Performance of Rock-socketed Drilled Shafts in Deep Soft Clay Deposits

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SYNOPSIS

In designing rock-socketed drilled shaft, bearing capacity evaluation is very important because the maximum values of base and side resistance are not generally mobilized at the same value of displacement. FHWA and AASHTO code suggest different ultimate bearing capacity formular according to rock type and shaft settlement. In domestic code suggest base resistance and side resistance can be added on condition that after confirming the result of field load test with axial load transfer test.

This paper shows that static load test and bi-directional load test result analysis of deep rock-socketed drilled shaft in three different sites. Load-settlement curve, t-z, and q-w curve in rock-socketed part were calculated and compared. t-z curve in weathered and soft rock showed no deflection softening behavior in pretty large strain (about 2-3 % of diameter). Ultimate resistance could be the summation of side resistance and base resistance in rock-socketed drilled shaft in domestic sites.

Key Words : Drilled shaft, Rock-socket, Side resistance, Base resistance, t-z curve, q-w curve

1. INTRODUCTION

Drilled shafts are used heavily for foundations for bridges and other transportation structures in Korea, principally because they are cost-effective relative to driven steel pipe pile. They are constructed by excavating into the rock, forming a cylindrical socket, and the sockets are concreted, usually with steel reinforcing. The "rock socket" resists structure loads, R_T, through a combination of side resistance, R_S, and base resistance, R_B, in the rock and in the "overburden" R_{SO}. A schematic of a typical rock-socketed drilled shaft is shown in Figure 1.

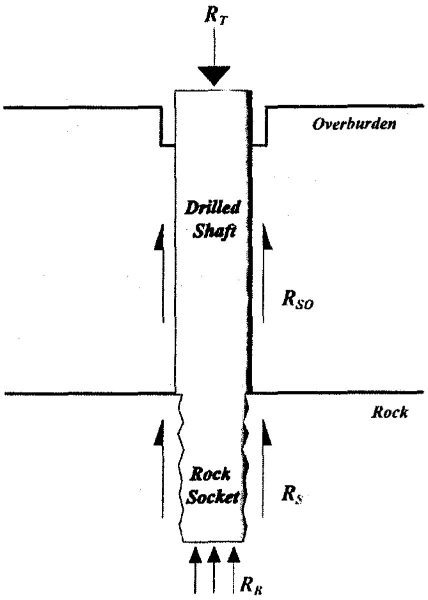

Fig. 1. Schematic of a typical rock-socketed drilled shaft(15)

1.1 Rock Socket Behavior

1.1.1 Side Resistance

Side resistance in rock sockets can develop in one of the three ways:

(1) through shearing of the bond between the concrete and the rock that develops when cement paste penetrates into the pores of the rock(bond),

(2) sliding friction between the concrete shaft and the rock when the cement paste does not penetrate into the pores of the rock and when the socket is smooth (friction), and

(3) dilation of an unbonded rock-concrete interface, with increase of in effective stresses in the rock asperities around the interface until those asperities shear off, one by one (dilation). Dilation behavior is also accompanied by friction behavior.

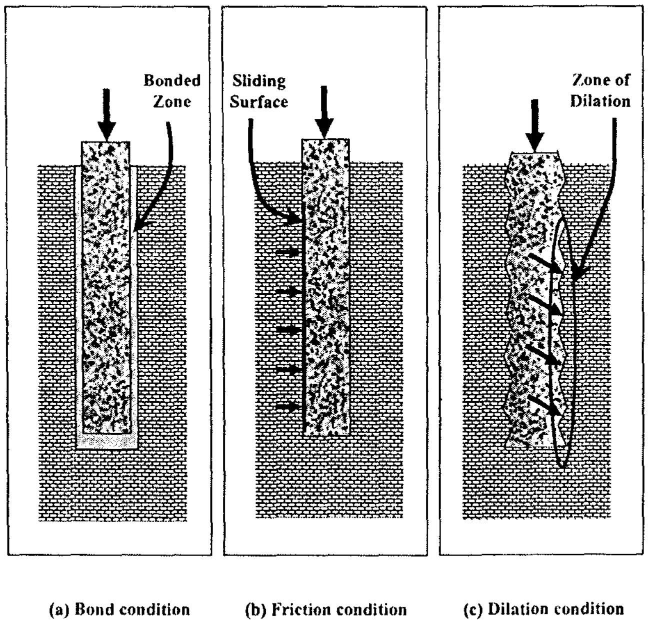

Fig. 2. Schematic representation of interface conditions in rock sockets(11)

1.1.2 Rough and Soft socket

The designer must decide whether the socket in a rock layer will be smooth or rough, since roughness of the borehole wall has a large effect on side resistance. It is currently recommended that unless the sides of the borehole will be artificially roughened during construction that the socket be considered smooth. If artificial roughening of the borehole wall is specified, the socket can be considered rough if keys at least 76mm in height are cut to a depth of at least 51mm into the borehole wall every 0.46m of depth for shafts with diameters equal to or larger than 0.61m in diameter.

