THE QUEST FOR QUALITY IN DEEP FOUNDATIONS (Continued)

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OVERVIEW

In Part 1 of this three-part series we explored the proposition that the natural desire to improve the way we design and build deep foundations provides a fundamental motive for achieving excellence and high quality. Unfortunately, perceived high cost and the lack of rewards or incentives to make the effort that quality work demands, often leads us to complacency and mediocrity. This complacency can result in substantial reductions in bored pile capacity that dwarfs the potential improvements available from high quality engineering design and construction.

In Part 2 we will examine some case histories that illustrate how seemingly minor factors in design or construction technique can cause capacity reductions of 80% or more. Faced with the daunting risks and uncertainties associated with deep foundations many engineers fall back on code-value engineering and from there to unnecessarily expensive foundations. A review of the impact of shaft diameter on measured side shear resistances leads to cautionary advice about how relying too much on intuition can affect the quality of our analytical work.

The Widening Gulf between Analysis and Construction - Is anyone paying attention?

Part 1 concluded with Ralph B. Peck's (1991) prediction of a gradual widening of the gap between theory and practice: *"Researchers will take refuge in increasingly esoteric investigations; practitioners will pay little attention to the research results. Reading learned journals will become less interesting and profitable to practitioners, scientific oriented workers will find themselves more or less writing to each other."*

Engineers often get absorbed in analytical techniques, fuss and fret over factors that might change things by 5% while ignoring factors that probably affect capacity by 50% or more. We know from personal experience that some engineers will pay more attention to an out-of-date gage calibration certificate than to an obvious lack of proper base cleaning of a bored pile.

The evidence suggests that many engineers fail to appreciate the importance of construction technique on bored pile capacity. Some engineers, and many contractors, sincerely believe that "the dry hole method" provides the best way to install a bored pile. The reason often given: we can examine a dry hole by eye or with a camera and such inspection ensures good end bearing conditions. (i.e. visual quality control) The evidence from our testing, however, suggests that the worst cases of poor end bearing capacity occur in dry holes (Schmertmann and Hayes, 1997, Schmertmann et al, 1998). There are several reasons for this, namely:

- Excessive stress relief in a dry hole can disturb a large zone below and around the base of the pile
- The inability to use air lifting or pumping procedures and the ineffectiveness of cleanout buckets in dry holes generally make them more difficult to clean.
- Apparent minor seepage at the base of a "dry hole" can dramatically loosen or soften the base

Mounting evidence from static load testing indicates that, unless the founding material consists of reasonably competent rock or hardpan, we should construct every bored pile for important structures using a stabilizing fluid that suits the conditions.

We have also found that both designers and contractors often do not appreciate the impact of some construction techniques on side shear. We know from our testing data that several construction processes can dramatically affect side shear namely:

- Poor concrete/rebar design
- Poor temporary casing techniques
- Improper use of slurry
- Shape, roughness, and vertical alignment

Poor Concrete/Rebar Design - Too much beef?

An example of the impact of mismatched rebar design and concrete, reported by Hayes and Simpson (2003) illustrates how incompatibility of design and constructability impacts foundation capacity. Test results demonstrated how tightly spaced spirals on a heavily reinforced cage, along with concrete having no retarder, reduced side shear by a factor of 10. Table 1 summarizes the test data from this case and one other to illustrate the dramatic reductions that can occur in these situations.

Table 1- Concrete Placement and Rebar Effects on Side Shear

Case Reference	Proper Technique Side Shear	Improper Technique Side Shear	Shear Reduction Factor
A – 723-1, 2	1867 kPa	187 kPa	10
B – 627	1915 kPa *	105 kPa	18
* Estimated unit side shear for shaft in 10,000 psi rock.			

Table 2-Casing Effects on Side Shear

Case Reference	Correct Use of Temp. Casing	Improper Use of Temp. Casing	Side Shear Reduction Factor
C – 932	1341 kPa	67 kPa	20
D – 748	>400 kPa	72 kPa	>5.5
E – 582	192 kPa *	10 kPa	20
* Estimated unit side shear for an uncased shaft in glacial till soils.			

