Note:

A SHORT NOTE ON THE EARTH PRESSURE AND MOBILIZED ANGLE OF INTERNAL FRICTION IN ONE-DIMENSIONAL COMPRESSION OF SOILS

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ABSTRACT

In this note an empirical expression for the coefficient of earth pressure at-rest K_0 in terms of the mobilized angle of internal friction ϕ'_{mob} in one-dimensional compression of normally consolidated soils is presented. It was obtained from published experimental data in a straightforward manner and without any assumptions. Although it is empirical, the equation has a better basis in terms of soil behaviour than the familiar Jaky's equation and, for the full range of K_0 (= K_{0NC}) values considered, it provides slightly better predictions than the Jaky and three other prediction equations.

Key words: K_0 coefficient, normally consolidated soil, mobilized angle of internal friction.

1. INTRODUCTION AND BACKGROUND

The at-rest earth pressure coefficient is usually considered to be a fundamental parameter in soil mechanics. Its symbol K_0 was originally introduced by Donath (1891), according to Brooker and Ireland (1965), to define the ratio between the horizontal and vertical pressures induced in the soil by a vertical load when the lateral strain is zero. However, in modern soil mechanics, the coefficient K_0 represents a global effective stress condition and it is defined as the ratio between the horizontal and the vertical effective geostatic stresses at a point in a semi-infinite soil mass.

Various methods, both in situ and in laboratory, have been proposed for the measurement of this parameter, but none are very satisfactory. In natural soil deposits, it is often difficult to distinguish between the natural variability and non-homogeneity of the soil deposit and, moreover, lack of repeatability of the measurement techniques can occur. Because of difficulties caused by sample disturbance, laboratory tests for K_0 are often conducted on reconstituted and remoulded specimens and are also affected by test errors and repeatability problem.

Theoretical expressions for K_{0NC} have been developed over the years. Probably the most famous is the one proposed by Jaky (1948), *i.e.*, $K_{0NC} = 1 - \sin\phi'$, which is an approximate version of the original expression (Jaky, 1944). Others have been developed by Rowe (1958), Hendron (1963), Burland and Roscoe (1969) and Burland and Federico (1999). Empirical correlations also have been developed for K_{0NC} (*e.g.*, Brooker and Ireland, 1965; Alpan, 1967; Yamaguchi, 1972; Massarsch, 1979).

In the mentioned expressions, both empirical and theoretical, the in situ effective stress ratio is most often expressed as a function of the angle of internal friction ϕ', *i.e.*, of the ultimate or failure stress condition. Anyway, this ratio represents stress conditions well below failure. As a consequence, a more appropriate measure of these conditions could be, in principle, the angle of internal friction mobilized at this state of stress. The note explores this possibility for one-dimensional compression of normally consolidated soils.

2. THE MOBILIZED ANGLE OF INTERNAL FRICTION ϕ'_{mob} DURING ONE-DIMENSIONAL COMPRESSION

During one-dimensional compression, the effective stress ratio developed as the vertical load is increased is a constant. A Mohr's circle of stress can be drawn representing the effective stress conditions during load and, for each load increment on the same soil, these circles are all tangent to a straight line (Fig. 1).

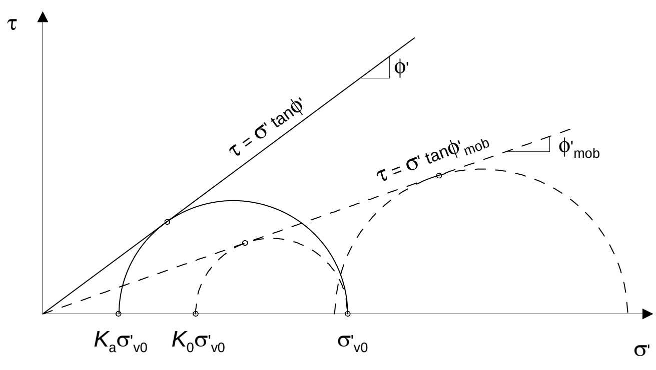

Fig. 1 Mohr circles of stress for soils at failure and under one-dimensional loading conditions