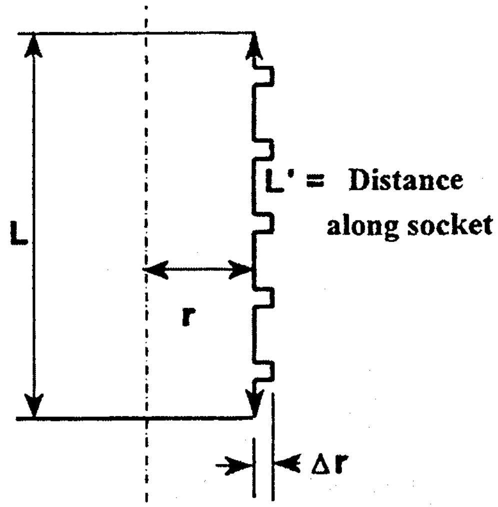

Fig. 3. Definition of geometric terms in rock socket(15)

Table 1. Rough rock socket condition

Unit (mm)	Rock (Rowe and Armitage, 1984)	Cohesive IGM (O'Neill et al., 1996)
h	≥ 10 mm	≥ 76 mm
Δr	≤ 10 mm	≥ 51 mm
L'	(50~200) mm	≥ 460 mm

1.1.3 Base Resistance

Bearing capacity theories have long been developed for deep foundations in soil; however they cannot be applied directly to rock because bearing capacity in rock in often controlled by fracture propagation, which is strongly controlled by the existence of joints and seams in the rock. O'Neill and Reese(1999) indicated that if the rock is massive and if the base of the socket is embedded in sound rock, the ultimate unit base resistance will be 2.5 times the median q_u of the rock to two socket diameters below the base of the socket. Where the rock is jointed below the base of the socket, the base resistance is reduced severely because the joint accelerate the development of fracture in the rock on which the socket is resisting.

1.2 Type of Rock

Rock can be classified by several different specification. Following table shows the general classification of rock in domestic site.

Table 2. Classification of rock (5) (1 MPa = 10kg/㎠ = 100 t/㎡)

		TCR (%)	RQD (%)	q_u (MPa)	N (blow/㎝)
Weathered Soil		0	0	≤ 1MPa	≤ 50/15
Weathered Rock		0 ~ 30	≤ 10	1 ~ 20	≥ 50/15
Soft Rock		30 ~ 60	10 ~ 25	10 ~ 40	-
Hard Rock	Regular Rock	60 ~ 80	25 ~ 50	30 ~ 70	-
	Hard Rock	≥ 80	50 ~ 75	60 ~ 130	-
	Very Hard Rock	≥ 80	≥ 75	≥ 120	-

2. ULTIMATE BEARING CAPACITY

2.1 Summation of Side Resistance and Base Resistance

It is important the R_B and R_S be evaluated at a common value of axial displacement, since the maximum values of base and side resistance are not generally mobilized at the same value of displacement. Fig. 4 shows the difference of calculated ultimate resistance and actual ultimate resistance. AASHTO proposed ultimate side resistance occurs about 10 mm relative displacement between shaft and surrounding rockmass and after then continues deflection-softening (Fig. 5).

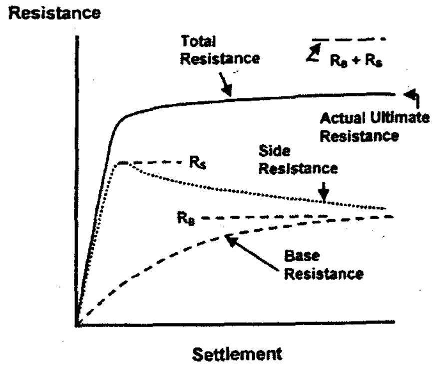

Fig. 4. Load transfer mechanism (15)

2.1.1 FHWA(1999)

FHWA proposed that if a hard, sound rock stratum exists at the base of the drilled shaft, and if only compression loads are applied, it may only be necessary to penetrate the rock a distance large enough to expose the sound rock, in which case R_S can be ignored in the rock socket. In cases significant penetration of the socket will be made, the issue of whether R_S should be added directly to R_B to obtain an ultimate value of R_T for compression loading an important matter. If the rock is ductile in shear (deflection softening does not occur), the R_B and R_S can be added directly.

- Hard Rock (Brittle) $R_T = R_B$
- Weathered Rock, Soft Rock (Ductile) $R_T = R_B + R_C$

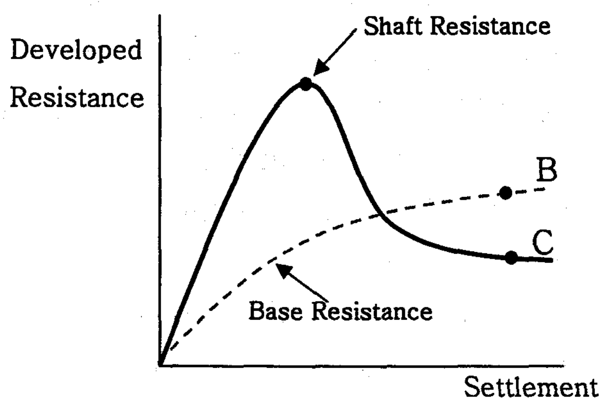

Fig. 5. Illustration of deflection-softening behavior of drilled shafts under compression loading (15)

2.1.2 AASHTO (1999)

AASHTO proposed axial compression load is carried solely by the side resistance on a shaft socketed into rock until a total shaft settlement on the order of 10 mm occurs. At this displacement, the ultimate side resistance is mobilized and slip occurs between the concrete and rock. As a result of this slip, any additional load is transferred to the tip.