Poor Temporary Casing Technique: Does it really make a difference?

Seemingly insignificant aspects of using temporary casing can also have a major detrimental impact on side shear capacity of bored piles. The results of comparison tests of casing technique (Table 2) further illustrate the important effects that construction details can have on side shear.

The construction sequence in Case D (Figures 2 to 5) provides an interesting example of the detrimental effect of inserting temporary casing in a cored slot, trapping sludge behind the casing. The trapped sludge remained at the limestone concrete interface during the concreting and subsequent removal of the temporary casing. An O-cell® test revealed a side shear in the limestone of only 72 kPa. Changing the construction sequence, in a subsequent test pile, by placing the temporary inner casing after the limestone had been drilled out resulted in a measured unit side shear exceeding 400 kPa. (The test ended well before reaching ultimate shear resistance). The "before and after" load-movement curves (Figure1) establish dramatically how seemingly innocuous changes in construction sequence can have a profoundly deleterious impact.

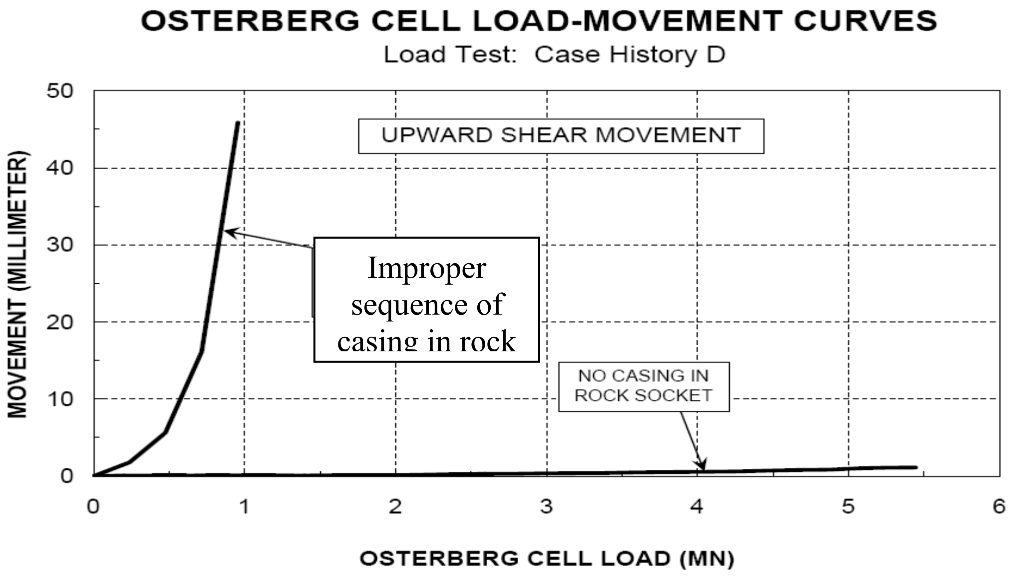
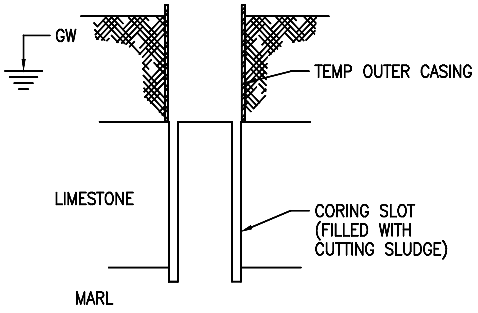


