Side Shear and End Bearing in Rock Sockets

FOR DRILLED SHAFTS

Geotechnical engineers have misconceptions about the relationships between side shear and end bearing in rock socketed drilled shaft foundations. Some sockets are designed using both end bearing and side shear. Others are designed using side shear and no end bearing. Some believe that when the ultimate side shear is reached and the bond between the concrete and rock is broken, all side shear is lost, and that a large part of the load is transmitted to the bottom in end bearing, even when the penetration into rock is many diameters. This is simply not true. The reasons are explored in this article.

WHAT DOES THEORY TELL US?

The distribution of load with depth for a concrete shaft socketed in rock depends on the shaft length/diameter ratio (z/d) and the rock/concrete ratio (Er/Ec). The results of linear elastic finite element studies for an embedment of two diameters are shown in Figure 1. About 92 percent of the applied load is taken in side shear and 8 percent in end bearing for equal rock and concrete modulus. When the rock modulus is five times that of the concrete (for very hard rock such as a hard limestone or granite), only about 4 percent of the load reaches the bottom. When the rock modulus is 1/3 that of the concrete (typical of shale), about 13 percent of the load reaches the bottom. If the bottom of the socket is very soft, the distribution of side shear along the shaft is not very different than if the bottom consisted of the same rock as the socket wall. For an embedment ratio of 1 and a rock/concrete modulus ratio of 4, only 18 percent of the load reaches the bottom. When the rock/concrete modulus is 1/4, 30 percent of the load reaches the bottom. Thus, even for relatively small embedments, the majority of the load is transferred in side shear for both weak and strong rocks. At larger embedments, nearly all the load is taken in side shear. When alternating equal layers of hard and soft rock are assumed, the side shear distribution is almost the same as if the entire rock socket is homogeneous with a modulus of the harder layer.

A nonlinear analysis was performed to study the mechanism of load transfer where progressive failure in bond occurs at the concrete rock interface. The Mohr-Coulomb failure criterion was used: s = c + n tan f; c is the limiting bond stress, n is the normal radial stress, and f is the angle of friction between the concrete and the rock. The shear strength was assumed as c = 300 lb/in2 and f = 30 degrees. The results are shown in figure 2. The solid line shows the dissipation of load with no bond failure. It is seen that when the crack or failure in bond has progressed to 1/3 or even 2/3 depth, the load dissipation is almost the same as with no bond failure. When bond failure occurs to the full depth, only 15 percent of the load reaches the bottom. The load required to cause the crack to advance to the bottom is 6.2 times the load at initial bond failure.

Though theoretical, these studies indicate that as the embedded depth increases, less load is transmitted to the bottom, and that at z/d=2 or more, very little load reaches the bottom. When the load is large enough to cause initial bond failure between the concrete and the rock, the side shear does not drop, but increases as the load is further increased until complete bond failure occurs.

WHAT DO LOAD TESTS SHOW?

By using the Osterberg Load Cell (O-Cell), large loads can be applied and side shear and end bearing can be measured separately, making it possible to test production sockets. An O-Cell is a jacklike hydraulic device placed at the bottom of a drilled shaft. After concrete is placed and cured, hydraulic pressure is applied to the O-Cell, causing an equal upward and downward force on the shaft. The upward force is resisted by side shear. The hydraulic pressure, the upward movement of the bottom and top of the shaft, and the downward movement of the bottom of the socket are measured and recorded on a data logger. They can also be viewed real-time. The test is continued until either the ultimate side shear, the ultimate bearing, or the capacity of the O-Cell is reached. If the shaft is to be used as a working shaft rather than a test shaft, the inside of the cell and the space between the cell and the side of the hole are grouted following the test. The maxi-

