Preliminary and Working Pile Load Tests in Simsima Limestone

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ABSTRACT

A load testing program was undertaken to determine the working capacity of drilled shafts in Simsima Limestone, the predominant founding stratum in Doha, Qatar. The drilled shafts ranged from 500 mm to 1,500 mm in diameter and gained capacity from both side and base resistance within the Simsima Limestone. The weathering profile of the Simsima Limestone is complex with the degree of weathering likely to increase or decrease with depth. For the purposes of design, the limestone was characterized into three grades of weathering (A, B, and C) and the pile load testing program focused on testing each of these zones.

The load testing program consisted of a series of instrumented Osterberg-cell tests, conventional jack and reaction-frame tests, high-strain dynamic tests, and tension tests to determine the pile-rock interactions within the various zones of Simsima Limestone. The load test results are presented and the findings summarized.

Construction of the working piles raised concerns about the effect of softening with time for the exposed limestone during drilling. Extrapolation of the results of the load testing program and additional high-strain dynamic testing were used to address these issues.

The results of the tests are presented and compared to the design assumptions with suggestions made to optimize future designs.

INTRODUCTION

Three adjacent project sites in Education City, Qatar located on the outskirts of Doha City were constructed simultaneously and in similar geologic conditions. As a result, the pile load tests from each of the sites can be combined to provide a larger database of load tests in the Simsima Limestone from which comparisons to the design assumptions are made. Because of the importance of the structures and the complex structural systems required for the projects a comprehensive load testing program was undertaken. The load testing program consisted of a series of instrumented Osterberg-cell tests, conventional jack and reaction-frame tests, and tension tests and was designed to test preliminary, sacrificial piles and a percentage of the working piles. Piles that were constructed outside the requirements of the specifications were evaluated with high-strain dynamic testing.

Several methods for the design of rock socket drilled shafts have been developed relating various geotechnical properties to values for side friction and end bearing resistances. The side friction and end bearing values used in design are be compared to the results of pile load tests in order to validate the design and identify areas where future designs can be optimized.

GEOLOGIC SETTING AND SUBSURFACE CONDTIONS

The Simsima Limestone is a member of the Upper Dammam Formation and is the prevalent near surface geologic unit in Doha, Qatar. Consequently, the Simsima Limestone is the founding level for many structures in Doha. The weathering profile of the Simsima Limestone is complex with the degree of weathering likely to increase or decrease with depth and zones of less weathered rock commonly overlying zones of more weathered rock. Previous efforts have been made to characterize the Simsima Limestone for the purposes of foundation design (Fourniadis, 2010) and were considered for this work as well.

CHARACTERIZATION OF THE SIMSIMA LIMESTONE

For the purposes of design, the Simsima Limestone was characterized into three site-specific grades (zones) of rock quality or weathering, grades A through C, with grade A being the most intact/competent rock and grade C representing the lowest quality rock. The characterizations of the grades are summarized in Tables 1a and 1b.

Table 1a. Simsima Limestone Grade Designation

Simsima Weathering Grade	Intact Rock UCS (MPa)	RQD	Mean Fracture Spacing (mm)	Fracture Conditions
A	15	50-75	200-600	Weathered
B	15	25-50	60-200	Gouge <5mm
C	15	<25	<60	Gouge >5mm

Table 1b. Simsima Limestone Grade Designation, Cont.

Simsima Weathering Grade	RMR	Class/ Description	φ (°)	Cohesion (kPa)
A	47	III/ Fair Rock	25-35	200-300
B	30	IV/ Poor Rock	15-25	100-200
C	12	V/ Very Poor Rock	<15	<100

BASIS OF DESIGN

The safe working load (SWL) for each pile was taken to be the smallest value of the following three equations:

$$Q_W = \frac{Q_s}{1.5}, Q_W = \frac{Q_s + Q_b}{2.5}, \text{and } Q_w = \frac{A_b f'_c}{4}, \text{ where} \quad (1)(2)(3)$$

Q_w = the safe working load on the piles (SWL);
Q_s = the ultimate side capacity of the pile;
Q_b is the ultimate base capacity of the pile;
A_b is the base are of the pile; and
f'_c is the concrete unconfined compressive strength.

The ultimate side capacity is given by the relationship proposed by Zhang (1997) for smooth socket walls:

$$Q_s = 0.4\sqrt{UCS}, \text{ where} \quad (4)$$

UCS = the mass unconfined compressive strength in MPa.

