SKIN FRICTION BETWEEN VARIOUS SOILS AND CONSTRUCTION MATERIALS

by

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SYNOPSIS

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 friction coefficients through laboratory experiments in which the factors influencing the results may be controlled quantitatively.

Several hundred experiments were carried out by the Author to determine the magnitude of skin friction, in which the following variables were considered:—

(1) Various construction materials: steel, wood, concrete.
(2) For each material two surface conditions were used: smooth and rough; which are described in such a way that they may be reproduced by anyone with a reasonable degree of accuracy.
(3) Various types of soil.
(4) Strictly controlled moisture content.
(5) Variation of the normal load between the friction surfaces.

The test results show that for cohesive soils both cohesion and internal friction should be considered in evaluation of skin friction. The results include ratios of adhesion to cohesion, and of angle of skin friction to the angle of internal friction for definite types of soil, moisture content, various construction materials and their surface finishes; so that for practical application it is necessary to test the soil in shear and to make a sieve analysis. From the shear test data skin friction can then be evaluated by use of the given coefficients.

Jusqu'à présent les valeurs employées pour le frottement superficiel, en ce qui concerne les projets, ont été les valeurs moyennes obtenues par des essais de chantier, avec référence qualitative seulement quant aux facteurs qui influencent la grandeur de ces valeurs, facteurs tels que: catégories de sols, sortes de matériaux de construction et leur fini superficiel et la teneur d'eau des sols.

La tendence moderne est de déterminer les coefficients de frottement superficiel par des expériences en laboratoire dans lesquelles les facteurs influençant la valeur des coefficients peuvent être contrôlés quantitativement.

Plusieurs centaines d'expériences furent effectuées par l'auteur en vue de déterminer la valeur des frottements superficiels en considérant les facteurs variables suivants:

(1) Différents matériaux de construction: acier, bois, béton.
(2) Pour chaque matériel, deux conditions de surface: lisse et rude; lesquelles sont décrités d'une telle manière que n'imposte qui peut les réproduire avec une certaine degré de précision.
(3) Plusieurs catégories de sols.
(4) Teneur d'eau rigoureusement côntrolée.
(5) La charge normale fut aussi variée.

Les résultats de ces expériences ont démontré que pour les sols cohérents tant la cohésion que le frottement interne doivent être considérés pour l'évaluation du frottement superficiel. Les résultats contiennent de plus, certains rapports tel que le rapport entre l'adhésion et la cohésion, et le rapport entre l'angle de frottement superficiel et l'angle de frottement interne pour certaines catégories de sols bien déterminés. Il y en a d'autres également, ayant trait, ceux-ci, aux diverses teneurs d'eau, aux différents matériaux de construction et à leurs finis superficiels. Si bien que pour pratiquer une application utile des présents résultats, il est nécessaire de fair subir à un sol donné un essai de cisaillement, ainsi qu'une analyse granulométrique. Partant des résultats obtenus dans un essai de cisaillement, on peut évaluer le frottement superficiel en employant les coefficients donnés.

INTRODUCTION

Recent developments in civil engineering, especially in soil mechanics and foundations of structures, permit the designing engineer to take a great step forward from "design by experience" to design by a well established theory verified by experiments. In the case of foundations or other structures which are in direct contact with the soil, there usually arises the question: What is the stress–strain relation if one starts to move in relation to the other? This mutual effect of soils and structures in the transmission of forces from one to the other through the contact surface is called skin friction.

Until recently the values of skin friction were obtained from field observations, or they were

calculated from the resistance of pile driving, sinking of caissons, etc. These previous values were values averaged along the pile or caisson, and it was impossible to relate them to the behaviour of soil layers. Furthermore, no relationship was given to the surface finish of different construction materials.

The purpose of the investigation carried out by the Author was to determine the values of skin friction between different types of soils and construction materials. The subject of this article is to make an analysis of these values and to give a relationship between skin friction and the strength of soils and also between skin friction and the surface characteristics of various construction materials.

APPLICATION OF GENERAL FRICTION THEORY

The basic idea of friction seems simple, but it has constituted a problem for many decades. Every field of engineering is concerned with friction, but so far in connexion with soils and solids no theory has been available. The theory of skin friction between solid materials has had some development, and some of the conclusions may be applied to the friction between soils and solids.

For solid materials it was found that the magnitude of friction always depended on whether the surface was dry or moist, or completely lubricated by some liquid. This led to the division of friction into such groups as dry and liquid friction. Between them is semifluid or composite friction. The magnitude of friction is to a very great extent dependent upon cleanliness, atmospheric dust and humidity, oxide and other films, surface finish, velocity of sliding, contact pressure, temperature, grain size, direction of grain, vibration and static loads, etc. One can see that the problem of friction between solids is more complicated than it may seem at first. The analysis and investigation of skin friction of soils, which are much less homogeneous materials, is obviously still more complicated. Bowden and Tabor (1950) showed that in the case of smooth steel plates the contact surface varied from 1/100,000 of the gross area at a low normal load to 1/400 at a high normal load. Merriman (1930) quotes a similar postulate for wood specimens; when the normal pressure reaches the allowable stress for wood the fibres bite into each other and the coefficient of friction increases. In the case of a lubricated solid surface, the liquid could only partly produce a lubricant film between the two surfaces because the two solid materials are in contact over a percentage of the gross area. Therefore, the friction force lies between the values that it would have for a solid-solid boundary and a solid-liquid boundary.

Soils are, according to their composition, in a state between solid and liquid materials. Therefore, the mechanics of friction will be partly like those of a solid and partly like those of a liquid. The skin resistance could not be higher than the ultimate shearing strength of soil, and it is important to find out the ratio between soil strength and skin resistance. Since the development of skin friction is due to displacement, the stress–strain curves can be obtained only from the experimental results. For granular soils the experimental stress–strain relation was expressed in a mathematical term by Kézdi (1959) and applied to determine the earth pressure and pile resistance. In the case of skin friction, the stress-strain curve was expressed as an exponential function of displacement:

$$\frac{\tau}{\sigma} = \tan \delta \left[1 - \exp\left(-k \frac{s}{s_0 - s}\right)\right]$$

where τ = shearing stress which produces "s" displacement
σ = normal stress
δ = angle of skin friction
s = displacement
s_0 = maximum displacement due to failure
k = constant for the soil.

DESCRIPTION OF TESTS

To determine the strength of soils, and skin friction between soils and construction materials, two kinds of equipment were used throughout this investigation; namely:

(*a*) strain controlled shear box,
(*b*) stress controlled shear box.

