RELATION BETWEEN STANDERD PENETRATION TEST AND SKIN RESISTANCE OF DRIVEN CONCRETE PILE IN OVER-CONSOLIDATED CLAY SOIL

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ABSTRAC

In this research the relation between skin resistances and standard penetration test of over consolidated clay soils has been studied. The research includes doing boreholes at Babil governorate in Iraq to get undisturbed samples and standard penetration test. Determination skin friction from direct shear test between smooth concrete and soil was explored in laboratory for design purposes and correlated with standard penetration test values. In many foundation design problems, the shear strength between soil and foundation materials were estimated or correlated without any direct methods for measurement.

Twelve strain controlled direct shear tests were performed simulate the shear strength interaction between smooth concrete and undisturbed over consolidated silty clay, determine the soil – foundation interface friction, considering the following variables :(1) over consolidation ratio OCR between 1.4 to 2.4 (2) Concrete, smooth surface, (3) Undisturbed samples, (4) Variation of the normal load between the friction surface. The results showed that both cohesion and internal friction should be considered in evaluating skin friction. The results of cohesion and angle of internal friction were correlate with the standard penetration test SPT –N . Interface friction angle was 14.5° , while the adhesion was 15.5 kPa. The ultimate shear strength was mobilized through 10%- 16% strain in the direction of shear surface. .

A fairly good correlation between the N_{70}-value and the interface friction parameters were established for determination unit skin friction for driven piles. Finally, based on the test results, a simple relation was proposed to relate the N_{70} value and interface friction of silty clay soils for a range of N_{70} between 12 and 20.

Keywords: undisturbed, O.C., silty clay، soil-concrete skin friction, direct shear test, SPT.

ألخلاصه

تضمن البحث إيجاد علاقة بين مقاومة الاحتكاك وقيمة ضربات فحص الاختراق القياسي لتربة طينية مفرطة الانضمام.حيث تم عمل حفر أختبارية لأستخراج نماذج تربة غير مشوشة من محافظة بابل –العراق وفحص الاختراق القياسي . حسب احتكاك التماس من تجارب القص المباشر بين الخرسانة (مواد الأساس)وتربه طينية لإيجاد علاقة مع قيمة ضربات فحص الاختراق القياسي N_{70} لأغراض تصميمية . غالباً ما تخمن معاملات مقاومة القص بين التربة ومواد الأسس في معظم مشاكل تصميم بدون طرق مباشِره للحساب.

أجري في هذا البحث اثنا عشر اختبار لمقاومه القص المباشر مسيطر الانفعال أخذين بنظر الاعتبار المتغيرات والظروف التالية: (١) تربه طينية عرينيه مفرطة الانضمام (نسبة الانضمام الأسبق OCR بين ١,٤-٢,٤) (٢) خرسانة ناعمة السطح (٣) نماذج غير مشوشة (٤) تغيّر الحمل العمودي على سطح التماس . عكست الاختبارات بشكل اعتيادي معاملات القص كما في تجارب التربة الطبيعية (التلاصق وزاوية الاحتكاك الداخلي)، كانت زاوية احتكاك التماس °١٤،٥ وبينما كان الالتصاق ١٥،٥kPa ، تأهبه مقاومة القص العظمى للتماس من ١٠ - ١٦% انفعال أزاحه باتجاه سطح القص.

تم استخلاص علاقة ملائمة بين قيمة ضربات فحص الاختراق القياسي -N_{70} ومعاملات احتكاك الالتصاق لحساب احتكاك التماس لركائز الدق،أخيرا وبالاعتماد على نتائج الفحص تم استخلاص علاقة بين احتكاك التماس ضربات فحص الأختراق القياسيN_{70} لتربة طينية غريني ، لقيم ضربات فحص الأختراق القياسي N_{70} بين (١٢-٢٠).

الكلمات ألمفتاحيه:غير مخلخلة،مفرطة الانضمام،طين غريني، احتكاك التماس التربة – الخرسانة، ،فحص القص المباشر،فحص القياسي .

INTRODUCTION

A relation between SPT and skin friction have no direct relationships .Recent developments in civil engineering, especially in soil mechanics and foundations engineering , take a great step forward from "design by experience" to design by a well-established theory verified by experiment(Site and lab Test). In the case of skin friction(between soil and foundation) , is the stress-strain relation if one starts to move in relation to another? This mutual effect of soils and structures in the transmission of the forces through the contact surface is called skin friction.

Until recently the values of skin friction used for design purposes were the average values obtained by field tests, with only qualitative reference to such factors influencing their magnitude as type of soil, type of construction material, and surface finish, moisture content of the soil, etc. The modern trend is to establish skin shear strength coefficients throughout laboratory experiments in which the factors influencing the results may be controlled quantitatively (Potyondy, 1961).

The soil parameters needed for static analysis of single and group piles capacity consist of the angle of internal friction(ϕ)and the cohesion (c). The strength parameters have been determined from laboratory triaxial tests on undisturbed samples with experience used to extrapolate this data to obtain the design parameters. Also , used in situ parameters of cone penetration test or pressure meter test and probably most pile design still relies heavily on standard penetration test N values in sand and field tests for shear strength in cohesive soil deposits (Bowles, 1997).

In the field of geotechnical engineering, it's well known that the designs of piles foundation depend upon end bearing and /or skin friction between the piles and soil. When a load is applied to the soil surface , the soil resists the applied loads by developing contact forces wherever they touch at their asperities. At each contact, the particles respond by deforming in three ways: compressing, bending, and sliding. Deformation due to sliding is usually the most significant and is non linear and irreversible, making the load – deformation behavior of soil non linear and irreversible as well (Lambe and Whitman, 2000).

There are various ways to determine the pile capacity and most of them rely on full scale field tests using full size piles, but such tests are expensive and the results may apply only to the site where the test was performed. The value of skin friction factor to use in determining the load capacity of piles is a subject of much debate and testing (Budhu, 2008).

Chandler, and Martins(1982) doing Load tests on a modal pile installed in speswhite kaolin, based on tests on nine normally consolidated and one over consolidated sample, show that the angle of shaft friction is independent of the stress ratio in the soil before loading and is only just less than the triaxial effective angle of shearing resistance of soil for normally and over consolidated kaolin. When loaded axially the lateral stresses on the pile shaft decrease with increasing shaft load for normally consolidated soil, and increase on loading in over consolidated soil

Many geotechnical problems involve estimation of stresses transferred along the interface between soils and solid surfaces. Considerable work have been done on the interfacial friction between cohesion-less soils (sands) and solid surfaces. The interfacial shear resistance between fine grained soils and solid surfaces depends on whether its mobilization takes place in drained or in undrained condition. Also most of these studies are on normally consolidated soils, the influence of over-consolidated soils has received little attention (Acar et. al 1982, Ampera and Aydogmus, 2005)

Several kinds of apparatus have been used to investigate the interfacial friction between fine-grained soil and solid surfaces, for example the direct shear apparatus and the simple shear apparatus. Model pile tests have also been used for this purpose.