The ultimate end bearing (base) capacity of the pile is given by the lower bound relationship proposed by Zhang and Einstein (1998):

$$Q_b = A_b 3.0\sqrt{UCS} \quad (5)$$

The ultimate side capacities used in the pile design were 500 kPa, 400 kPa, and 200 kPa for Simsima Limestone grades A, B and C, respectively. The ultimate base capacity used in the pile design was 7,000 kPa for grade A and 5,900 kPa for grade B Simsima Limestone. No pile bases were to be founded in grade C.

OVERVIEW OF LOAD TEST PROGRAM

The load testing program was undertaken at three separate stages to evaluate the validity of the design and the acceptability of the constructed piles. The three stages were:

1. Prior to construction on preliminary, sacrificial test piles in order to confirm the basis of design;
2. During construction on working piles to confirm the constructed state; and
3. After completion of the pile installation program to confirm the capacity of piles that were constructed by methods that did not conform to the specifications.

PRELIMINARY LOAD TESTS

Ten preliminary piles were tested across the three sites immediately prior to the working pile installation program. Four of the tests were conventional jack and reaction-frame tests and six tests were Osterberg-cell tests. The preliminary test piles were designed to test the ultimate rock-pile interface in each of the weathered zones of the Simsima Limestone as well as the end bearing capacity of the piles. All piles were instrumented with several levels of strain gauges and extensometers. The piles were planned to be tested to failure so that the ultimate capacities could be determined. A borehole was drilled within two meters of each of the test pile locations to confirm the subsurface conditions.

Jack and Reaction-Frame Tests

Four conventional jack and reaction-frame tests were performed on instrumented preliminary test piles in advance of the pile installation program. Instrumentation was placed to measure the load distribution along the pile lengths within specific grades of the Simsima Limestone. To isolate the zone of Simsima Limestone to be tested, a dual casing system was used comprising a larger diameter casing installed around the pile casing such that a small void was present between the two casings. Three to four levels of strain gauges and two levels of extensometers were installed in the piles, with four instruments being installed at each level. One level of strain gauges was installed immediately below the dual casing level to measure the effectiveness of the dual casing system in eliminating resistance from the overburden.

In all four of the jack and reaction-frame tests the base resistance in the piles was minimized and only the side friction was to be tested. To remove the base resistance, a soft-toe comprised of 300 mm of expanded polystyrene was installed at the bottom of the rebar cage. The soft-toe was required to provide less than 350 kPa of resistance. A typical cross section of the preliminary jack and reaction-frame tests is shown in Fig. 1a.

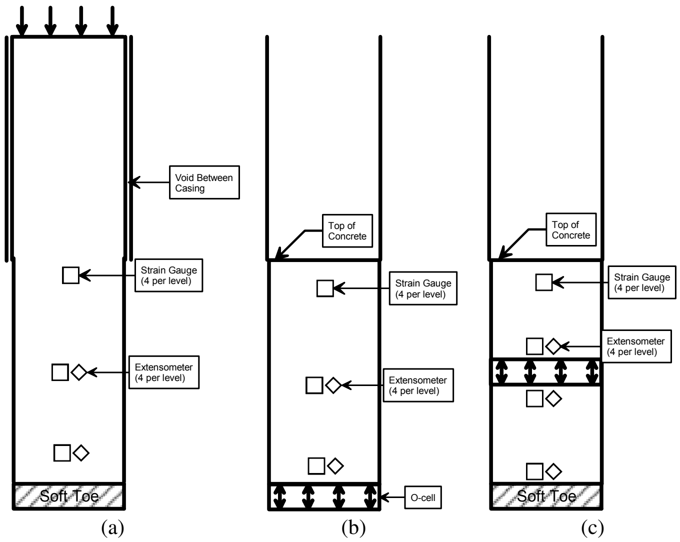

Fig 1. Cross-Sections of Typical Preliminary Load Test Setups (a) Jack and Reaction-Frame with soft-toe, (b) O-cell at Base of Pile, and (c) O-cell at mid-point of shaft with soft-toe.

A typical plot of load versus settlement at the top of the pile is shown in Fig. 2. Three load cycles were performed with the top load at each cycle corresponding to 100% of SWL, 150% of SWL, and predetermined maximum test load. Figure 3 shows the typical load distribution along the pile for the final load cycle and shows some residual stresses at the zero load. The measured loads at the upper and lower levels of strain gauges indicates that the dual casing system and soft-toe performed adequately.