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The inclination angle of this line is of course smaller than the ultimate internal friction angle ϕ' and it represents the mobilized angle of internal friction ϕ'_{mob} in one-dimensional compression. Using the obliquity relations (*e.g.*, Taylor, 1948) for the geometry of the Mohr's circle, the at-rest coefficient of earth pressure K_0 (= K_{0NC}) can be geometrically related to the mobilized angle of internal friction ϕ'_{mob} through the following expression:

$$K_0 = \frac{1-\sin\phi'_{mob}}{1+\sin\phi'_{mob}} = \tan^2\left(45° - \frac{\phi'_{mob}}{2}\right) \quad (1)$$

Equation (1) for the coefficient of earth pressure at-rest in terms of the mobilized angle of internal friction was originally introduced by Terzaghi (1923)[1] and then discussed by Rowe (1954, 1957, and 1958). Although the mobilized friction angle ϕ'_{mob} in one-dimensional compression cannot be directly measured, it can be correlated with the ultimate effective stress friction angle ϕ' of soils. This connection can lead to a practical and useful relationship between K_0 and ϕ', as shown in the following.

3. EXISTING EXPRESSIONS OF ϕ'_{mob} AS A FUNCTION OF ϕ'

A few correlations of the mobilized friction angle ϕ'_{mob} in one-dimensional compression with the friction angle ϕ' of soils were found in the literature.

According to Rowe (1957), the mobilized angle of internal friction can be assumed equal to the Hvorslev angle of true friction ϕ_e (for the sake of completeness, Rowe (1958) proposed a different empirical correlation: ϕ'_{mob} = 3/4ϕ_e). With this assumption and using several experimental data reported in the literature (Gibson, 1953; Bjerrum, 1954; Holtz and Krizek, 1971), Abdelhamid and Krizek (1976) correlated the angle of true friction ϕ_e with the angle of shear strength ϕ', obtaining:

$$\phi'_{mob} \equiv \phi_e = 1.15\,(\phi' - 9°) \quad (2)$$

where the symbol ≡ stands for "coincident with".

Equating the values of K_0 coming from Jaky (1948) simplified expression ($K_{0NC} = 1 - \sin\phi'$) to Equation (1), Bolton (1991) found:

$$\phi'_{mob} = \phi' - 11.5° \quad (3)$$

for ϕ' varying in the range 30° to 45°.

Simpson (1992), describing the design of retaining structures in cohesive soils, investigated the K_{0NC} value predicted by his BRICK model. For one-dimensional consolidation the strain path is a straight line inclined at 45° in the space (ε_v, γ), where ε_v is the volumetric strain and γ is the engineering shear strain. According to Simpson, "this implies that the elastic shear strain which will govern shear stress will be cos 45° times that developed during failure in pure shear. Hence the angle of friction mobilized in one-dimensional consolidation will be given by:"

$$\sin\phi'_{mob} = \frac{1}{\sqrt{2}}\sin\phi' \quad (4a)$$

In the range 20° ~ 35°, this equation can be rewritten as:

$$\phi'_{mob} \cong 0.69\phi' \quad (4b)$$

The corresponding expressions for K_{0NC} can be obtained substituting the Eqs. (2), (3), and (4a) into Eq. (1).

4. PROPOSED EQUATION FOR ϕ'_{mob} AND DISCUSSION

Table 1 summarizes a number of experimental data, some of them rather "old", on K_{0NC} taken from literature as well as the soils w_l, I_p, and ϕ'. These data are relative mainly to reconstituted samples and have been obtained through a variety of experimental techniques characterized by different precision, especially as regards the control of the condition $\varepsilon_r = 0$ during the consolidation phase. The K_{0NC} values have been substituted in Eq. (1) and the derived mobilized angles of internal friction ϕ'_{mob} have been compared with the corresponding experimental angles ϕ' of internal friction, as shown in Fig. 2. This leads to the following relation between the two angles:

$$\phi'_{mob} = 0.64\ \phi' \quad (5)$$

with a coefficient of determination R^2 equal to 0.84. This allows to write, without any assumptions, a new empirical equation for normally consolidated soils:

$$K_{0NC} = \frac{1-\sin 0.64\ \phi'}{1+\sin 0.64\ \phi'} \quad (6)$$

It is interesting to note that the proposed correlation (Eq. (5)) for cohesive soils is quite similar to Eq. (4b) and also to the equation $\phi'_{mob} = 0.67\,\phi'$ derived by Hayat (1992) for sands using data from literature. Moreover, Eq. (6) can be also written as:

$$K_{0NC} \cong \frac{1-\sin\frac{2}{3}\phi'}{1+\sin\frac{2}{3}\phi'} = \tan^2\left(45° - \frac{\phi'}{3}\right) \quad (7)$$

Note that Eq. (7) is the same expression derived, in a rather questionable way, by Wierzbicki (1958 and 1963).

The predictive capability of the proposed relation (6) for the at-rest coefficient of earth pressure K_{0NC} has been checked by the comparison between predicted and measured values, as reported in Fig. 3, whereas Figs. 4 to 7 show the same comparison as regards the well known Jaky (1948) simplified expression and Eqs. (2), (3) and (4a), respectively. The statistical results, summarized in Table 2 in terms of coefficient of determination R^2 and standard deviation s_d, indicate a slightly better predictive quality of Eq. (6) in comparison with the other equations. It is worth noting that some of the considered experimental friction angle values (namely: ϕ' = 10°, ϕ' = 47.7° and ϕ' = 53.8°) are quite unusual. If these values are removed from the data set of Table 1, the predictive capability of all the equations decreases, but the statistical measures relevant to Eq. (6) still remain the best (R^2 = 0.78 and s_d = 0.08).

Figure 8 shows the K_{0NC} theoretical predictions from the above mentioned equations as a function of ϕ' and the comparison with the experimental K_{0NC} values.

[1] More precisely, Terzaghi refers to an angle ϕ_0 of "internal friction within the back-filling of a perfectly rigid wall". This angle is smaller than the maximum angle of internal friction ϕ_2, which represents the "resistance to shearing along a definite plane".