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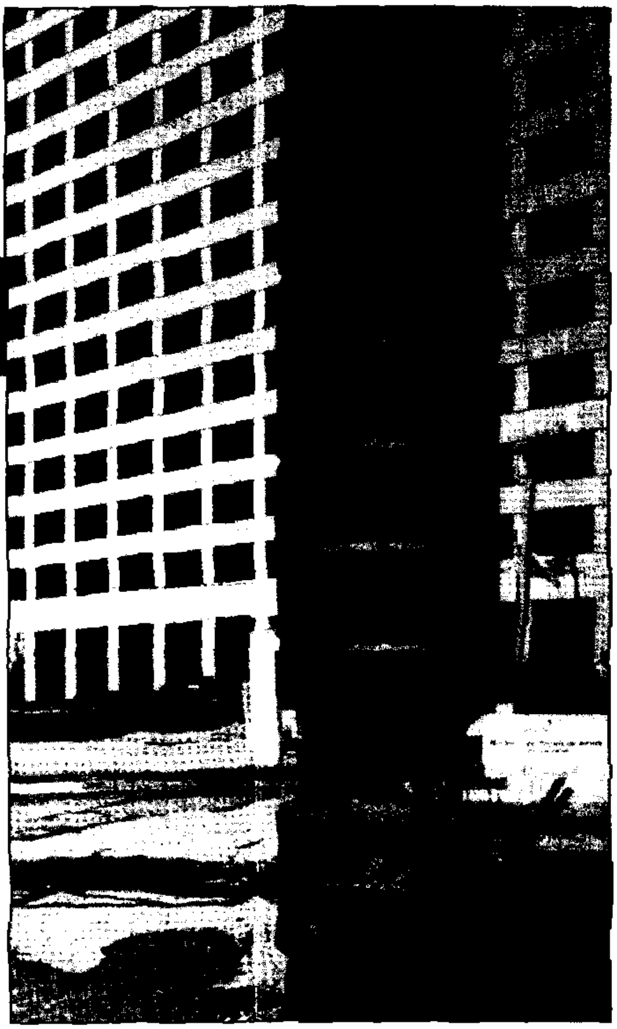
mum capacity of the largest O-Cell is 3,000 tons. For large-diameter shafts, multiple cells are used for larger loads. Shafts up to 10 ft. have been tested. The largest test load to date is 15,000 tons. Approximately 400 tests have been made, of which approximately 70 have been in rock sockets.

Figure 3 shows the geologic profile and the load deflection curves for a test shaft associated with a bridge over the Ohio River. The upper curve represents upward deflection of the O-Cell, and the bottom curve represents downward deflection of the O-Cell base. The shaft was socketed in shale with limestone and coal seams, having an average compressive strength of 500 lb/in2. Because of deep scour, only the load capacity of the socket was considered. As shown in figure 3, concrete was placed some distance above the top of the shale. However, strain gage readings showed that the load taken in the concrete above the shale was very small. Therefore, the results were considered to represent the properties of the rock socket. The load was carried to six times the design load and ultimately was not reached in side shear or end bearing. The maximum tested unit side shear in the rock socket was 135 lb/in2, which is much larger than designers would expect for shale with limestone and coal seams. Considering the confining effect of the overburden weight, considerably more side shear strength can be expected than suggested from unconfined compression tests of rock cores.



This tool is typically used in very hard rock where deep sockets are designed. This tool requires considerable weight in order to operate efficiently. The picture shows the parasite weights that are attached to the kelly bar and provide the required weight to the drilling tool.

For a test in hard limestone in Kentucky where the compressive strengths of rock cores from the borings varied from 15,000 to 20,000 lb/in2, concrete was filled to only one half the depth of the socket. For this reduced socket length, neither ultimate in side shear nor end bearing was reached. At the maximum load of 900 tons, the upward deflection was only 0.057 inches and the downward deflection only 0.17 inches. The side shear unit stress was 265 lb/in2. Since this was considerably more than the engineer assumed, the second part of the test (to fill and test the upper half of the socket) was cancelled. The design load (tension and compression) was only 200 tons. As only half the depth of the rock was tested, the proven side shear and end bearing resistance for the full shaft are both more than 1,800 tons. The actual capacity is therefore 9 times the design load in tension and 18 times the design load in compression (assuming properties for the top half of the socket are similar to the bottom).We can conclude from these and other rock socket tests, that more economical designs could have been achieved if the tests had been performed ahead of time to obtain parameters for the design, rather than after the fact to confirm the design.

