A Comparison between the CBR and the Shear Strength Methods in the Design of Flexible Pavements

Comparaison entre la méthode CRB et la méthode de résistance au cisaillement pour le calcul de chaussées souples

by G. WISEMAN, Ing., S. M., D. Sc. (Tech.), Senior Lecturer,
and
J. G. ZEITLEN, S. M., Professor of Soil Engineering Israel Institute of Technology, Haifa, Israel

Summary

The in-situ strength of subgrade clays at over 150 test locations at three airfields in Israel was measured using vane, penetration, CBR, and unconfined compression tests. The sensitivity of the subgrade clays encountered was investigated and the clays were found to possess from little to no sensitivity to remoulding. The ratio of maximum to minimum torque in the vane test was found to be unreliable as a measure of sensitivity for the clays. The static penetration resistance was correlated with the vane test and was found to be a good measure of shear strength when used with the appropriate bearing capacity factor. The rate of strain was found to be of considerable significance in evaluating shear strength.

It is shown analytically that for any combination of pavement thickness, total wheel load and tyre pressure the CBR required of the subgrade is numerically equal to eight times the maximum shear stress (kg per sq. cm) induced in the subgrade. For the clays tested, the in-situ CBR was found to be numerically equal to 4 times the in-situ shear strength (kg per sq. cm), and the use of a constant value for this coefficient is justified analytically. Based on the above two findings, the ratio of the maximum shear stress induced in the subgrade to the shear strength which is implied when using the CBR design curves is analyzed.

The application of the concepts developed by the authors to the design of the total thickness of flexible pavements for clay subgrades, similar to those studied, is discussed.

Sommaire

La résistance sur place (*in situ*) des argiles a été mesurée en plus de 150 endroits sur trois aérodromes en Israël, à l'aide du CBR et de compression simple et à l'aide de scissomètres et de pénétromètres. La sensibilité au remaniement des argiles étudiées s'est montrée faible et même nulle. La relation entre les torsions maximum et minimum de l'essai au scissomètre s'est montrée peu sûre comme mesure de la sensitivité. Une corrélation a pu être établie entre la résistance à la pénétration statique et le résultat de l'essai au scissomètre. Et la résistance à la pénétration statique fournit une bonne valeur de la résistance au cisaillement si on l'emploie avec le facteur correct de la force portante. On a constaté que la vitesse de déformation avait une grosse influence sur la valeur de la résistance au cisaillement.

On en a déduit analytiquement que pour n'importe quelle combinaison d'épaisseur de revêtement de la charge totale par roue et de la pression des pneus, le CBR demandé au sous-sol a une valeur numérique de huit fois (approximativement) celle de la contrainte maximum de cisaillement (kg/cm^2) produite dans celui-ci. Pour les argiles examinées le CBR *in-situ* a été trouvé égal, numériquement, à 4 fois la résistance *in situ* au cisaillement (kg/cm^2) et l'emploi d'une valeur constante pour ce coefficient est justifié du point de vue analytique.

Sur la base des deux résultats susmentionnés, le rapport entre la contrainte maximum dans le sous-sol et sa résistance au cisaillement qu'implique l'emploi du CBR, est analysé.

L'application des notions, développées par les auteurs à la détermination de l'épaisseur totale des routes sur des argiles semblables à celles étudiées, est examinée.

1. Introduction

The most widely used method of airfield flexible pavement design is the empirical one developed and verified under field conditions by the United States Corps of Engineers and known as the California Bearing Ratio method. However, a method of design for pavements on clay subgrades was proposed by R. GLOSSOP and H. Q. GOLDER (1948), based on shear strength. Basically the method consists of designing for a minimum thickness of flexible pavement so that the maximum induced shear stress does not exceed the shear strength. The problem of the factor of safety to be used is a particularly difficult one in view of the repetitive and transient nature of the loading. Analysis of this problem is made by comparison of the shear strength method with the method using the CBR pavement design thickness curves.

