This Design Guide has been developed to provide geotechnical and structural engineers with guidance in estimating geotechnical axial capacity of drilled shafts in rock. For the purposes of the design guide, the term "rock" is intended to refer to the geological deposits common to Illinois including limestone, dolomite, sandstone, as well as hard shale deposits with an unconfined compressive strength (q_u) exceeding 100 ksf. Drilled shafts in softer shale deposits having a q_u less than 100 ksf should be analyzed in accordance with the design guide for the Axial Capacity of Drilled Shafts in Shale.

Publication FHWA-NHI-10-016, "Drilled Shafts: Construction Procedures and LRFD Design Methods" (FHWA-DS) was published in 2010 and presented updated methods for determining the geotechnical axial capacity of drilled shafts in rock. These updates have since been incorporated into the AASHTO LRFD Bridge Design Specifications (LRFD) effective with the 7th edition (2014). The updated methods and equations are described below and are also reflected in a design spreadsheet available at: *Drilled Shaft Axial Capacity - Rock*

In order to determine the axial geotechnical capacity of drilled shafts in rock using the AASHTO LRFD formulas provided herein, engineers need to be able to determine the Geological Strength Index (GSI) of the rock according to the lithology and structure and surface conditions described for the rock samples. As such, it is important that geotechnical engineers and field personnel are familiar with the GSI classification method discussed in LRFD 10.4.6.4 and that adequate descriptions are provided on the Rock Core Log and/or in the Structure Geotechnical Report.

Geotechnical Axial Resistance of Drilled Shafts in Rock:

Per LRFD 10.8.3.5.4a, the factored geotechnical axial resistance of drilled shafts in compression in rock, R_R, may be determined considering either only side or tip resistance or a combination of both side and tip resistances. Maximum side and tip resistances are typically mobilized at different deformations with maximum side resistance often peaking prior to mobilizing the full tip resistance. This must be taken into consideration when determining capacities based upon a combination of side and tip resistance and is discussed in further detail in the following "Settlement Analysis" section. Factored geotechnical axial resistance of drilled shafts in compression should be determined using the following formulas:

$R_R = \phi R_n = \phi_{qs} R_s + \phi_{qp} R_p$ (LRFD Eqn. 10.8.3.5-1)

R_s = nominal shaft side resistance

= $q_s A_s$ (LRFD Eqn. 10.8.3.5-3)

R_p = nominal shaft tip resistance

= $q_p A_p$ (LRFD Eqn. 10.8.3.5-2)

ϕ_{qs} = geotechnical resistance factor for side resistance

= 0.55 for Strength Limit State (LRFD Table 10.5.5.2.4-1)

= 1.0 for Service Limit State (LRFD 10.5.5.1)

= 1.0 for Extreme Limit State (LRFD 10.5.5.3)

ϕ_{qp} = geotechnical resistance factor for tip resistance

= 0.5 for Strength Limit State (LRFD Table 10.5.5.2.4-1)

= 1.0 for Service Limit State (LRFD 10.5.5.1)

= 1.0 for Extreme Limit State (LRFD 10.5.5.3)

A_s = area of shaft side surface (ft^2)

q_s = unit side resistance (ksf)

A_p = area of shaft tip (ft^2)

q_p = unit tip resistance (ksf)

The Strength Limit State geotechnical resistance factors listed above assume a redundant use of 2 or more drilled shafts at a given substructure unit. When a foundation unit is supported by a single drilled shaft, the Strength Limit State Resistance Factors shall be reduced by 20 percent per LRFD 10.5.5.2.4. Per FHWA-DS 14.4.3, group effects typically do not need to be investigated as the strength of the rock mass is anticipated to be greater than the drilled shaft/rock interface.