For the determination of physical properties of soils well-known standard equipment was used.

The strain controlled shear box had a shearing area of 5·59 sq. in., and it was used mainly to determine the shearing strength of soils in the supplementary investigations and for the determination of skin friction of saturated soils.

The stress controlled shear box had a shearing area 12·4 sq. in. and it was drained on both sides. For the measuring of horizontal movement a dial gauge was fixed to the base plate. The specimens of construction materials were placed in the lower portion of the box, and the soil was placed in the upper half.

SELECTION OF CONSTRUCTION MATERIALS AND SOILS

In the choice of construction materials for the tests, consideration was given to the application of the results in the field of civil engineering. In the present stage of engineering, important construction materials are steel, concrete and wood, and thus it was logical to select them for the investigation of skin friction.

The quality of the steel was that of a common, commercial mild steel, which is widely used for piles, sheet piles, etc. For practical purposes, two kinds of surface finish were used in this investigation. The completely smooth surface polished by fine sandpaper represents one extreme case. The other case was produced by artificially rusting the specimen and afterwards removing the loose rust (see Figs 10 and 11 facing p. 350).

It was much harder to choose the type and quality of the wood specimens. In engineering practice pine is possibly the most commonly used. Sound white pine was used, and it was cleaned from any unnatural surface irregularities and defects. Each test piece was shaped by planning to minimize the effect of roughness attributed by other than the natural texture of the wood. The finish thus obtained was similar to the surface of the plywood sheets used in shuttering. Unfortunately, no tests or references are available in connexion with the effect of hardness of wood on the contact with different soils. It may be supposed that granular materials produced a certain indentation into the surface of the wood, and this may be increased with the intensity of the normal load. The tests were carried out in two directions to the grain of wood, parallel and at right angles (see Fig. 12).

For concrete specimens two different grain sizes of aggregates were used. For fine surface the maximum grain size was 2·5 mm and for a rough surface 7·5 mm. The first concrete was poured into a plywood form, and the second one on flat rough ground. The first specimen represents the smooth concrete surface made in a planed wood form, and the second specimen represents the rough concrete surface poured against the side of an excavation (see Figs 13 and 14).

The next problem was to find what results apply between the two extreme types of soils, sand and clay, in which the tested soil has both cohesion and internal friction. The chief factor that determined the selection of the present soils was that they had to be commonly available and could easily be made up into specimens. By analysis of the distributed grain sizes of soils it can be seen that between clay and sand are the silts, or silty soils; and obviously, these soils should be investigated.

To get an intermediate soil, the two extreme types of soils, sand and clay, were mixed. In a supplementary investigation it was found that the best ratio was 50% sand and 50% clay, and this ratio represents the individual behaviours of sand and clay.

After the selection of the basic types of soils, the next step was to determine their general

physical and strength properties. This was an important procedure, because it was necessary to know these values for the purposes of later discussions and comparison with other results.

Very seldom in practice is there found a sandy soil which has a uniformity coefficient close to unity. Therefore it was advisable to choose a "well-graded" sand. Originally 2–3% of it had grains larger than 2 mm; but in order to be in the sand category these larger grains were taken off. The uniformity coefficient of this sand was 3·8 and the final grading curve is shown in Fig. 1. The specific gravity was 2·66, and the maximum void ratio obtainable had a value of 0·72, relative density 0·21; the minimum void ratio obtainable had the value of 0·51, and the relative density 0·72. The angle of internal friction in the dry condition was 44° 30′, and in the saturated condition it was 39°.

Following the general principle of choosing the average of common soils, a glacial clay with its plasticity index falling into the medium range was chosen. The specific gravity was 2·71, and the Atterberg limits were PL=21·80; LL=37·80; PI.=16·0. In view of later discussion, it was necessary to investigate a certain range of relative consistencies (C) of cohesive soils. For the mixture of clay and sand it was found that this value was between 1·0 and 0·52; therefore in the preliminary investigation of remolded and undrained clay this

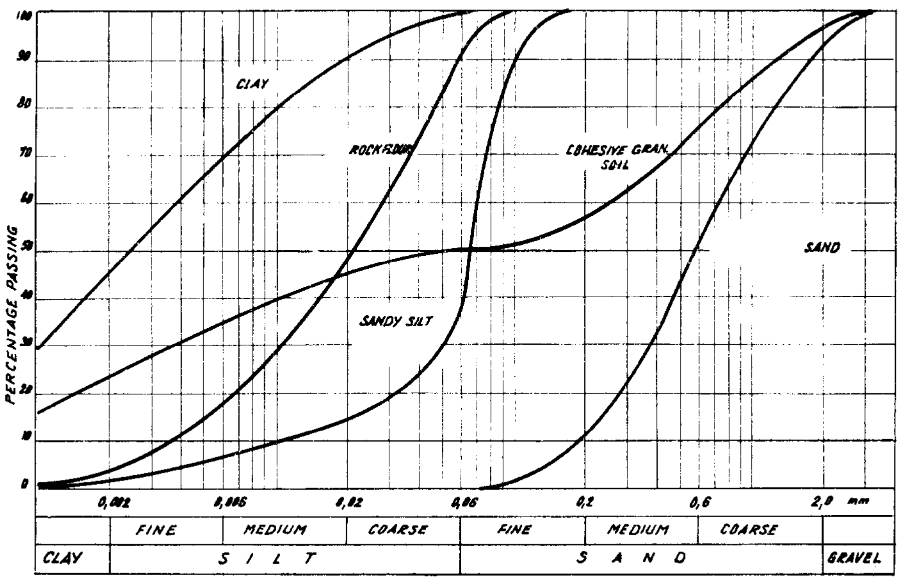

Fig. 1 Grading curves for soils

range was used with three different consistency indices, C_1=0·94, C_2=0·73, and C_3=0·52. The equivalent moisture content at these values of relative consistencies was w_1=22·8%, w_2=26·1%, and w_3=29·5%. It was impossible to keep these moisture values constant; therefore, the shearing stresses were first plotted with reference to a certain normal load against the moisture content. From these curves the respective shearing values due to the constant moisture were plotted separately.

