Table of Contents

Chapter Twenty
FOUNDATIONS

A critical consideration for the satisfactory performance of any structure is the proper selection and design of foundations that will provide adequate resistance, tolerable lateral and vertical movements and aesthetic compatibility. This Chapter discusses MDT-specific criteria for the design of structural foundations for spread footings, driven piles and drilled shafts.

20.1 GENERAL

Chapter Twenty is based upon the LRFD design approach. The following summarizes the concepts in the LRFD Specifications.

20.1.1 LRFD Specifications

Considering basic design principles for foundations, the **LRFD Bridge Design Specifications** has implemented a major change compared to those principles in the former AASHTO **Standard Specifications for Highway Bridges**. The LRFD Specifications makes a clear distinction between the strength of the native materials (soils and rocks) supporting the bridge and the strength of the structural components transmitting force effects to these materials. The distinction is emphasized by treating the former in Section 10 "Foundations" and the latter in Section 11 "Abutments, Piers and Walls." It is necessitated by the substantial difference in the reliability of native materials and man-made structures. The foundation provisions of the LRFD Specifications are essentially strength design provisions with a primary objective to ensure equal, or close to equal, safety levels in all similar components against structural failure. The target safety levels for each type of foundation are chosen to achieve a level of safety comparable with that inherent in those foundations designed with the former Standard Specifications This approach differs from that for superstructures, where a common safety level has been selected for all superstructure types.

Historically, the primary cause of bridge collapse has been the washout of native materials. Other substructure/foundation failures, other than those precipitated by vessel or vehicular collision, are virtually non-existent. Accordingly, the LRFD Specifications introduced a variety of strict provisions in scour protection, which may result in deeper foundations.

To ensure maximum efficiency, the LRFD Specifications requires that components of the substructure foundation be analyzed and proportioned no differently from those of the superstructure. In practical terms, this means that force effects in the substructure and between the substructure and foundation are determined by analysis, as appropriate, and factored according to Section 3 of the LRFD Specifications. Loads generated by earth pressures can be determined with assistance from Section 11. Then, the nominal and factored resistance of the substructure is computed according to Section 10. The geotechnical resistance factors provided in Tables 10.5.5-1, -2 and -3 of the LRFD Specifications are approximately half of those provided for structural components. This is the justification for the new design philosophy, which permits the designer to tailor the level of design sophistication to the size, importance and appearance of the bridge.

Sections 10 and 11 of the LRFD Specifications are largely based on NCHRP Report 343 **Manuals for the Design of Bridge Foundations**.

20.1.2 Required Information

Prior to the design of the foundation, the designer must have a knowledge of the environmental, climatic and loading conditions expected during the life of the proposed unit. The primary function of the foundation is either 1) to spread concentrated loads over a sufficient area to provide adequate bearing capacity and limitation of movement, or 2) to transfer loads from unsuitable foundation strata to suitable strata. Therefore, a knowledge of the subsurface soil conditions, location and quality of rock, ground water conditions, and scour and frost effects is necessary.

For the Geotechnical Section to perform its analysis, the Section must know the magnitude and types of loads that require support. The accepted practice is for the Bridge Bureau to report the loads and reactions from the bridge superstructure at the ground line. The Geotechnical staff will analyze only the portion of the shaft or pile below ground. The Geotechnical Engineer will forward the L-Pile files to the bridge designer for the Bridge Bureau's use in designing portions of the bridge substructure that are above ground line, using the same program as Geotechnical. The bridge designer uses the L-Pile files to examine pile top deflections and the behavior of the substructure under different loading combinations. If any modifications are necessary, such as embedment depth or pile diameter, then the geotechnical designer will perform a new foundation analysis.

Conceptual axial loads need to be provided with the Core Request for the Geotechnical Section to determine proper methods for core sampling. Service loads should be reported. After sampling and analysis, Geotechnical will deliver core logs to the Bridge Bureau and a recommendation for foundation type.

Geotechnical will analyze the foundation with these loads. The Section will send a preliminary report to the Bridge Bureau plus an L-Pile file containing the soils and foundation information. The bridge designer may use the file to analyze the substructure elements above ground.

If the final axial design loads are within 10% of the preliminary loads used for the foundation analysis, no further foundation analysis is necessary. If the final design loads are outside this envelope, discuss with Geotechnical the necessity and advisability for further foundation analysis. Figure 20.1A shows the format and information provided in a typical core request.

20.1.3 Selection of Foundation Type

Section 13.4.8 discusses those types of foundations used by MDT and the general criteria which influence the selection of a foundation type.

Typically, the selection of a foundation type is based on the foundation investigation and recommendations by the Geotechnical Section. The selection is made by examining the test boring data, the existing ground lines, whether or not the proposed roadway is below, at or above the existing ground line, and hydraulic considerations such as scour depth or the desirability of a multidirectional pier.

20.1.4 Location of Bottom of Foundation

Figure 20.1B provides guidance on basic design criteria for the elevations of footings and pile tips.

20.1.5 Foundation Approval

The Bridge Bureau selects a structural foundation type based on boring information and the Geotechnical Section's recommended foundation type. The information presented for consideration of foundation type should include the following:

1. logs of subsurface investigation;

2. plan and elevation showing proposed foundations with applicable test borings plotted at the proper location and elevation;

3. allowable foundation pressure or type, size and maximum allowable load of piles. When piles are proposed, the estimated pile tip elevation at each foundation must be shown; and

4. finished ground elevation at the face of substructure.

MONTANA DEPARTMENT OF TRANSPORTATION
Helena, Montana 59620

MEMORANDUM

TO: Kent Barnes, P.E.
Materials Engineer

FROM: Joseph P. Kolman, P.E.
Bridge Engineer

DATE: August 29, 2001

SUBJECT: F-NH 1-3(20)247
Cut Bank West
CN 1310

Please furnish Borings and Foundation Recommendations for the proposed replacement bridge over Cut Bank Creek on US 2 just west of the community of Cut Bank. The new bridge will be constructed on a new alignment located just downstream from the existing bridge. The Alignment Review activity is complete and the alignment approved.

The attached bridge layout represents our proposed structure concept for this site. The preliminary estimated DL + LL reactions at the ground line are indicated below. Lateral loads will be furnished later as the design progresses.

