Advances in the Design and Construction of Drilled Shafts in Rock

John Turner, Ph.D., P.E., PG, D.GE

DAN BROWN AND ASSOCIATES

ADSC Mid-Atlantic Section
Drilled Shaft Seminar
April 27, 2017

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: $\sum \gamma_i Q_i \le \sum \varphi_i R_i$

$$\sum \varphi_i R_i = \sum_{i=1}^{n} \varphi_{S,i} R_{SN,i} + \varphi_B R_{BN}$$

Unit Side Resistance in Rock

$$\frac{f_{SN}}{p_a} = C\sqrt{\frac{q_u}{p_a}}$$

Most recent analysis of existing data shows that for design of “normal” rock sockets:

C = 1.0 mean value

“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**

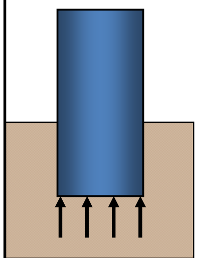
AASHTO: Reduction for Lower Quality Rock

Reduce side resistance on the basis of RQD:

RQD%	Reduction Factor	
	Closed Joints	Open or Gouge-Filled Joints
100	1.00	0.85
70	0.85	0.55
50	0.60	0.55
30	0.50	0.50
20	0.45	0.45

Experience suggests the above is applicable only when a rock socket cannot be excavated without support

Rock: Base Resistance



in terms of uniaxial compressive strength:

$$q_{BN} = N^*_{cr} \times q_u$$

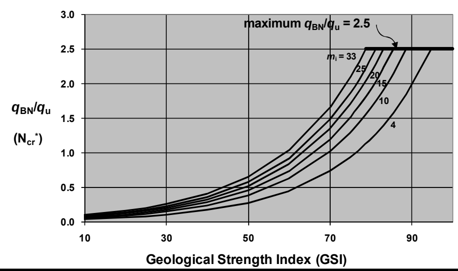
N^*_{cr} = bearing capacity factor

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
GEC 10

Combining Side and Base Resistances

'Strain Compatibility' between side and base resistance of rock sockets

- *often cited as a reason to neglect one or the other*
- *Is it real?*

AASHTO 7th Ed.

10.8.3.5.4a-General

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

". . . Where end bearing in rock is used as part of the axial compressive resistance in the design, the contribution of skin friction in the rock shall be reduced to account for the loss 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."

AASHTO 7th Ed.

C10.8.3.5.4d – Commentary (added in 2015)

. . before making a decision to omit tip resistance, careful consideration should be given to applying available methods of quality construction and inspection that can provide confidence in tip resistance. Quality construction practices can result in adequate clean-out at the base of rock sockets, including those constructed by wet methods. Inspection tools, such as the Shaft Inspection Device (SID), probing tools, borehole calipers, and others, can be applied more effectively to ensure quality of rock sockets prior to concrete placement (Crapps and Schmertmann 2002, Turner 2006). In many cases, the cost of quality control and assurance is offset by the economies achieved in socket design by including tip resistance. Load testing provides a means to verify tip resistance in rock.

Illustrative Case 1: Goethals Bridge

Elizabeth, NJ to
Staten Island, NY

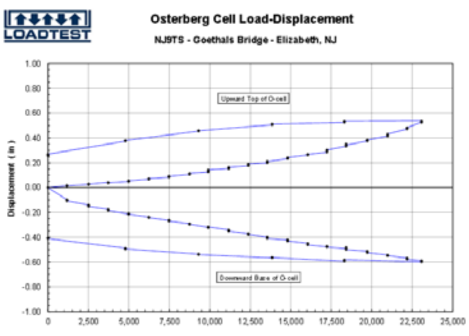
Passaic Formation (Triassic-Jurassic)