To obtain a medium range cohesive granular soil the clay was mixed with sand in different ratios by weight; and the quoted percentage represents the ratio of sand to the total weight. Therefore the notation will by used ratio of mixing $p=\frac{\text{sand}}{\text{sand}+\text{clay}}$ for cohesive granular soil. An investigation was carried out to determine the change of the shearing resistance of mixed soil as a function of the ratio of mixing. The shearing stress was determined at

$p_1=16{\cdot}7\%$, . . . $p_5=83{\cdot}3\%$ with reference to the moisture content given by C_1, C_2, and C_3. The normal load over the plane of failure was varied from 550 lb/sq. ft to 6,000 lb/sq. ft. The change of cohesion and angle of internal friction are plotted in Figs 3 and 4.

From these curves, it can be seen that the shearing resistance changed greatly with the change of moisture content. In the change of cohesion the critical p value was 50%. Under this value the cohesion did not change very much, but over $p=50\%$ it increased sharply. The change of internal friction did not show such a sharp change, but the ϕ values at $p=50\%$ reached one half of their values for sand. Therefore the obvious choice for later investigations was the $p=50\%$ ratio. With respect to application of the final results, the Author believes that these results can be used without difficulty for mixing soils in another ratio. The liquid limit of this soil was 22%, its plastic limit was 12·4% and the moisture content due to C_1, C_2, and C_3 was $w_1=13\%$, $w_2=15\%$, and $w_3=17\%$ (Fig. 2).

For the testing of silty soil a crushed granite which had been passed through a No. 200 sieve was used. During this discussion it will be referred to as "rock flour" or "silt". The grading

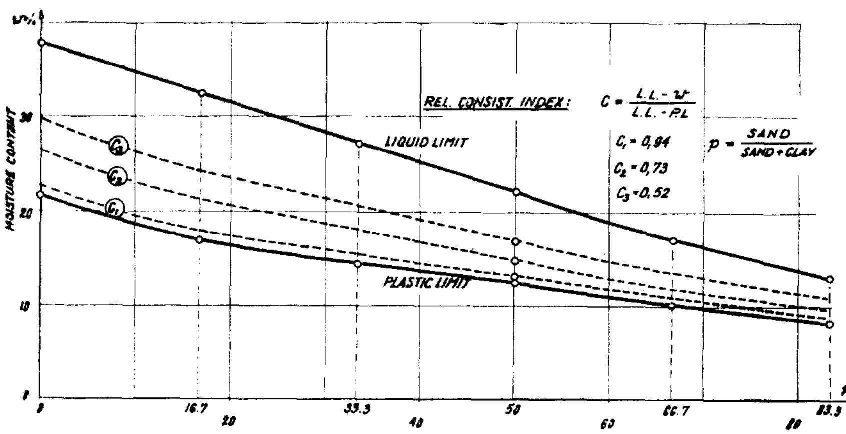

Fig. 2. Change of PL and LL of cohesive granular soil in relation to sand content

curve is shown in Fig. 1, and the coefficient of uniformity is 7·7. The specific gravity of the particles was 2·73, and the liquid limit was 26%. It was impossible to make a plastic limit test for that material. For determining the shear resistance of silt, dry, wet, and saturated tests were done on the strain controlled shear box. The highest shear resistance ($\phi=32°$) was in the wet condition with cohesion of 188 lb/sq. ft. In the subsequent investigation this case was omitted, because the cohesive behaviour was at 4–5% moisture content, and as the moisture increases the cohesion falls to zero. For practical purposes it is most important to investigate the dry and fully saturated conditions, which give the extreme cases.

In the case of cohesive granular soil, clay, and saturated silt, it was found that the uniformity of the distribution of moisture content changed during the test. In the determination of the shearing resistance of soils the moisture content increased slightly at the bottom and top of the specimen in the shear box. A similar occurrence was observed at the plane of shearing. For example, in one case, the uniform moisture content was 14·00%, and after the test it was increased to 14·82% in the failure plane.

In the investigation of skin friction, the Author found that similar redistribution occurred, but the various construction materials had different effects on the moisture content of the contact surface. In order to avoid absorption of moisture by building materials, the specimens made of wood and concrete were saturated in water for 48 hours, and just before the testing were surface dried. The average moisture content of wood was 41% and 3% of concrete.

(*a*) Between the soil and steel specimen the moisture was higher than the average of the soil sample. In the case of rough steel this increment was 2–3% and in the case of smooth steel 4·5–5·5%.

(*b*) In the case of wood, in testing parallel to the grain, the increase was 0–1·5%, while testing at right angles to the grain there was no increment at all.

(*c*) In the case of concrete the moisture content on the contact surface decreased, on smooth concrete by 7–10%, and 5–8% on rough concrete.

For determination of moisture content on the contact surface a thin slice was taken from the soil, the thickness of which was about 2 mm.

ANALYSIS OF EXPERIMENTAL RESULTS

1. *Skin friction on steel*

(*a*) *Sand.* Table 1 and Fig. 5 shows the values of internal and skin friction of dry and saturated sand. It can be seen that, when the normal load was increased from 1,000 lb./sq. ft to 3,000 lb./sq. ft, the angle of internal friction decreased from 44° 30′ to 43° 30′. When the sand was completely saturated, the angle of internal friction of the sand was 39°—37° and the skin friction on smooth steel 24° 50′–23° 30′. The relation between δ and ϕ shows the

Table 1

Stress and strain controlled shear box results of skin friction for sand

Dry sand [Dense, w=0·8%; Density: 0·66]

Normal load	1,000 lb/sq. ft				3,000 lb/sq. ft			
Material	ϕ	δ	δ/ϕ	$\frac{\tan\delta}{\tan\phi}$	ϕ	δ	δ/ϕ	$\frac{\tan\delta}{\tan\phi}$
Smooth steel	44° 30′	24° 10′	0·543	0·457	43° 30′	24° 00′	0·55	0·47
Rough steel	44° 30′	34° 00′	0·765	0·68	43° 30′	33° 40′	0′78	0·70
Wood par. to grain	44° 30′	35° 00′	0·790	0·71	43° 30′	33° 20′	0·766	0·69
Wood at right angles to grain	44° 30′	39° 00′	0·880	0·82	43° 30′	38° 30′	0·885	0·84
Smooth concrete	44° 30′	39° 30′	0·89	0·84	43° 30′	38° 30′	0·885	0·84
Rough concrete	44° 30′	44° 00′	0·99	0·98	43° 30′	42° 30′	0·98	0·97

Saturated sand [Dense]

