Drilled Shafts in Rock: Experience from Recent Projects

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DAN BROWN AND ASSOCIATES
Key Points

- Reliable analytical tools for selecting design values of side and base resistances have evolved and are supported by results of load tests.

- Side and base resistances *can be* combined.

- Design rock sockets to be as large as needed *and not larger*.

- Keys to successful design and construction are:
  - site characterization
  - construction means and methods that allow the contractor to control quality (QC) and which facilitate verification of quality (QA).
Design Equations: Axial Compression

Reference:

*Drilled Shafts: Construction Procedures and LRFD Design Methods*  
FHWA GEC 10, 2010

LRFD Design Equation:

\[ \sum \gamma_i Q_i \leq \sum \phi_i R_i \]

\[ \sum \phi_i R_i = \sum_{i=1}^{n} \phi_{S,i} R_{SN,i} + \phi_B R_{BN} \]
Most recent analysis of existing data shows that for design of “normal” rock sockets:

\[
\frac{f_{SN}}{p_a} = C \sqrt{\frac{q_u}{p_a}}
\]

- \(C = 1.0\) mean value
- \(C = 0.63\) lower bound, encompasses 90% of data
- \(C = 0.50\) absolute lower bound to encompass 100% of data
“Normal” Rock Socket:

Can be excavated using conventional rock tools (augers, core barrels) without caving and without the use of casing or other means of support (e.g., grouting ahead of excavation)

- $C = 1.0$ recommended
- $q_u$ limited to compressive strength of concrete
Reduction for Lower Quality Rock

Reduce side resistance on the basis of RQD:

<table>
<thead>
<tr>
<th>RQD%</th>
<th>Closed Joints</th>
<th>Open or Gouge-Filled Joints</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>1.00</td>
<td>0.85</td>
</tr>
<tr>
<td>70</td>
<td>0.85</td>
<td>0.55</td>
</tr>
<tr>
<td>50</td>
<td>0.60</td>
<td>0.55</td>
</tr>
<tr>
<td>30</td>
<td>0.50</td>
<td>0.50</td>
</tr>
<tr>
<td>20</td>
<td>0.45</td>
<td>0.45</td>
</tr>
</tbody>
</table>

Experience suggests the above are applicable only when socket cannot be excavated without support.
in terms of uniaxial compressive strength:

\[ q_{BN} = N_{cr}^* \times q_u \]

\( N_{cr}^* \) = bearing capacity factor

For design in “competent” rock:

\[ q_{BN} = 2.5 \, q_u \]
Base Resistance in Jointed or Fractured Rock Mass

• Strength of fractured rock mass, and bearing resistance, can be characterized using the *Hoek-Brown* strength criterion

• Appendix C of the GEC 10
‘Strain Compatibility’ between side and base resistance of rock sockets

- often cited as a reason to neglect one or the other
- Is it real?
Drilled shafts in rock subject to compressive loading shall be designed to support factored loads in:

- Side-wall shear comprising skin friction on the wall of the rock socket; or
- End bearing on the material below the tip of the drilled shaft; or
- A combination of both

The difference in the deformation required to mobilize skin friction in soil and rock versus what is required to mobilize end bearing shall be considered when estimating axial compressive resistance of shafts embedded in rock. Where end bearing in rock is used as part of the axial compressive resistance in the design, the contribution of skin friction that occurs once the shear deformation along the shaft sides is greater than the peak rock shear deformation, i.e., once the rock shear strength begins to drop to a residual value.
10.8.3.5.4a - Commentary

Design based on side-wall shear alone should be considered for cases in which the base of the drilled hole cannot be cleaned and inspected or where it is determined that large movements of the shaft would be required to mobilize resistance in end bearing.

Design based on end-bearing alone should be considered when sound bedrock underlies low strength overburden materials, including highly weathered rock.