2.1.3 Bridge Design Manual (Substructure) (1999)

a) After static load test with axial load transfer analysis, then

$R_T = R_B + R_S$

b) On condition that reference load test data is filed up

$R_T = R_B + R_S$

2.1.4 Structure Foundation Design Manual (2003)

a) After static load test with axial load transfer analysis, then

$R_T = R_B + R_S$

b) Otherwise, axial bearing capacity of rock-socketed drilled shaft

$R_T = R_S$

2.2 Evaluation of Bearing capacity

2.2.1 AASHTO (1996)

a) R_S (RQD > 65%)

$$R_S = \pi B_r D_r \ (0.144 \ q_{SR}) \qquad (1)$$

where, B_r = socket diameter (ft)

D_r = socket length (ft)

q_{SR} = ultimate unit side resistance (psi)

b) R_B

$$R_B = N_{ms} q_u A_t \qquad (2)$$

where, N_{ms} = non-dimensional unit

A_t = cross-sectional area of rock-sockets

q_u = unconfined compression strength of rock core

2.2.2 FHWA (1999)

FHWA classified two different cases Rock and Intermediate geomaterials (IGM).

In case of intermediate geomaterials (IGM);

- cohesive

$0.5 \text{ MPa} \leq q_u < 5.0 \text{ MPa}$

- cohesionless

N ≥ 50

In case of rock; $q_u \geq 5.0$ MPa

a) R_S

- Smooth sockets (Horvath and Kenny (1979))

$$f_{\max} = 0.65 \ p_a \ [q_u/p_a]^{0.5} \leq 0.65 \ p_a \ [f_c'/p_a]^{0.5} \quad (3)$$

where, f_c' = compressive strength of concrete (28 days)

p_a = atmospheric pressure

- Rough Sockets (Horvath et al , 1983)

$$f_{\max} = 0.8 \ [\frac{\Delta r}{r}(\frac{L'}{L})]^{0.45} q_u \quad (4)$$

b) R_B

- Massive Rock (RQD ≅ 100%) and cohesive IGM (Rowe and Armitage 1987)

$$q_{\max} = 2.5 \ q_u \quad (5)$$

- RQD ≅ (70~100)% and q_u ≥ 0.5 MPa (Zhang and Einstein

(1988))

$$q_{\max}(\text{ MPa}) = 4.83[q_u(MPa)]^{0.51} \qquad (6)$$

- Jointed Rock and Cohesive IGM
  Method 1 : Carter and Kulhawy (1988)

$$q_{\max} = [s^{0.5} + (ms^{0.5} + s)^{0.5}]q_u \qquad (7)$$

where, 'ｓ' and 'ｍ' are jointed rockmass parameter

Method 2 : Canadian Foundation Engineering manual (1992)

$$q_{\max} = 3 \ q_u K_{sp} \cdot d \qquad (8)$$

where, K_{sp} = Bearing capacity factor

d = depth factor

$= 1 + 0.4\dfrac{H_s}{B_0} \le 3.4$

$$K_{sp} = \frac{3 + S_d / B}{10\sqrt{1 + 300\dfrac{t_d}{S_d}}}$$

H_s = depth of drilled shaft socket

S_d = vertical spacing between joint

t_d = thickness of joint

The Range of validity above equation are,

$B > 0.3m$, $0.05 < S_d/B < 2.0$, and $0 < t_d/S_d < 0.02$

2.2.3 AASHTO (2004)

a) R_s (Carter and Kulhawy (1988))

$$f_s = 0.15q_u \quad (q_u \le 1.9MPa) \qquad (9\text{-a})$$

$$f_s = 0.21\sqrt{q_u} \quad (q_u > 1.9MPa) \tag{9-b}$$

b) R_B is same as equation of 1996

2.2.4 Bridge Design manual (substructure (2002))
: is using same as FHWA (1988) and AASHTO (1999)

2.2.5 Structure Foundation Design manual (2003)

a) R_B

$$q_{\max} = (\frac{1}{5} \sim \frac{1}{8})q_u \tag{10}$$

Table 3. Suggested $q_{\max}$ (6)

	Empirical Equation
Coates (1967)	$q_{\max} = 3q_u$
Terg (1962)	$q_{\max} = (5-8)q_u$
Rowe & Armitage (1987)	$q_{\max} = 2.5q_u$
Zhang & Einstein (1998)	$q_{\max} = 4.83\sqrt{q_u}(MPa)$

b) R_S

$$f_S = (2.3-3)\sqrt{f_c'} \quad (B>0.4m) \tag{11-a}$$

$$f_S = (3 \sim 4)\sqrt{f_c'} \quad (B<0.4m) \tag{11-b}$$

where, f_c' is smaller of f_c' and q_u (unit psi)

3. FILED LOADING TEST

One static load test (Namhang bridge in Pusan port) and two bi-directional load test results (Machang bridge and second Incheon bridge) are analyzed for load-settlement curve, t-z, and q-w curve in rock-sockets.

3.1 Static loading test (Namhang Bridge in Pusan Port)

1500mm diameter of drilled shaft was rock-socketed into the severely weathered rock 22 meter out of 44 meter total length. SPT N number of weathered rock layer was more than 50/2. Maximum 3000 ton of static loading was applied on the top of drilled shaft using 26 reaction anchors. Average compression strength of concrete was 36.1 MPa.