Normal load	1,000 lb/sq. ft				3,000 lb/sq. ft			
Material	ϕ	δ	δ/ϕ	$\frac{\tan\delta}{\tan\phi}$	ϕ	δ	δ/ϕ	$\frac{\tan\delta}{\tan\phi}$
Smooth steel	39° 00′	24° 50′	0·64	0·57	37° 00′	23° 30′	0·64	0·57
Rough steel	—	—	—	—	—	—	—	—
Wood par. to grain	39° 00′	33° 20′	0·85	0·82	37° 00′	33° 00′	0·89	0·86
Wood at right angles to grain	39° 00′	34° 30′	0·89	0·85	37° 00′	34° 30′	0·93	0·91
Smooth concrete	39° 00′	34° 40′	0·89	0·85	37° 00′	33° 20′	0·90	0·87
Rough Concrete	—	—	—	—	—	—	—	—

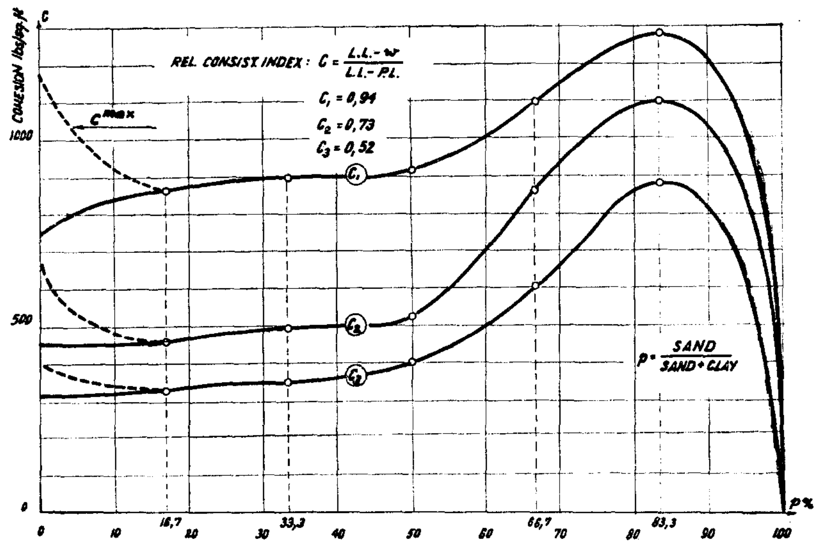

Fig. 3. Change of cohesion in relation to sand content of cohesive granular soil

same value under different loads. Further on in the discussion of these values only the lowest will be referred to, because in practice the others are on the safe side.

(*b*) *Clay.* In tests of skin friction, only two moisture contents were used, w_1 and w_2. The results of the investigation of skin friction are shown in Fig. 6 and Table 2. The curve of

Table 2

Values of shear strength and skin friction of clay

		Friction on	ϕ or δ	c or c_a lb/sq. ft	$c$max $c_a$max lb/sq. ft	$\frac{\delta}{\phi}$	$\frac{c_a}{c}$	$\frac{c_a\text{max}}{c\text{max}}$
Cons. Index: 0·94, Moist. Cont.=22·8%	1	Soil	16° 30′	750	1,175	—	—	—
	2	Smooth steel	9° 00′	200	600	0·55	0·27	0·51
	3	Rough steel	10° 00′	350	350	0·61	0·47	0·84
	4	Wood par. to grain	11° 00′	300	1,020	0·67	0·40	0·87
	5	Wood at right angles to grain	13° 50′	390	1,000	0·82	0·52	0·85
	6	Smooth concrete	16° 10′	425	1,175	0·97	0·57	1·00
Cons. Index: 0·73, Moist. Cont. 26·1%	1	Soil	11° 30′	460	675	—	—	—
	2	Smooth steel	6° 30′	140	360	0·56	0·30	0·53
	3	Rough steel	5° 50′	265	580	0·50	0·58	0·86
	4	Wood par. to grain	7° 00′	210	600	0·61	0·46	0·89
	5	Wood at right angles to grain	8° 00′	230	620	0·69	0·50	0·92
	6	Smooth concrete	9° 30′	240	675	0·82	0·52	1·00

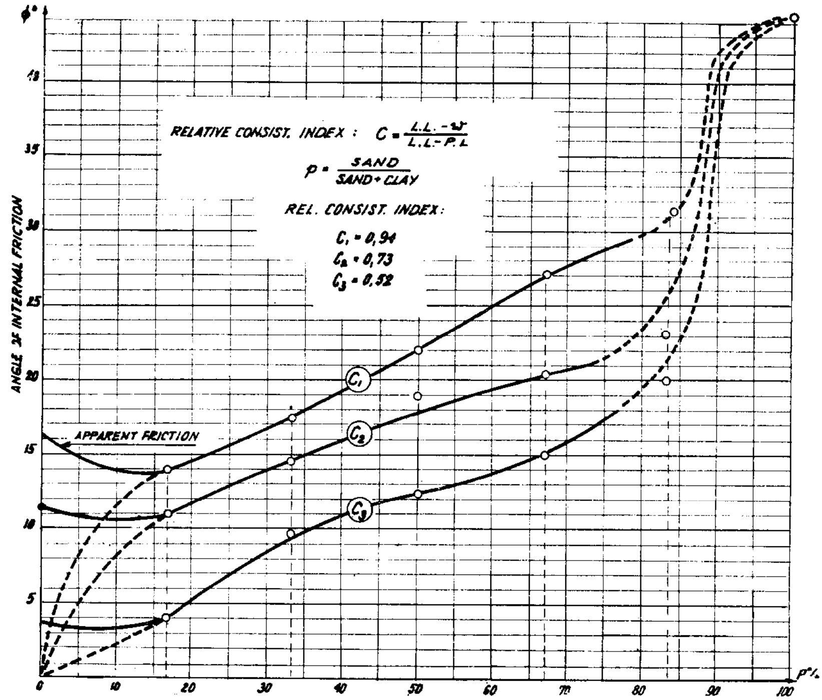

Fig. 4. Change of internal friction in relation to sand content of cohesive granular soil

skin resistance may be divided into two portions; the first, in which the skin resistance increases with the increasing of the normal load; and the second, which shows that the skin friction does not change at all with the increasing of normal load over the shearing plane. The shape of the first part, can be explained by the variation of the density of the soil with the normal load, change in the saturation and pore pressure produced by the normal load, as well as the expulsion of air from the contact area. The second part levels off as the density reaches the maximum. The Author found, from a few supplementary investigations, that this limit between the two ranges could be reduced by a very high pre-loading for 48 hours before the