Figure 1 – The Impact of Temporary Casing on Side Shear

<u>Construction Sequence – Case “D”</u>



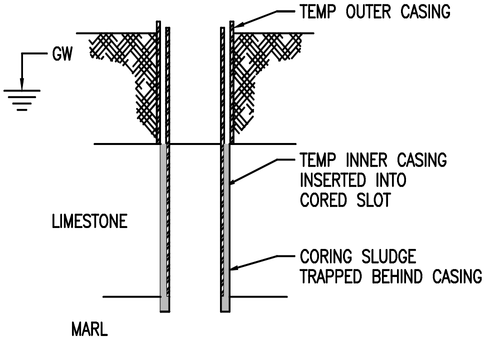
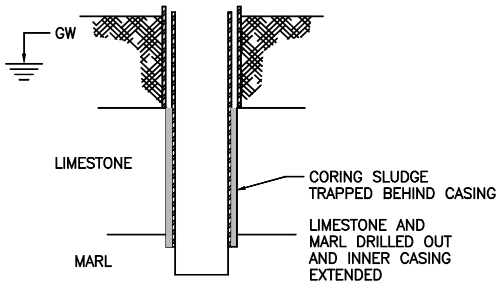


Figure 2- Coring through limestone **Figure 3- Temp. casing inserted into slot**



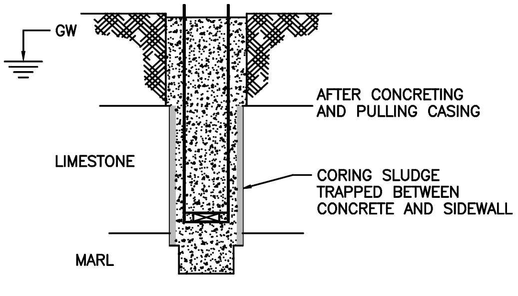


Figure 4- Limestone drilled out

Figure 5- O-cell® assembly inserted, concrete placed and temporary casing removed.

Engineering Misconceptions and Intuition – Can we make sense of an apparent pardox?

The foregoing examples of the impact of construction technique should make us very aware of the crucial importance of quality assurance testing of the foundation "product" in exposing design and construction techniques that may reduce capacity by 50% to 90% or more. They also indicate the counterintuitive nature of some engineering perceptions. Intuition suggests, for example that a "dry hole" should produce better end-bearing or that temporary casing would not have a significant effect on side shear capacity.

Engineers setting up a testing program for quality assurance of bored piles should take careful note of the sometimes misleading reliance on intuition. For many years, as a consulting engineer, the author intuitively assumed that he could extrapolate the side shear test data from a smaller bored pile to a larger diameter bored pile, all else remaining equal. Intuitively one would assume that such measured unit side shears would reasonably represent a site characteristic for all bored piles. Well, as it turns out, maybe not.

The important work of Seidel and Collingwood (2001) and remarks by M.W. O'Neill (1998) related to the influence of side wall roughness on all side shear capacity of bored piles in rock sockets, indicated to the author that a significant inverse relationship might exist between unit side shear and pile diameter. Seidel and Collingwood formulated the shaft resistance coefficient (SRC) as follows:

$$q_s \approx SRC = \eta_c \cdot \frac{E_m}{(1+\nu)q_u} \cdot \frac{\Delta r}{d_s} \qquad (2)$$

wherein Δr denotes mean roughness height; d_s denotes the socket diameter; η_c denotes a construction method reduction factor, ν denotes Poisson's Ratio, E_m denotes rock mass modulus and q_u denotes the unconfined compressive strength of the rock. (E_m/q_n commonly defines the modular ratio).

Earlier work by Horvath et al (1983) also produced an empirical equation for shaft resistance as follows:

$$q_s = 0.8\left(\frac{\Delta r_h}{0.5d_s}\frac{L_t}{L_s}\right)^{0.45} .\sigma_{cw} \qquad (3)$$

where q_s denotes unit shaft resistance, σ_{cw} denotes unconfined compressive strength of the rock or the concrete shaft, whichever is smaller and L_s denotes socket length, d_s denotes socket diameter, r_h denotes mean roughness height and L_t denotes traversed length along the rough surface of the socket. Both of these formulations suggest an inverse relationship between unit shear resistance, q_s and shaft diameter, d_s. Out of curiosity the author compiled unit shear data from O-cell® tests carried out in rock sockets in shale at various USA locations as shown in Table 3.