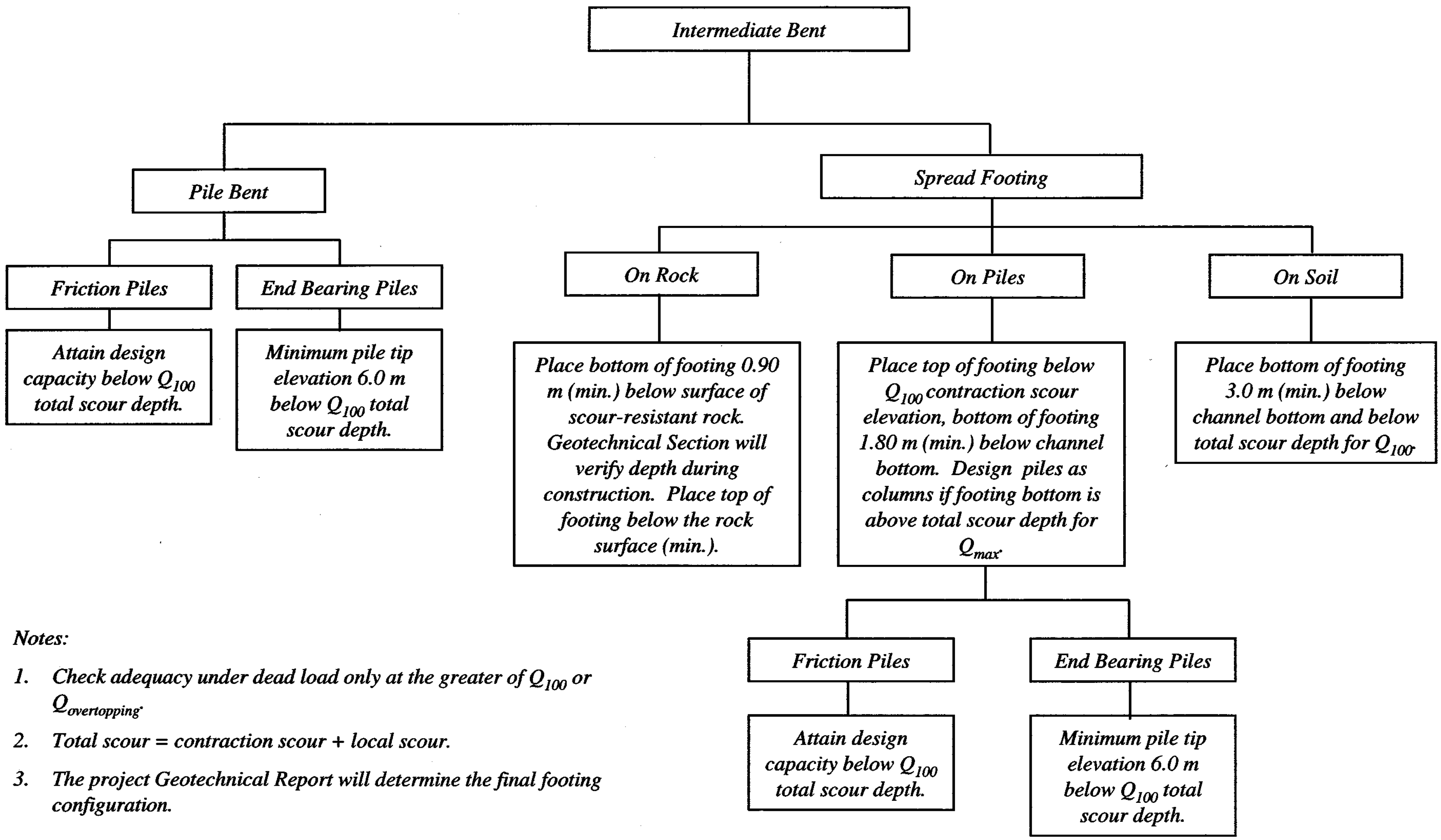
End Bents: 634 kn/pile (estimate based on 5 beam lines and 2 piles/beam at each end)
Int. Piers: 9127 kn/drilled shaft (we plan to use 1 drilled shaft/pier)

Project Location:
Township/Range/Section
Glacier County:
T33N, R6W, Section 11 (See attached Map)

JPK:rwm:1310core-req

Attachments
CC: M.P. Johnson – Great Falls
R.E. Williams – Road Design
K.M. Barnes, w/1 attached
J.J. Moran, Geotechnical Section, w/1 attach
A.Kornec, Core Drill Section, w/1 attach
G.J. Stockstad, Environmental Services, w/1 attach
R.W. Modrow, Bridge Bureau, w/1 attach
Bridge File, w/1 attach

Figure 20.1A



Notes:

1. *Check adequacy under dead load only at the greater of Q_{100} or $Q_{overtopping}$.*
2. *Total scour = contraction scour + local scour.*
3. *The project Geotechnical Report will determine the final footing configuration.*

FOUNDATION DESIGN CRITERIA
(Intermediate Bents)

Figure 20.1B

20.2 SPREAD FOOTINGS AND PILE CAPS

Spread footings are typically thick, reinforced concrete slabs sized to meet the structural and geotechnical loading requirements for the proposed structural system. Spread footings are used to support piers, bents, abutments and retaining walls where suitable soils or rock are located at a relatively shallow depth. Suitable material is usually construed as being material where the last two blow counts are 35 or greater on a standard SPT test at a depth of less than 3 m. Factors affecting the size of the footings are the structural loading versus the ability of the soil to resist the applied loads.

Where suitable materials lie below the depth that can be excavated economically or where no firm layers were identified in the subsurface exploration, a deep foundation may be used. Piles and drilled shafts are the most common types of deep foundations used in Montana. See Section 20.3 and 20.4 for further discussion on piles and drilled shafts.

Deep foundations typically use an intermediate member called a pile cap and piles or drilled shafts to transfer the structural loads to strata that is capable of resisting the loads. Pile caps appear similar to spread footings but differ in that they transmit the loads to piles or drilled shafts instead of directly to the soil below the cap.

20.2.1 Minimum Dimensions/Materials

The following criteria shall apply:

1. Footing and Cap Thickness:
   a. Spread Footings: 600 mm
   b. Pile Caps: 750 mm
2. Class of Concrete: Typically Class DD, except for underwater placements where Class DS is used.
3. Compressive Strength: 28 day (for structural design):
   a. DD: f'_c = 21 MPa
   b. DS: f'_c = 17 MPa
4. Reinforcing Steel: f_y = 420 MPa

20.2.2 Footing Thickness and Shear Design

Reference: LRFD Articles 5.8.3, 5.13.3.6 and 5.13.3.8

The footing thickness may be governed by the development length of the footing dowels (footing to wall or column) or by concrete shear requirements. Generally, shear reinforcement in footings should be avoided. If concrete shear governs the thickness, it is usually more economical to use a thicker footing unreinforced for shear instead of a thinner footing with shear reinforcement. Footing thicknesses will be increased in 50-mm increments.

20.2.3 Depth and Cover

Reference: LRFD Articles 2.6.4.4.2 and 10.6.1.2

The vertical location of a footing should satisfy the following criteria. These criteria are summarized in Figure 20.1B.

20.2.3.1 Bottom of Footings

The following applies:

1. Footings on Soil. The bottom of footings on soil shall be set at least 3.0 m below the channel bottom and below the total scour depth determined for Q_{100}.
2. Footings on Rock. Small embedments (keying) should be avoided because blasting to achieve keying frequently damages and renders it the sub-footing rock structure more susceptible to scour. If footings on

smooth massive rock surfaces require lateral restraint, steel dowels should be drilled and grouted into the rock below the footing level. The bottom of the footings should be at least 0.9 m below the surface of scour-resistant rock with the top of the footings at least below the rock surface.

3. Footings on Erodible Rock. Weathered or other potentially erodible rock formations need to be carefully assessed for scour. The Geotechnical Section should be consulted for the spread footing foundation. The decision should be based upon an analysis of intact rock cores, including rock quality designations and local geology, hydraulic data and anticipated structure life. An important consideration may be the existence of a high-quality rock formation below a thin weathered zone. For deep deposits of weathered rock, the potential scour depth for the design flood for scour should be estimated and the footing base located so that the top of the footing is below the estimated contraction plus local scour. The excavation above the top of the spread footing is usually backfilled with the same material that was excavated.