Side Resistance of Drilled Shafts in Rock:

Except as mentioned below for fractured rock, maximum unit side resistance, q_s (ksf), should be computed as follows:

$$\frac{q_s}{p_a} = C\sqrt{\frac{q_u}{p_a}}$$ (LRFD Eqn. 10.8.3.5.4b-1)

For fractured rock that caves and cannot be drilled without some type of artificial support, maximum unit side resistance, q_s (ksf), should be computed as follows:

$$\frac{q_s}{p_a} = 0.65\,\alpha_E \sqrt{\frac{q_u}{p_a}} \qquad \text{(LRFD Eqn. 10.8.3.5.4b-2)}$$

p_a = atmospheric pressure
= 2.12 ksf

C = regression coefficient
= 1.0 for normal conditions (See LRFD C10.8.3.5.4b)

q_u = unconfined compressive strength of rock (ksf) $\leq f'_c$

f'_c = concrete compressive strength

α_E = joint modification factor given in AASHTO LRFD Table 10.8.3.5.4b-1

Tip Resistance of Drilled Shafts in Rock:

If the rock below the base of the drilled shaft to a depth of 2.0B is either intact or tightly jointed and the depth of the socket is greater than 1.5B, the maximum unit tip resistance, q_p (ksf), should be computed as follows:

$$q_p = 2.5\,q_u \qquad \text{(LRFD Eqn. 10.8.3.5.4c-1)}$$

If the rock below the base of the drilled shaft to a depth of 2.0B is jointed, the joints have random orientation, and the condition of the joints can be evaluated, the maximum unit tip resistance, q_p (ksf), should be computed as follows:

$$q_p = A + q_u \left[m_b \left(\frac{A}{q_u}\right) + s\right]^a \qquad \text{(LRFD Eqn. 10.8.3.5.4c-2)}$$

$$A = \sigma'_{vb} + q_u \left[m_b \left(\frac{\sigma'_{vb}}{q_u}\right) + s\right]^a \qquad \text{(LRFD Eqn. 10.8.3.5.4c-3)}$$

q_u = unconfined compressive strength of rock (ksf)

σ'_{vb} = vertical effective stress at the socket bearing elevation (tip elevation) (ksf)

LRFD Eqn. 10.8.3.5.4c-1 should be used as an upper bound to LRFD Eqn 10.8.3.5.4c-2. Also, it is recommended that the q_u value used to calculate tip resistance reflect the weighted average within a depth of 2 shaft diameters below the tip elevation.

s, a, and m_b are Hoek-Brown strength parameters for the fractured rock mass determined from the Geological Strength Index (GSI) as shown below. Additional information regarding the GSI can be found in LRFD 10.4.6.4.

$$s = e^{\left(\frac{GSI - 100}{9 - 3D}\right)} \qquad \text{(LRFD Eqn. 10.4.6.4-2)}$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}}\right) \qquad \text{(LRFD Eqn. 10.4.6.4-3)}$$

$$m_b = m_i\, e^{\left(\frac{GSI - 100}{28 - 14D}\right)} \qquad \text{(LRFD Eqn. 10.4.6.4-4)}$$

e = 2.718 (natural or Naperian log base)

GSI = geological strength index described in LRFD Figures 10.4.6.4-1 and 10.4.6.4-2

D = disturbance factor, see discussion below (dim)

m_i = constant provided in LRFD Table 10.4.6.4-1

The disturbance factor, D, is discussed in LRFD C10.4.6.4 and ranges from zero (undisturbed) to 1 (highly disturbed), and is an adjustment for the rock mass disturbance induced by the excavation method. Rock coring techniques may typically be assumed for the excavation method with a disturbance factor of zero. If alternative techniques are used, D, should be adjusted accordingly.

Settlement Analysis:

The Service Limit State “settlement”, or axial displacement of the drilled shaft that occurs as the side and tip resistance is mobilized, should be determined. The estimated displacement should be reported along with the geotechnical axial resistance so that the structural designer can select a drilled shaft depth that provides sufficient factored axial capacity and results in a tolerable service settlement for the structure being designed. In addition, the settlement analysis can be used to determine compatible combinations of side and tip resistance. LRFD 10.8.3.5.4 indicates that when using a combined approach, contribution of side resistance should be reduced to a residual value to account for loss of skin friction once peak rock shear deformations have been exceeded. Similar discussion is contained in LRFD 10.8.3.5.4d. Given that IDOT has limited testing or data for establishing such residual values, it is recommended that nominal resistances that use combined shear and tip resistance be limited to the load-deformation response corresponding with the lesser of the maximum side or tip resistance calculated in accordance with LRFD.