Reddish brown siltstone, w/ interbedded sandstone and shale

13

9-ft Dia Test Shaft w/ permanent casing to rock, 8.5-ft dia rock socket

Load Test at Goethals 8.5-ft Diameter Socket



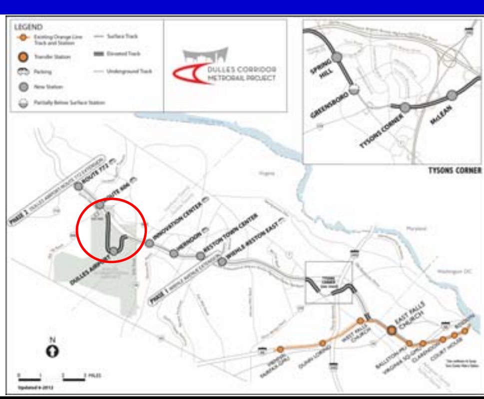
Results of O-Cell Test, NJ 9-ft Shaft

Socket Diameter	8.5	ft	
Socket length	25	ft	
Avg side resistance above O-cell	36	ksf	@ .53 inch
Base resistance	335	ksf	@ .60 inch
Design concrete f_c	5,000	psi	

Mean q_u ≈ 8,000 psi > design f_c' = 5,000 psi
by GEC 10: f_{SN} = 39 ksf, with C = 1 and using concrete strength
Compared to *mobilized* f_{SN} = 36 ksf at .53 inch

Bearing zone: q_u ≈ 8,000 psi > design f_c' = 5,000 psi
Based on ACI design eq. for nominal strength of R/C, q_{BN} would be limited to ≈ 520 ksf
Compared to 335 ksf mobilized at .60 inches (0.6% diameter)
Design q_{BN} = 300 ksf

Illustrative Case 2: Dulles Metro Silver Line



Illustrative Case 2: Dulles Metro Silver Line*

- *Photos and load test information for Dulles Metro courtesy of Schnabel Engineering*

Monoshafts in Balls Bluff Formation Siltstone

Balls Bluff Formation (Triassic-Jurassic)

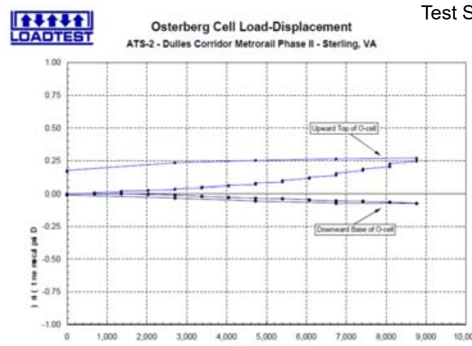
Reddish brown siltstone with interbedded v. fine to mdm grained sandstone and silty shale and shale

20

Three Load Tests on 6-ft Dia Test Shafts permanent casing to rock

Load Test at Dulles on 6-ft Diameter Socket

Test Shaft No. 2



Summary of Results of O-Cell Tests Dulles 6-ft Shaft

	TS-1	TS-2	TS-3
Socket Length (ft)	30.0	22.5	22.2
Avg Mobilized Unit Side Resistance (ksf)	15.8	22.8	20.9
Max Mobilized Unit Side Resistance (ksf)	27.4	28.6	31.6
Upward Displacement (in)	0.21	0.31	0.20
Mobilized Unit Base Resistance (ksf)	293	299	288
Downward Displacement (in)	1.41	0.07	0.13
Design Concrete Strength, f_c' (psi)		4,000 psi	

Summary Analysis of Load Test Results Dulles 6-ft Shafts

For Test Shaft 1:

Mean q_u ≈ 3,200 psi < design f_c' = 4,000 psi
by GEC 10: f_{SN} = 31 ksf, with C = 1 and using rock strength (q_u)
Compared to *mobilized* f_{SN} = 27 to 32 ksf at .20 to 0.31 inch

Bearing zone: q_u ≈ 4,000 psi ≈ design f_c' = 4,000 psi
Based on ACI design eq. for nominal bearing strength of concrete, q_{BN} would be limited to ≈ 290 ksf
Compared to 288 to 299 ksf *mobilized* in test shafts

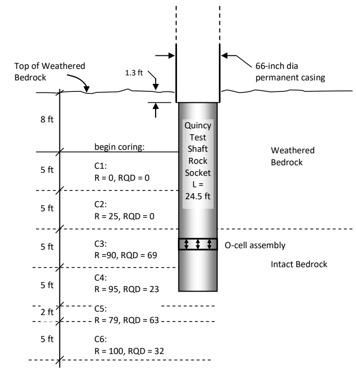
For comparison: Design Allowable q_B = <u>72.5 ksf</u> for RQD < 50
q_B = <u>36.0 ksf</u> for RQD ≥ 50