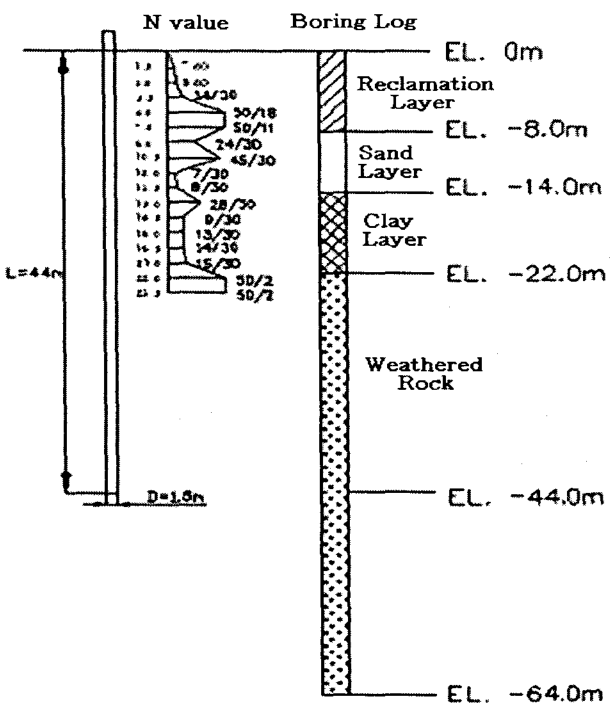

Fig. 6. Subsurface condition of Namhang bridge (6)

Table 4. Drilled shaft condition in Namhang bridge

	Length	Type	Diameter	Maximun loading
Namhang bridge	44m	Weathered rock-socketed	1.5m	3,000 ton

Load- Settlement curve as shown in Fig. 7 did not show any yielding or failure behavior on 3000 ton static loading. 18 mm of total settlement occurred and elastic compression of shaft was 14 mm. 84.6 percent of loading was resisted by side resistance and only 15.4 percent by base resistance.

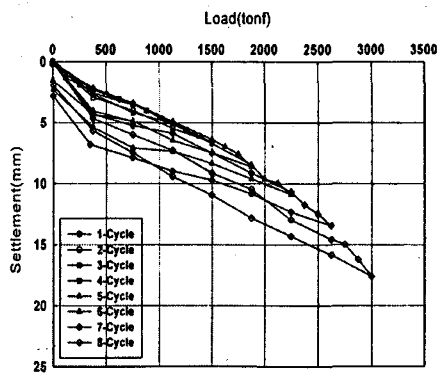

Fig. 7. Load-settlement curve of Namhang bridge (6)

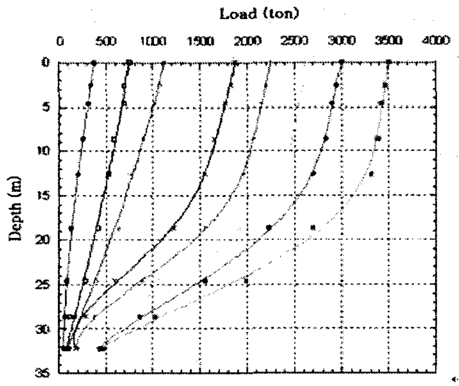

Fig. 8. Load transfer curve of Namhang bridge(6)

t-z curve of Namhang bridge is shown in Fig. 9. The range of unit side resistance in weathered rock layer was 0.16 to 0.3 MPa (from lower to upper part of layer) with 18 mm settlement, which is more than 1 percent of diameter. Also t-z curve did not show yielding behavior on the more than 1 percent of diameter settlement.

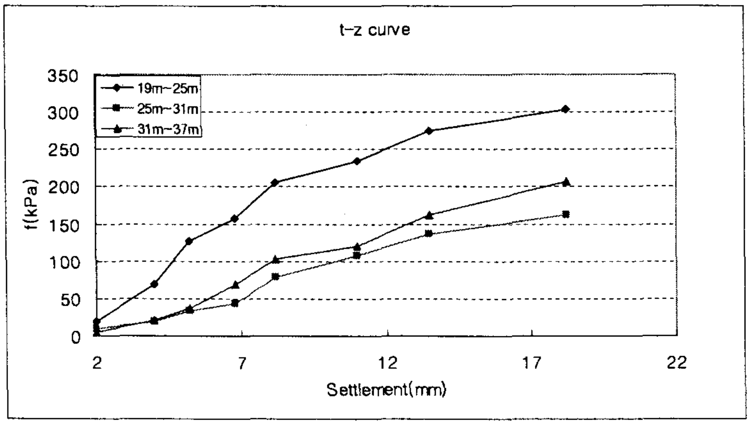

Fig. 9. t-z curve in Namhang bridge

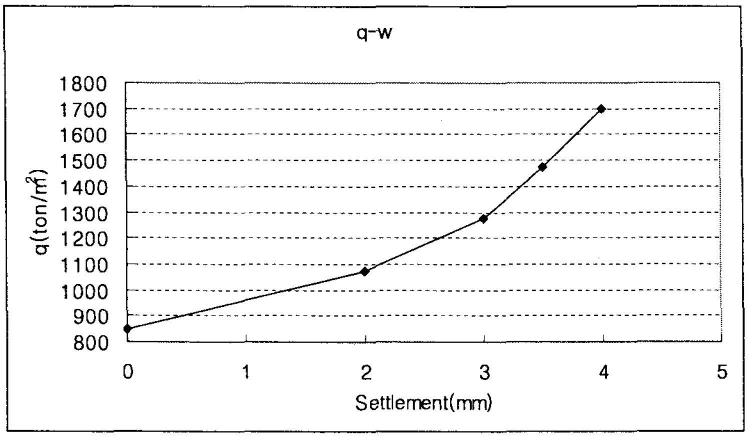

Fig. 10. q-w curve in Namhang bridge

3.2 Bidirectional loading test

Two sites were selected for bidirectional loading test. In Machang bridge site, one hydraulic jacking system, installed on the base of shaft, was used and 2350 ton of loading was applied. 1.5 meter diameter shaft was socketed in rockmass layer 16 m out of total 50.4 m in length. Base of shaft was socketed into soft rock layer, in which average RQD was 13 and unconfined compression of rock core 26.9 MPa.