Two critical rock mass properties used to evaluate settlement are the elastic modulus and poisson’s ratio. Per LRFD 10.4.6.5, the elastic modulus of the rock mass, E_m, shall be taken as the lesser of the intact modulus of a sample of rock core, E_R, or the E_m value calculated in accordance with LRFD Table 10.4.6.5-1 using the GSI data as shown below. IDOT typically does not determine E_R data with laboratory tests on rock core samples and recommends that the data be taken from LRFD Table C10.4.6.5-2.

$$E_m = \sqrt{\frac{q_u}{100}}\, 10^{\frac{GSI - 10}{40}} \text{ (GPa) for } q_u \leq 100 \text{ MPa}$$

$$E_m = 10^{\frac{GSI-10}{40}} \text{ (GPa) for } q_u > 100 \text{ MPa}$$

$$E_m = \frac{E_R}{100} e^{\frac{GSI}{21.7}}$$

Note: For the above E_m equations, q_u needs to be in units of Mpa (1Mpa = 20.9 ksf).

The poisson's ratio for intact rock is summarized in LRFD Table C10.4.6.5-2. For the purpose of this design guide, the mean – 1 standard deviation is recommended when determining the poisson's ratio and E_R using the LRFD tables.

LRFD C10.8.3.5.4D references FHWA-DS and NCHRP Synthesis 360, "Rock-Socketed Shafts for Highway Structure Foundations", for guidance in evaluating the axial load-deformation response of rock socketed shafts. These references provide approximate closed form solutions which generalize a bilinear load transfer behavior of axially loaded drilled shafts in rock. These solutions compare reasonably well to more sophisticated nonlinear finite element analyses. This bilinear relationship is illustrated in Figure 1 and is estimated using the following formulas. Formulas are provided for the conditions of a "shear socket" that considers no contribution from tip resistance; and, for a "complete socket" that considers combined side and tip resistance.

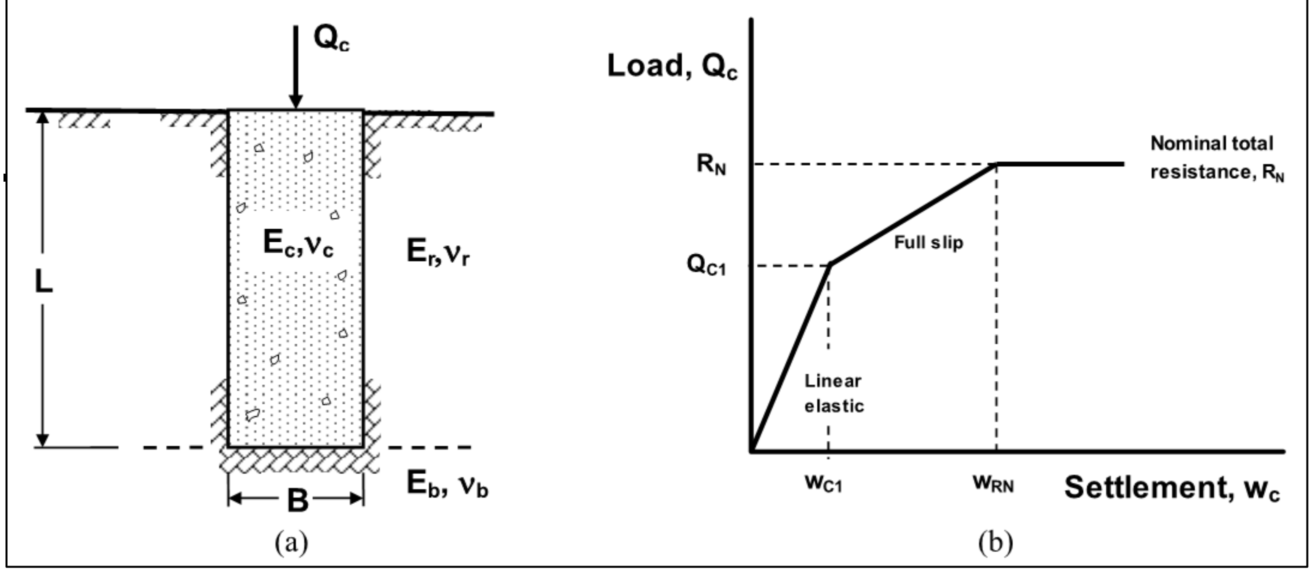

Figure 1 – Generalized Load-Displacement Model for Drilled Shafts in Rock

Constants:

$(\mu L)^2 = \left(\frac{2}{\zeta\lambda}\right)\left(\frac{2L}{B}\right)^2$

L = length of the rock socket (in.)