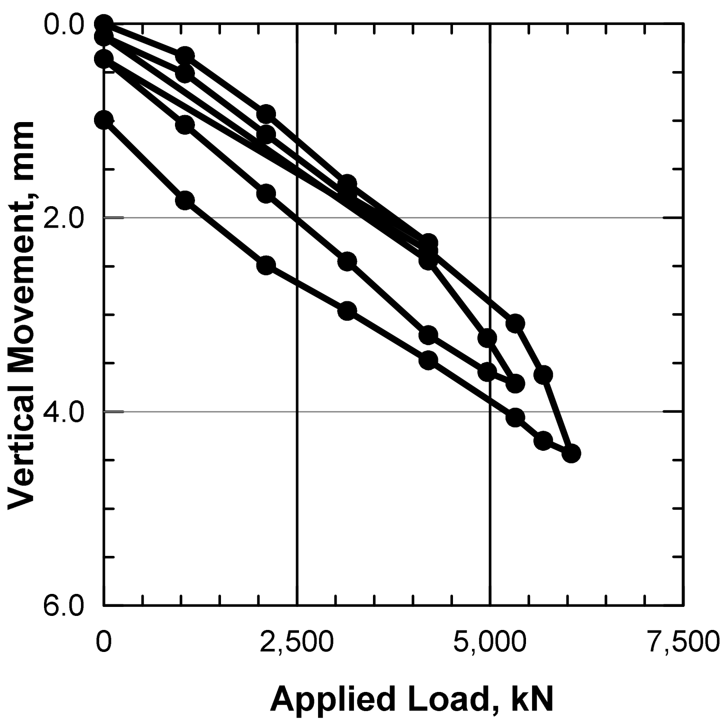

Fig. 2. Typical Load Displacement Curve, from PC3

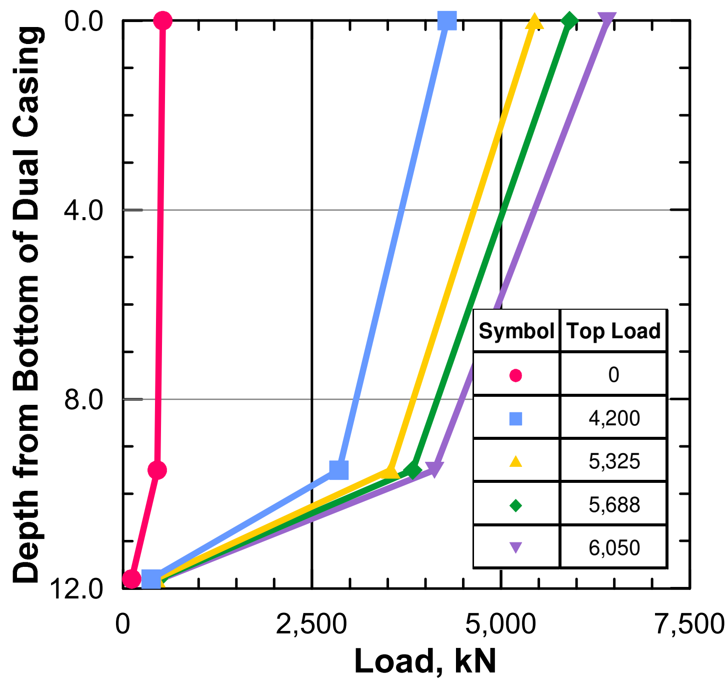

Fig. 3. Side Friction Load Distribution, from PC3

Table 2 summarizes the results of the four preliminary jack and reaction-frame tests, including the maximum mobilized side friction attained in each of the tests and the grade of Simsima Limestone that was tested.

Table 2. Results of Preliminary Load Tests by Jack and Reaction Frame

Test ID	PC1	PC2	PC3	PC4
Diameter (mm)	750	750	750	1,000
Maximum Load (kN)	5,200	8,950	6,050	10,650
Maximum Deflection (mm)	3.0	7.6	4.4	8.2
Prevalent Simsima Grade	A	A	B	B
Maximum Mobilized Side Friction (kPa)	511	1,877	668	1,155

Osterberg-Cell Tests

Six instrumented Osterberg Cell (O-cell) tests were performed on preliminary test piles in advance of the pile installation program. The O-cell tests were contractor proposed alternates to the conventional jack and reaction-frame tests. The instrumentation was placed to measure either the load distribution along the pile lengths within specific grades of the Simsima Limestone or the base resistance. As shown on Figs. 1b and 1c, the O-cell was either installed at the base of the pile or at the mid-height of the test section. Placing the O-cell at the base of the pile allows for testing the base resistance of the pile as well as the side friction above the O-cell. If the O-cell was placed at the mid-height of the test section in order to test two zones of Simsima Limestone, then a soft-toe was installed to minimize the contribution of the base resistance. Figures 4 and 5 show the typical load displacement curves from an O-cell test and the measured side shear distribution, respectively.