4. Footings Placed on Tremie Seals and Supported on Soil. The location of the top of the footing to be placed on a seal is determined in the same manner as a footing placed directly on the ground. That is, the bottom of the footing is below the estimated scour depth at the design flood. The elevation at the bottom of the footing is the same as the top of the seal. The required seal depth is then calculated assuming that the contractor will have to dewater the cofferdam to place the footing "in the dry." The seal mass counteracts the buoyant forces that occur when the cofferdam is dewatered. This depth is typically 40% of the head from the bottom of the seal to the normal water elevation. This 40% is simply the ratio of $\gamma_{water}/\gamma_{concrete}$. To help accommodate construction uncertainties while locating the cofferdam in the channel, the length and width of the seal are 1 m greater than the dimensions of the footing. This allows for minor "adjustments," if necessary, to position the footing for the pier correctly.

20.2.3.2 Top of Footings

The top of the footing on dry land shall have a minimum of a 300-mm permanent earth cover.

20.2.4 Bearing Resistance and Eccentricity

Reference: LRFD Article 10.6.3

For spread footings, the Geotechnical Section will provide the factored nominal bearing resistance to the Bridge Bureau. (The nominal bearing resistance is what was traditionally called the allowable bearing capacity.) The maximum factored design bearing pressure is shown on the Structural Plans for the footing.

20.2.4.1 Soils under Footings

Reference: LRFD Article 10.6.3.1.5

In contrast to the approach in the former Standard Specifications, a reduced effective footing area based upon the calculated eccentricity is used to include these effects. Uniform design bearing pressure is assumed over the effective area. This uniform-pressure model acknowledges the plastic nature of soil. An example is provided in Figure 20.2C.

The location of the resultant of the center of pressure based upon factored loads should be within the middle ½ of the base.

20.2.4.2 Rock

Reference: LRFD Article 10.6.3.2.5

Following the traditional approach, a triangular or trapezoidal pressure distribution is assumed for footings on rock. This model acknowledges the linear-elastic response of rock.

The location of the resultant center of pressure based upon factored loads should be within the middle ¾ of the base.

20.2.5 Sliding Resistance

The approximate coefficients of friction in Figure 20.2A may be used to check the sliding resistance unless more exact coefficients are established for a particular case.

Keys in footings to develop passive pressure against sliding are not very effective and their economic justification is often over estimated. However, when it becomes necessary to use a key, the designer shall submit studies to the Bridge Area Engineer during preliminary design.

Concrete On:	Coefficient of Friction
Wet Clay or Silty Clay	0.33
Sand, Silty or Clay Gravel	0.40
Coarse Grain Soil with Silt	0.45
Dry Clay	0.50
Coarse Grain Soil without Silt	0.55
Gravel and Sand	0.60
Sound Rock or Concrete	Place footings against rock

Reference: 1978 MDT Structures Design Manual.

SLIDING RESISTANCE
Figure 20.2A

20.2.6 Settlement

Reference: LRFD Articles 3.12.6, 10.6.2.2 and 10.7.2.3

Differential settlement (SE) is considered a load in the LRFD Specifications. Generally, due to the methods used to determine allowable foundation loads by MDT, force effects due to differential settlement need not be investigated. If varying conditions exist, settlement will be addressed in the Geotechnical Report and the following effects should be considered:

1. Structural. The differential settlement of substructures causes the development of force effects in continuous superstructures. These force effects are directly proportional to structural depth and inversely proportional to span length, indicating a preference for shallow, large-span structures. They are normally smaller than expected and tend to be reduced in the inelastic phase. Nevertheless, they may be considered in design if deemed significant, especially those negative movements which may either cause or enlarge existing cracking in concrete deck slabs.

2. Joint Movements. A change in bridge geometry due to settlement causes movement in deck joints which should be considered in their detailing, especially for deep superstructures.

3. Profile Distortion. Excessive differential settlement may cause a distortion of the roadway profile that may be undesirable for vehicles traveling at high speed.

4. Appearance. Viewing excessive settlement may create a feeling of lack of safety.

20.2.7 Reinforcement

Reference: LRFD Articles 5.10.8 and 5.13.3

Unless other design considerations govern, the reinforcement in footings should be as follows:

1. Longitudinal Steel. Place longitudinal distribution bars on top of the primary transverse steel for the top mat of footing reinforcement. Place the transverse steel on top of the longitudinal steel for the bottom mat of footing reinforcement.

2. Embedment Length. Bar embedment lengths shall be shown on the plans. In spread footings, hooks may be omitted on transverse footing bars unless bond calculations dictate otherwise.

   Vertical steel extending upwards out of the footing shall also extend down to the bottom footing steel and shall be hooked on the bottom end regardless of the footing thickness.

3. Spacing. In spread footings, the spacing of reinforcing steel shall not exceed 150 mm in either direction.

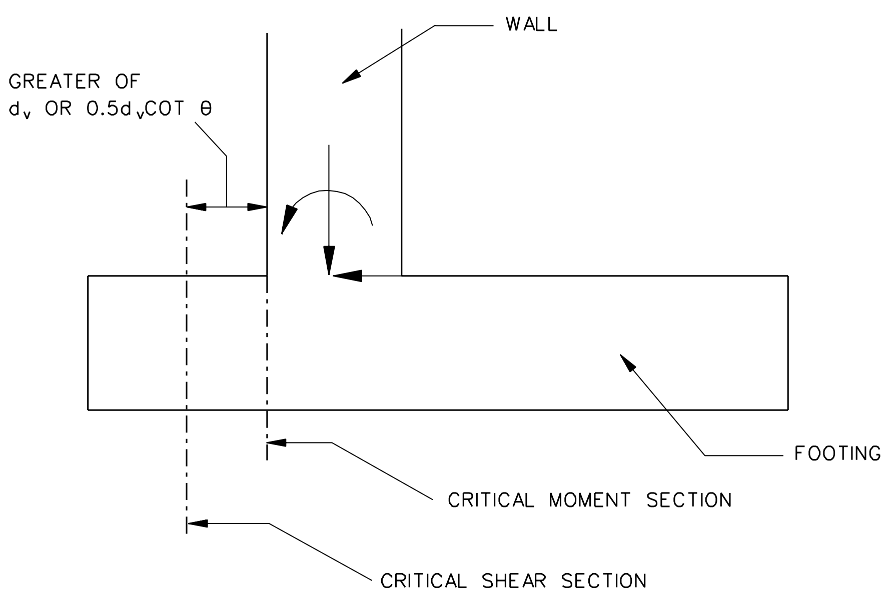
4. Other Reinforcement Considerations. Article 5.13.3 in the LRFD Specifications specifically addresses concrete footings. For items not included, the other relevant provisions of Section 5 should govern. For narrow footings, to which the load is transmitted by walls or wall-like piers, the critical moment section shall be taken at the face of the wall or pier stem and the critical shear section a distance equal to the larger of "d_v" (effective shear depth of the footing) or "0.5 $d_v \cot \theta$" from the face of the wall or pier stem where the load introduces compression in the top of the footing section. See Figure 20.2B. For other cases, either Article 5.13.3 is followed, or a two-dimensional analysis may be used for greater economy of the footing.

20.2.8 Joints

Footings do not generally require expansion joints. Where used, footing construction joints should be offset 600 mm from expansion joints or construction joints in walls and should be constructed with 75-mm deep keyways placed in the joint.

20.2.9 Stepped Footings

The difference in elevation of adjacent stepped footings should not be less than 150 mm. The lower footing should extend 600 mm under the adjacent higher footing, or an approved anchorage system may be used.



CRITICAL SECTIONS FOR MOMENT AND SHEAR FOR WALLS OR WALL-LIKE PIERS

Figure 20.2B

20.2.10 Additions to Existing Footings

At the interface between existing and new footings, existing concrete should be removed as needed to provide adequate development length for lap splicing of existing reinforcement, or an approved anchorage system may be used.