Such comparisons have in the past been hindered by :

(a) an uncertain position of the CBR method in relation to tyre pressure; and

(b) the limited quantity of evidence relating CBR to shear strength for a range of clay subsoils (SCALA, 1956; ROBENSON and LEWIS, 1958). Publication by W. J. TURNBULL and R. G. AHLVIN (1957) of a mathematical expression of the CBR relationships, including tyre pressure as a continuous variable, eliminates the first difficulty. The work, partially reported here, correlating *in situ* CBR to *in situ* shear strength for clay subsoils provides a relationship between CBR and shear strength for a range of clay sub grades and helps to eliminate the second difficulty.

The correlation studies reported are based on *in situ* studies at three airfields in Israel, involving more than 150 test pit locations under existing pavements (WISEMAN, 1959).

2. Description of soils

The soils encountered in the subsoil studies at the three sites were swelling type clays, of intermediate to high plasticity (Table 1). The natural moisture content was in general equal to or greater than the plastic limit. The degree of saturation was about 90 per cent.

Table 1

Classification tests. Airfield subgrades

Tests	Airfields		
	A	*B*	*C*
Atterberg Limits			
Liquid Limit (per cent)	39-54	48-62	65-80
Plastic Limit (per cent)	17-22	20-25	28-32
Plasticity Index (per cent)	19-32	31-41	37-51
Shrinkage Limit (per cent)	13	7-10	7-9
Specific Gravity of Solids	2·72	2·70	2·63
Mechanical Analysis			
Sand (# 4-# 200) (per cent)	4	5	0
Silt (# 200-5μ) (per cent)	46	32	43
Clay (< 5μ) (per cent)	50	63	57
Clay (< 2μ) (per cent)	40	55	46
Free Swell Test (per cent)	63-75	64-92	97-114

3. Sensitivity of the subsoil clays

By manual inspection the subsoil clays investigated appeared to be insensitive to remoulding. However, the sensitivity as expressed by the ratio of maximum to minimum shear strength at the end of the test as measured by the field vane test had an average value of 2 for clays of high plasticity. The sensitivity for the clays of intermediate plasticity ranged from 2 for undisturbed shear strengths of 1·0 kg per sq. cm to 4 for undisturbed shear strengths of 3·0 kg per sq. cm. The laboratory vane test on undisturbed samples showed sensitivities as defined above of an average value of 4.

These relatively high values for the sensitivity of a clay which upon manual inspection appeared to be insensitive leads to the belief that probably, for the relatively stiff clays tested and for these very shallow depths, the ratio of the maximum to minimum shear strength during a vane test is not a valid measure of the sensitivity of the clay to remoulding. The reason for the very low minimum shear strengths measured is probably the lack of intimate contact and transfer of normal pressure across the previously sheared cylindrical surface.

Through the use of the laboratory vane apparatus it is possible to express the sensitivity as the ratio between the shear strength of an undisturbed specimen and the shear strength of the same specimen when remolded and recompacted at the same water content to the same density.

Laboratory vane tests were performed in this manner on seven typical undisturbed samples, which ranged in shear strength from 1·0 to 2·0 kg per sq. cm. The lack of sensitivity of the clay was proved out by these tests in which the sensitivity ranged from 1·1 to 0·9 with an average value of 1·0. It is interesting to note that even for the remoulded specimens the ratio of the maximum to minimum shear strength in the vane test continued to be relatively high.

4. Penetration Tests as a Measure of Shear Strength

Static penetrometers have frequently been used to measure shear strength of both clays and sands. Various theories relate the ultimate bearing capacity of circular footings to the shear strength of clay. SKEMPTON (1951) in his work on the bearing capacity of saturated clays, showed that for circular footings, as the depth of penetration increased from zero, the ratio of bearing capacity to shear strength (Nc) increased from 6·2 to 9·0. For several years the authors have been using static penetrometers to evaluate the shear strength with clays of high degrees of saturation, having a natural moisture content of about the plastic limit. On several occasions when the static penetration resistance has been correlated in the field with vane tests, the bearing capacity factor (Nc) has been found to be about 10 for end bearing plates of areas ranging from 0·3 to 4·0 sq. cm of area. In Fig. 1 are shown the results of a statistical study