The results may be valuable and provide some rationale for reported correlations between N and skin friction from piles.

The objective of this study is correlate the evaluate results of adhesive and angle of internal friction skin friction using direct shear test box with the standard penetration test results on undisturbed over consolidated silty clay soil .

TEST PROGRAM

The test program consist of doing boreholes at Babil governorate in Iraq to get undisturbed samples and in-situ standard penetration test. Conducted classification test and determination skin friction from direct shear test between smooth concrete and native soil was explored in laboratory for design purposes and correlated with Standard penetration test blows values .Conducted 12 direct shear box tests on specimens of over consolidated cohesive silty clay soil and smooth concrete slice (foundation materials). The soil was placed in the bottom part of the direct shear box test and the concrete was placed above it (in the upper part of the box) as shown in **Fig.1**. Test series (S1 – S5) were performed on five undisturbed cohesive silty clay soils, a total of twelve Direct Shear Tests were carried out , **Table 1** shows the details of the test series performed.

These 12 tests were conducted in such a way that in the first six tests the rate of strain was higher than the second group, **Table 2** shows the details values of shear strength and interface friction parameters of the test series performed.

The shear strength along the surface of contact of the soil and the foundation are given by Coulomb equation (Ampera and Aydogmus, 2005) as in eq. (1) below :

$$InterfaceFriction = f_c c + \sigma \tan(f_\phi \phi) \qquad (1)$$

Where $f_c = \dfrac{c_a}{c}$ (mean= 0.25)

$f_\phi = \dfrac{\delta}{\phi}$, (mean= 4.4)

σ′ = Effective normal stress.
c = Cohesion of soil
C_a = Adhesive bet. Soil and clay
ϕ = Angle of internal friction of soil
δ = Angle Interface friction

It's worth to mention that all soil tests were carried out in accordance with (ASTM standards). For the determination of physical properties of soil well-known standard equipment was used. It is worth to mention that laboratory , field ,and in situ tests of the study conducted in 2007 .

SAMPLE PREPARATION FOR ITERFACE TEST

The soil part of the specimen was undisturbed cohesive silty clay soil extracted directly from Shelby tube to prepare the test sample, for the direct shear test, (Five undisturbed samples were obtained from various locations and different depths . The boring equipment used in carrying out the field work was rotary drills rigs, with thin wall tube samplers Shelby tube for taking undisturbed samples .Disturbed samples were obtained to determine the classification of soils, the samples that were secured by the Standard Split Spoon Sampler were also used as disturbed samples. The water table was found at the time of boring be 3-4 m deep) using extruder sampler (6 × 6× 2 cm) specified for direct shear test as shown in **Fig. 1 and Fig.2** . The other part was a slice of concrete cube which was cast using job mix (1:1.5:3) and cured for 28 days. Then the soil pushed to the bottom half of the Direct Shear Box before tighten the two halves of box. Later the concrete slice (foundation material) was put in the upper half of the shear box . Finally the test was conducted in the usual manner (Das, 2002, and Lambe and Whitman, 2000).

Slices of concrete cubes made to fit the Shear Box device dimension by making projection of 6 mm in the direction of applied shear and less than 1 mm in the opposite direction. Note that the soils are denoted by series symbols (S1 – S5).

ANGLE OF ITERFACE FRICTION δ

The angle of wall friction δ can be estimated from **Table 3** or directly measured for important projects. Any direct measurements between the soil and wall material should use pressure that is on the order of what is expected in the prototype, since δ is some what pressure dependent.

If ϕ < δ, you assume a frictionless interface (but there may be adhesion, since a

ϕ<δ soil would have cohesion).Interface friction apparently depends not only on the soil properties but also on the amount and direction of foundation movement. Indications are that maximum wall friction may not occur simultaneously with maximum shearing resistance along the rupture surface and that wall friction is not a constant along the wall—probably because the relative soil-wall movement is not constant as shown in **Fig.3**.

Considerable engineering judgment must be applied to obtain realistic values of wall friction since they are pressure-dependent. Values of δ = 0.6ϕ to 0.8ϕ are reasonable for concrete walls where forms are used giving a relatively smooth back face. **Table 3** gives several values of δ for other wall-to-soil materials (Bowels, 1997) .

For steel, concrete, and wood the values shown are for a normal pressure an of about 100 kPa. Decrease the values about 2 degrees for each 100 kPa increase in sand (Acar et al. ,1982).

ITERFACE ADHESION C_a

Interface adhesion develops from any

cohesion in the soil. In the upper region it is expected a tension crack may form (or form during dry periods as the ground naturally shrinks). The value of adhesion C_a below the tension crack depth is usually taken at from 0.5 to 0.7 s_u with a maximum value not much over 50 kPa (Bowels, 1997).

THE STANDARD PENETRATION TEST (SPT)

The standard penetration test, developed around 1927, is currently the most popular and economical means to obtain subsurface information (both on land and offshore). It is estimated that most conventional foundation design in world is made using the SPT. The method has been standardized as ASTM D 1586 since 1958 with periodic revisions to date. The test consists of the following: (ASTM D 1586-99)

1. Driving the standard split-barrel sampler a distance of 460 mm into the soil at the bottom of the boring.
2. Counting the number of blows to drive the sampler the last two 150 mm distances (total = 305 mm= 1 ft) to obtain the N number.
3. using a 63.5-kg driving mass (or hammer) falling "free" from a height of 760 mm. Several hammer configurations are available.

The exposed drill rod is referenced with three chalk marks 150 mm apart, and the guide rod is marked at 760 mm (for manual hammers).

The assemblage is then seated on the soil in the borehole (after cleaning it of loose cuttings). Next the sampler is driven a distance of 150 mm to seat it on undisturbed soil, with this blow count being recorded (unless the system mass sinks the sampler so no N can be counted). The sum of the blow counts for the next two 150-mm increments is used as the penetration count N unless the last increment cannot be completed. In this case the sum of the first two 150-mm penetrations is recorded as N.

The boring log shows refusal and the test is halted if

1. 50 blows are required for any 150-mm increment.
2. 100 blows are obtained (to drive the required 300 mm).
3. 10 successive blows produce no advance.

When the full test depth cannot be obtained, the boring log will show a ratio as

70/100 or 50/100

Indicating, that 70 (or 50) blows resulted in a penetration of 100 mm. Excessive equipment wear, as well as greatly reduced daily drilling meterage, results when blow counts are high. Standardization of refusal at 100 blows allows all drilling organizations to standardize costs so that higher blow counts result in a negotiation for a higher cost/length of boring or a requirement for some type of coring operation.