Where the substructure of an existing structure is extended, the old footing with respect to the new footing should be shown on the new Footing Plan Sheet.

20.2.11 Cofferdams

The purpose of a cofferdam is to provide a protected area within which an abutment or a pier can be built. A cofferdam in general is a structure consisting of steel sheeting driven into the ground and below the bottom of the footing elevation and braced to resist pressure. It should be nearly watertight and capable of being dewatered.

Generally, cofferdams are designed and detailed by the Contractor and reviewed by the Construction Bureau.

20.2.12 Field Integrity Testing

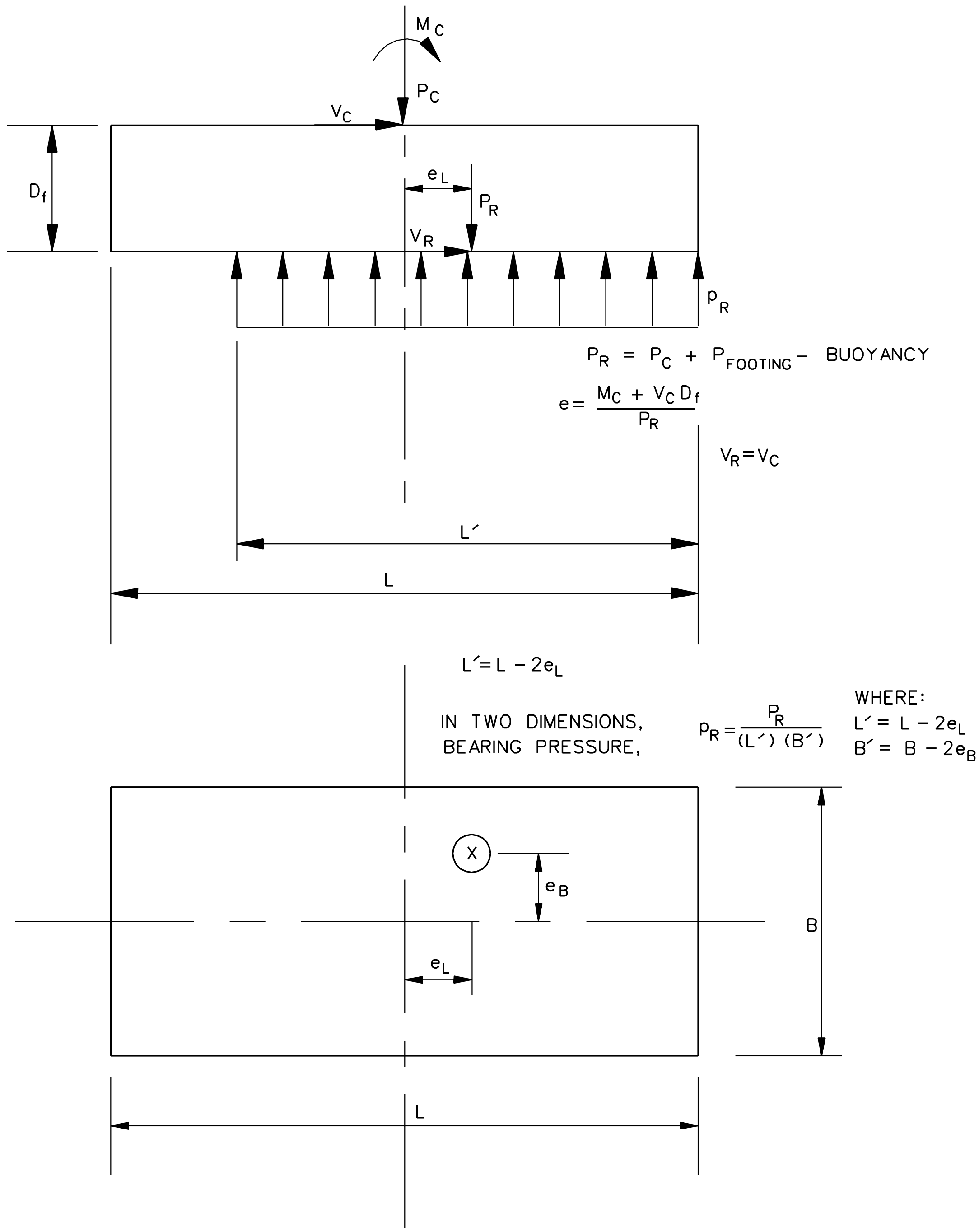
All excavations for spread footings are tested to check the integrity of the subsoil and to determine if it is necessary to adjust the footing elevations.

20.2.13 Design Example of Analysis of a Spread Footing on Competent Soil

See Figure 20.2C for a schematic example of a footing on soil to support an interior pier at a stream crossing.

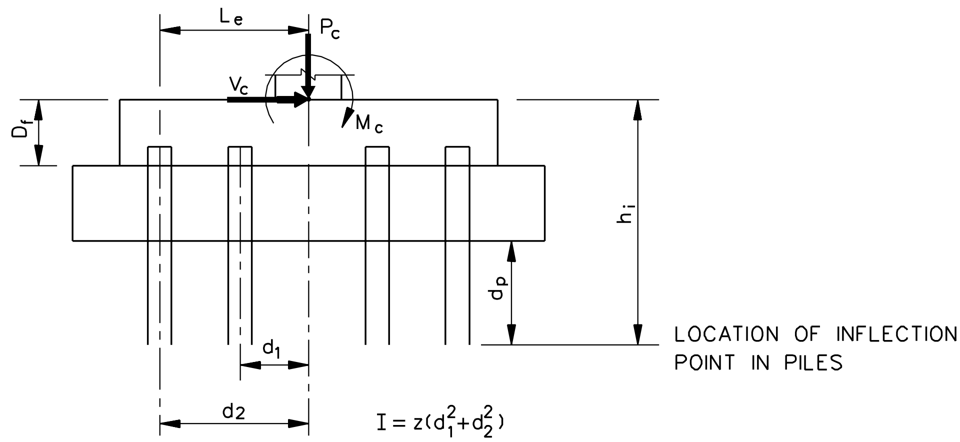
20.2.14 Design Example of Analysis of Pile-Supported Footings

See Figure 20.2D for a schematic example of the analysis of a pile-supported footing to support an interior pier at a stream crossing (fixed pile connection). See Figure 20.2E for a similar footing assuming a pinned pile connection.



ANALYSIS OF SPREAD FOOTING ON COMPETENT SOIL

Figure 20.2C



Assumptions: Pile cap is rigid. Pile connections are fixed and shear forces per pile are significant.