In Incheon bridge site, two hydraulic jacking systems were used one in the base of shaft, the other in between weathered rock and soft rock layer as shown in Fig. 12. 2.35 meter diameter shaft was socketed in rockmass layer 19 m in length and upper 18 m was reclaimed by earth in stead of concrete. To investigate maximum side resistance of each rock layer, multi-level loading system was applied as shown in Table 5. The range of SPT number was 8-11 out of 100 blows in weathered rock layer. In soft rock layer, rock quality designation value was zero but the range total core ratio 77 to 81. The range of unconfined compression strength of rock core was 15 to 19.4 MPa.

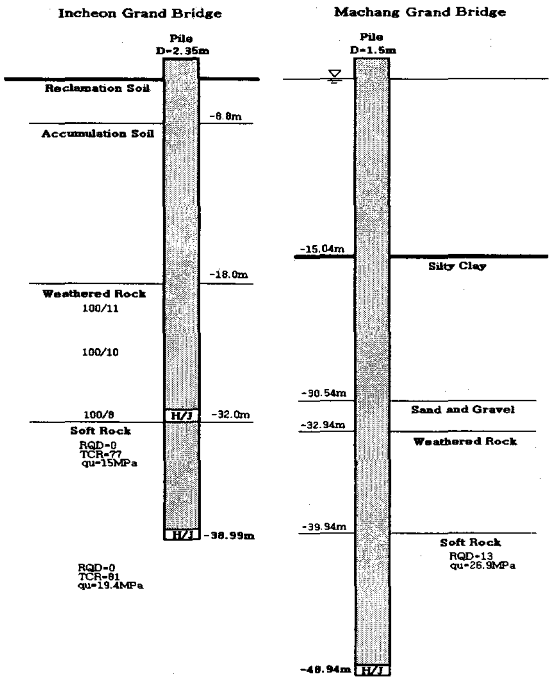

Fig. 11. Subsurface conditions of Machang and Incheon bridge(5, 6)

Table 5. Multi-level loading result for Incheon bridge (5)

Stage	Loading method	Upper Jack max. Displacement(mm)		Lower Jack max. Displacement(mm)	
		Top	Bottom	Top	Bottom
1	Upper Jack-open/close Lower Jack-8500 ton			3.57	24.76
2	Upper Jack-10,750 ton Lower Jack-open	103.55	22.51		
3	Upper Jack Lower Jack		27.63	27.15	48.35

3.2.1 Behavior of Machang bridge

In case of Machang bridge, upper plate moved up 5.64 mm while lower plate moved down 32.47 mm on 2350 ton loading. With these data t-z curve and q-w curve were obtained. The range of unit side resistance of soft rock layer was 0.2 to 1.5 MPa, while 0.1 to 0.2 MPa for weathered rock. This result would be different if static loading were applied on the top of drilled shaft but t-z curve in soft rock layer can be accepted. Unit side resistance near the base of shaft (15 m from the top of rock socket) was more than 1.4 MPa which is similar with Machang bridge site. Maximum unit base resistance showed 13.3 MPa in 32.47 mm (2.16 percent of diameter) settlement.

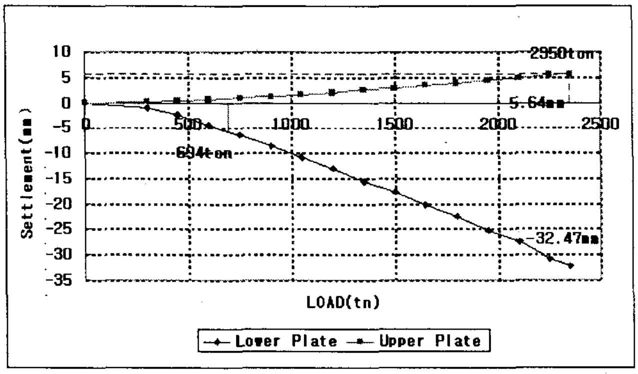

Fig. 12. Load-displacement curve of Machang bridge(3)

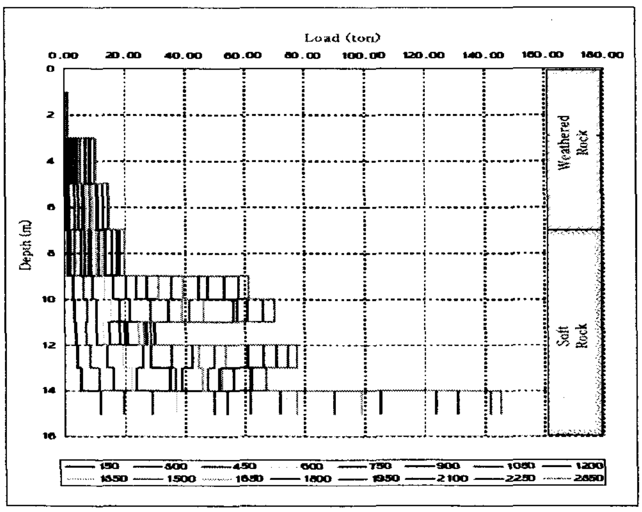

Fig. 13. Side resistance distribution of Machang bridge(3)