The standard blow count N_{70} can be computed from the measured N as follows:

$$N_{70} = C_N \times N \times \eta_1 \times \eta_2 \times \eta_3 \times \eta_4 \qquad (2)$$

Where

η_i = adjustment factors from (bowels, 1997)

N_{70} = adjusted N

C_N = adjustment for effective overburden pressure p'_o (kPa) computed, $C_N = \left(\dfrac{95.76}{p'_\circ}\right)^{1/2}$

Meyerhof (1976) produced correlations between base and frictional resistances and **N**-values. It is recommended that **N**-values first be normalized with respect to effective overburden stress:

Normalized **N** = $\mathbf{N_{mea.}}$ × 0.77 log (1920/σ′$_v$) (3)

the influence of over- consolidated soils has received little attention

Pile type	Soil type	Ultimate base resistance q_b (kPa)	Ultimate shaft resistance q_s (kPa)
Driven	Gravelly sand Sand	40(L/d) **N** but < 400 **N**	2 $\mathbf{N_{avg}}$
	Sandy silt Silt	20(L/d) **N** but < 300 **N**	
Bored	Gravel and sands	13(L/d) **N** but < 300 **N**	$\mathbf{N_{avg}}$
	Sandy silt Silt	13(L/d) **N** but < 300 **N**	

DIRECT SHEAR TEST

The direct shear test is the oldest and simplest form of shear test arrangement. The test equipment consists of a metal shear box in which the soil specimen is placed. The box is split horizontally into halves. Normal force on the specimen is applied from the top of the shear box. Shear force is applied by moving one –half of the box relative to the other to cause failure in the soil specimen. A conventional strain controlled direct shear box machine with specimen dimensions of (6×6×2 cm) was used. Series of shear strength test on 12 samples were conducted in such a way that the soil is placed in the bottom part of the Direct Shear Box device and the concrete (foundation material) is placed above it, i.e. in the top part of the shear box. The test were carried out at two constant rate of strain 1.2 mm/min and 0.3 mm/min. to make two different condition.

The tests were carried out in soaked condition using normal pressure ranging from 26.2 kPa to 349 kPa, as illustrated in **Table 2**.

Typical results of sample number five are shown in **Fig. 4**. and **Fig. 5** shows typical strain-shear stress for over consolidated clay.

RESULTS AND DISCUSSION

The soil was classified as over consolidated silty clay soil of firm to stiff strength $s_u \cong 30 - 60kPa$.

Fig. 6 show the results of direct shear test between an over consolidated cohesive soil and smooth concrete (foundation material) in soaked conditions at a stain rate of 1.2 mm/min.

The results showed that both cohesion and internal friction should be considered in evaluating skin friction , as illustrated in **Table 4**. The ultimate shear strength was mobilized at about 10 % strain of sample dimension. From the results shown in **Fig. 7** the following best relationship was obtained according to R^2:

Linear equation(kPa)

$$C_a = 0.2802(\sigma_n) + 14.196 \quad (4)$$

By using the linear equation and putting two different normal stresses(σ_n =50 & 100 kPa) to get two adhesion shear stresses(C_a). determine from the slope of the line the angle of interface friction. and by putting zero normal stress we get the adhesion shear stresses for **Fig. 7**.

Fig. 8 and, illustrates the results between shear stress and normal stress for the group B six specimens, the results show a friction angle and adhesion between the tested soil and concrete. The adhesion achieved from the cohesion properties of soil and angle of friction (interface friction) obtained from rough surface of concrete (foundation material). The ultimate shear strength was mobilized at about 10 % strain of sample dimension . From the results shown in **Fig.9** the following best relationship was expressed according to R^2:

Linear equation(kPa)

$$C_a = 0.301(\sigma_n) + 13.72 \quad (5)$$

Comparison of results of **Figs. 6 and 7** with the results shown in **Figs. 8 and 9** indicates that tests at lower rate of strain ,i. e ,0.3 mm/min had increased the adhesion and decreased the angle of internal friction slightly. This is due to the low applied strain that permits the soil to consolidate and to increase the contact area with the concrete face.

Fig.10 shows the scatter of the results graph of all twelve tests. From the results the friction angle and adhesion between the tested soil and concrete can be expressed by best relationship :

Linear equation(kPa)

$$C_a = 0.301(\sigma_n) + 13.72 \quad (6)$$

A list of possible relationships for estimating the interface friction parameters using various equations developed in this study is summarized in **Table 5**. During this study, all possible relationships were tried; however, naturally in some of these relationships the evaluated adhesion stress were low, high or negative value. The equations given in **Table 5** are the ones which had correlation coefficient (R^2> +0.5).

Figs. 11and **12** illustrate the relations among standard penetration test blows value (N_{70}), adhesion ,and interface friction angle. They can be used to relate the N_{70} value and interface friction of silty clay soils for N_{70} between 12 and 20,such relations can be expressed by:

Adhesion, kPa

$$C_a = 1.12 \times N_{70} \quad (7)$$

Interface friction angle, (º)

$$\delta = 0.964 \times N_{70} \quad (8)$$

The shear strength parameters between a soil and a foundation material can be conveniently determined by a direct shear test. Normal pressure is the most effective parameter on the shear between the soil and concrete.

The direct shear test is simple to perform, but it has some inherent shortcoming .The reliability of the results may be questioned because the soil is not allowed to fail along the weakest plane but is forced to fail along the plane of split of the shear. This shortcoming is related to the original test for soil only to determine their strength parameters.

Despite of this shortcoming, this is a great advantage of the direct shear test; where the shear strength between the soil and the foundation material can be obtain during ordinary site investigation for pile foundation construction.

CONCLUSIONS

This work has presented the results of an experimental Laboratory and in-site work on five different soil samples used as underneath soil for a construction site. From the results of this work, the following conclusions can be withdrawn:

1. Interface angle of friction is $\delta^{\circ} \cong 4.4\phi$, and $\delta = 0.964 \times N_{70}$.
2. The adhesion between over consolidated clay and concrete is C_a=0.25 c ,and $C_a,(kPa) = 1.12 \times N_{70}$.
3. In cohesive soils it's preferable to use standard penetration test together with other tests as direct shear test to estimate the pile load capacity or the length of pile proposed.
4. In the case of over consolidated cohesive soils the adhesion and interface friction part should be taken and is mobilized at a 10 % strain into in evaluating interface shear strength.
5. The shape of stress-strain curve of soil concrete interaction is without peak value.

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LIST OF SYMBOLS

C: Cohesion.
C_a: Adhesion between soil and concrete.
d :Shaft diameter.
G_s : Specific gravity
e_o :Void ratio.
L: Embedded length.
L.L: Liquid limit.
N_{70}: Corrected blows of SPT.
N_{avg} : Average value along shaft .

OCR: Overconsolidation ratio= $\frac{P'_c}{P'_\circ}$

P'_c: Maximum preconsolidation pressure.
$P'_\circ$: Effective average geostatic pressure.
P.L: Plastic limit.
P. I: Plasticity Index.
SPT: Standard penetration test (in-situ test).
s_u:Undrained shear strength.
w: Water content %.
Ø: Angle of internal friction.
δ: Interface friction angle between soil and concrete.
τ_s : Interface Shear strength.