Table 3- Data from Loadtest Files Used for Side Shear Study

Project	Material	Shaft Dia.	Max. Test Load	Unit End Bearing (EB)		Unit Side Shear (SS)		Ratio EB/SS
				Max. Meas'd	At 12.5 mm	Max Meas'd	At 5 mm	
		m	MN	kPa	kPa	kPa	kPa	
8785	Clay Shale	1.98	43.9	5267	5267	306	306	17.2
8998	Clay Shale	1.22	21.4	8427	8427	418	418	20.2
8547	Shale	1.37	15.3	4745	3064	517	517	9.2
8932	Shale	2.44	16.1	6876	958	718	479	10 est'd
8879	Shale	1.83	14.7	4668	4668	747	747	6.3
8932	Shale	2.44	160.6	19152	7661	958	383	20.0
8626	Shale	1.07	19.6	9260	9260	982	982	9.4
8817	Blue Shale	1.52	49.4	6416	6416	1025	1025	6.3
8929	Brittle Shale	1.37	34.0	8523	7182	1087	1087	7.8
8626	Shale	1.07	19.6	7613	9911	1187	814	6.4
8958	Massive Shale	1.22	41.8	17093	6033	1197	862	14.3
8739	Claystone	1.07	25.0	11874	11874	1211	1211	9.8
8640	Shale	1.07	16.1	5138	5138	1336	896	3.8
8958	Massive Shale	1.22	30.7	12353	12353	1344	838	9.2
8817	Blue Shale	1.52	65.4	13502	13502	1570	1197	8.6
8516	Black Shale	1.17	34.8	9624	9624	1880	1880	5.1
8640	Shale	1.07	32.5	11539	11539	2062	862	5.6
8621	Shale	1.02	40.7	27723	27723	2385	2394	11.6
8528	Shale	1.22	43.2	18051	18051	3064	1939	5.9
8528	Shale	1.22	49.4	18769	23940	3069	2107	6.1
8854	Clay Shale	1.07	38.0	18099	8379	3800	503	4.8

This preliminary compilation of the data, plotted on Figures 6 to 8 with a linear regression trend line, appears to support the proposition that an inverse relationship between unit side shear and socket diameter does exist, at least in shale rock. Apparently the effect can reduce unit shear capacity by as much as 40% to 50% in soft to medium shales and even more in harder shales. Further analysis and research, currently underway, will examine this phenomenon in more detail.

In the meantime the evidence mounts – ignore it at your peril!

The only case history known to the author, that provided evidence of the pile diameter (or roughness ratio) effect, evolved from the foundation study (quality assurance) for a high rise structure in Miami. In their paper describing the study Frizzi and Meyer (2004) concluded that - "The use of other correlations in the literature based on lighter loaded, smaller diameter shafts (i.e. smaller scale tests) yield un-conservative side shear predictions."