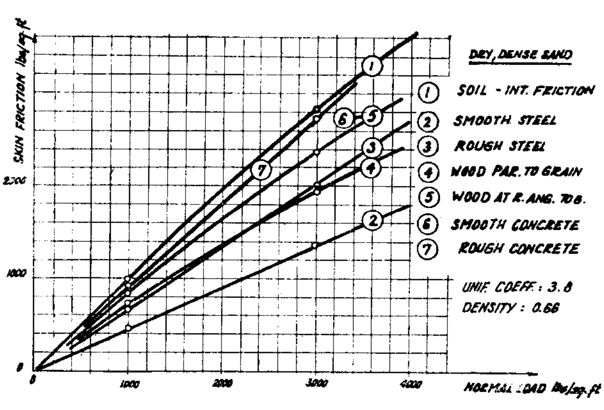

Fig. 5. Skin friction/normal load relationship of dry sand

skin friction test was carried out. By this preloading the δ value was increased almost to the value of ϕ, but the initial adhesion never reaches the value of the cohesion of the soil.

(*c*) *Cohesive granular soil.* The skin friction tests were carried out with different moisture contents, and the test results were plotted against the corresponding moisture content. From these figures, the skin shearing under different loads due to w_1, w_2, and w_3 were plotted. The first important result that was established from these tests was that the skin-friction diagram followed the shape of the shearing diagram of the soil, and each value was proportional to the shearing value of the soil, but it never reached the soil-strength values. The results also show that the roughness of the surface and the moisture content have a great influence on the skin friction (Fig. 7, curves 2 and 3; Figs 8 and 9).

Table 3

Values of shearing strength of cohesive granular soil

Values of skin friction between construction materials and cohesive granular soil

w	No. of line in Fig.	Friction on	c or c_a lb/sq. ft	ϕ or δ	c_a/c	δ/ϕ	$\frac{\tan\delta}{\tan\phi}$
w_3 Moisture Content = 17%	1	Coh. Gran. soil	385	13° 00′	—	—	—
	2	Smooth steel	10	7° 30′	0·03	0·58	0·57
	3	Rough steel	100	10° 00′	0·26	0·77	0·76
	4	Wood par. to grain	40	13° 50′	0·10	1·06	1·06
	5	Wood at right angles to grain	105	14° 00′	0·27	1·08	1·08
	6	Smooth concrete	185	13° 00′	0·48	1·00	1·00
	7	Rough Concrete	300	13° 30′	0·78	1·04	1·04
w_2 Moisture Content = 15%	1	Coh. gran. soil	520	19° 10′	—	—	—
	2	Smooth steel	55	8° 30′	0·11	0·44	0·43
	3	Rough steel	290	13° 00′	0·56	0·68	0·66
	4	Wood par. to grain	100	16° 00′	0·19	0·84	0·83
	5	Wood at right angles to grain	225	17° 00′	0·43	0·89	0·88
	6	Smooth concrete	360	16° 40′	0·69	0·87	0·86
	7	Rough concrete	420	18° 00′	0·81	0·94	0·93
w_1 Moisture Content = 13%	1	Coh. gran. soil	920	22° 00′	—	—	—
	2	Smooth steel	105	9° 50′	0·11	0·45	0·41
	3	Rough steel	425	18° 30′	0·46	0·84	0·83
	4	Wood par. to grain	265	17° 40′	0·29	0·81	0·79
	5	Wood at right angles to grain	500	21° 40′	0·54	0·99	0·98
	6	Smooth concrete	615	19° 00′	0·67	0·86	0·85
	7	Rough concrete	750	20° 30′	0·81	0·93	0·93

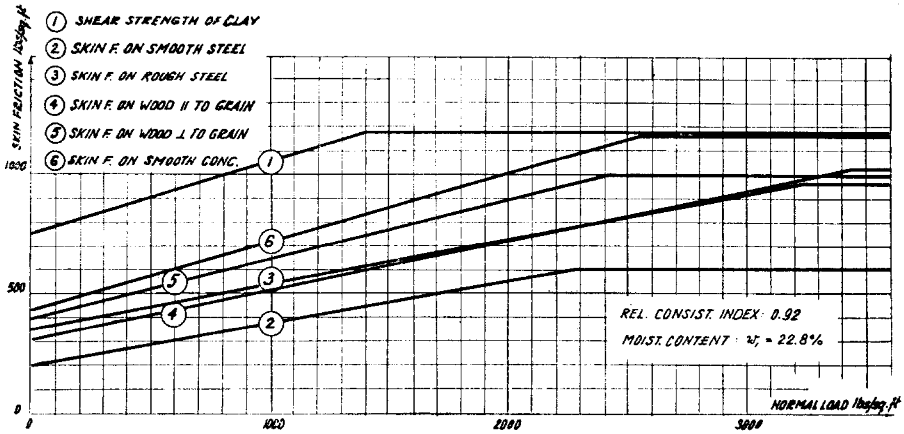

Fig. 6. **Skin friction/normal load relationship of clay**

(*d*) *Silt* (*rock flour*). The skin friction between silt and steel was investigated with the silt in both dry and saturated conditions. In the case of dry silt the large stress controlled shear box was used with smooth and rough specimens. For saturated silt only smooth steel was applicable because it could be tested only in the small strain controlled shearing box. The test results are shown in Table 4. It is interesting to note that the smooth steel has a good skin resistance on silt. The δ/ϕ values are 0·79 for dry, and 0·68–0·75 for saturated silt. The value of 0·68 was obtained when the vertical load was 4,000/sq. ft; therefore, in the case of a lower load (less density) this value should be corrected. This value drops very rapidly with the decrease of density, and the Author suggests the value of $\delta/\phi = 0{\cdot}2$–$0{\cdot}3$ in the case of loose saturated silt. Also a few tests were carried out to investigate the effect of moving water and the influence of vibration. These tests, though not very extensive, appear to show that both conditions are able to destroy the structure of silt, and therefore bring the shearing resistance practically to zero. In the case of rough steel and dry silt the skin friction was close to the internal friction of the soil itself. The values of ϕ and δ decrease somewhat with the increase of the normal load.