FINAL THOUGHTS

Theoretical studies and load tests show that rock sockets offer much more side shear than designers are typically willing to rely on. Factors other than embedment or modulus ratio can also have considerable influence on socket capacity. These include:

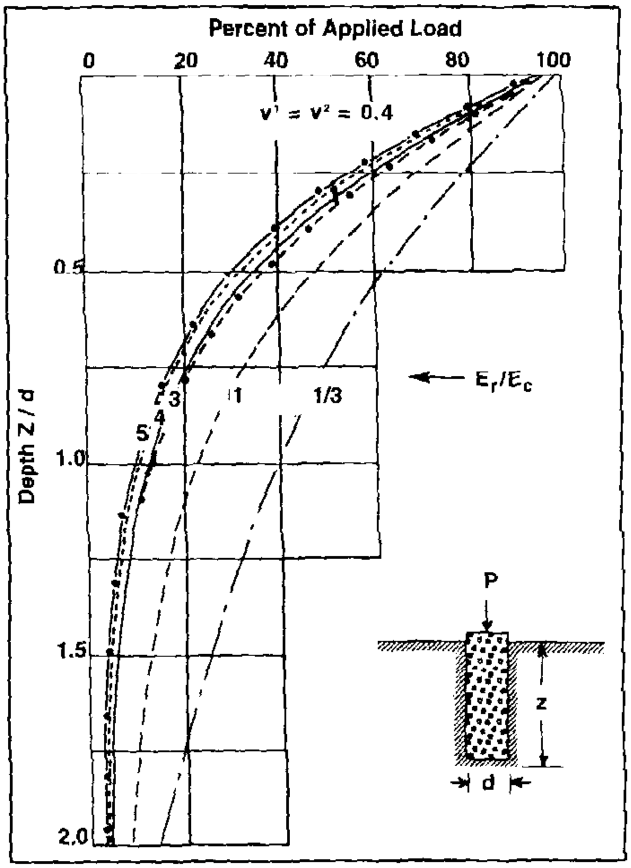


Figure 1. Distribution of Side Shear in a Rock Socket Embedment/Diameter Ratio of 2