Footing is considered rigid if $L_e/D_f \leq 2.2$
$P_R = P_c + P_{footing} + P_{seal} - \text{Buoyancy}$

To obtain forces in piles, sum moments about inflection point:

$$P_{max} = \frac{P_R}{\#\text{ of piles}} + \frac{(V_c h_i + M_c)\, d_2}{I_z}$$

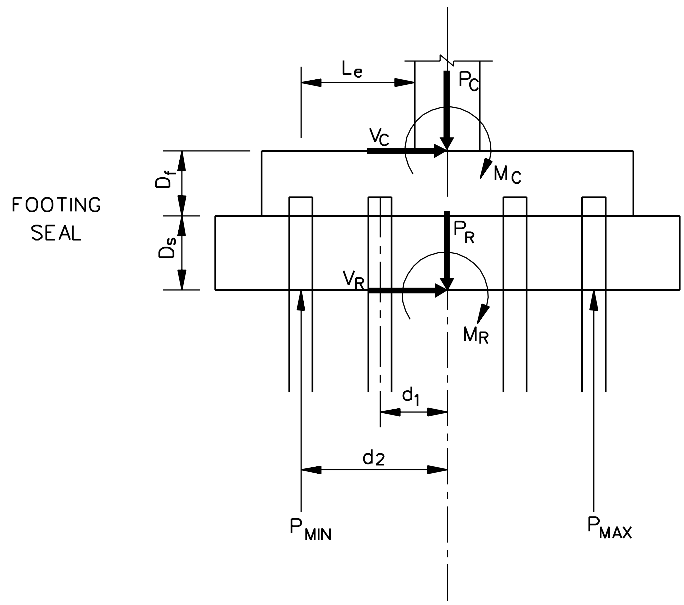
$$P_{min} = \frac{P_R}{\#\text{ of piles}} - \frac{(V_c h_i + M_c)\, d_2}{I_z}$$

$$V_{pile} = \frac{V_c}{\#\text{ piles}}$$

$$M_{pile} = V_{pile}\, d_p$$

PILE FOOTING ANALYSIS
(Fixed Pile Connection)

Figure 20.2D



Assumptions: Pile cap is rigid. Pile connections are pinned or shear force in pile is small.

$P_R = P_c + P_{footing} + P_{seal} - \text{Buoyancy}$

$V_R = V_c - V_{\text{passive soil pressure on footing and seal}}$ *Note: Passive soil pressure is typically ignored.*

$M_R = M_c + V_c\,(D_f + D_s)$

Footing is considered rigid if $L_e/D_f \leq 2.2$

Pile Loads:

$$P_{max} = \frac{P_R}{\#\text{ piles}} + \frac{M_R\ d_2}{\Sigma d_i^2}$$

$$P_{min} = \frac{P_R}{\#\text{ piles}} - \frac{M_R\ d_2}{\Sigma d_i^2}$$

PILE FOOTING ANALYSIS
(Pinned Pile Connection)

Figure 20.2E

20.3 PILES

20.3.1 Pile Selection and Design

Where underlying soils provide inadequate bearing capacity or excessive settlement, piles may serve to transfer loads to deeper suitable strata. Piles may function through skin friction and/or through end bearing. Required bearing capacity, soil conditions and economic considerations determine pile type. MDT traditionally uses steel pipe piles, steel H-piles and fluted steel piles.

The following applies to piles:

1. Verify the design of all piles by engineering analysis.

2. Unless project conditions require otherwise, use one pile size and type throughout a project. Mixing pile types or sizes on a project increases cost and the likelihood of construction errors.

3. Require a two-component epoxy paint meeting the requirements of the **Standard Specifications** to protect any part of the pile exposed to the environment. The paint must extend at least 600 mm below the channel bottom or ground line.

4. Preliminary Design Assumption. For preliminary design purposes only, select the number of piles on the basis of allowing a maximum service-load stress of 62.0 MPa.

20.3.2 Types

20.3.2.1 Steel Pipe Piles

Reference: LRFD Articles 6.9.5 and 6.12.2.3

In addition to the information contained in the LRFD references, the following applies to the use of steel pipe piles in MDT projects:

1. Usage. Steel pipe piles may serve as bearing piles, as friction piles, or as a combination of the two.

2. Diameter. MDT normally uses pipe piles either 406 mm or 508 mm in diameter, with a wall thickness of 12.7 mm in either case. MDT may accept other sizes with prior approval from the Bridge Area Engineer.

3. Concrete Fill. The contractor will fill each pipe pile with Class DD concrete after driving with concrete having a compressive strength of at least f'_c = 21.0 MPa.

4. Neglect Concrete for Design. Except in rare, extreme design problems, MDT does not include the concrete in analyzing steel pipe pile capacity for design. Design the pipe pile without the concrete. The concrete will provide additional conservation to the design.

20.3.2.2 Steel H-Piles

The following will apply to steel H-piles:

1. Usage. These are generally used either where the pile obtains most of its bearing resistance from end bearing on rock or as recommended in the Geotechnical Report.

2. Size. Pile size designations may be HP310 or HP360; HP310 is typical.

20.3.2.3 Fluted Steel Piles

The following will apply to fluted steel piles:

1. Usage. Fluted steel piles are generally used only in deep, soft materials.

2. Size. The gage of the pile wall thickness may be 9 (3.8 mm), 7(4.6 mm), 5 (5.3 mm) or 3 (6.1 mm) gage.

20.3.3 Pile Length

Reference: LRFD Articles 10.7.1.10, 10.7.1.11 and 10.7.1.12

If a pile foundation is determined to be the appropriate solution to the structural and geotechnical specifics at the site, the length of the piles will be estimated based on information in the Geotechnical Report. The following is provided to guide the designer through the decision-making process in determining pile length:

1. Minimum Length. In special cases, it will be necessary to specify the minimum length of piles in the plans. Piles should be a minimum of 3.0 m in length and, unless refusal is encountered, penetrate into hard cohesive or dense granular original soil not less than 3.0 m. If the depth to suitable rock strata is less than 3.0 m, MDT practice is to seat the pile in holes cored in the rock. A minimum core depth of 1.0 m into scour-resistant rock is recommended. Where piles less than 3.0 m in length are anticipated, consideration shall also be given to lowering the elevation of the bottom of footing and providing spread footings instead.

2. Tip Elevation for Friction Piles. Show the minimum pile tip elevation from the Geotechnical Report on the drawing of the structural element.

3. Tip Elevation for Point Bearing Piles. Show the minimum pile tip elevation from the Geotechnical Report on the drawing of the structural element. The bottom of the tip is usually placed some distance into the formation material to ensure that it is through any weathered surficial material and into competent rock.

4. Pile Tip Elevation Guidelines. Figure 20.1B provides guidance for use in determining minimum pile tip elevations.

20.3.4 Design Details

Reference: LRFD Article 10.7.1

The following will apply to the design of piles:

1. Battered Piles. The use of battered piles must be justified by analysis. When used, a pile batter of 12 vertical to 2 horizontal is considered desirable. However, piles may be battered to a maximum of 4 vertical to 1 horizontal where substantial resistance is not otherwise attainable. For the outside row of piles in footings, a batter should be provided on alternating piles. Where closely spaced battered piles are used, the pile layout should be checked to ensure that battered piles do not intersect. Battered piles should not be employed where extensive downdrag load is expected because this load causes flexure in addition to axial force effects. Battered piling can not be used within cofferdams.

2. Spacing. Spacing of piles is specified in Article 10.7.1.5 in the LRFD Specifications. Center-to-center spacing should not be less than the greater of 750 mm or 2.5 times the pile diameter or width of pile. The distance from the side of any pile to the nearest edge of footing shall be greater than 250 mm.

3. Embedment. Embed piles a minimum of 500 mm into the footing after all damaged pile material has been removed. If pile reinforcement is extended into the footing, satisfying the provisions of LRFD Article 5.13.4.1, the embedment length may be reduced. Pile connections with high tensile loads or moments require additional design considerations.