Illustrative Case 3: Fore River Bridge

Quincy to Weymouth, MA

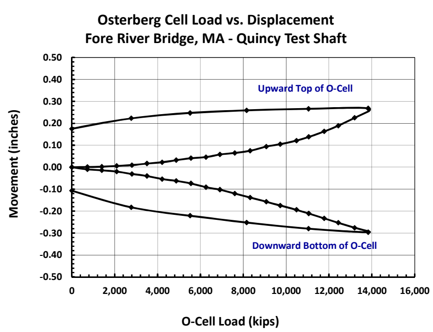
Weymouth Formation Argillite (Cambrian)

Quincy Test Shaft



26

Load Test at Fore River Bridge 5.5-ft Diameter Socket



Results of Quincy O-Cell Test at FRB

Diameter	5.5	ft	
Socket length	24.5	ft	
Avg side resistance above O-cell	53	ksf	@ .27 inch
Base resistance	296	ksf	@ .30 inch
Design concrete f_c	4,000	psi	

Over test shaft, average q_u ≈ 5,080 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} = 53 ksf at .27 inch

Bearing zone: q_u ≈ 6,000 psi > design f_c' = 4,000 psi
Based on ACI design eq. for nominal strength of R/C, q_{BN} would be limited to ≈ 420 ksf
q_{BN} = 0.4 (6,000 psi) = 2,400 psi = 345 ksf
Compared to 296 ksf mobilized at .30 inches (0.5% of diameter)

Additional Projects Illustrating the Following Aspects of Rock Socket Behavior

1. Validity of design equations for nominal unit side and base resistances
2. Mobilization of side and base resistances at compatible displacements

The Bridge at Antlers

I-5 North of Redding, CA

Sacramento River – Lake Shasta

Bragdon Formation (Mississippian)
- Metasandstone, metashale, and metaconglomerate
- Sloped bedding/foliation, 25-45 degrees from horizontal

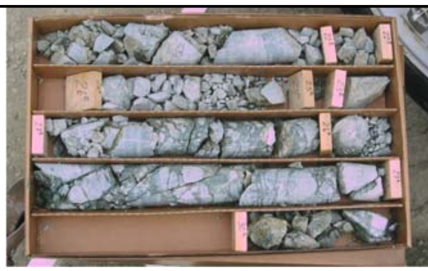
Results of O-Cell Test at Antlers

Diameter	6.5	ft	
Socket length	35	ft	
Avg side resistance above O-cell	33	ksf	@ .11 inch
Base resistance	532	ksf	@ .53 inch
Design concrete $f_{c'}$	4,000	psi	

Over test shaft, average q_u ≈ 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 ≈ 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 ≈ 420 ksf
Compared to 532 ksf mobilized at .53 inches (0.7% of diameter)

Pitkins Curve, Highway 1, Big Sur Coast, CA



Franciscan mélange and BIM-rocks consists of alternating layers of:

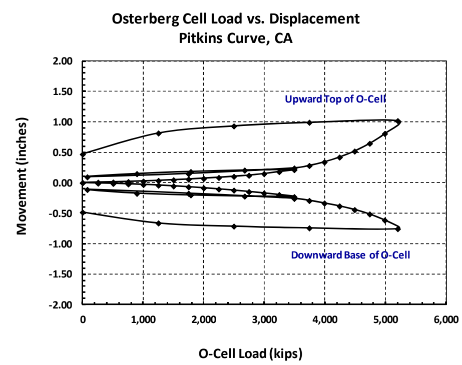
1. JRms: Jurassic/Cretaceous metasediments

sandstones and mudstones exhibiting low-grade metamorphism; tectonically deformed resulting in shear zones and variable fracturing.