Table 1 Properties of the soil used *

Soil Series	Depth (m.)	Sample No.	LL %	PI %	w %	γ_d kN/m^3	G_s	** OCR= $\frac{P'_c}{P'_\circ}$	Consolidation Test				Sieve Analysis % Passing No. 200	Soil Type	Triaxial compression UU-test		SPT N-Value
									e_o	P'_c kPa	c_c	c_r			C kPa	Ø Deg	
S1	2.0-3.0	1, 2	40	18	17.2	15.7	2.69	2.6	0.739	120	0.18	0.031	97.4	CL	95	4.0	14
S2	2.5-3.0	3, 4	55	28	24.6	15.2	2.71	1.72	0.835	90	0.19	0.034	98.3	CH	149	5.5	18
S3	3.0-4.0	5, 6	48	25	27.2	13.9	2.69	1.4	0.851	85	0.16	0.031	96.3	CL	112	1.5	16
S4	2.0-3.0	7, 8, 9	44	21	21.5	15.4	2.68	2.05	0.824	95	0.14	0.026	85.5	CL	56	4.5	12
S5	5.5-6.5	10, 11, 12	43	23	25.9	14.4	2.68	1.4	0.738	125	0.17	0.031	97.0	CL	83	5.5	15

***Adopted from (Ahmad and JahanGer, 2008)**

$$**OCR = \frac{P'_c}{P'_\circ}$$

$$S1 \Rightarrow \sigma'_{v\circ} = P'_\circ = \sum \gamma' \times Z_{a\,var\,ege} = 2.5 \times (15.7 \times 1.172) = 46.001 kPa$$

$$P'_c = 120 kPa$$

$$\Rightarrow OCR = \frac{120}{46} = 2.6$$

$$S5 \Rightarrow \sigma'_{v\circ} = P'_\circ = \sum \gamma' \times Z_{a\,var\,ege} = [4 \times (14.4 \times 1.259) + 2 \times (14.4 \times 1.259 - 9.81)] = 89.11 kPa$$

$$\Rightarrow OCR = \frac{125}{89.11} = 1.4$$

Table 2– Values of shear strength and interface friction parameters

Sample No.	Normal Stress (kPa)			Rate of Strain (mm/min)	Shear Strength		Average Interface coefficients used	
	1	2	3		Adhesion C_a (kPa)	Angle of interface friction δ' (°)	C_a(kPa)	δ'(°)
1	41.9	71.0	129.3	1.20	36.13	14.86	14.5	15.5
2	41.9	71.0	129.3	1.20	13.2	17.43		
3	41.9	71.0	129.3	1.20	16.35	10.5		
4	45	76.4	139	1.20	5.15	17.9		
5	45	76.4	139	1.20	4.0	18.53		
6	45	76.4	139	1.20	5.2	17.6		
7	45	76.4	139	0.30	12.93	16.4		
8	114	192.4	349	0.30	39.85	13.4		
9	114	192.4	349	0.30	21.49	15.10		
10	114	192.4	349	0.30	24.13	15.4		
11	114	192.4	349	0.30	45.8	8.3		
12	26.2	40.8	70	0.30	3.1	13.6		

Table 3 Friction angles between various materials and soil or rock (Bowels, 1997)

Interface materials	Friction angle, δ, degrees†
Mass concrete or masonry on the following:	
Clean sound rock	35°
Clean gravel, gravel-sand mixtures, coarse sand	ϕ
Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	ϕ
Clean fine sand, silty or clayey fine to medium sand	ϕ
Fine sandy silt, nonplastic silt	ϕ
Very stiff and hard residual or preconsolidated clay	ϕ
Medium stiff and stiff clay and silty clay	ϕ
Steel sheet piles against the following:	
Clean gravel, gravel-sand mixture, well-graded rock fill with spalls	22°
Clean sand, silty sand-gravel mixture, single-size hard rock fill	17
Silty sand, gravel, or sand mixed with silt or clay	14
Fine sandy silt, nonplastic silt	11
Formed concrete or concrete sheetpiling against the following:	
Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	22–26
Clean sand, silty sand-gravel mixture, single-size hard rock fill	17–22
Silty sand, gravel, or sand mixed with silt or clay	17
Fine sandy silt, nonplastic silt	14
Various structural materials	
Masonry on masonry, igneous and metamorphic rocks:	
Dressed soft rock on dressed soft rock	35°
Dressed hard rock on dressed soft rock	33
Dressed hard rock on dressed hard rock	29
Masonry on wood (cross grain)	26
Steel on steel at sheet-pile interlocks	17
Wood on soil	14–16‡

*May be stress-dependent (see text) for sand.
†Single values ±2°. Alternate for concrete poured on soil is $\delta = \phi$.
‡May be higher in dense sand or if sand penetrates wood.

Table 4 Values of shear strength and skin friction for silty clay material

Sample No.	c kPa	Adhesion C_a (kPa)	$f_c = \frac{c_a}{c}$	Mean f_c	Ø Deg	Angle of interface friction δ′ (°)	$f_\phi = \frac{\delta}{\phi}$	Mean f_ϕ
1	95	36.13	0.39	0.25	4.0	14.86	3.7	4.4
2	95	13.2	0.13		4.0	17.43	4.2	
3	149	16.35	0.11		5.5	10.5	1.9	
4	149	5.15	0.04		5.5	17.9	3.25	
5	112	4.0	0.04		1.5	18.53	12.3	
6	112	5.2	0.05		1.5	17.6	11.7	
7	56	12.93	0.24		4.5	16.4	3.64	
8	56	39.85	0.72		4.5	13.4	2.9	
9	56	21.49	0.39		4.5	15.10	3.3	
10	83	24.13	0.29		5.5	15.4	2.8	
11	83	45.8	0.55		5.5	8.3	1.5	
12	83	3.1	0.04		5.5	13.6	2.4	

Table 5 Summary of relationships developed to evaluate interface friction parameters

Relations developed to evaluate interface friction parameters			
Equation	**R^2**	**Regression Equation**	**For example σ_n =100 kPa**
Group A			
Linear	0.513	$C_a = 0.280262 * \sigma_n + 14.19$	C_a= 42.19 kPa
Log	0.501	$C_a = 22.6933 * \log(\sigma_n) + -60.42$	C_a= -14.62 kPa(Neglect Eq.)
Exponential	0.549	$C_a = \exp(0.00771 * \sigma_n) * 18.26$	C_a= 39.51 kPa
Power	0.542	$\log(C_a) = 0.628 * \log(\sigma_n) + 0.835$	C_a= 123 kPa(over estimated)
Group B			
Linear	0.884	$C_a = 0.301289 * \sigma_n + 13.72$	C_a= 43.82 kPa
Log	0.914	$C_a = 44.320 * \log(\sigma_n) + -151.69$	C_a= - 63 kPa(Neglect Eq.)
Exponential	0.689	$C_a = \exp(0.00579 * \sigma_n) * 19.42$	C_a= 34.67kPa
Power	0.910	$\log(C_a) = 0.964 * \log(\sigma_n) + -0.759$	C_a=14.75 kPa(under estimated)
Scatter graph, Group A&B			
Linear	0.861	$C_a = 0.302 * \sigma_n + 12.901$	C_a=43.17 kPa
Log	0.833	$C_a = 40.039 * \log(\sigma_n) + -133.12$	C_a= -53kPa(Neglect Eq.)
Exponential	0.678	$C_a = \exp(0.00571 * \sigma_n) * 20.62$	C_a= 36.5kPa
Power	0.822	$\log(C_a) = 0.845 * \log(\sigma_n) + -0.143$	C_a= 35.3kPa