4. Downdrag (DD) Loads. When a pile penetrates a soft layer subject to settlement, the force effects of downdrag or negative loading on the foundations must be evaluated. Downdrag acts as an additional permanent axial load on the pile. If the force is of sufficient magnitude, structural failure of the pile or a bearing failure at the

tip is possible. At smaller magnitudes of downdrag, the pile may cause additional settlement. For piles that derive their resistance mostly from end bearing, the structural resistance of the pile must be adequate to resist the factored loads including downdrag. Battered piles should be avoided where downdrag loading is possible due to the potential for bending of the pile. Downdrag forces can be mitigated by preboring and filling the prebored hole with pea gravel, or by building the approach fill far enough in advance of the pile driving for the fill to settle out.

5. Uplift Forces. Uplift forces can be caused by lateral loads, buoyancy or expansive soils. Piles intended to resist uplift forces should be checked for resistance to pullout and structural resistance to tensile loads. The connection of the pile to the footing must also be checked.

6. Laterally Loaded Piles. The resistance of laterally loaded piles must be estimated according to approved methods. Several methods exist for including the effects of piles and surrounding soil into the structural model for lateral loadings including seismic loads. These methods are discussed in Section 20.4.2.

7. Group Effect. Minimum spacing requirements are not related to group effect. Group effects are specified in the LRFD Specifications in Article 10.7.3.7.3 and in Article 10.7.3.10.

8. Pile Tips. Use pile tips to minimize damage to the piles.

9. Pile Loads. The pile load shall be shown in the Plans. This information will help ensure that pile driving efforts during the construction process will result in a foundation adequate to support the design loads. The load to which piles are to be driven shall be greater than or equal to the total factored load. The governing strength limit state load combination from LRFD Table 3.4.1-1 shall also be indicated. Pile design loads are typically limited to less than 900 kN to help maintain the competition among local contractors, who would otherwise be forced to rent larger equipment if they had to drive to higher pile capacities.

10. Pile Load Tests. Where pile design loads are high or where the pile quantity is large, pile load tests may be justified. The designer should consult MDT's Geotechnical Section if considering pile load testing. Test locations should be shown in the plans or described in the special provisions.

20.4 DRILLED SHAFTS

Reference: LRFD Article 10.8

20.4.1 Design

The following will apply to the design of drilled shafts:

1. Usage. Drilled shafts may be an economical alternative to driven piles. Drilled shafts should also be considered to resist large lateral or uplift loads when deformation tolerances are relatively small. Drilled shafts derive load resistance either as end-bearing shafts transferring load by tip resistance or as friction shafts transferring load by side resistance.

2. Socketed Shafts. A schematic drawing of a rock-socketed shaft is shown in Figure 20.4A. Where casing through overburden soils is required, design the shaft as one size and do not step down when going into formation material.

3. Column Design. Because even soft soils provide sufficient support to prevent lateral buckling of the shaft, it may be designed according to the criteria for short columns in Article 5.7.4.4 of the LRFD Specifications. If the drilled shaft is extended above ground to form a pier or part of a pier, it should be analyzed and designed as a column. The effects of scour around the shafts must be considered in the analysis.

4. Reinforcement. The shaft will have a minimum of 0.8 percent of the gross concrete area and will extend from the bottom of the shaft into the footing. If the drilled shaft is extended above ground level, reinforcement should satisfy the requirements of Article 5.7.4.2 in the LRFD Specifications.

20.4.2 Pile and Drilled Shaft Modeling

Several possibilities exist for including the effects of piles and surrounding soil into the structural model for lateral loadings including seismic loads. Two of these methods are summarized in Figure 20.4B and include:

1. equivalent cantilever model, and
2. equivalent soil springs model.

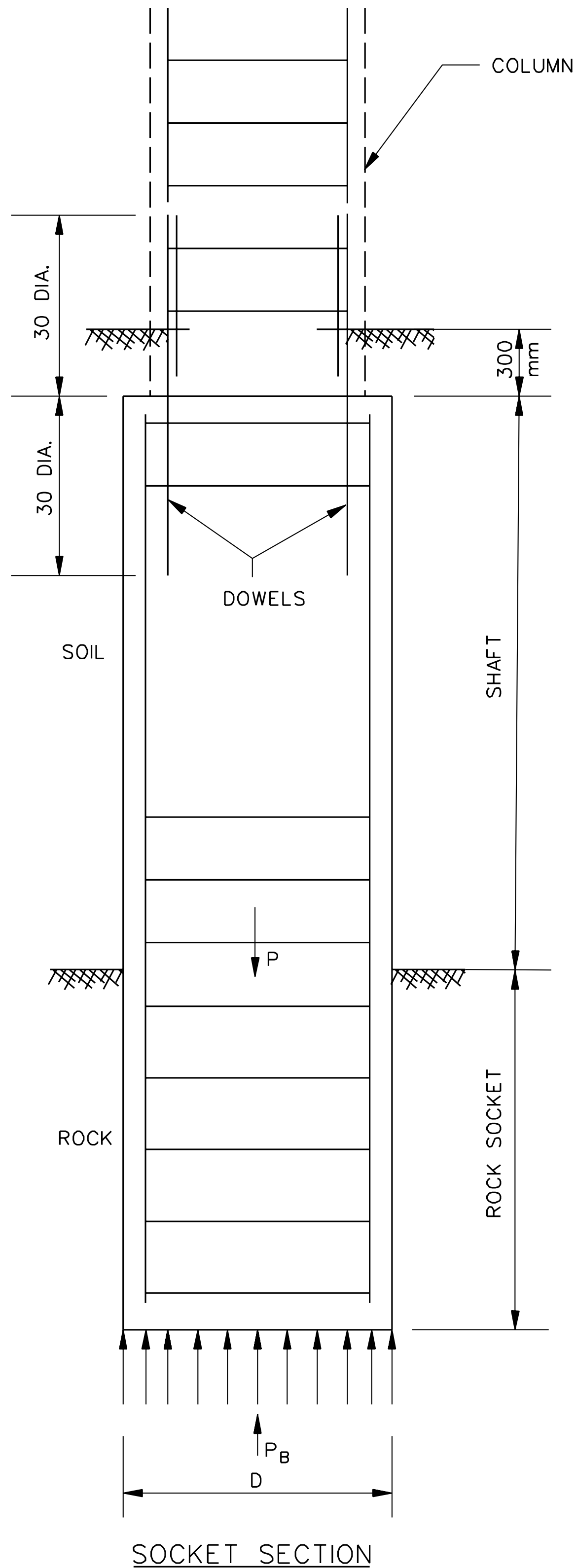
The simplest approach is to assume that an equivalent cantilever column can be used to model the pile. The section of the cantilever is the same as that of the pile but its length (depth to "fixity") is adjusted so as to give either the same stiffness at ground level or the same maximum bending moment as in the actual soil-pile system.

The length to fixity of the equivalent cantilever can be determined from charts such as those in Figures 20.4C and 20.4D, which are for large diameter concrete piles, or from equations relating the stiffnesses of the pile and soil given in Figure 20.4E. The soil constants, K_h and n_h, for use in the equations are subsequently given in Figure 20.4F.

In most cases, the use of either the charts or the relative stiffness formulation will give satisfactory results, eliminating the need for a detailed foundation model. Note that the charts give only the effective depth for stiffness considerations, and pile moments based on this length will be overestimated. It should also be noted that the two methods (charts, relative stiffnesses) give different results for the effective depth to fixity. This is in part a reflection of the uncertainty associated with foundation engineering. However, both methods provide a rational and simple way for including foundation flexibility in the analysis of bridges, and results using either method will be closer to the actual behavior than will results from a model which rigidly fixes the bridge at ground level.