2. JRmb: Jurassic/Cretaceous metabasalt;

low-grade metamorphosed (greenstone) blocks embedded in the JRms

Load Test at Pitkins Curve Bridge 3.5-ft Diameter Socket



Results of O-Cell Test at Pitkins Curve

Diameter	3.5	ft
Socket length	35	ft
Avg side resistance in rock	28	ksf
Base resistance	396	ksf
Concrete $f_{c'}$:	4,000	psi

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

Over test shaft, average q_u ≈ 7,300 psi > 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

O-Cell Test at Pitkins Curve

Bearing zone: q_u ≈ 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 ≈ 420 ksf

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

Compared to 396 ksf mobilized at .75 inches downward displacement (1.8% of diameter)

The New Mississippi River Bridge (MRB)
Saint Louis

High strength competent limestone

Test shaft socket diameter same as production shaft diameter = 11 ft

O-Cell Test on 11-ft Diameter Socket at New MRB

Nominal Diameter	11	ft	as-built 11.5 ft
Socket length	23.3	ft	
Avg unit side resistance	44	ksf	@ .14 in
Base resistance	460	ksf	@ .14 in

Along test shaft, average q_u ≈ 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

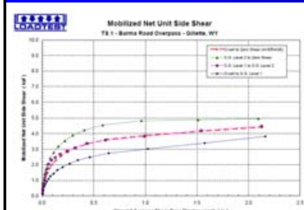
Bearing zone: q_u ≈ 12,000 psi > f_c' = 5,000 psi
Based on ACI design eq. for nominal strength of R/C q_{BN} would be limited to ≈ 520 ksf
Compared to 460 ksf mobilized at .14 inches (0.1% of diameter)

Reference: Axtell and Brown, *DFI Journal*, Dec 2011

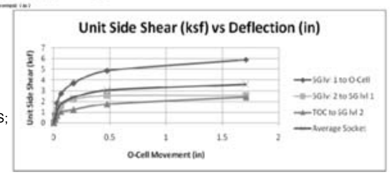
. . . and others

KC ICON Missouri River (Bond Bridge)	shale
Nashville (ADSC SE Chapter)	limestone
Lawrenceville, GA (ADSC SE Chapter)	Piedmont PWR and gneiss
Burma Road Overpass, WY	weak sandstone

Typical side load transfer behavior in rock



no evidence of strain softening



US 36 over Republican River, KS; grey thinly laminated shale

Are There Exceptions?

Geomaterials in which side and/or base resistance mobilization is either very sensitive to construction or is otherwise unreliable?

YES

Some examples

- *Argillaceous clay shales prone to sidewall smearing, e.g., Denver, Dallas*
- *Franciscan Complex rocks in CA referred to as mélange, BIM rocks: base resistance is all over the map*

However, socket behavior and design in these environments should not be generalized to all rock sockets. Experience is telling us these are exceptions, not the rule.

RQD and Rock Sockets: *Be Careful*

From Deere and Deere (1988) "The Rock Quality Designation (RQD) Index in Practice".

ABSTRACT: The Rock Quality Designation (RQD) index was introduced 20 years ago at a time when rock quality information was usually available only from geologists' descriptions and percent of core recovery. The RQD is a modified core recovery percentage in which unrecovered core, fragments and small pieces of rock, and altered rock are not counted so as to downgrade the quality designation of rock containing these features. Although originally developed for predicting tunneling conditions and support requirements, its application was extended to correlations with *in situ* rock mechanical properties and, in the 1970's, to forming a basic element of several classification systems. Its greatest value, however, remains as an exploratory tool where it serves as a red flag to identify low-RQD zones which deserve greater scrutiny and which may require additional borings or other exploratory work. Case history experience shows that the RQD red flag and subsequent investigations often have resulted in the deepening of foundation levels and the reorientation or complete relocations of proposed engineering structures, including dam foundations, tunnel portals, underground caverns, and power facilities.

Example: Low RQD, High Socket Resistances

Test shaft w/ tip in C4: q_b = 319 ksf @ δ = .21 inch
f_s = 31 ksf

RQD does not account for orientation of discontinuities, in this case horizontal

43

What Does it Take to Obtain and Count on Mobilization of Base Resistance?

A clean base and some means to measure it, *i.e.* Quality Control and Quality Assurance

QC Tools:

Contractors' Means
- **cleanout buckets**
- **airlift**

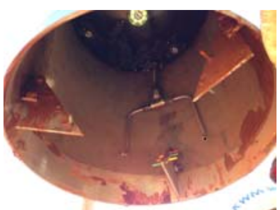
Specifications

Installation Plan

Verifying Base Resistance (*cont*)

QA Tools:

Shaft Inspection Device (SID)
Weighted tape
Sonic caliper
Competent inspection



Summary of Key Points

- Reliable analytical tools for selecting design values of side and base resistances for rock sockets 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)

Questions ?

Thank you