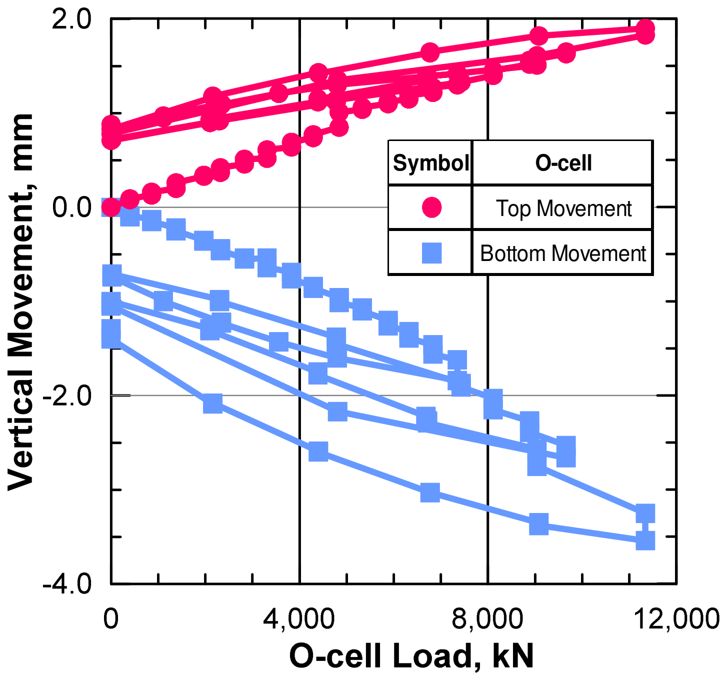

Fig. 4. Typical O-cell Load Displacement Curve, from O4

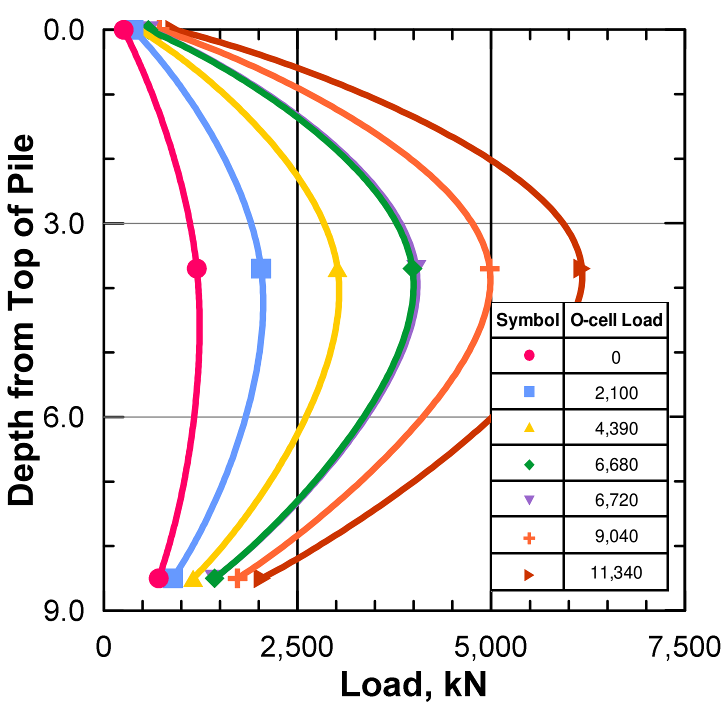

Fig. 5. Typical Side Shear Distribution, from O4

Tables 3a and 3b summarize the results of the O-cell tests, showing the maximum mobilized side or base resistance and the corresponding grades of Simsima Limestone, where the terms above and below refer to above and below the O-cell.

Table 3a. Results of Preliminary Load Tests by O-cell

Test ID	O1		O2		O2	
Diameter (mm)	750		750		750	
Maximum Cell Load (kN)	11,150		11,500		11,500	
Maximum Deflection (mm)	Side	Base	Above	Below	Above	Below
	4.0	48	2.4	18	0.8	0.9
Prevalent Simsima Grade	A	A	B	C	B	B
Maximum Mobilized Side Friction (kPa)	1,538	25,170 (base)	2,183	2,110	1,073	3,077

Table 3b. Results of Preliminary Load Tests by O-cell, cont.