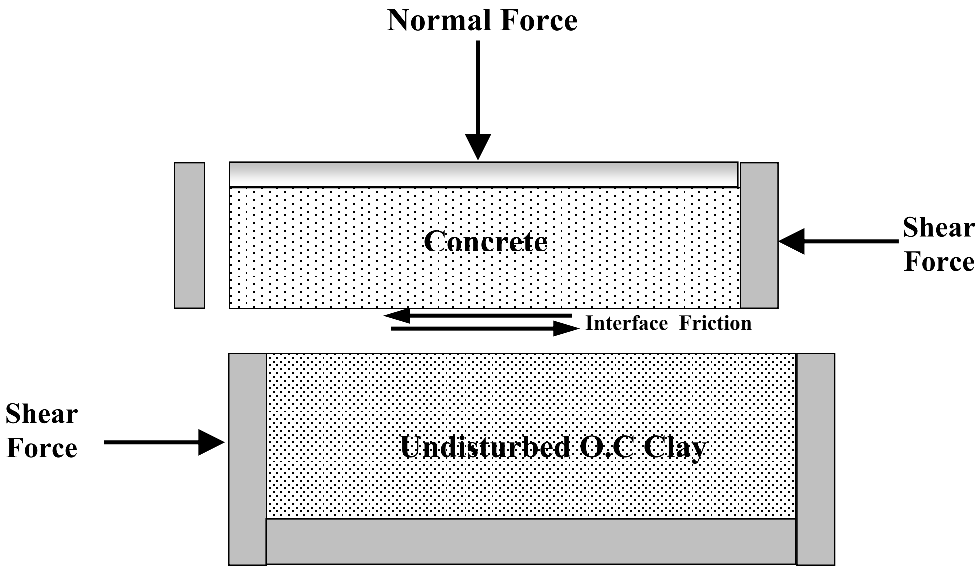

Fig. 1 Schematic diagram of direct shear test During test

a) DST sampler extrude

b) DST box assembly

c) Concrete and Soil Interaction

Fig. 2 Direct shear test   plate

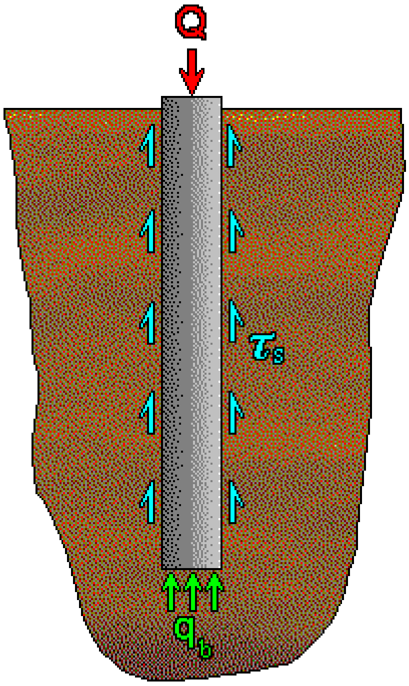

Fig. 3 Schematic diagram of pile load capacity shows interface friction stress

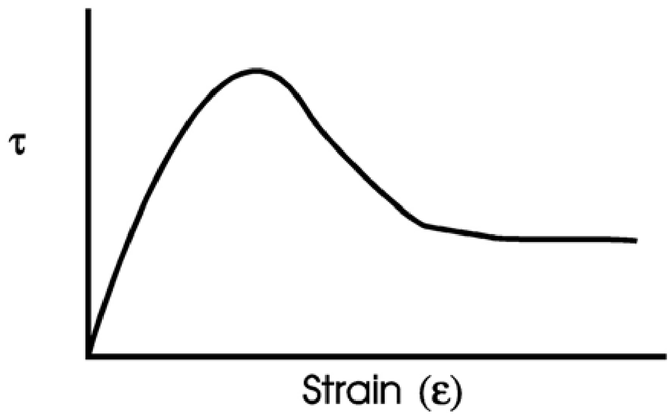

Fig. 4 Strain for over consolidated clay (Winterkorn and Fang, 1975)