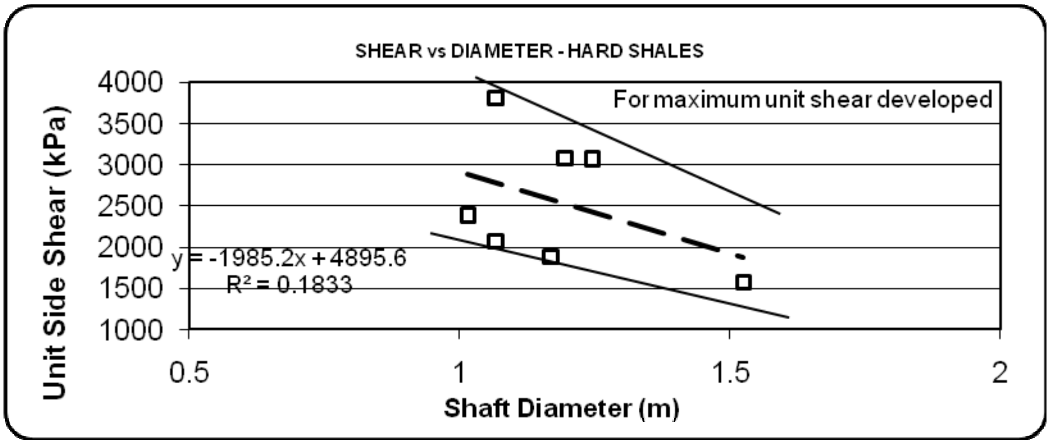


Figure 6- Diameter effect on shear in hard shale

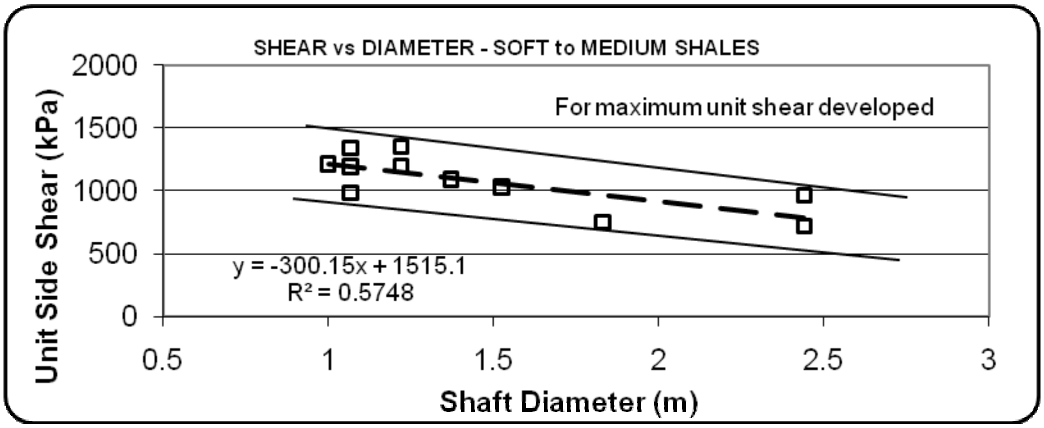


Figure 7- Diameter effect on shear in medium shale

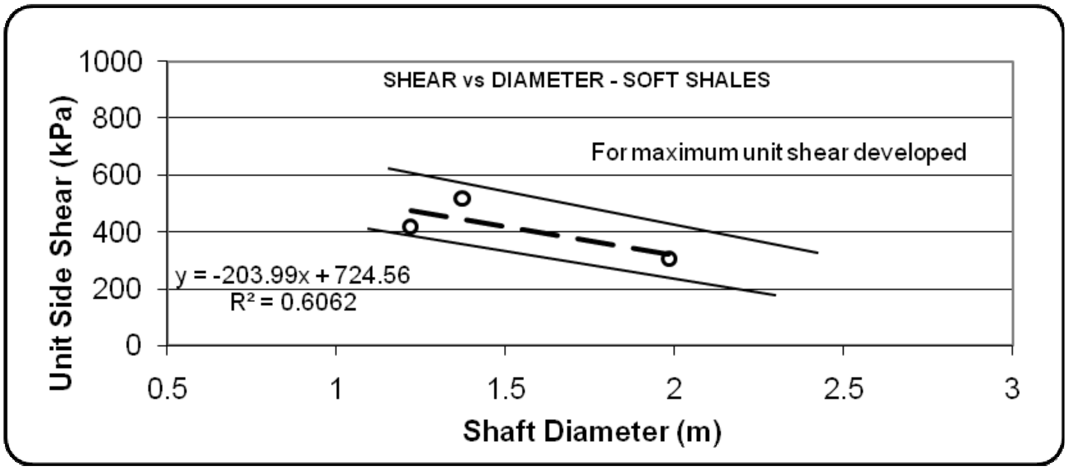


Figure 8- Diameter effect on shear in soft shale

The importance of this "diameter effect" often comes into play when designers of quality assurance test programs consider whether or not to use a reduced-diameter (or model) test shaft instead of a full size prototype for a load test. Engineers usually reason that testing a smaller pile would reduce costs or that testing a full size pile to three or four times design load exceeds normal testing capabilities (the required test loads being too high). Clearly, however, using the data from testing a smaller pile without making a currently uncertain correction for roughness may lead to an unsafe extrapolation of the measured unit shear to larger diameter piles.