Table 1 Experimental and predicted values of K_{0NC}

Soil	w_l (%)	I_p (%)	φ' (°)	K_{0NC} Experimental data	Proposed Eq. (6)	Abdelhamid & Krizek (1976)	Bolton (1991)	Simpson (1992)	Jaky (1944)	Reference
Remolded Boston Blue Clay	33	15	27.5	0.54	0.536	0.468	0.568	0.508	0.538	Ladd (1965)
Remolded Weald Clay	46	24	26	0.61	0.555	0.499	0.600	0.527	0.562	Skempton & Sowa (1963)
Remolded Vicksburg Buckshot Clay	63	39	24	0.54	0.581	0.543	0.644	0.553	0.593	Ladd (1965)
Undisturbed Kawasaki Clay I and II	80*	38*	37	0.52	0.427	0.305	0.398	0.403	0.398	Ladd (1965)
Undisturbed Brobekkvein Oslo Clay	39	18	30.5	0.47	0.499	0.410	0.509	0.472	0.492	Simons (1960)
Undisturbed Skabo Clay	52	29	30	0.47	0.505	0.419	0.518	0.478	0.500	Landva (1962)
Hokkaido silt 1 (slurry)	52	21	37.2	0.45	0.425	0.302	0.395	0.401	0.395	Mitachi & Kitago (1976)
Hokkaido silt 2 (slurry)	51	21	35.1	0.45	0.447	0.333	0.428	0.422	0.425	Mitachi & Kitago (1976)
Hokkaido Clay (slurry)	72	32	36.1	0.47	0.436	0.318	0.412	0.412	0.411	Mitachi & Kitago (1976)
Spestone Kaolinite (slurry)	72	32	22.6	0.64	0.600	0.575	0.677	0.573	0.616	Parry & Nadarajah (1974)
Kawasaki clay-mixture M-10 (slurry)	28	11	39.2	0.42	0.404	0.274	0.365	0.382	0.368	Nakase & Kamei (1988)
Kawasaki clay-mixture M-15 (slurry)	35	15	38.7	0.40	0.409	0.281	0.373	0.387	0.375	Nakase & Kamei (1988)
Kawasaki clay-mixture M-20 (slurry)	43	19	40.6	0.41	0.391	0.256	0.346	0.370	0.349	Nakase & Kamei (1988)
Kawasaki clay-mixture M-30 (slurry)	55	29	40.8	0.41	0.389	0.253	0.343	0.368	0.347	Nakase & Kamei (1988)
Kawasaki clay M-50 (slurry)	84	51	41.6	0.43	0.381	0.243	0.332	0.361	0.336	Nakase & Kamei (1988)
Whitefish Falls	---	---	27	0.48	0.542	0.478	0.578	0.514	0.546	DeLory & Salvas (1969)
Wallaceburg	---	---	23	0.51	0.595	0.566	0.668	0.567	0.609	DeLory & Salvas (1969)
Marine Clay	---	---	34	0.51	0.459	0.350	0.446	0.433	0.441	Koutsoftas & Ladd (1985)
Vicksburg Buckshot Clay (slurry)	57	36	26.7	0.50	0.546	0.484	0.585	0.518	0.551	Donaghe & Townsend (1978)
Louisiana EABPL Clay	79	53	21.7	0.64	0.613	0.597	0.699	0.585	0.630	Donaghe & Townsend (1978)
Sidney Kaolin	50	16	30.7	0.48	0.497	0.407	0.505	0.469	0.489	Poulos (1978)
Hydrite 10 Kaolinite (flocculated sample)	62	28	17.8	0.75	0.670	0.701	0.802	0.645	0.694	Abdelhamid & Krizek (1976)
Hydrite 10 Kaolinite (dispersed sample)	62	28	16.9	0.69	0.684	0.727	0.828	0.659	0.709	Abdelhamid & Krizek (1976)
Hydrite PX Kaolinite	---	---	16.9	0.65	0.684	0.727	0.828	0.659	0.709	Edil & Dhowian (1981)
Australian Kaolin 1	75	40	23	0.56	0.595	0.566	0.668	0.567	0.609	Moore & Cole (1977)
Australian Kaolin 2	58	32	30	0.44	0.505	0.419	0.518	0.478	0.500	Moore & Cole (1977)