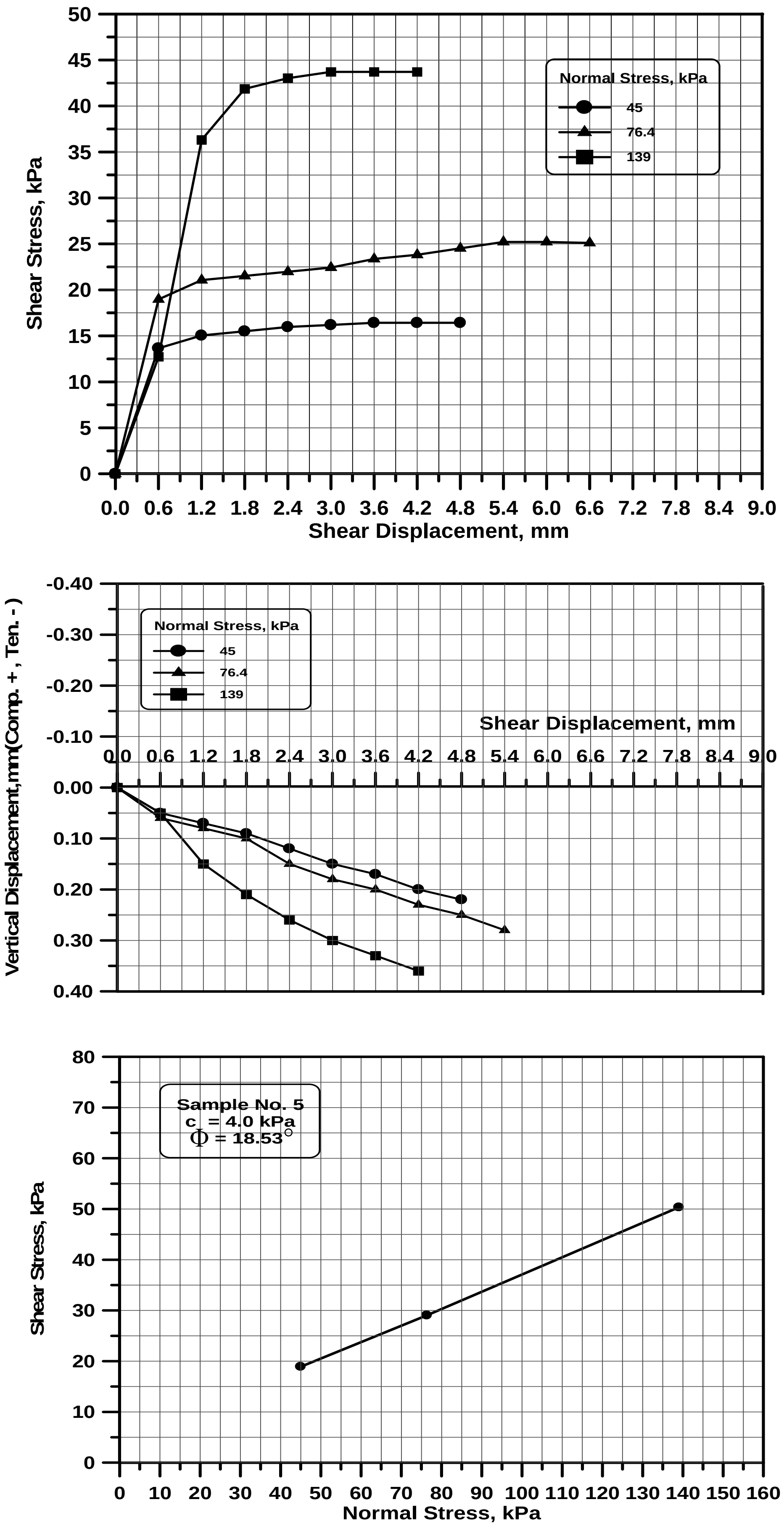

Fig. 5 Results of direct shear test for sample number 5

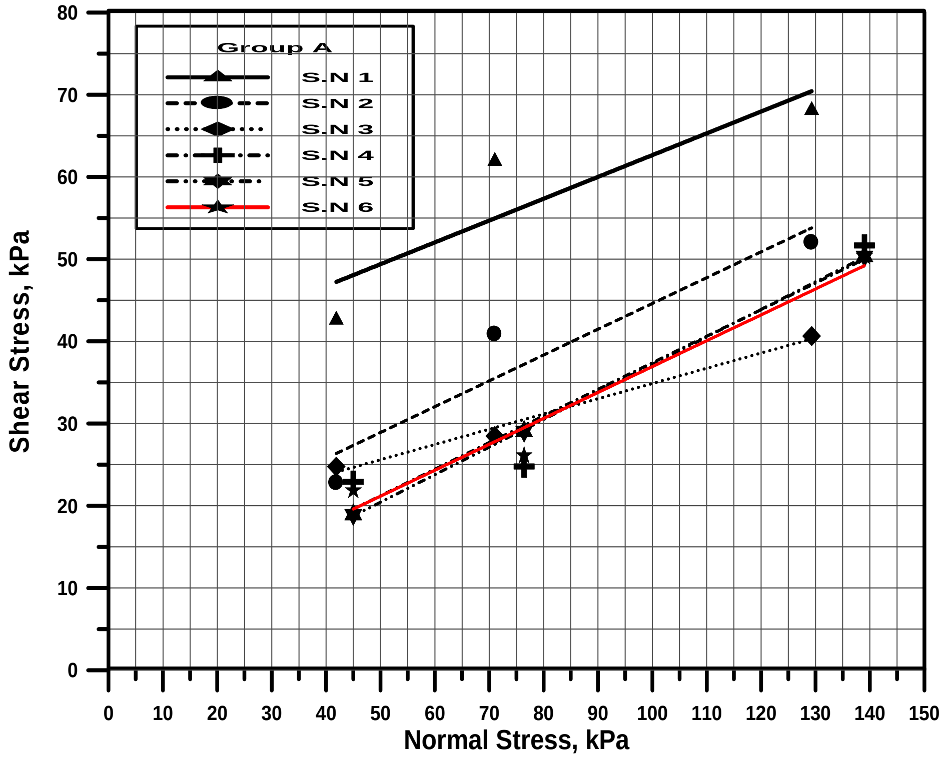

Fig. 6 Results of six direct shear test of O.C clay soil-Precast concrete in undrained condition group A

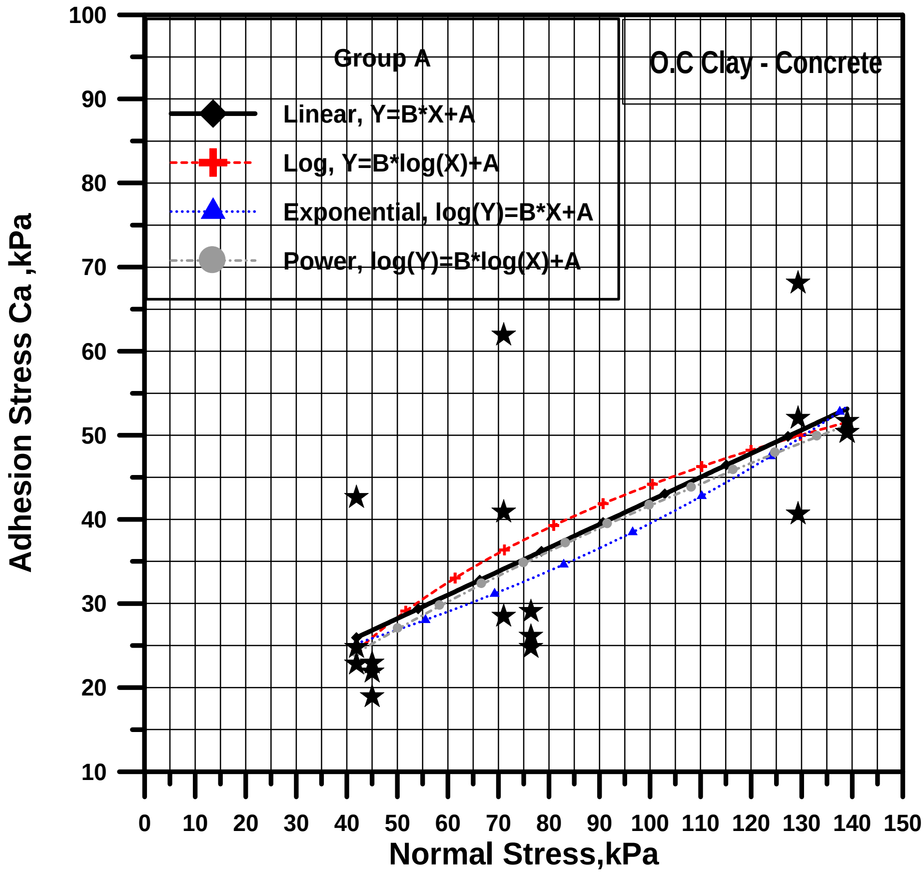

Fig. 7 Determination of skin friction parameters (C_a & δ) of O.C clay – Precast concrete group A