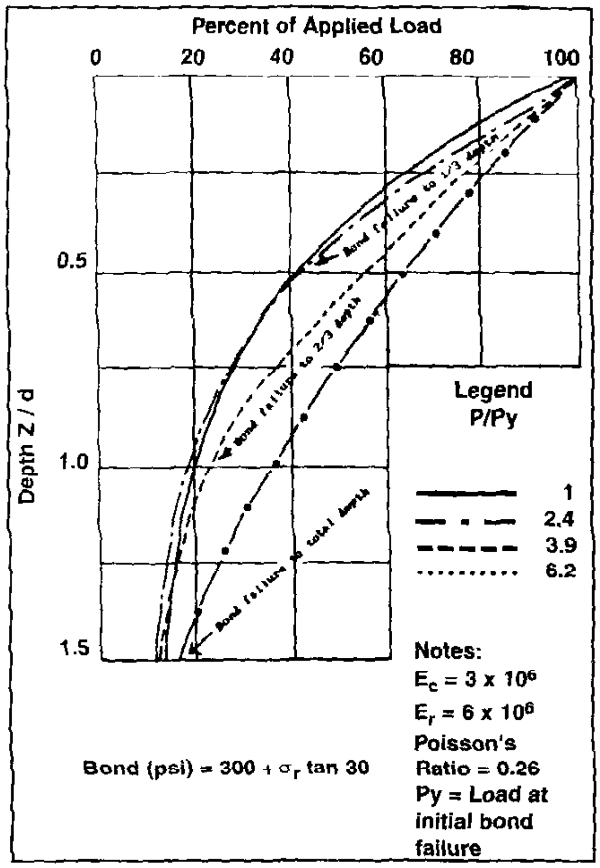


Figure 2. Effect of Bond Failure on Distribution of side Shear wih depth

1. Roughness of the socket walls. Sufficient roughness can be achieved by attaching cutting teeth to the coring device.

2. Penetration of water and/or drilling mud into soft and pervious rock sockets. This can reduce side shear and end bearing.

3. Timing of shaft concrete placement. Holes drilled dry in shale and left open for some time may oxidize and then become soft from adsorbing water from the placed concrete.

4. Cleaning of the shaft bottom. End bearing capacity is reduced when inadequate cleaning occurs.

5. Shaft displacement. In some cases the expected displacement of the shaft is the design factor, rather than the maximum safe shaft load. ○

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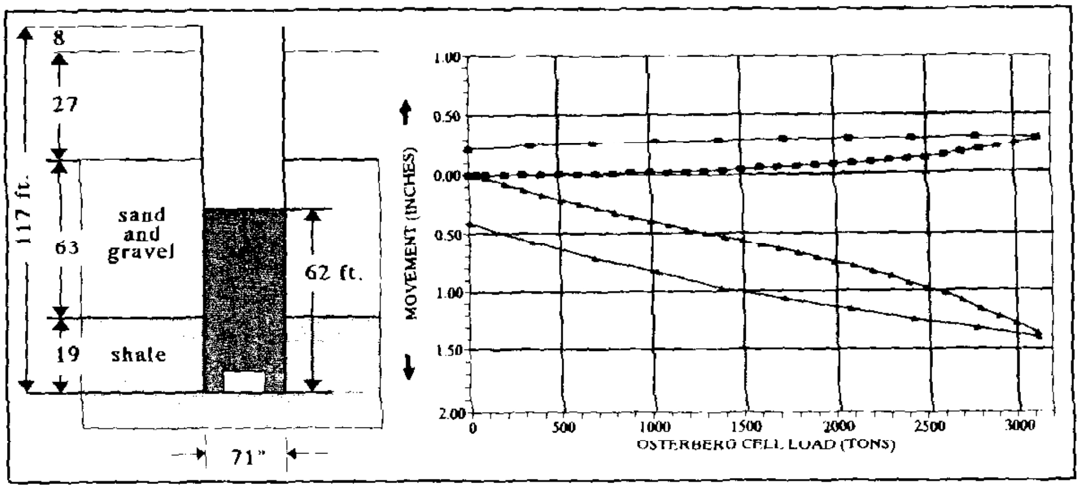

Figure 3. Load-deflection curves for a load test on a shaft for the foundations of a bridge over the Ohio River in Kentucky