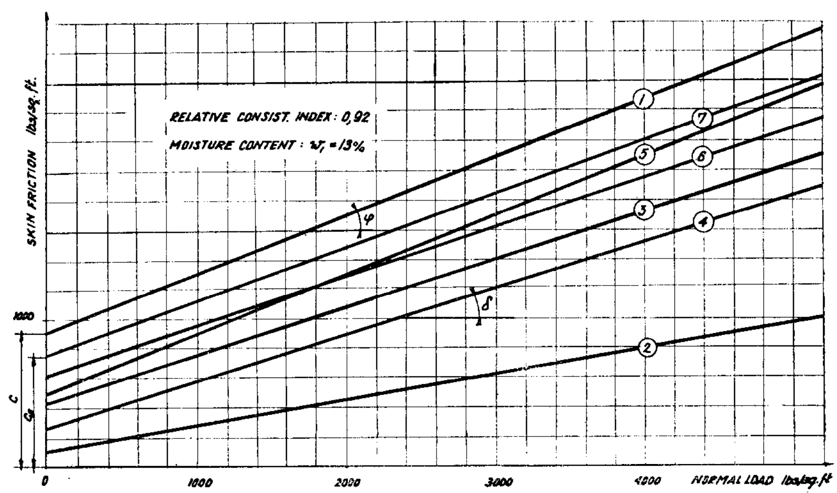

Fig. 7. **Skin friction/normal load relationship of cohesive granular soil**

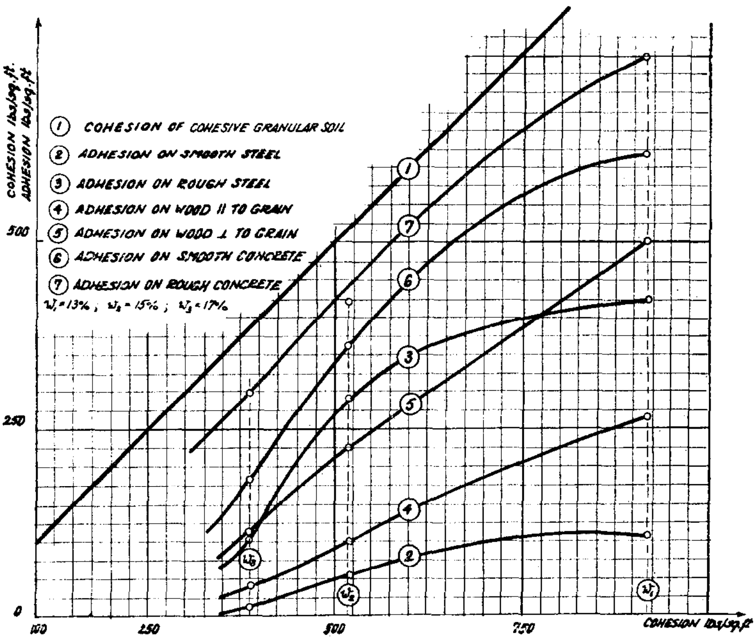

Fig. 8. Change of c_a/c ratio of cohesive granular soil.

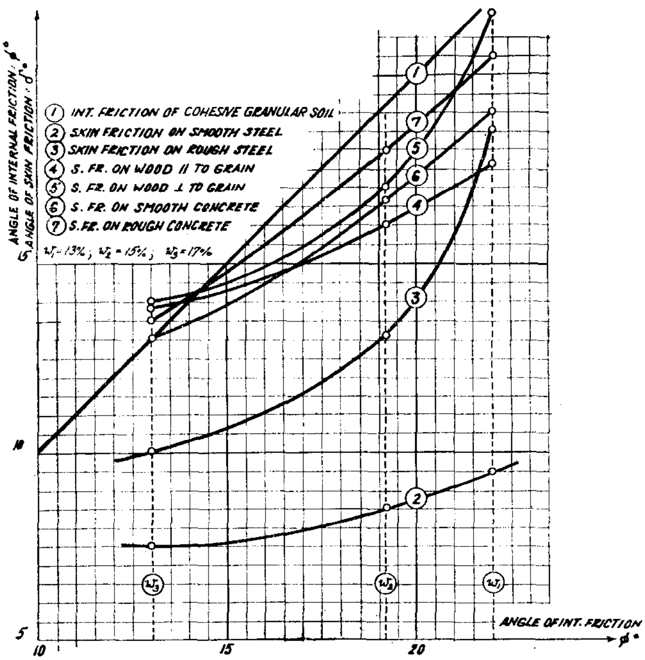

Fig. 9. Change δ/ϕ ratio of cohesive granular soil

2. *Skin friction on wood*

Relative to the direction of shear force, two positions of grains of wood specimens were used; grain parallel and grain at right angles.

(*a*) *Sand.* The sand was tested in dry and saturated conditions. The values of internal friction of sand and its angles of skin friction on wood are given in Table 1. The interlocking between the wood and sand was greater when the grains were across the direction of movement.

(*b*) *Clay.* The clay was tested at two moisture contents, w_1, and w_2. The results of the investigation of skin friction are shown in Fig. 6 and Table 2.

(*c*) *Cohesive granular soil.* Values of adhesion and friction are plotted in Figs 8 and 9. From these diagrams it can be seen that in the case of shearing parallel to the grain the δ/ϕ ratio increases when the moisture content is higher. These values at w_1 are 0·81, at w_2 are 0·84, and at w_3 are 1·06. The value of 1·06 shows that the skin friction could be higher than the internal friction of soil. This can be explained by the fact that the grains are pressed into the wood specimen, and thus have more resistance against sliding and rolling. This assumption is proved by the comparison of results at various moisture contents. In the case of higher moisture content the distribution of the normal load between the grains and the fine parts is not uniform; the coarser grains set up a skeleton system between each other which takes on more load and this causes impressions on the wood. In the case of lower moisture content the grains take less load and do not indent the wood. When the direction of shearing was at right angles to the grain, the δ value showed the same increment for low moisture as for high moisture condition, for which the ratio of δ/ϕ was 1·08.

(*d*) *Silt* (*rock flour*). Values of skin friction are shown in Table 4.

3. *Skin friction on concrete*

The concrete specimens, as was mentioned before, had two kinds of roughness; but in the case of clay, saturated sand, and saturated silt only smooth concrete was used.

(*a*) *Sand.* The sand was investigated in two conditions: dry and saturated. In both cases the normal load was 1,000 lb./sq. ft and 3,000 lb./sq ft. The obtained values are given in Table 1. When the load was increased, the δ and ϕ values generally slightly decreased; the ϕ value however, was more sensitive to the load than the δ.

(*b*) *Clay.* The clay was investigated only at two moisture contents, w_1 and w_2. The experiments showed that after a certain increase of the normal load, the skin friction on smooth concrete was equal to the maximum shearing strength of clay. This limit changed with the density of clay (degree of saturation).