Kaolin	---	---	23.2	0.64	0.592	0.561	0.663	0.564	0.606	Parry & Wroth (1976)
Spestone Kaolin	76	37	20.7	0.66	0.627	0.622	0.724	0.600	0.647	Sketchley & Bransby (1973)
Kaolin	---	---	23	0.69	0.595	0.566	0.668	0.567	0.609	Burland (1967)
Kaolin	55	23	23.3	0.51	0.591	0.559	0.660	0.563	0.604	Singh (1971)
London Clay	95	65	20	0.65	0.637	0.641	0.742	0.611	0.658	Skempton & Sowa (1963)
London Clay	65	38	17.5	0.66	0.675	0.710	0.811	0.649	0.699	Brooker & Ireland (1965)
Weald Clay	41	21	22	0.54	0.609	0.590	0.692	0.581	0.625	Brooker & Ireland (1965)
Weald Clay	---	---	26.2	0.58	0.552	0.494	0.595	0.524	0.558	Skempton & Sowa (1963)
Bearpawe Shale	101	78	15.5	0.70	0.706	0.770	0.870	0.682	0.733	Brooker & Ireland (1965)
Bearpawe Shale	82	64	21	0.65	0.623	0.615	0.717	0.596	0.642	Singh et al. (1973)
Drammen Clay 1	60	31	31.7	0.49	0.485	0.389	0.487	0.458	0.475	Berre & Bjerrum (1973)
Drammen Clay 2	33	10	30	0.49	0.505	0.419	0.518	0.478	0.500	Berre & Bjerrum (1973)
Drammen Clay	55	27	30.7	0.49	0.497	0.407	0.505	0.469	0.489	Brown et al. (1977)
New York Varved Clay	65/35	39/12	20.9	0.67	0.624	0.617	0.719	0.597	0.643	Leathers & Ladd (1978)
Hackensack Valley Varved Clay	65/40	35/25	19	0.65	0.652	0.668	0.769	0.626	0.674	Saxena et al. (1978)
South African Clay	---	---	28.7	0.48	0.521	0.444	0.544	0.493	0.520	Knight & Blight (1965)
Portsmouth Clay	35	15	32	0.47	0.482	0.384	0.481	0.455	0.470	Simon et al. (1974)
Beaumont Clay	67	41	24	0.55	0.581	0.543	0.644	0.553	0.593	Mahar & Ingram (1979)
Boston Blue Clay	41	21	26.8	0.54	0.545	0.482	0.582	0.517	0.549	Kinner & Ladd (1973)
Goose Lake Flour	32	16	27.5	0.50	0.536	0.468	0.568	0.508	0.538	Brooker & Ireland (1965)
Albuquerque Clay-Sand	25	11	30.5	0.56	0.499	0.410	0.509	0.472	0.492	Calhoun & Triandafilidis (1969)
Backebol Clay	90	60	30	0.49	0.505	0.419	0.518	0.478	0.500	Massarsch & Broms (1976)
Bombay Clay	115	70	24	0.63	0.581	0.543	0.644	0.553	0.593	Kulkarni (1973)
Khor-Al-Zubair Clay	55	35	27.3	0.49	0.538	0.472	0.572	0.510	0.541	Hanzawa (1977a)
Fao Clay	39	20	36.9	0.44	0.428	0.306	0.400	0.404	0.400	Hanzawa (1977b)
Norvegian Clay	26	8	10	0.75	0.799	0.961	1.054	0.781	0.826	Bjerrum (1961)
Moose River Muskeg	---	---	47.7	0.30	0.326	0.176	0.257	0.313	0.260	Adams (1965)
Portage Peat	---	---	53.8	0.30	0.278	0.122	0.195	0.273	0.193	Edil & Dhowian (1981)
Kyoto Clay	88	57	32.5	0.45	0.476	0.375	0.472	0.449	0.463	Akai & Adachi (1965)
Lagunillas Clay	61	37	26.8	0.53	0.545	0.482	0.582	0.517	0.549	Lambe (1964)
New England Marine Clay	---	20	32	0.50	0.482	0.384	0.481	0.455	0.470	Ladd (1976)
Haney Clay	---	---	30	0.55	0.505	0.419	0.518	0.478	0.500	Campanella & Vaid (1972)
Newfield Clay	31	13	28.6	0.50	0.522	0.446	0.546	0.494	0.521	Singh (1971)