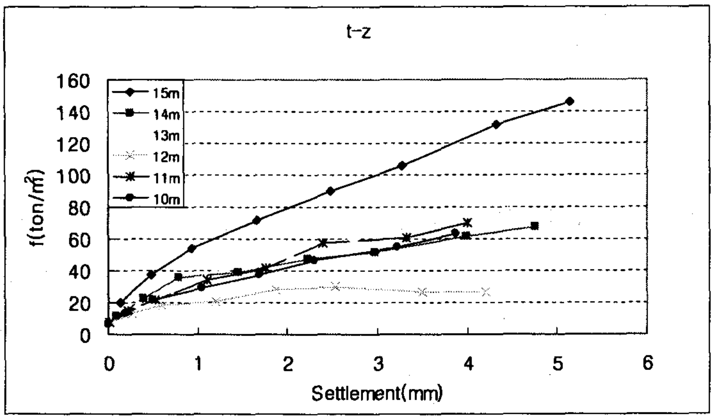

Fig. 14. t-z curve in soft rock of Machang bridge

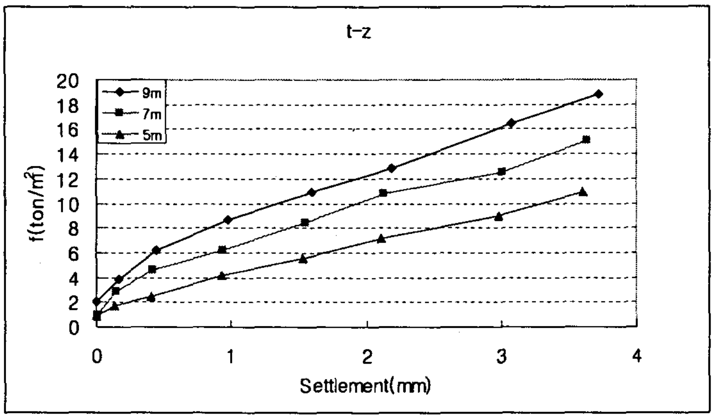

Fig. 15. t-z curve in weathered rock of Machang bridge

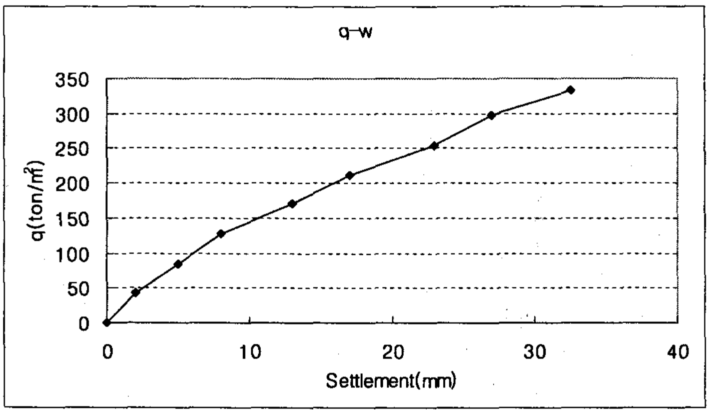

Fig. 16. q-w curve in Machang bridge

3.2.2 Behavior of Incheon bridge

Multi-level loading system was applied in Incheon bridge site to get maximum side resistance of each rock layer. First lower jacking system was pressurized with upper jacking system closed. On second stage upper jacking system was pressurized with lower jacking system open. Loading procedure and maximum displacement are shown in Table 5. The range of unit side resistance for weathered rock layer was 0.5 to 2.5 MPa with 105 mm (4.5 percent of diameter) settlement, while 1.2 to 2.5 MPa for soft rock with 23 mm (1 percent of diameter) settlement. Maximum unit base resistance showed 18 MPa with 17.5 mm (0.8 percent of diameter) settlement.

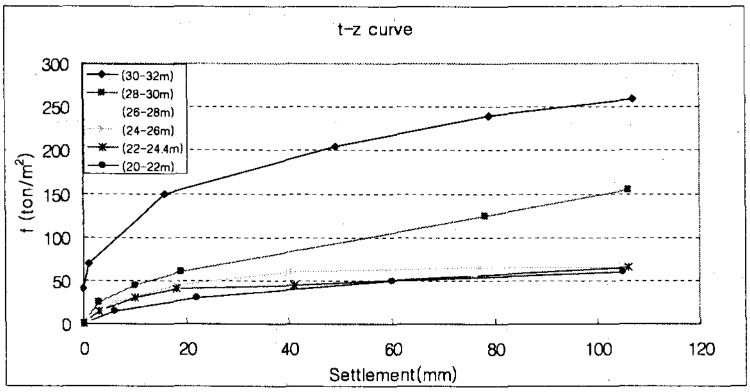

Fig. 17. t-z curve in weathered rock of Incheon bridge(5)

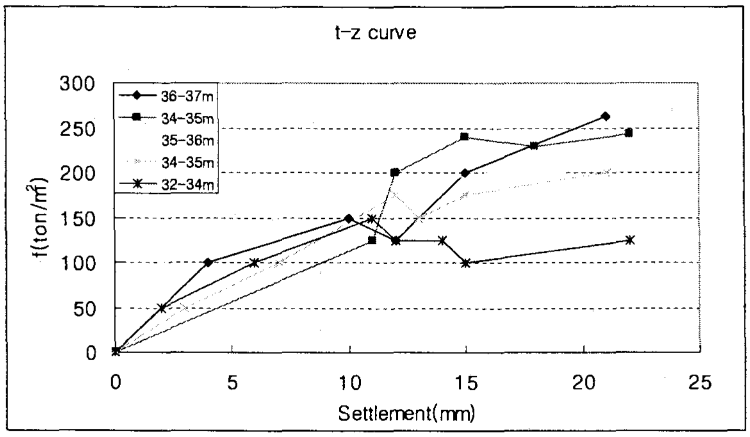

Fig. 18. t-z curve in soft rock of Incheon bridge(5)