Do Model Piles Make Sense in Today's Deep Foundation World?

The advent of the O-cell® method (Osterberg 2001) for pile load testing makes it possible to apply (formerly inconceivable) high loads to large diameter bored piles. We no longer have to test "model" piles. Beginning in 1993 the magnitude of applied static test loads has increased steadily from 58 MN to the current world record of 284 MN achieved on the New Incheon Bridge in March, 2005. The chart in Figure 9 summarizes the trend.

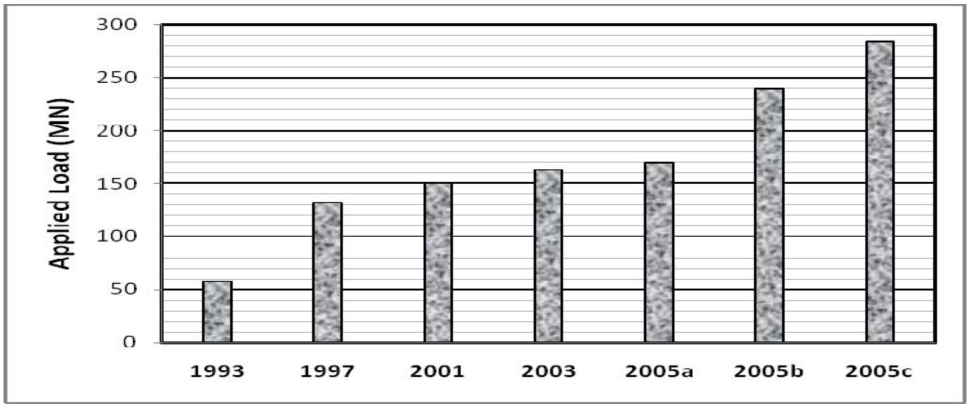


Figure 9- Historical Trend in World Record Test Loads

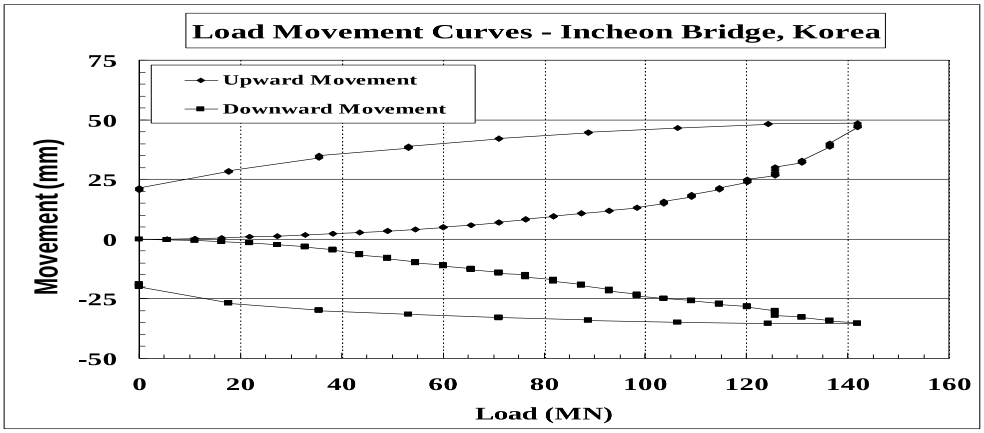


Figure 10 - Load-movement curves, Incheon Bridge, South Korea

These important tests provide convincing evidence that we can achieve quality assurance, not only of design procedures but also of construction technique, even for very high capacity bored piles. *In Part 3 we will consider how better work and higher quality can add value and lower costs.*