B = rock socket diameter (in.)

ζ = $\ln\left[5\left(1\text{-}v_r\right)L/B\right]$

v_r = average poisson's ratio for rock mass providing side resistance

λ = E_c / G_r

E_c = modulus of elasticity of concrete drilled shaft (LRFD 5.4.2.4)

G_r = elastic shear modulus of rock mass providing side resistance

= E_r / [2(1+v_r)]

E_r = average modulus of elasticity of rock mass providing side resistance

ξ = G_r / G_b

G_b = elastic shear modulus of rock mass providing tip resistance

= E_b / [2(1+v_b)]

E_b = average modulus of elasticity of rock mass providing tip resistance

$= \frac{2B}{\Sigma\left(\frac{L_i}{E_i}\right)}$

Note: The above equation for E_b calculates an equivalent value using the methodology for determining the equivalent stiffness of springs in a series and should produce a value conservatively less than taking a direct weighted average. L_i and E_i are the layer thickness and modulus of elasticity of the rock mass of individual rock layers within a distance of 2B below the shaft tip.

v_b = average poisson's ratio for rock mass providing tip resistance

$\lambda_1 = \dfrac{-\beta + \sqrt{(\beta^2+4\alpha)}}{2\alpha}$ $\lambda_2 = \dfrac{-\beta - \sqrt{(\beta^2+4\alpha)}}{2\alpha}$

$\alpha = a_1 \left(\dfrac{E_c}{E_r}\right)\left(\dfrac{B^2}{4}\right)$ $\beta = a_3 \left(\dfrac{E_c}{E_r}\right) B$

$a_1 = (1+v_r)\zeta + a_2$ $a_3 = \left(\dfrac{v_c}{2\tan\psi}\right)\left(\dfrac{E_r}{E_c}\right)$

$a_2 = \left[(1-v_c)\left(\dfrac{E_r}{E_c}\right)+(1+v_r)\right]\left(\dfrac{1}{2\tan\phi\tan\psi}\right)$

v_c = poisson's ratio of the concrete shaft

= 0.2 (LRFD 5.4.2.5)

ψ = angle of dilation of the rock at the sidewall interface (in the absence of more exact information, use a small value such as 1 degree)

$\tan\phi\tan\psi = 0.001\left(\dfrac{q_u}{p_a}\right)^{2/3}$

Note: Calculate "$\tan\phi\tan\psi$" for each layer and then use a weighted average over the socket length "L".

q_u = average unconfined compressive strength of rock providing side resistance (ksf)

p_a = atmospheric pressure

= 2.12 ksf

Shear Socket Condition

- Linear Elastic Portion of Load-Displacement Curve

$$\frac{E_r B w_c}{2 Q_c} = \frac{2 E_r \cosh(\mu L)}{\pi \mu B E_c \sinh(\mu L)} \qquad \text{(Eqn. SS-1)}$$

- Full Slip Portion of Load-Displacement Curve

$$w_c = F_1\left(\frac{Q_c}{\pi E_r B}\right) - F_2 B \qquad \text{(Eqn. SS-2)}$$

Q_c = load applied to the top of the rock socket

$F_1 = a_1 B(\lambda_2 C_2 - \lambda_1 C_1) - 4 a_3$ $F_2 = a_2\left(\dfrac{c}{E_r}\right)$

$C_1 = \dfrac{e^{(\lambda_2 L)}}{e^{(\lambda_2 L)} - e^{(\lambda_1 L)}}$ $C_2 = \dfrac{e^{(\lambda_1 L)}}{e^{(\lambda_2 L)} - e^{(\lambda_1 L)}}$

$c = 0.1\, p_a\left(\dfrac{q_u}{p_a}\right)^{2/3}$

Note: Calculate "c" for each layer and then use a weighted average over the socket length "L".