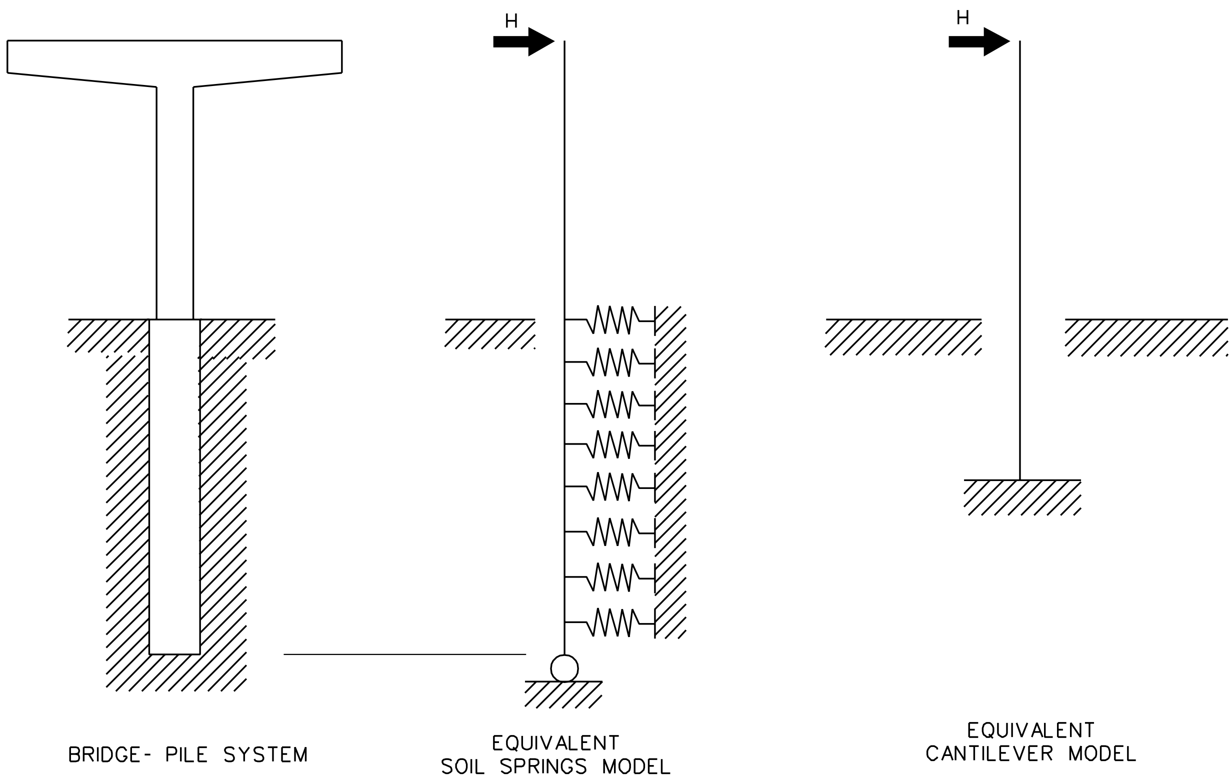
Typical ranges for the effective length to fixity (for stiffness) are from 3 to 9 pile diameters, the low end of the range being for very stiff sites. It should be noted that this depth to fixity is potentially a function of the direction of loading, because pile group effects may be different longitudinally and transversely. In the absence of more specific information, the effective modulus of horizontal subgrade reaction (K_h) for each pile may be assumed to vary linearly from 25% of the K_h value for a single pile, when the spacing in the direction of load is 3 pile diameters, to the K_h value for a single pile, when the spacing is 8 pile diameters.

The second technique noted above involves the use of p-y curves to represent the soil. This is the equivalent soil springs model. The advantage of this approach is the avoidance of the need to calculate equivalent spring constants as in the above method. The disadvantage is the substantial increase in the size and complexity of the structural model. The solution's accuracy is primarily a function of the spacing between nodes used to attach the soil springs to the pile (the closer the spacing, the better the accuracy), and is not so dependent on the pile itself. Simple beam column elements are usually adequate for modeling the pile behavior. The computer program LPILE is used by MDT to model equivalent soil springs.