ROCK SOCKET LOAD TESTS UTILIZING THE OSTERBERG CELL						
Location	**Rock Type**	**Rock Properties**	**Side Shear Reached**	**End Bearing Reached**	**Test / Design**	**Comments**
Burgin, Kentucky	Hard Limestone	Comp. Strength 15,000-20,000 psi	19 tsf (0.06 inches)	127 tsf (0.17 inches)	> 9.0	Ultimate Loads Not Reached
Decatur, Alabama	Med. Hard Limestone	Comp. Strength 15,000-22,000 psi	20 tsf (0.05 inches)	370 tsf (1.5 inches)	> 5.0	Ultimate Loads Not Reached
Edison, New Jersey	Weathered Shale	60-70% Recovery RQD = 7%	3.7 tsf (1.7 inches)	34 tsf (0.5 inches)	> 3.0	Ultimate Reached in Side Shear
Latham, New York	Shale		22 tsf (0.18 inches)	216 tsf		Ultimate Loads Not Reached
Hong Kong	Weathered Granite		18 tsf (0.09 inches)	194 tsf (0.28 inches)	> 4.5	Ultimate Loads Not Reached
Rochester, New York	Bedded Sandstone		13 tsf (1.8 inches)	161 tsf (0.08 inches)		Ultimate Reached in Side Shear
Ohio River Kentucky	Shale with coal seams	350-500 psi	9.3 tsf (0.3 inches)	113 tsf (1.4 inches)	> 6.0	Ultimate Loads Not Reached
Milton, Mass	Argilite Shale	Comp. Strength 3200 psi RQD 17	Ultimate 15 tsf @ 0.55 inches	Ultimate 250 tsf @ 0.65 inches	10	Ultimate Reached in Side Shear and End Bearing
Chicago, Illinois	Hard Limestone	Comp. Strength 12,200 psi RQD 45-55	20 tsf (0.23 inches)	725 tsf (0.13 inches)		Ultimate Loads Not Reached
Milwaukee, Wisconsin	Fractured Limestone	RQD 60	9.55 tsf (0.36 inches)	115 tsf (1.23 inches)		Ultimate Loads Not Reached
Springfield, VA	Weathered green & black granite	86% Recovery RQD 50	1.99 tsf (0.17 inches)	Ultimate 239 tsf (3.71 inches)	> 4.0	Ultimate Reached in End Bearing
Hannibal, MO	Lime Rock		16.1 tsf (.059 inches)	100.3 tsf (.114 inches)	> 7.1	Ultimate Loads Not Reached
Hannibal, MO	Shale		7.46 tsf (0.20 inches)	93.0 tsf (4.31 inches)	> 4.1	Ultimate Reached in End Bearing
Grand Rapids, MI	Mod. Weathered Gypsum w/ Clay Shale	Comp. Strength Approx. 3,000 - 8,000 psi	17.0 tsf (.297 inches)	81.4 tsf (.230 inches)	> 5.1	Ultimate Loads Not Reached
Lexington, Missouri	Gray - Black Shale w/ thin Limestone & Coal Seams	Comp. Strength Approx. 121.5 psi SPT 100 REC	8.03 tsf (0.22 inches)	101.5 tsf (0.54 inches)	> 5.6	Ultimate Loads Not Reached
Lexington, Missouri	Gray - Black Shale w/ thin Limestone & Coal Seams	Comp.Strength Approx. 336.5 psi	8.85 tsf (0.31 inches)	72 tsf (2.39 inches)	> 4.7	Ultimate Reached in End Bearing
Albany, New York	Sound, black shale (Snake Hill Formation)	REC 95% ROD 23%	35.1 tsf (0.22 inches)	289.5 tsf (0.30 inches)	> 6.7	Ultimate Loads Not Reached
Providence, RI	Dark gray fractured graphitic shale	SPT 100	3.06 tsf (2.07 inches)	61.1 tsf (2.07 inches)	7.1	Ultimate Loads Approached
Aspen, Colorado	Shale Bedrock	RQD 85% REC 100%	4.75 tsf (0.07 inches)	163 tsf (0.09 inches)	> 7.1	Ultimate Loads Not Reached
Portsmouth, N.H.	Granite		11.75 tsf (.082 inches)	228 tsf (.077 inches)	> 2.7	Ultimate Loads Not Reached
Atlanta, Georgia	Fractured & partially-weathered rock		9 tsf (.204 inches)	165 tsf (1.92 inches)	> 3.2	Ultimate Reached in End Bearing
Aspen, Colorado	Hard, reddish-brown Siltstone	RQD 90 REC 100%	7.71 tsf (2.51 inches)	61.5 tsf (1.96 inches)	7.2	Ultimate Reached in Side Shear and End Bearing

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