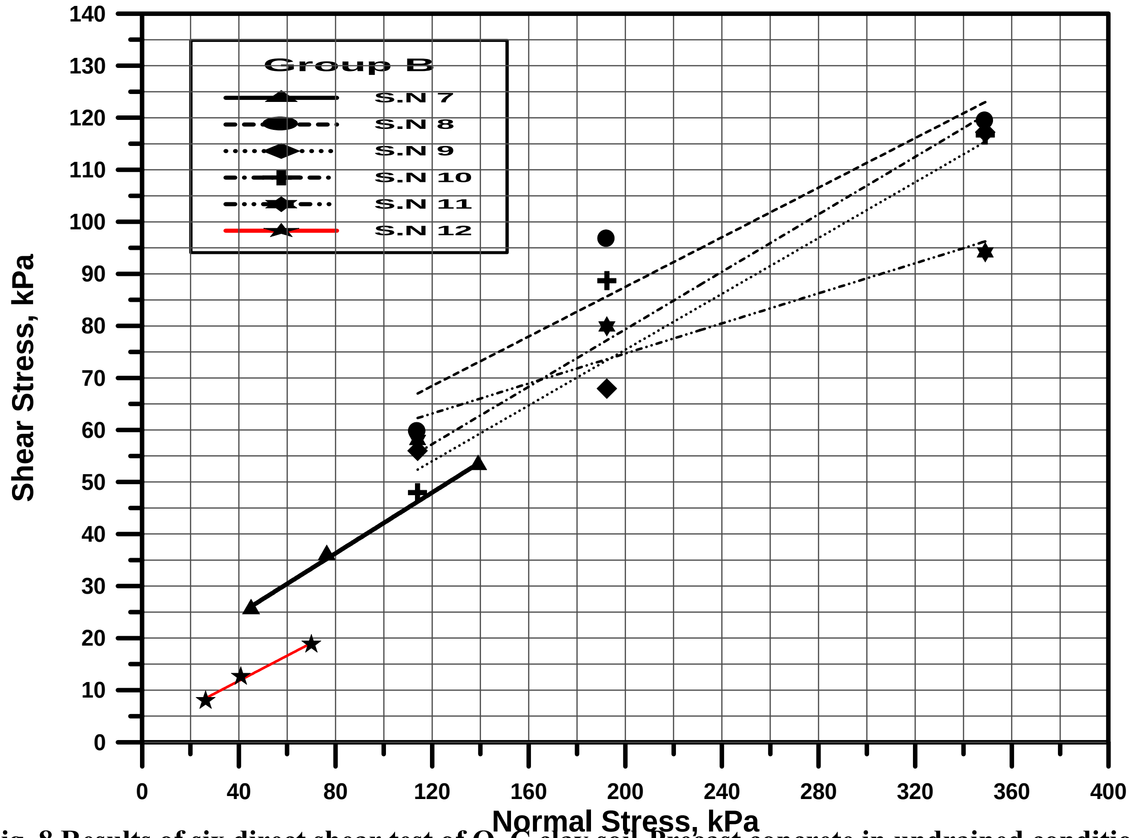

Fig. 8 Results of six direct shear test of O. C clay soil-Precast concrete in undrained condition group B

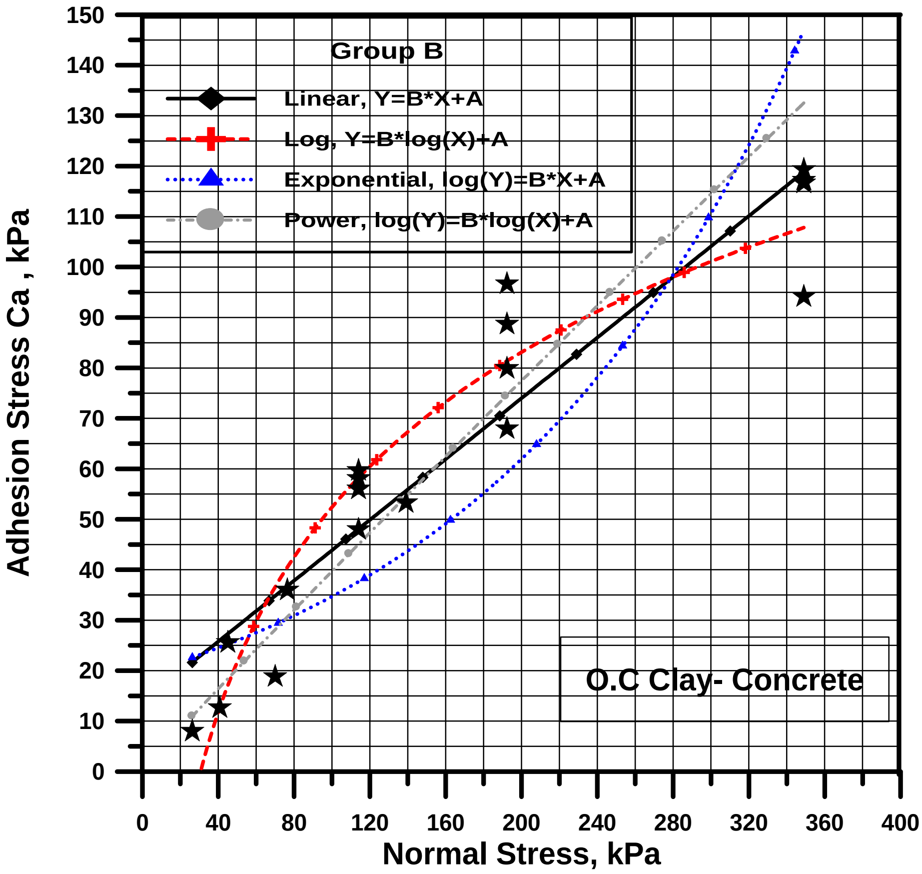

Fig. 9 Determination of skin friction parameters (C_a & δ) of O. c Clay – Precast concrete in group B