(*c*) *Cohesive granular soil.* For the purpose of comparison, the value of adhesion was plotted separately against cohesion and the angle of skin friction against the internal friction of soil in Figs 8 and 9.

The δ value is strongly affected by the moisture content; it increases with the decrease of moisture content. The measurement of the true moisture content at the failure surface could not be done with accuracy, because of the roughness of the failure surface, and because of that the average moisture of the sample was always higher than the true value on the contact shear surface.

(*c*) *Silt* (*rock flour*). In the case of dry silt, the two types of surface finish behaved in the same manner. In saturated condition the skin friction was decreased with the increasing of the normal load and the δ/ϕ ratio was changed from 1·00 to 0·96.

CONCLUSIONS

The Author has analysed the results of experimentation on the change of skin friction as a function of grain distribution of soils, moisture content, normal load, type of construction material, and difference of surface finish. In every case the skin friction was lower than the

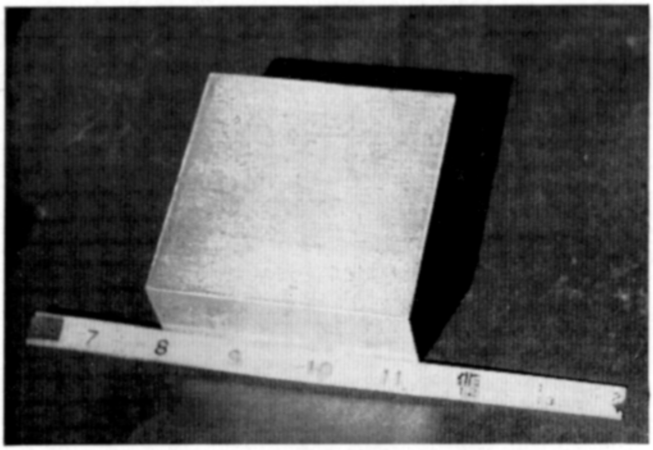

Fig. 10. Specimen of smooth steel

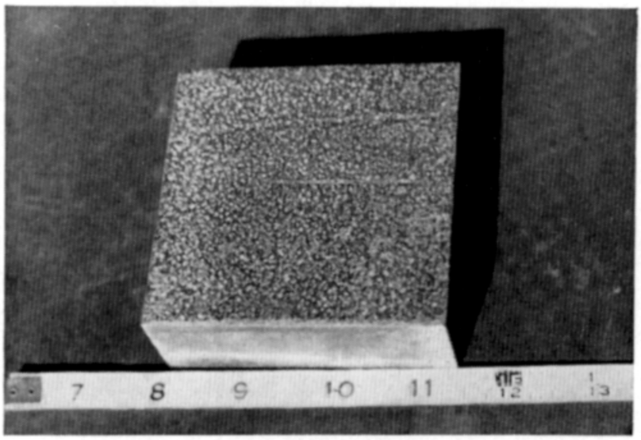

Fig. 11. Specimen of rough steel

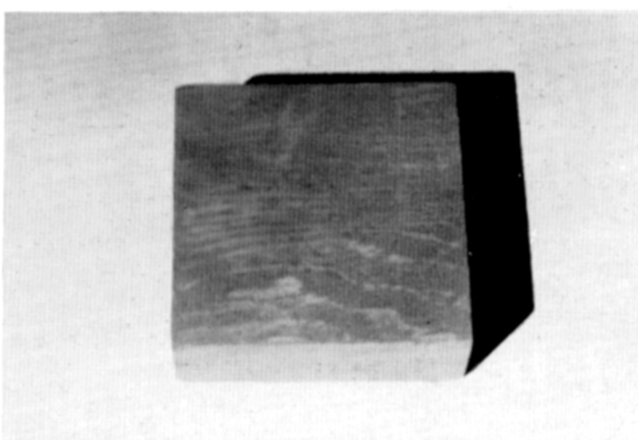

Fig. 12. Specimen of wood

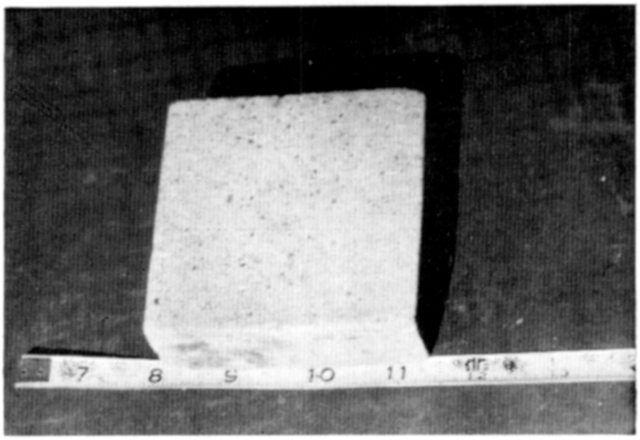

Fig. 13. Specimen of smooth concrete

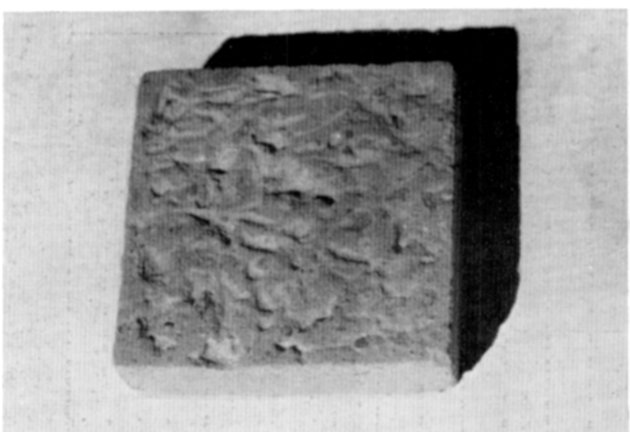

Fig. 14. Specimen of rough concrete

Table 4

Stress and strain controlled shear box results of skin friction for silt

Dry rock flour (silt–dense)