<u>Complete Socket Condition</u>

- Linear Elastic Portion of Load-Displacement Curve

$$\frac{G_r\, B\, w_c}{2\, Q_c} = \frac{1 + \left(\frac{4}{1\text{-}v_b}\right)\left(\frac{1}{\pi\lambda\xi}\right)\left(\frac{2L}{B}\right)\left(\frac{\tanh(\mu L)}{\mu L}\right)}{\left(\frac{4}{1\text{-}v_b}\right)\left(\frac{1}{\xi}\right) + \left(\frac{2\pi}{\zeta}\right)\left(\frac{2L}{B}\right)\left(\frac{\tanh(\mu L)}{\mu L}\right)} \qquad \text{(Eqn. CS-1)}$$

Once Q_c is determined, the following formula can be used to determine the portion of the total load (Q_c) that is transferred to the base (Q_p):

$$\frac{Q_b}{Q_c} = \frac{\left(\frac{4}{1\text{-}v_b}\right)\left(\frac{1}{\xi}\right)\left(\frac{1}{\cosh(\mu L)}\right)}{\left(\frac{4}{1\text{-}v_b}\right)\left(\frac{1}{\xi}\right) + \left(\frac{2\pi}{\zeta}\right)\left(\frac{2L}{B}\right)\left(\frac{\tanh(\mu L)}{\mu L}\right)}$$

- Full Slip Portion of Load-Displacement Curve

$$w_c = F_3\left(\frac{Q_c}{\pi\, E_r\, B}\right) - F_4\, B \qquad \text{(Eqn. CS-2)}$$

$$F_3 = a_1 B(\lambda_1 C_3 - \lambda_2 C_4) - 4\, a_3 \qquad F_4 = a_2\left(\frac{c}{E_r}\right)\left[1 - a_1 B\left(\frac{\lambda_1 - \lambda_2}{D_4 - D_3}\right)\right]$$

$$C_3 = \frac{D_3}{D_4 - D_3} \qquad C_4 = \frac{D_4}{D_4 - D_3}$$

$$D_3 = \left[\pi\left(1\text{-}v_b^2\right)\left(\frac{E_r}{E_b}\right) + 4\, a_3 + a_1\, \lambda_2\, B\right] e^{(\lambda_2 L)} \qquad D_4 = \left[\pi\left(1\text{-}v_b^2\right)\left(\frac{E_r}{E_b}\right) + 4\, a_3 + a_1\, \lambda_1\, B\right] e^{(\lambda_1 L)}$$

$$c = 0.1\, p_a\left(\frac{q_u}{p_a}\right)^{2/3}$$

Note: Calculate "c" for each layer and then use a weighted average over the socket length "L".

$$\frac{Q_b}{Q_c} = P_3 + P_4\left(\frac{\pi\, B^2\, c}{Q_c}\right) \qquad \text{(Eqn. CS-3)}$$

$$P_3 = \frac{a_1\left(\lambda_1 - \lambda_2\right) B\, e^{\left(\left[\lambda_1 + \lambda_2\right] L\right)}}{D_4 - D_3} \qquad P_4 = \frac{a_2\left(e^{(\lambda_2 L)} - e^{(\lambda_1 L)}\right)}{D_4 - D_3}$$

The intersection of the linear elastic and full slip regions shown in Figure 1 can be determined by substituting Equation SS-2 into SS-1 and CS-2 into CS-1 and solving for Q_c.

Q_c used in Equation CS-2 is considered the maximum nominal resistance (i.e., R_n) considering combined tip (i.e, Q_b or R_p) and side resistance (i.e, R_s). Q_c should typically be determined iteratively using Equation CS-3 such that the ratio of Q_b/Q_c does not result in values of side or tip resistance that exceed R_s or R_p calculated in accordance with LRFD.

For scenarios where it is desirable to estimate the settlement for the maximum nominal tip resistance, R_p, calculated in accordance with LRFD, the above methodology for “complete socket condition” may be used with the properties for side resistance set to a minimal or near zero value.