Test ID	O4		O5		O6	
Diameter (mm)	750		1000		1000	
Maximum Cell Load (kN)	11500		22500		20000	
Maximum Deflection (mm)	Above	Below	Above	Below	Side	Base
	1.9	3.5	1.5	0.5	2.2	90
Prevalent Simsima Grade	B	C	A	A	A	A
Maximum Mobilized Side Friction (kPa)	1,767	1,137	3,867	4,865	1,545	12,732 (base)

WORKING PILE LOAD TESTS

Thirteen working piles were tested using jack and reaction-frame tests. Four of the working pile tests were instrumented with three levels of strain gauges and two levels of telltales and nine of the piles were not instrumented. In all cases, the top of pile load-deflection was measured.

The maximum test load correlated to 100% of the design verification load (DVL) on the piles plus 50% of the SWL. The DVL is equivalent to the SWL plus any additional load required in order to overcome skin friction above the pile cutoff level. If the piles were tested from the design cutoff level, then the DVL is equal to the SWL. The maximum allowable deflection was set to 16mm at maximum test load. The piles were tested in two cycles, first loading to 100% of DVL and then unloading to zero load and reloading to 100% DVL plus 50% SWL. Loading increments were typically 25% of DVL. The load deflection curve of a typical test is shown on Fig. 6.

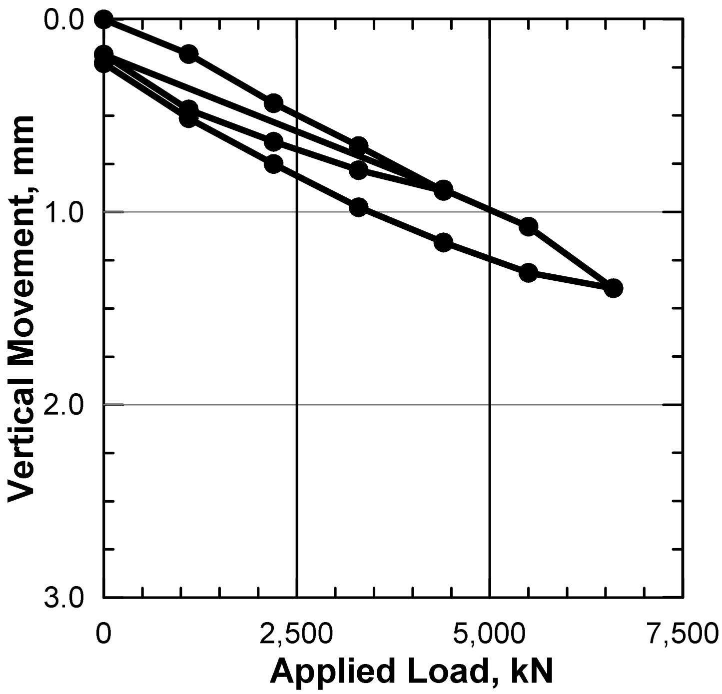

Fig. 6. Typical Load Deflection Curve, from C2

Tables 4a and 4b summarize the results of the working pile load tests showing the maximum mobilized side resistance and the corresponding prevalent grades of Simsima Limestone.

Table 4a. Results of Working Pile Load Tests

Test ID	C1	C2	C3	C4	C5	C6
Diameter (mm)	500	750	750	750	750	500
Maximum Load at Top (kN)	1,500	4,400	4,400	4,400	4,400	3,516
Maximum Deflection (mm)	0.9	1.4	1.4	1.4	1.3	1.5
Prevalent Simsima Grade	A	C	A	A	B	A
Maximum Mobilized Side Friction (kPa)	N/A	371	363	376	358	N/A

Table 4b. Results of Working Pile Load Tests, cont.

Test ID	C7	C8	C9	C10	C11	C12	C13
Diameter (mm)	1,200	1,500	1,000	750	1,000	500	1,200
Maximum Load at Top (kN)	18,777	25,706	7,463	9,941	7,000	2,700	15,000
Maximum Deflection (mm)	6.7	3.4	2.3	3.0	2.2	1.2	3.5
Prevalent Simsima Grade	A	A	A	A	A	A	A
Maximum Mobilized Side Friction (kPa)	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Four working piles were also tested in tension by pulling on a bar embedded in the pile. Only one tension test pile was instrumented. The results are summarized on Table 5.