Where the shaft is drilled some depth into sound rock, a combination of sidewall shear and end bearing can be assumed.
Case 1: The Bridge at Antlers

I-5 North of Redding, CA

Sacramento River – Lake Shasta
Rendition of Proposed Replacement Bridge

Piers 2 and 5: 2 columns
Piers 3 and 4: 4 columns

Each column supported on drilled shaft w/ 11.5-ft dia rock socket
Bragdon Formation (Mississippian)

- Metasandstone, metashale, and metaconglomerate
- Sloped bedding/foliation, 25-45 degrees from horizontal
Load Test at Antlers
6.5-ft Diameter Socket

RCD ‘Pile Top’ Rig

78-inch Test Shaft at Pier 5
Load Test at Antlers
6.5-ft Diameter Socket

Osterberg Cell Load vs. Displacement
Antlers Bridge, CA

Upward Top of O-Cell
Downward Bottom of O-Cell
Results of O-Cell Test at Antlers

<table>
<thead>
<tr>
<th>Feature</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter</td>
<td>6.5 ft</td>
</tr>
<tr>
<td>Socket length</td>
<td>35 ft</td>
</tr>
<tr>
<td>Avg side resistance above O-cell</td>
<td>33 ksf @ .11 inch</td>
</tr>
<tr>
<td>Base resistance</td>
<td>532 ksf @ .53 inch</td>
</tr>
<tr>
<td>Design concrete $f_{c'}$</td>
<td>4,000 psi</td>
</tr>
</tbody>
</table>

Over test shaft, average $q_u \approx 8,500$ psi > design $f_{c'} = 4,000$ psi by GEC 10: $f_{SN} = 35$ ksf, with $C = 1$ and using concrete strength

Compared to mobilized $f_{SN} = 33$ ksf at approximately .1 inch

Bearing zone: $q_u \approx 9,700$ psi > design $f_{c'} = 4,000$ psi

Based on ACI design eq. for nominal strength of R/C, $q_{BN}$ would be limited to $\approx 420$ ksf

Compared to 532 ksf mobilized at .14 inches
Production shafts: 12-ft casing, 11.5 ft diameter sockets
Case 2: Pitkins Curve, Highway 1, Big Sur Coast, CA
Purpose of Bridge:
Span the Pitkins Curve Landslide
Subsurface profile consists of alternating layers of:

1. **JRms**: Jurassic/Cretaceous metasediments;
   sandstones and mudstones exhibiting low-grade metamorphism as indicated by phyllitic and schistose features; tectonically deformed resulting in shear zones and variable fracturing.

2. **JRmb**: Jurassic/Cretaceous metabasalt;
   low-grade metamorphosed (greenstone) knockers embedded in the JRms, possibly as a result of faulting between JRms and JRmb unit to the west.
<table>
<thead>
<tr>
<th>Layer</th>
<th>Thickness (ft)</th>
<th>% of profile</th>
<th>Description</th>
<th>REC %</th>
<th>RQD</th>
<th>$q_u$ (tsf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5.5</td>
<td>5%</td>
<td>phyllitic schist</td>
<td>60, 66</td>
<td>0, 52</td>
<td>653.7</td>
</tr>
<tr>
<td>2</td>
<td>2.5</td>
<td>2%</td>
<td>schist</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>11.5</td>
<td>10%</td>
<td>quartz vein w/ thin moderately hard to hard shale; quartz vein, extremely hard, fractured</td>
<td>8, 93</td>
<td>5, 20</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>8.5</td>
<td>8%</td>
<td>schist, fresh, hard, moderately fractured, Ca-healed</td>
<td>93, 100</td>
<td>46, 74</td>
<td>284.3</td>
</tr>
<tr>
<td>5</td>
<td>7.5</td>
<td>7%</td>
<td>phyllitic schist, soft to mdm hard, intensely fractured</td>
<td>32, 98, 100</td>
<td>0, 56, 20</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>8</td>
<td>7%</td>
<td>schist, hard to very hard</td>
<td>32, 98, 100</td>
<td>0, 56, 20</td>
<td>145.8</td>
</tr>
<tr>
<td>7</td>
<td>1.5</td>
<td>1%</td>
<td>intensely fractured</td>
<td>50</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>18</td>
<td>16%</td>
<td>breccia/sheared schist phyllite</td>
<td>10, 86, 18</td>
<td>0, 0, 0</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>16.5</td>
<td>15%</td>
<td>schistosed phyllite, hard, fresh, moderately to intensely fractured, thinly foliated, Ca-healed</td>
<td>96, 95, 100</td>
<td>0, 16, 20</td>
<td>169.6, 138, 203.5</td>
</tr>
<tr>
<td>10</td>
<td>33.5</td>
<td>30%</td>
<td>phyllitic schist, hard to v. hard, Ca-healed foliation and irregular fractures</td>
<td>100, 100, 50, 100</td>
<td>32, 80, 84, 90, 84, 30, 45</td>
<td>697.6, 1,128, 206.5, 444.5</td>
</tr>
</tbody>
</table>

\[ \Sigma = 113 \]
Design for Axial Loading:

• High degree of uncertainty regarding both side and base resistances in Franciscan meta-sedimentary and metabasalt rock
• Uncertainty regarding construction in fractured rock with variable water inflow

Seems like a prime candidate for . . . . . .
Osterberg Cell Load vs. Displacement
Pitkins Curve, CA

Movement (inches)

0.00
0.50
1.00
1.50
2.00

0 1,000 2,000 3,000 4,000 5,000 6,000

O-Cell Load (kips)

Upward Top of O-Cell
Downward Base of O-Cell
### Results of O-Cell Test at Pitkins Curve

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter</td>
<td>3.5 ft</td>
</tr>
<tr>
<td>Socket length</td>
<td>35 ft</td>
</tr>
<tr>
<td>Avg side resistance in rock</td>
<td>28 ksf</td>
</tr>
<tr>
<td>Base resistance</td>
<td>396 ksf</td>
</tr>
<tr>
<td>Concrete $f_{c'}$ :</td>
<td>4,000 psi</td>
</tr>
</tbody>
</table>

*Sidewall rock was caving during construction of test shaft; used ‘plug-ahead’ method in order to complete excavation.

Over test shaft, average $q_u \approx 7,300$ psi $> \text{design } f_{c'} = 4,000$ psi

Average RQD over socket length = 25%

by GEC 10: with C = 1 and using concrete strength, with reduction factor for fractured (and caving) rock of .47, $f_{SN} = 16.5$ ksf,

Compared to *mobilized* $f_{SN} = 28$ ksf with no strain softening
Bearing zone: $q_u \approx 4,700$ psi > design $f'_c = 4,000$ psi

Based on ACI design eq. for nominal strength of R/C, $q_{BN}$ would be limited to $\approx 420$ ksf

Based on analysis for fractured rock (Hoek Brown), estimated $q_{BN} \approx 0.7 \ q_u \approx 470$ ksf

Compared to 396 ksf mobilized at .75 inches downward displacement
Production shafts: 5-ft diameter
4 shafts per pier
‘Plug-ahead’ method for drilling socket in caving rock
Pitkins Curve Bridge and Rain Rocks Rock Shed
Case 3: The New **Mississippi River Bridge (MRB)**
Saint Louis
### DESCRIPTION OF MATERIAL

<table>
<thead>
<tr>
<th>DEPTH IN FEET</th>
<th>DRY UNIT WEIGHT (pcf)</th>
<th>SPT BLOW COUNTS</th>
<th>CORER CORE RECOVERY</th>
<th>GRADE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>100%</td>
<td></td>
<td>77%</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>100%</td>
<td></td>
<td>88%</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>100%</td>
<td></td>
<td>100%</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>100%</td>
<td></td>
<td>100%</td>
<td></td>
</tr>
<tr>
<td>25</td>
<td>100%</td>
<td></td>
<td>100%</td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>100%</td>
<td></td>
<td>94%</td>
<td></td>
</tr>
<tr>
<td>35</td>
<td>100%</td>
<td></td>
<td>100%</td>
<td></td>
</tr>
<tr>
<td>40</td>
<td>100%</td>
<td></td>
<td>100%</td>
<td></td>
</tr>
<tr>
<td>45</td>
<td>100%</td>
<td></td>
<td>100%</td>
<td></td>
</tr>
</tbody>
</table>

**Hard, gray, very finely crystalline, thick bedded, fresh LIMESTONE with chert nodules**

- Unconfined Compressive Strength = 26,592 psi
- Unconfined Compressive Strength = 29,829 psi
- Unconfined Compressive Strength = 22,616 psi
- Unconfined Compressive Strength = 21,919 psi
- Unconfined Compressive Strength = 19,767 psi
- Clay seam (0.25 inches)
- With shale partings from 29.0 to 29.4 feet
- With shale partings from 30.7 to 31.5 feet
- Shale seam from 43.0 to 43.2 feet
- Boring terminated at 44 feet
Socket Excavation by Coring at New MRB

Load test carried out as part of ATC proposed by contractor to use large-diameter drilled shafts, reducing size of cofferdam and footings at pylons.