* average value

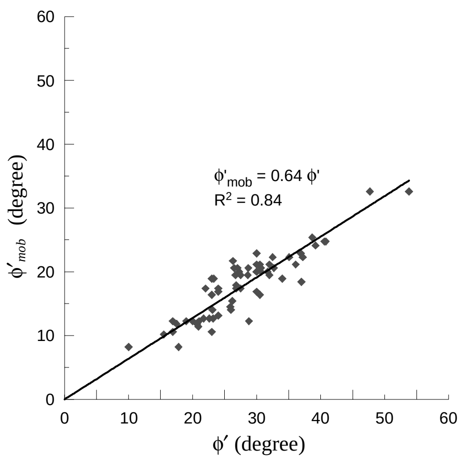

Fig. 2 Correlation between ϕ'_{mob} and ϕ'

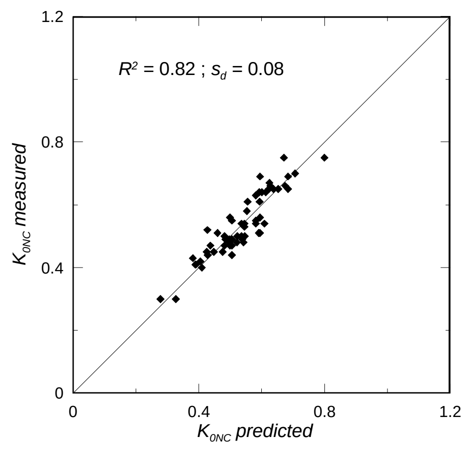

Fig. 3 Correlation between measured K_{0NC} values and the ones obtained from Eq. (6)

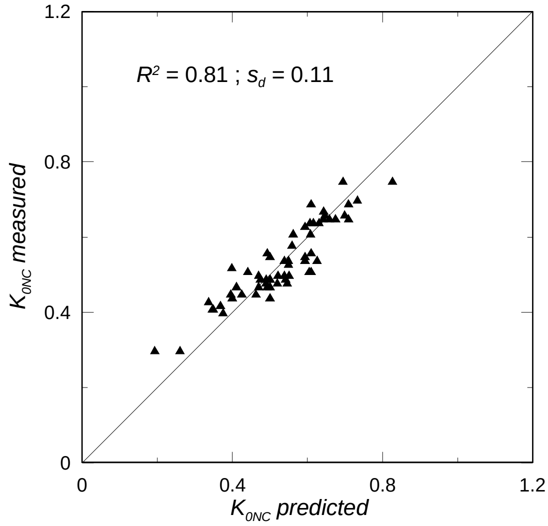

Fig. 4 Correlation between measured K_{0NC} values and the ones obtained from Jaky (1948) simplified equation

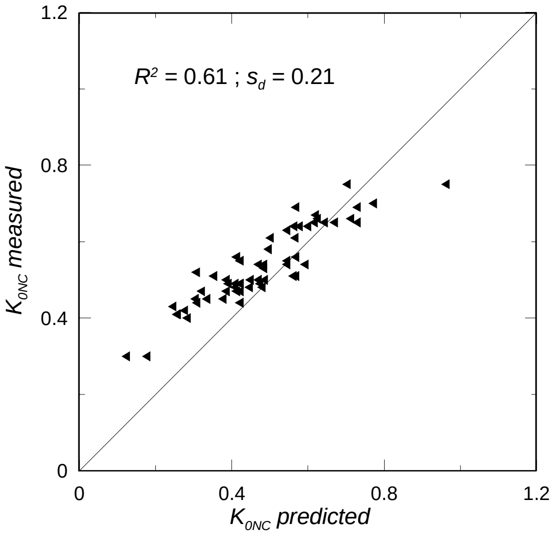

Fig. 5 Correlation between measured K_{0NC} values and the ones obtained from Eq. (2)

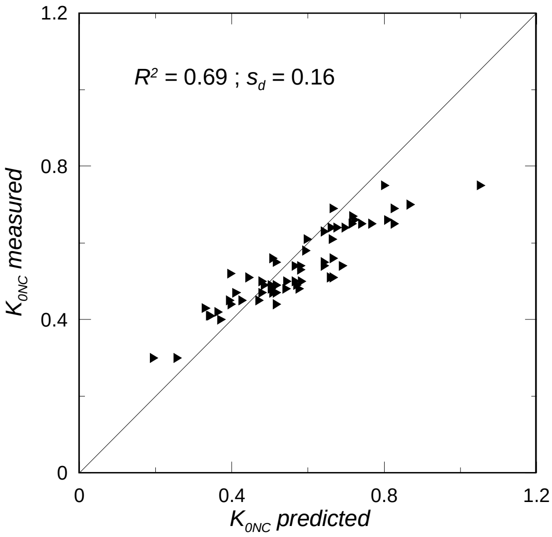

Fig. 6 Correlation between measured K_{0NC} values and the ones obtained from Eq. (3)