Table 5. Results of Tension Load Tests

Test ID	T1	T2	T3	T4
Diameter (mm)	750	750	1,200	500
Maximum Load (kN)	-1,500	-2,345	-5,796	-1,440
Maximum Deflection (mm)	-0.9	-5.0	-5.6	-2.1
Prevalent Simsima Grade	A	C	A	A
Maximum Mobilized Shaft Friction (kPa)	-93	N/A	N/A	N/A

LOAD TESTS ON NON-CONFORMING PILES

Eleven high-strain dynamic tests were performed in order to evaluate the capacity of piles that were constructed by methods that did not conform to the specifications. The project required that piles be constructed (i.e. the concrete cast) within 12 hours of the start of drilling to avoid softening of the rock exposed inside the shaft. The dynamic tests were performed by dropping an 11 or 13 ton hammer onto the piles with a Pile Driving Analyzer (PDA) attached. The mobilized skin resistance and toe resistance was determined from CAPWAP analyses. Tables 6a and 6b summarize the results of the dynamic load tests.

Table 6a. Results of High-Strain Dynamic Load Tests

Test ID	D1	D2	D3	D4	D5	D6
Diameter (mm)	500	750	500	500	1000	500
Maximum Load (kN)	5126	5179	5003	5200	12666	3309
Maximum Deflection (mm), from CAPWAP	2.3	0.7	2.0	1.9	2.6	1.6
Prevalent Simsima Grade	C	C	C	C	B	B
Maximum Mobilized Side Friction (kPa)	2443	1595	2213	3033	3522	1988

Table 6b. Results of High-Strain Dynamic Tests, cont.

Test ID	D7	D8	D9	D10	D11
Diameter (mm)	500	1000	1200	1000	500
Maximum Load (kN)	3257	12090	16227	13163	2901
Maximum Deflection (mm), from CAPWAP	2.3	2.3	2.3	2.3	1.9
Prevalent Simsima Grade	B	B	A	B	B
Maximum Mobilized Side Friction (kPa)	1779	3537	3939	3503	1634

COMPARISON OF ALL TESTS

Mobilized Base Resistance

Base resistance was measured in two O-cell tests and evaluated in high-strain dynamic tests. The average value of base resistance that was reached during the O-cell tests was 18,951 kPa, while the 11 high-strain dynamic tests mobilized an average of 2,244 kPa. A design ultimate value of 7,000 kPa was used for grade A Simsima Limestone. The ultimate base capacity was not reached during the testing program.

Mobilized Side Resistance

Statistical analyses of the load tests were performed to evaluate the mobilized side friction values. Figure 7 shows that the high-strain dynamic tests were able to mobilize the highest side friction values. Tension loads were relatively minor and as such the tension tests mobilized very little side friction. The ultimate capacity of the shaft-rock interface was not reached in any of the tests despite the test loads exceeding the design ultimate capacity values that were determined by conventional design measures.

The average side friction developed by the high-strain dynamic tests was 2,653 kPa based on 11 tests. The average side friction developed by the O-cell tests was 1,814 kPa based on 21 strain gauge levels. The average side friction developed by the jack and reaction-frame tests was 489 kPa based on 18 strain gauge levels. The average side friction developed by the tension tests was 75 kPa based on 2 strain gauge levels.