Normal load	1,000 lb/sq. ft				3,000 lb/sq. ft			
Material	ϕ	δ	δ/ϕ	$\frac{\tan \delta}{\tan \phi}$	ϕ	δ	δ/ϕ	$\frac{\tan \delta}{\tan \phi}$
Smooth steel	40° 00′	31° 30′	0·79	0·73	39° 10′	31° 00′	0·79	0·74
Rough steel	40° 00′	39° 50′	1·00	0·99	39° 10′	37° 20′	0·95	0·94
Wood par. to grain	40° 00′	37° 00′	0·92	0·90	39° 10′	36° 15′	0·92	0·90
Wood at right angles to grain	40° 00′	39° 20′	0·98	0·98	39° 10′	38° 40′	0·98	0·98
Smooth concrete	40° 00′	39° 50′	0·99	0·99	39° 10′	39° 10′	1·00	1·00
Rough concrete	40° 00′	40° 00′	1·00	1·00	39° 10′	39° 10′	1·00	1·00

Saturated rock flour (silt–dense)

Normal load	4,000 lb/sq. ft				8,000 lb/sq. ft			
Material	ϕ	δ	δ/ϕ	$\frac{\tan \delta}{\tan \phi}$	ϕ	δ	δ/ϕ	$\frac{\tan \delta}{\tan \phi}$
Smooth steel	29° 50′	20° 10′	0·68	0·66	32° 30′	24° 30′	0·75	0·71
Rough steel	—	—	—	—	—	—	—	—
Wood par. to grain	29° 50′	26° 00′	0·87	0·84	32° 30′	30° 10′	0·92	0·91
Wood at right angles to grain	29° 50′	28° 50′	0·97	0·95	32° 30′	31° 00′	0·95	0·94
Smooth concrete	29° 50′	29° 50′	1·00	1·00	32° 30′	31° 10′	0·96	0·95
Rough concrete	—	—	—	—	—	—	—	—

shearing strength of the soil. It is therefore important to determine the ratio between skin friction and shearing stress. However, because the three types of soil investigated behave in different ways, the Author believes that it is more practical to give the values of the "Coulomb line" for skin friction. In this equation the total skin resistance can be expressed in a similar form to that of Coulomb, if $fc = c_a/c$ and $f_\theta = \delta/\phi$, the equation of skin friction becoming:

$$\text{skin friction} = f_c c + \sigma \tan (f_\phi \phi).$$

Because the cohesive soils have constant cohesion at maximum density, another coefficient should be given for purely cohesive soils in the range where the shearing stress is independent of normal load. This coefficient is $f_c^{max} = c_a^{max}/c^{max}$. These f_c, f_0, and f_c^{max} coefficients are given in Table 5.

For the designing engineer the safety against failure is essential. Therefore, in cases where the skin friction works as the bearing capacity of soils, the skin friction values should be reduced. Two basic theories could be applied, the plastic and elastic. In the ultimate strength design the factor of safety means the ratio of applied external load to the ultimate capacity of the structure. Therefore, in connexion with skin friction, the ultimate skin resistance must be divided by the factor of safety.

In the elastic theory it can be done in two ways:—

Table 5

Proposed coefficients of skin friction between soils and construction materials

$[f\phi = \delta/\phi,\ fc = \frac{c_a}{c},\ fc\text{max} = \frac{c_a\text{max}}{c\text{max}}$; without factor of safety]

Construction material			Sand		Cohesionless silt			Cohesive granular soil		Clay		
			$0{\cdot}06 < D <$ 2·0 mm		$0{\cdot}002 < D < 0{\cdot}06$			50% Clay + 50% Sand		$D \leqq 0{\cdot}06$ mm		
	Surface finish of construction material		Dry	Sat.	Dry	Sat.		Consist. I. $=1{\cdot}0$–$0{\cdot}5$		Consist. Index: 1·0–0·73		
			Dense		Dense	Loose	Dense					
			$f\phi$	$f\phi$	$f\phi$	$f\phi$	$f\phi$	$f\phi$	fc	$f\phi$	fc	fcmax
Steel	Smooth	Polished	0·54	0·64	0·79	0·40	0·68	0·40	—	0·50	0·25	0·50
	Rough	Rusted	0·76	0·80	0·95	0·48	0·75	0·65	0·35	0·50	0·50	0·80
Wood	Parallel to grain		0·76	0·85	0·92	0·55	0·87	0·80	0·20	0·60	0·4	0·85
	At right angles to grain		0·88	0·89	0·98	0·63	0·95	0·90	0·40	0·70	0·50	0·85
Concrete	Smooth	Made in iron form	0·76	0·80	0·92	0·50	0·87	0·84	0·42	0·68	0·40	1·00
	Grained	Made in wood form	0·88	0·88	0·98	0·62	0·96	0·90	0·58	0·80	0·50	1·00
	Rough	Made on adjusted ground	0·98	0·90	1·00	0·79	1·00	0·95	0·80	0·95	0·60	1·00

(*a*) Plotting the stress–strain curve in regard to the Coulomb line. The yield point in the stress–strain curve (or reduced value of the yield point) projected back to the values of normal load (normal stress) will intersect the reduced safety value. In this case the sum of the cohesion and the friction part of the skin resistance is reduced by the safety factor.

(*b*) Approaching the allowable value of skin friction by reducing the adhesion and the angle of skin friction with safety factors separately.

The results of this investigation show that the selection of cohesive granular soils between clay and sand represent very well the characteristics of both extreme soil types. Not only the shearing resistance, but the skin friction too, falls into this intermediate range of composite soils. This comparison is based on the same relative consistency index for clay and for mixed soil.

The mixing had a favourable effect on the clay because it increased the cohesion and the internal friction. The clay added to the sand decreased the internal friction of the sand, especially when the clay content exceeded 15%. In consequence of the results obtained in these investigations, the Author suggests that the adhesion part of skin resistance might be neglected in cases where a cohesive granular soil is composed of less than 15% clay. Up to this limit, the composite soil behaves as a granular soil. When a cohesive soil has 5–8% sand content it can be considered as a pure cohesive soil, and the constant part of the shearing curve should be taken into account. Between these two cases is the cohesive granular soil which has cohesion and internal friction. The skin friction tests showed that in the evaluation of skin resistance, both adhesion and friction played important parts.

Some authors suggest a comparison between the angle of skin friction and the angle of internal friction; others suggest a comparison of their tangents. The Author, however, made a quite wide range investigation of skin friction, which showed that it is more convenient to use the angle values of friction both in the designing and in field engineering. In some cases the angle value could be used directly (angle of inclination of earth pressure); in other cases an interpolation for c_a, c and δ, ϕ values will be easier.

In summarizing it must be noted that four major factors determine skin friction; the moisture content of soils, the roughness of surface, the composition of soils, and the intensity of normal load. By these tests the Author has shown, that in the case of cohesive soils the adhesion and friction part should be taken into account in evaluating skin friction.

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