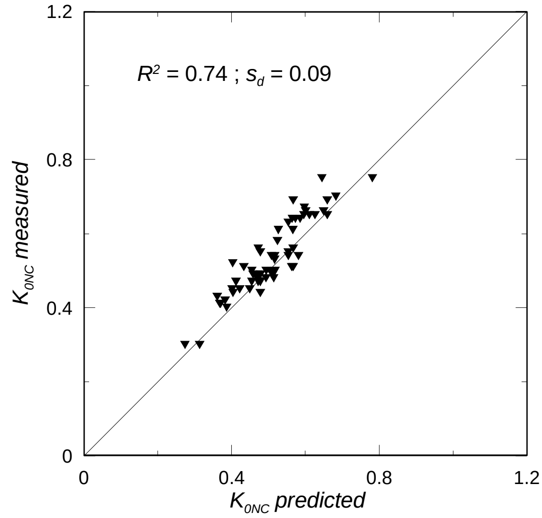

Fig. 7 Correlation between measured K_{0NC} values and the ones obtained from Eq. (4a)

Table 2 Comparison between R^2 and s_d values relative to different K_{0NC} equations

	Coefficient of determination, R^2	Standard deviation, s_d
Proposed Eq. (6)	0.82	0.08
Jaky (1948)	0.81	0.11
Abdelhamid and Krizek (1976)	0.61	0.21
Bolton (1991)	0.69	0.16
Simpson (1992)	0.74	0.09

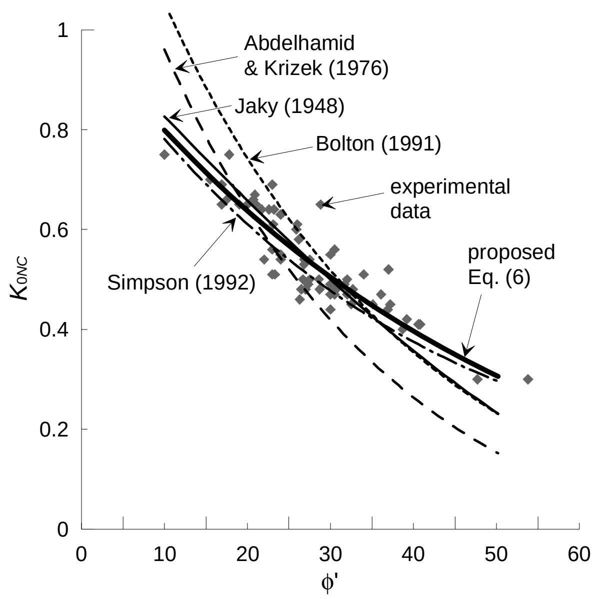

Fig. 8 K_{0NC} theoretical predictions as a function of ϕ' compared with experimental values

That the Jaky's equation predicts K_{0NC} quite well is surprising, since this equation was derived using questionable theoretical assumptions (Michalowski, 2005) and, as was mentioned earlier, has no obvious connection in terms of soil behaviour. The proposed Eq. (6), merely derived from experimental correlation between ϕ'_{mob} and ϕ', has the same simplicity as the Jaky's equation, but without any theoretical assumptions and it allows better predictions of the at-rest stress ratio K_{0NC} in normally consolidated soils.

5. CONCLUSIONS

The at-rest coefficient of earth pressure of normally consolidated soils K_{0NC} can be determined from the mobilized angle of internal friction ϕ'_{mob} in one-dimensional compression directly from the geometry of Mohr's circle of stress. After a brief presentation of the existing equations for ϕ'_{mob} as a function of ϕ', a new empirical expression for the mobilized angle of internal friction is proposed. This equation was determined from statistical analysis of experimental data found in literature. The corresponding equation of the at-rest coefficient of earth pressure has been obtained and its predictive capability has been tested. For a full range of ϕ', the values of K_{0NC} predicted by this new equation, although rather close to the ones determined by Jaky's equation, have a better agreement with the experimental data.

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