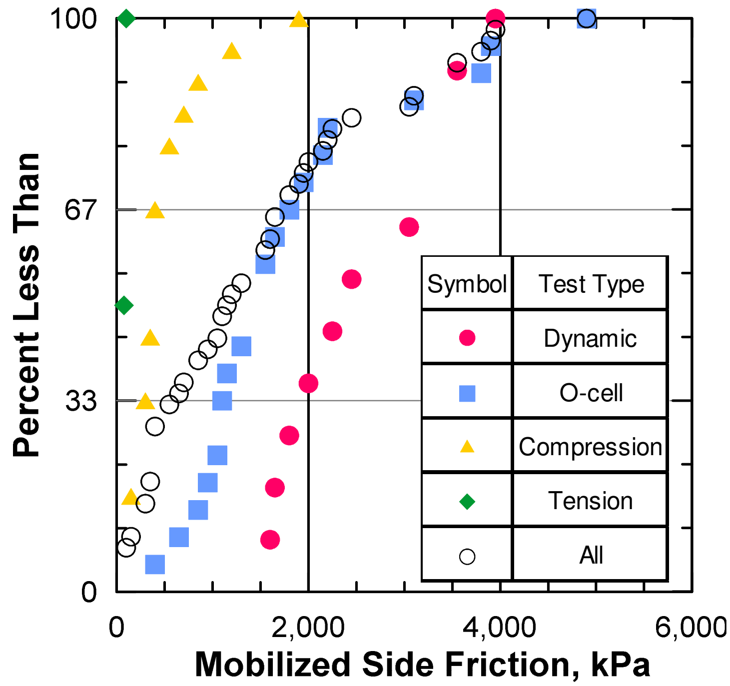

Fig 7. Cumulative Distribution of Mobilized Side Friction values by Test Type

Figure 8 shows that the grade A Simsima Limestone achieved the highest mobilized side friction values. This result was expected as grade A is the most competent zone.

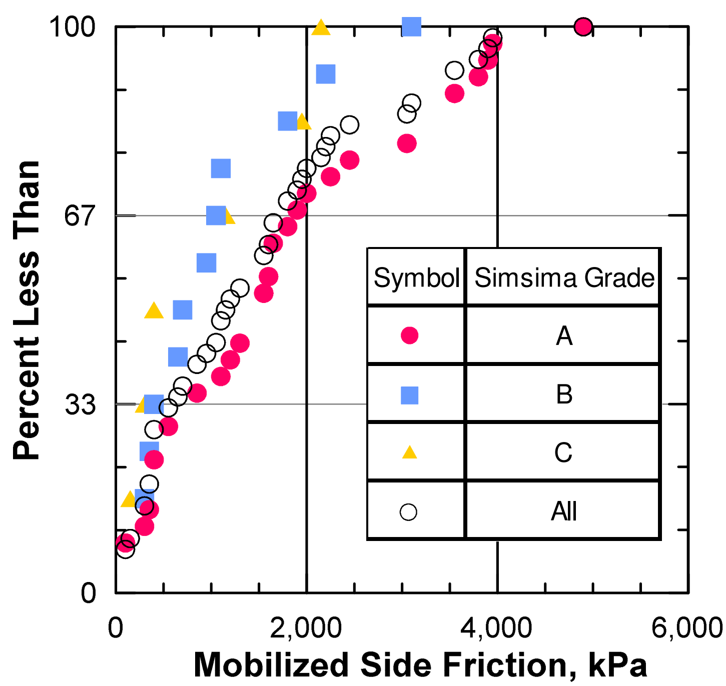

Fig 8. Cumulative Distribution of Mobilized Side Friction value by Grade of Simsima Limestone

The average side friction developed in grade A Simsima Limestone was 1,510 kPa, based on 19 data points. The average side friction developed in grade B Simsima Limestone was 1,475 kPa based on 21 data points. The average side friction developed in grade C Simsima Limestone was 1,380 kPa based on 12 data points. The design ultimate side friction values were 500 kPa, 400 kPa, and 200 kPa for grades A, B, and C Simsima Limestone, respectively. The measured values exceeded the design values and yet still do not represent ultimate values as none of the piles reached the ultimate state during testing.

CALIPER LOGGING

Caliper logging was performed on several of the test piles to assess the roughness profile of the drilled shafts. A smooth profile was assumed during design; however caliper logging indicated that a rougher side wall profile could have been assumed. The results of a typical test are shown on Fig. 9.