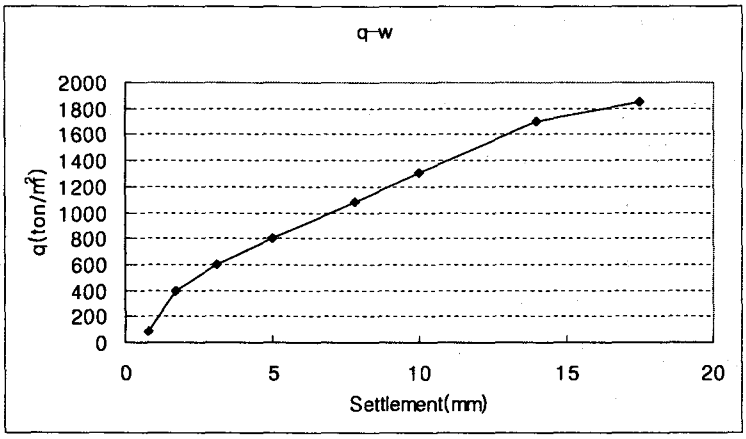

Fig. 19. q-w curve in soft rock of Incheon bridge(5)

3.3 Discussion and Conclusions

In designing rock-socketed drilled shaft, bearing capacity evaluation is very important because the maximum values of base and side resistance are not generally mobilized at the same value of displacement. AASHTO suggested axial compression load is carried solely by the side resistance on a shaft socketed into rock until a total shaft settlement on the order of 10 mm. FHWA suggested that total resistance could be different on condition that if surrounding rock mass were ductile or brittle. Ooi and Carter (1987) suggested that maximum side resistance occurs around 1 mm, after then shows deflection softening (Fig. 20). Fig. 21 shows the normalized side resistance transfer for drilled shaft in cohesionless soil.

Average t-z curve in domestic rockmass layer (Fig. 22) shows unit side resistance increase with more than one percent of diameter displacement and did not show any deflection softening. Especially in case of weathered rock layer of Incheon bridge, more than 4 percent of diameter of displacement occurred, which mean very ductile. Unit side resistances of soft rock layer are generally larger than those of weathered rock. Average unit side resistances of soft rock layer are about three time larger than those of weathered rock layer with same displacement (one percent of diameter), The range of unit side resistance of rock mass was 2 MPa (weathered rock layer of Namhang) to 22 MPa (soft rock layer of Incheon).

The range of unit base resistance was 133MPa (Machang) to 185 MPa (Incheon). Also unit base resistance increases with more than two percent of diameter settlement.

So the total resistance could be the summation of side resistance and base resistance in domestic weathered rock and soft rock layer.

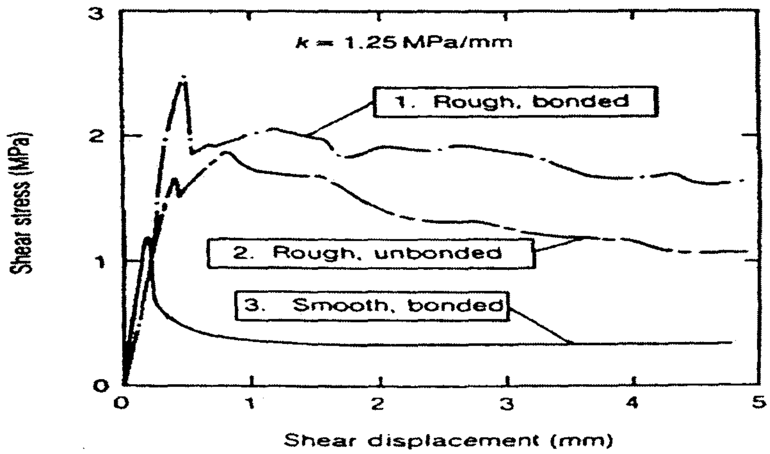

Fig. 20. Relationship between shear stress and shear displacement in Rockmass (Ooi and Carter, 1987)

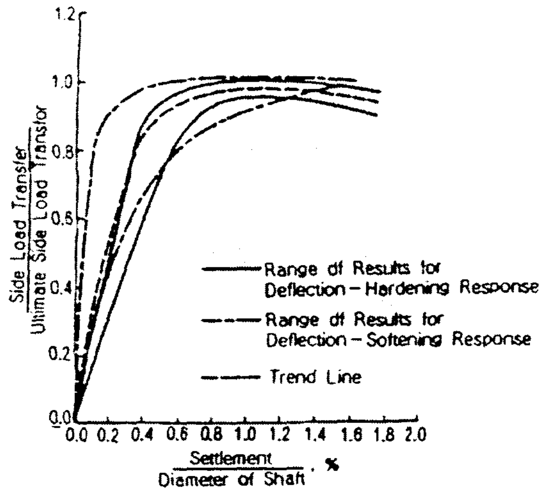

Fig. 21. Normalized side load transfer for drilled shaft in soil (Reese and O'Neill, 1988)