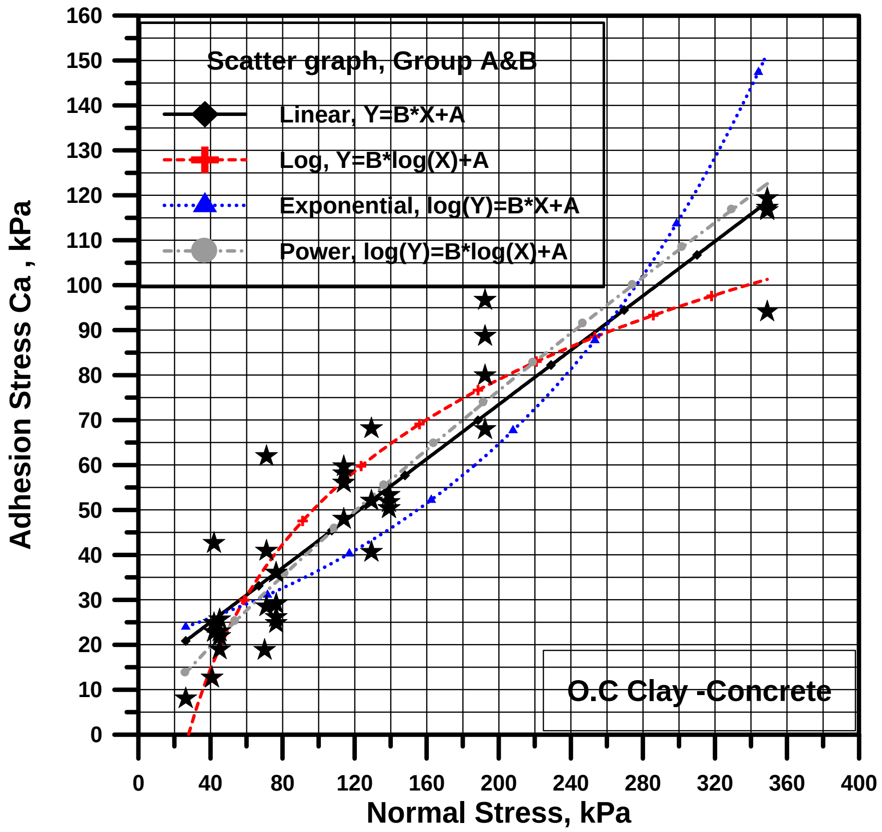

Fig. 10 Determination of skin friction parameters (C_a & δ) of O. C clay – Precast concrete (all test) Scatter graph

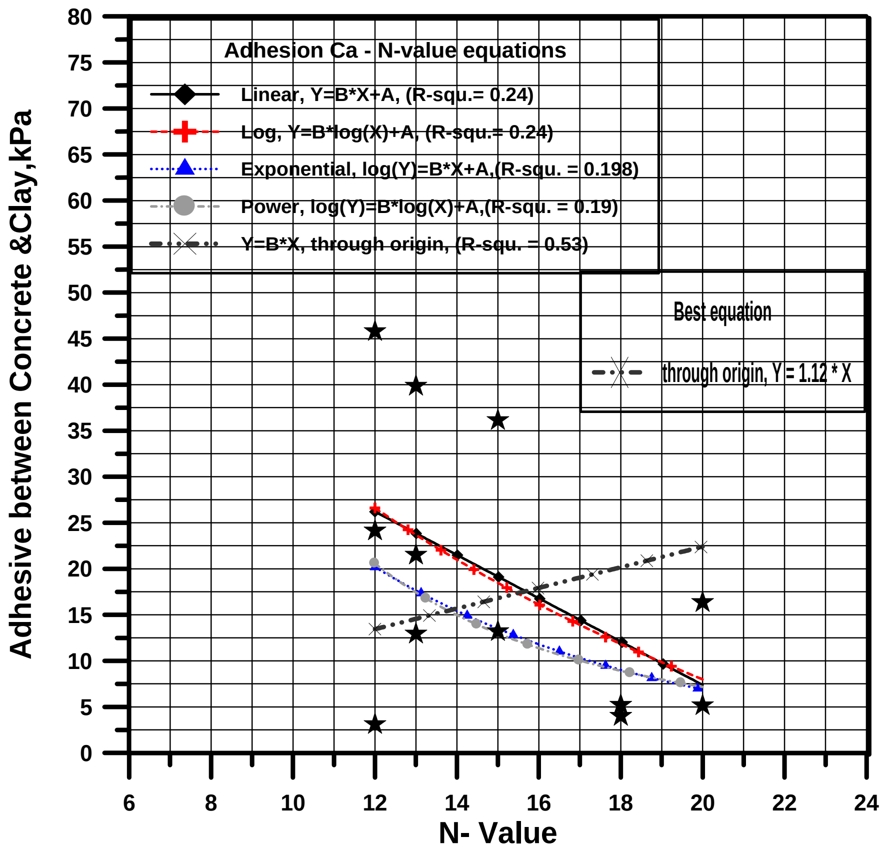

Fig. 11 Adhesive C_a versus N-value of O. C clay – Precast concrete

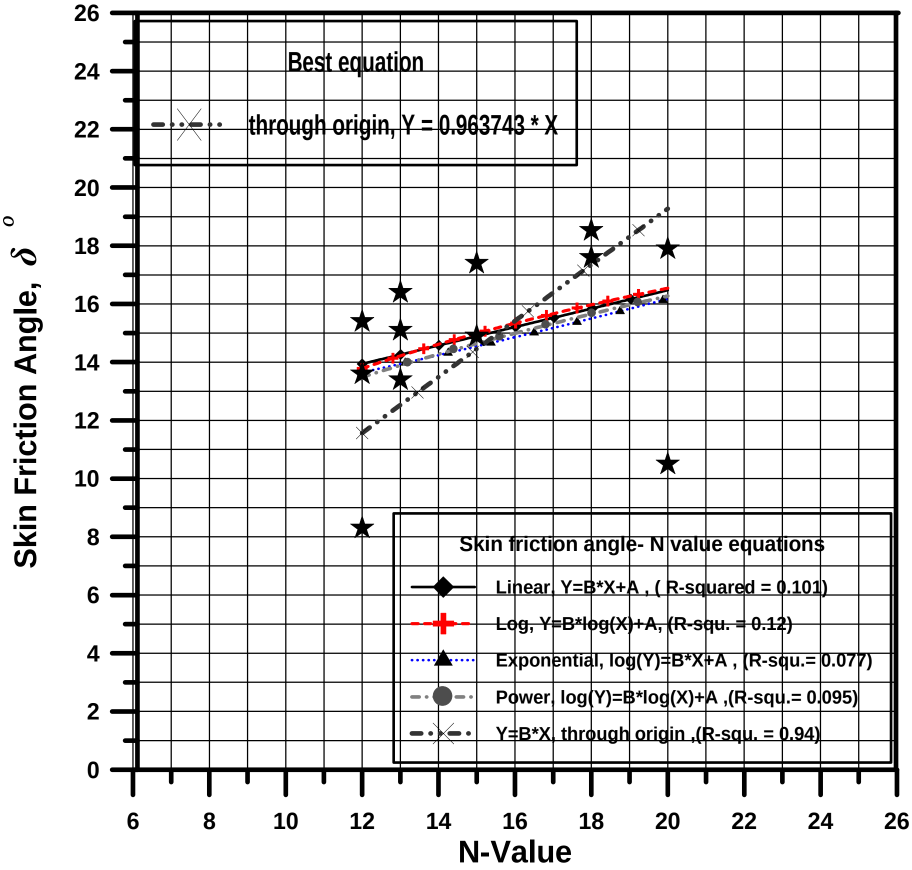

Fig. 12 skin friction angle δ versus N.value of O. C clay – Precast concrete