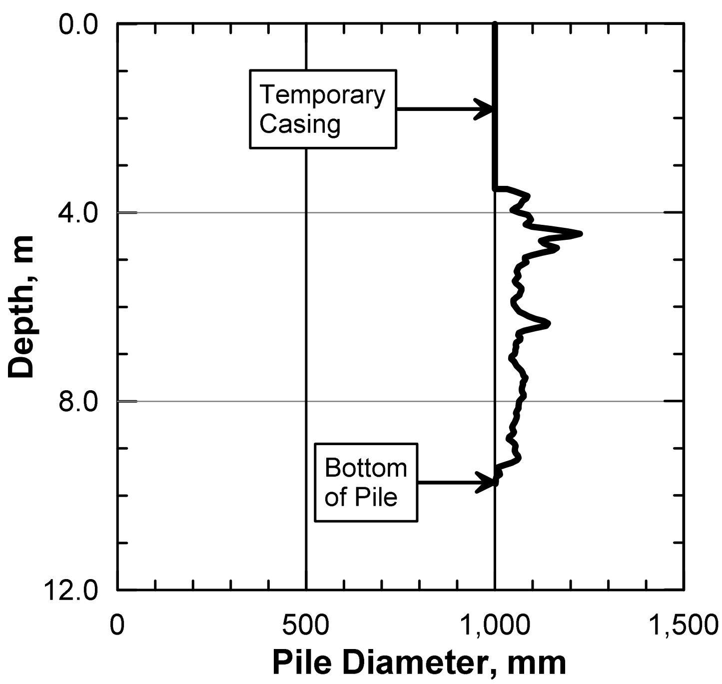

Fig. 9. Example of Caliper Logging Results, from O6

Comparing the roughness profile to the roughness classes proposed by Pells et al. (1980) indicates that the shaft wall roughness would fall into the highest roughness class (R4). R4 is defined as having grooves or undulations of depth greater than 10 mm and width greater than 10 mm, at spacings between 50 to 200 mm. A smooth-sided socket has been assumed during design. No explicit shaft wall roughening was performed.

CONCLUSIONS

1. A comprehensive load testing program, including preliminary load tests to confirm the design, tests on working piles to confirm the constructed state, and tests after construction to investigate piles that are out of specification provides confidence in the foundation design.
2. Preliminary load tests must be performed far in advance of installation of the working piles in order to optimize the design.
3. Confirmation testing of piles that exceeded the 12-hour limit for installation indicated that the piles met required capacity. Whether the Simsima Limestone softened after being exposed to water and air was not evaluated however if softening did occur, it was not sufficient to reduce the load bearing capacity to unacceptable levels.
4. The Simsima Limestone in this area of Qatar is capable of supporting high skin friction loads. This study was unable to reach the maximum capacity of the Simsima Limestone despite loading the limestone to up to three times the expected values determined from design methods.
5. In general, the pile-rock interface had a higher capacity than was anticipated during design. This may be a result of any of the following:
   a. The roughness of the side walls of shafts was greater than expected. Several piles were logged with calipers and indications are that the side walls were rougher than assumed.
   b. Unconfined compression strength tests on intact samples of the Simsima Limestone as low as 2 MPa were discovered during the subsurface investigation and informed the design. These values may not have been representative.
   c. At some locations, rock socket strength was determined to be limited by the strength of the concrete and a concrete strength of 40/50 MPa was assumed. During construction, the concrete strength was as high as 75 MPa, which amounts to a 50% increase.
   d. To simplify the construction process, only four pile designs were utilized: 500 mm diameter, 750 mm diameter, 1,000 mm diameter, and 1,200 mm diameter, each with a corresponding set length (i.e. all 750 mm diameter piles were 10 m long). The piles were installed such that the most economical of the four piles were installed at column locations, however by default, extra capacity will be available in piles that do not encounter the worst case rock conditions.
   e. Strength design processes, such as a factor of safety, were utilized to determine the capacity of the piles. Once a more complete database of site-specific design parameters is created, a design based on allowable movements could lead to more efficient design.
   f. For the purposes of design, side friction contribution from the soil and rock above the top of the Simsima Limestone were ignored. In some cases, these upper units provided additional support to the installed working piles.

REFERENCES

Fourniadis, I. [2010], “Geotechnical Characterization of the Simsima Limestone (Doha, Qatar)”, *Proc. GeoShanghai 2010 Intern. Conf on Geoenvironmental Engrg. and Geotechnics: Progress in Modeling and Applications*, Shanghai, China, June 3-5, pp. 273-278.

Nam, M. S. and Vipulanandan, C. [2008], “Roughness and Unit Side Resistances of Drilled Shafts Socketed in Clay Shale and Limestone”, Journal of Geotechnical and Geoenvironmental Engrg, Vol 134(9), pp. 1272-1279.

Pells, P.J.N., Rowe, R. K., and Turner, R. M. [1980], “An Experimental Investigation into Side Shear for Socketed Piles in Sandstone”, *Proc. Intl Conf. on Structural Foundations on Rock*, Sydney, Australia, Vol. 1, pp. 291-302.

Sadiq, A.M. and Nasir, S.J. [2002], “Middle Pleistocene Karst Evolution in the State of Qatar, Arabian Gulf”, Journal of Cave and Karst Studies, Vol. 64(2), pp. 132-139.

Willams, A. F. and Pells, P. J. N.[1981], “Side Resistance Rock Sockets in Sandstone, Mudstone, and Shale” Canadian Geotechnical Journal, Vol. 18(4), pp. 502-513.

Zhang, L. and Einstein, H. H. [1998], “End Bearing Capacity of Drilled Shafts in Rock”, Journal of Geotechnical and Geoenvironmental Engrg, Vol. 124(7), pp. 574-584.