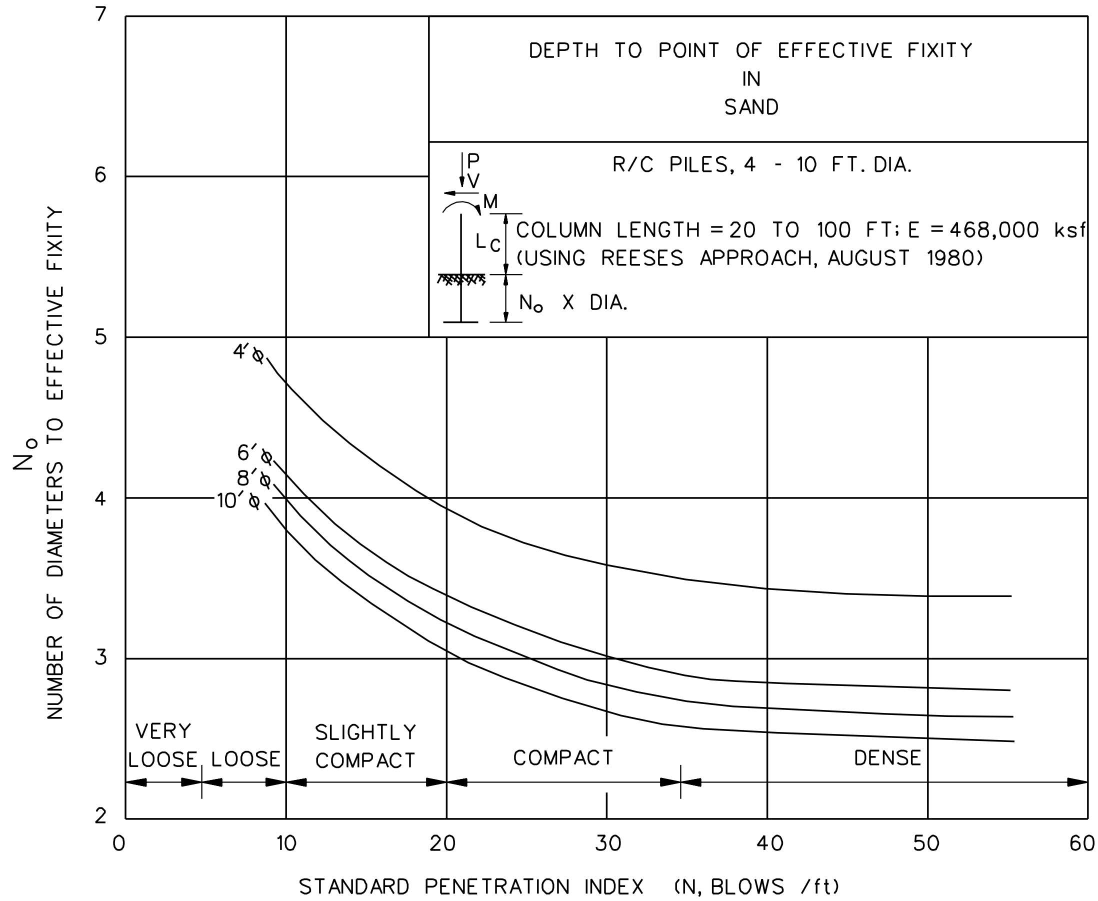
DRILLED SHAFTS

Figure 20.4A



METHODS OF REPRESENTING PILE FOUNDATION STIFFNESS

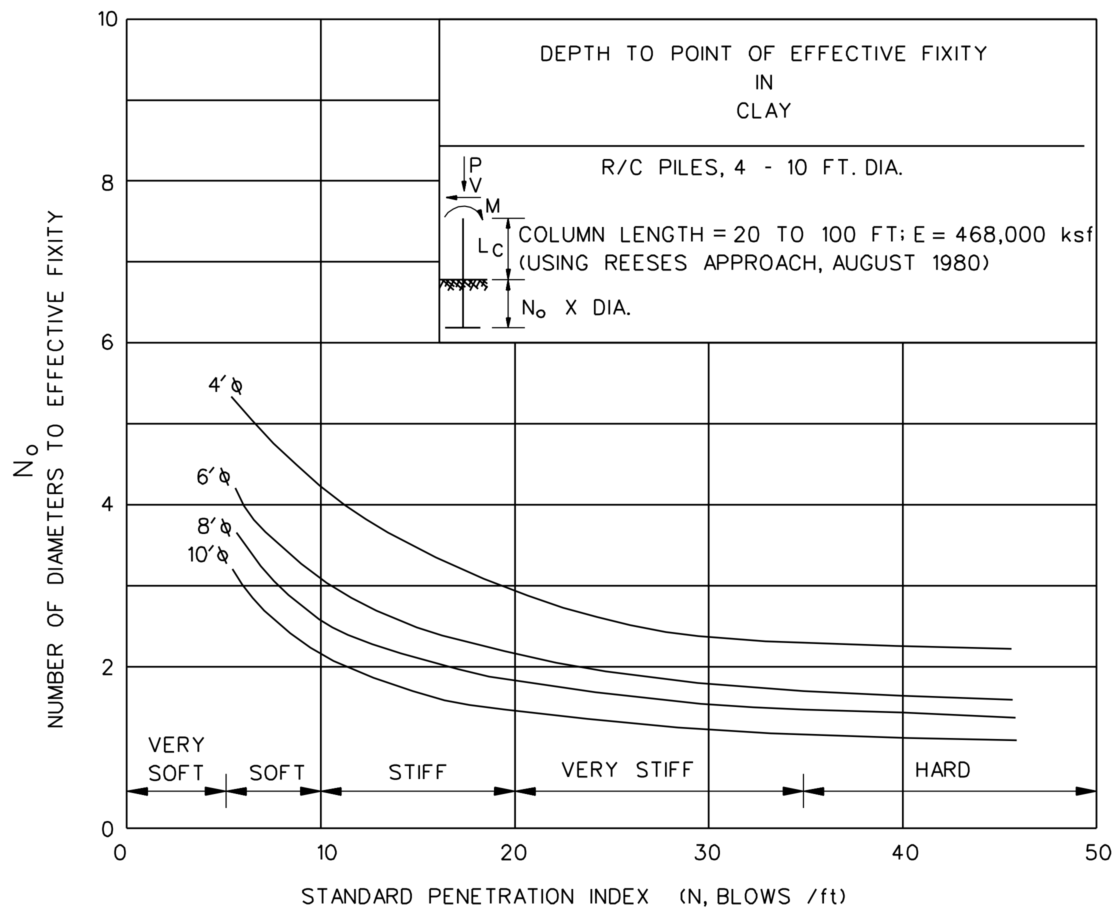
Figure 20.4B



0-28	28-30	30-36	36-41	APPROXIMATE φ IN DEGREES

DEPTH TO POINT OF EFFECTIVE FIXITY FOR DRILLED SHAFTS IN SAND

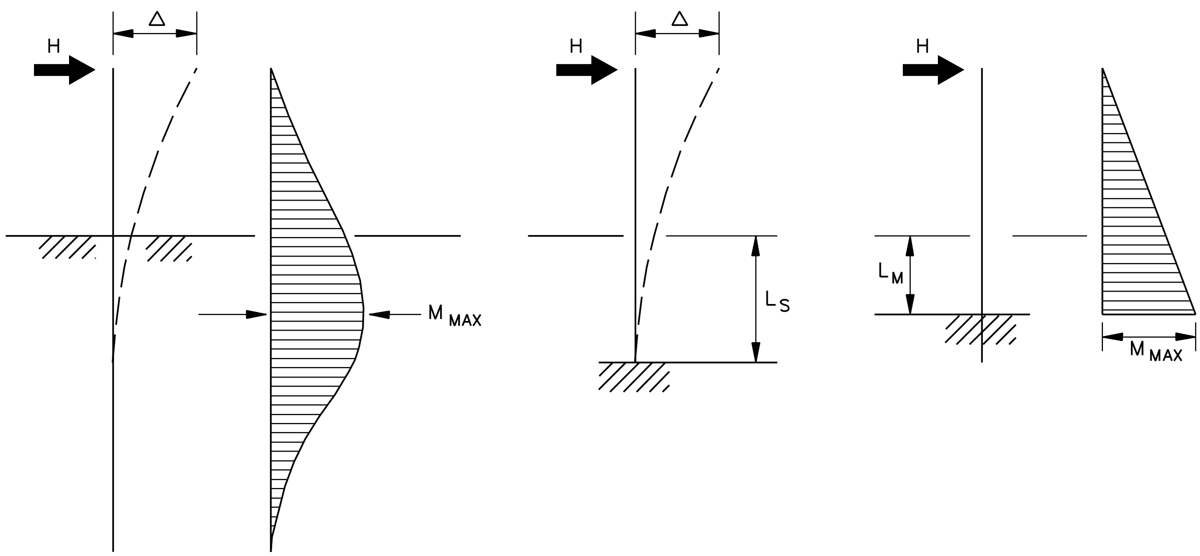
Figure 20.4C



0-0.5	0.5-1	1-2	2-4	4-6 SHEAR STRENGTH, ksf

DEPTH TO POINT OF EFFECTIVE FIXITY FOR DRILLED SHAFTS IN CLAY

Figure 20.4D



	L_S	L_M
Cohesive Soil Constant (K_h)	$1.4 \sqrt[4]{\frac{EI}{K_h}}$	$0.44 \sqrt[4]{\frac{EI}{K_h}}$
Cohesionless Soil Constant (n_h)	$1.8 \sqrt[5]{\frac{EI}{n_h}}$	$0.78 \sqrt[5]{\frac{EI}{n_h}}$

EQUIVALENT CANTILEVERED METHOD USING RELATIVE STIFFNESS FACTORS

Figure 20.4E

	Site Data			Design Parameters			
Soil Type	N (blows/ft)	Undrained Shear Strength (ksf)	ϕ' (degrees)	n_h (kips/ft^3)		K_h (kip/ft^2)	E_S (kip/ft^2)
Cohesionless Soils							
- Dense	30-50		45	100	60		
- Loose	4-10		30	15	6		
Cohesive Soils							
- Hard	20-60	3-15				375	520
- Medium	8-15	1-2				125	170
- Soft	2-4	0.3-0.6				30	40

N = standard penetration test resistance

K_h = modulus of horizontal subgrade reaction

n_h = constant of horizontal subgrade reaction $= \dfrac{dK_h}{d_z}$

E_s = soil modulus of elasticity

ϕ' = effective soil internal angle of friction

SUGGESTED SOIL STIFFNESS PARAMETERS FOR PRELIMINARY SEISMIC ANALYSIS

Figure 20.4F