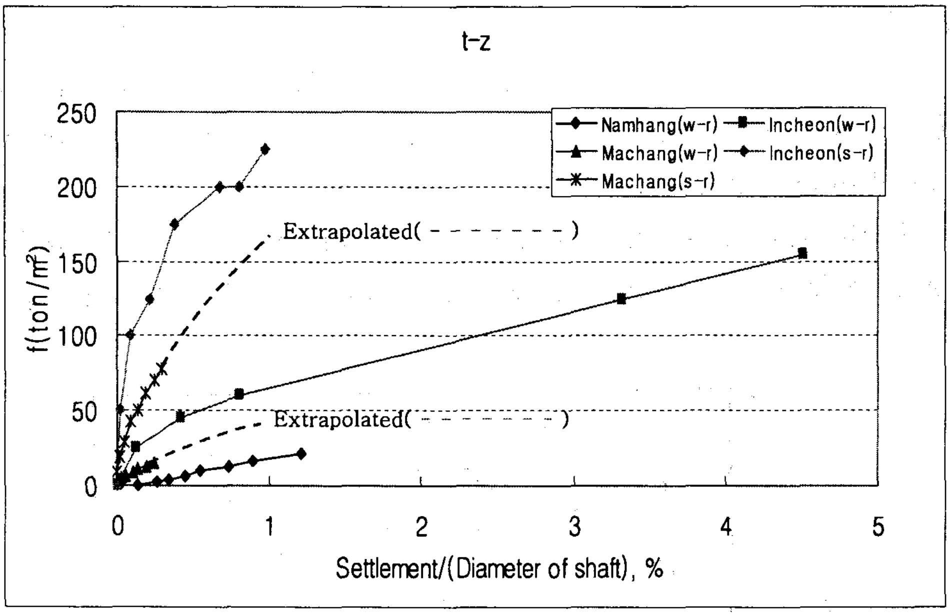

Fig. 22. Average t-z curve in weathered and soft rock layer

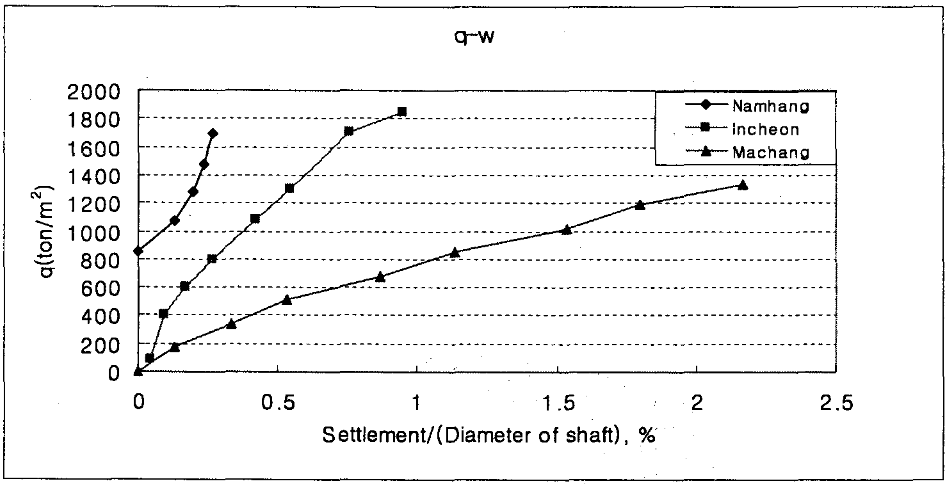

Fig. 23. q-w curve in weathered and soft rock layer

Referrences

1. 대한토목학회(1996), "도로교표준시방서", 제IV편 하부구조편, 제 9장, pp.207~314.
2. 대한토목학회(2001), "도로교설계기준해설(하부구조 편)", 건설정보사, pp. 680~734.
3. (주)백경지앤씨(2005), "마창대교 건설공사 현장타설말뚝 재하시험 보고서".
4. (주)백경지앤씨(2006), "부산신항 진입철도 건설공사 현장타설말뚝 재하시험 보고서".
5. 부경대학교 공학연구원 산업과학기술연구소(2003), "남항대교 건설공사 현장타설말뚝 재하시험 보고서".
6. 한국도로공사 인천대교건설사업소 (2006), "인천대교 건설공사 국고구간 현장타설말뚝 지지력 평가 기준(안)".
7. 한국지반공학회(2002), "구조물 기초 설계기준", 제 4장, pp.174~280.
8. Canadian Geotechnical Society(1992), "Canadian Foundation Engineering Manual", 3nd Edition, Canadian Geotechnical Society Technical Committee on Foundations, Ottawa.
9. Cater, J. P. and Kulhawy, F. H.(1987), "Analysis and Design of Drilled Shaft Foundations Socketed into Rock", Research Report 1493-4, Geotechnical Engineering Group, Cornell University, Ithaca, New York, January.
10. Horvath, R. G. and Kenny, T. C. (1979), "Shaft Resistance of Rock-Socketed Drilled Piers", in Proceedings, Symposium on Deep Foundations (Fuller, ed.), ASCE, Atlanta, October, pp. 182~214.
11. Nam, M. S. (2004), "Improved Design for Drilled Shafts in Rock", PhD Dissertation, University of Houston, Houston, Tx..
12. O'Neill, M. W. and Reese, L. C. (1999), "Drilled Shafts : Construction Procedures and Design Methods", FHWA-IF-99-025, Federal Highway Administration, August.
13. Osterberg, J. O. and Gill, S. A.(1973), "Load transfer mechanisms for piers socketed in hard soils or rock", Proc. 9th Canadian Sym. on Rock Mech., Montreal, pp. 235~262.
14. Reese, L. C. and O'Neill, M. W. (1988), "Drilled Shafts Student Workbook", NHI Course No. 13214, Federal Highway Administration, August.
15. O'Neill, M. W. and Reese, L. C. (1999), "Drilled Shafts: Construction Procedures and Design Methods", Publication No. FHWA-IF-99-025, Federal Highway Administration, Washington, D. C.