Test shaft socket diameter same as production shaft diameter = 11 ft.
Foundation Construction New MRB

Base cleanout by airlift;
Base sounded by weighted tape

Pylon foundation construction inside coffercell; 6 shafts per pylon
NOTE: In calc., assumed a 2:1 of shaft below the plate. This yield
We assume the
### O-Cell Test on 11-ft Diameter Socket at New MRB

<table>
<thead>
<tr>
<th></th>
<th></th>
<th>as-built 11.5 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Nominal Diameter</strong></td>
<td>11 ft</td>
<td></td>
</tr>
<tr>
<td><strong>Socket length</strong></td>
<td>23.3 ft</td>
<td></td>
</tr>
<tr>
<td><strong>Avg unit side resistance</strong></td>
<td>44 ksf</td>
<td>@ .14 in</td>
</tr>
<tr>
<td><strong>Base resistance</strong></td>
<td>460 ksf</td>
<td>@ .14 in</td>
</tr>
</tbody>
</table>

Along test shaft, average $q_u \approx 24,000$ psi $> f_c' = 5,000$ psi
by GEC 10: $f_{SN} = 39$ ksf, with $C = 1$ and using concrete strength
Compared to *mobilized* $f_{SN} = 44$ ksf

Bearing zone: $q_u \approx 12,000$ psi $> f_c' = 5,000$ psi
Based on ACI design eq. for nominal strength of R/C $q_{BN}$ would be limited to $\approx 520$ ksf
Compared to 460 ksf mobilized at .14 inches

Case 4: kcICON Missouri River Bridge
I-29/35 Kansas City
Large diameter drilled shafts in shale
Shafts socketed into Stratum II:
  Shale of the Pleasonton Group (Pennsylvanian)

Five borings at main pylon
\[ q_u : \ 800 \text{ to } 8,750 \text{ psi} \quad \text{mean} \approx 2,000 \text{ psi} \]
Upper 3 to 5 feet weathered
Below weathered zone, excellent quality
RQD > 70%

Shale recovered from drilling bucket at tip of test shaft
6-ft diameter test shaft excavated with rock augers and buckets under polymer slurry

‘Back-scratcher’ for preparation of socket sidewall
### O-Cell Test on 6-ft Diameter Socket at kcICON

<table>
<thead>
<tr>
<th>Diameter</th>
<th>6 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Socket length</td>
<td>30.1 ft</td>
</tr>
<tr>
<td>Unit side resistance top 4 ft</td>
<td>12 ksf</td>
</tr>
<tr>
<td>Unit side resistance below 4 ft</td>
<td>16 ksf @ 0.2 to 0.3 in</td>
</tr>
<tr>
<td>Base resistance</td>
<td>275 ksf @ 1.5 in</td>
</tr>
</tbody>
</table>

At test shaft, average $q_u \approx 1,200$ psi
by GEC 10: $f_{SN} = 19$ ksf, with $C = 1$
Compared to *mobilized* $f_{SN} = 16$ ksf

Bearing zone: $q_u \approx 2,000$ psi
by GEC 10: $q_{BN} = 2.5 \cdot q_u = 720$ ksf @ 2.4 to 3 inches displacement
Compared to 275 ksf mobilized at 1.5 inches
As importantly, Axtell et al. (2011) note:

“. . . the test data showed no evidence of strain softening and therefore strain compatibility was not a factor in combining side and base resistance. This tendency is likely related to the dilatancy at the shaft/rock interface. It is also noted that load test measurements in similar (even softer) shale materials from nearby projects referenced previously as reported by Miller (2003) showed ductile behavior at significantly larger displacements.”
Typical side load transfer behavior in weak rock
Key Points

• Reliable analytical tools for selecting design values of side and base resistances have evolved and are supported by results of load tests

• Side and base resistances can be combined

• Design rock sockets to be as large as needed . . . . . and not larger

• Keys to successful design and construction are:
  
  site characterization
  
  construction means and methods that allow the contractor to control quality (QC) and permit verification of quality (QA)
Rock On

Weymouth Argillite
from Fore River Bridge

Thank you