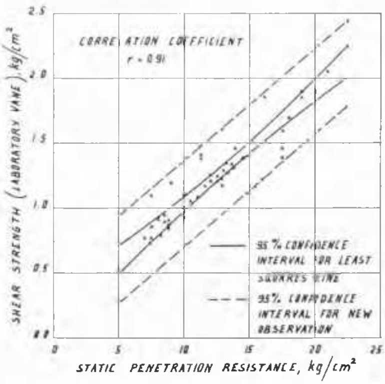

Fig. 1 Laboratory vane measurements plotted against static penetrometer figures.

Relation entre le scissomètre de laboratoire et le pénétromètre statique.

on undisturbed samples taken from the sites investigated by the authors. There is a high degree of correlation.

The CBR test when carried to failure may be considered as a test which also measures the penetration resistance of the clay subsoil and hence should also yield information as to the shear strength of the clay subgrade, using an appropriate bearing capacity factor $\left(\frac{D}{B} = 0{\cdot}25\,;\ Nc = 6{\cdot}8\right)$. The CBR tests performed by the authors were carried out at the standard rate. In nearly all cases the shape of the load deformation curve obtained was such as to indicate that bearing capacity failure had occurred at or before a penetration of 0·5 in. In each test pit location the shear strength by static penetration resistance was determined independently of the field CBR tests on the subsoil so that a comparison of the results from the two types of tests is possible, and indicate the influence of the rate of loading on the shear strength measured. The static penetration tests were performed quickly and the soil failed in less than one minute.

A statistical analysis of the test results in 89 test pits showed that the shear strength of the clay subsoil as measured by the penetration resistance in the CBR test is equal to 75 per cent of the shear strength as measured by the field static penetrometer.

In an attempt to evaluate independently the influence of rate of strain on the shear strength, a series of tests on laboratory compacted specimens was carried out. On these specimens the penetration resistance was determined using the CBR plunger at various rates of penetration. For each specimen the shear strength was also determined with the aid of a laboratory vane, failure occurring in less than 1 minute. The shear strength as measured by the penetration resistance was 95 per cent of the shear strength as measured by the laboratory vane when the rate of penetration was 0·5 in. min, whereas the shear strength when the rate of penetration was the CBR standard of 0·05 in. min was only 75 per cent of the shear strength as measured by the laboratory vane.

Fom the above and other laboratory studies it was concluded that the difference between the shear strength of the clay subsoil as determined from the field CBR test and the shear strength as measured by the field static penetrometer, is essentially due to the difference in the rate of loading in the two tests.

5. Relationship between in-situ CBR and in-situ Shear Strength

The theory of elasticity gives an expression for the settlement of a rigid circular plate as a function of the contact stress, radius of loaded area, Poisson Ratio, and the modulus of elasticity. It may be assumed that for clay subsoils, at a penetration of 0·1 inch in the CBR test, the soil still behaves approximately as an elastic medium, with a Poisson's ratio of 1/2. The corresponding expression for the CBR number may be written as follows :

$$\text{CBR} = 0{\cdot}127E \text{ where } 0{\cdot}127 \text{ has units of sq. cm per kg} \quad \ldots\ldots (1)$$

and

$E =$ the modulus of elasticity in kg per sq. cm.

Introducing the shear strength of the subgrade, s, the following expression is obtained :

$$\text{CBR} = C_1 s \qquad (2)$$

where

$$C_1 = 0{\cdot}127\,\frac{E}{s}\ \text{(sq. cm/kg)} \qquad (3)$$

From the above it can be seen that if for a range of consistancy of the clay the ratio of the modulus of elasticity to the shear strength of the clay is constant, it is reasonable to assume that C_1 is also constant. C_1 may be evaluated by determining either $\frac{\text{CBR}}{s}$ or $\frac{E}{s}$.

In using the above expression attention must be paid to compatibility of rates of strain. The problem of the influence of rate of strain has already been discussed or the rate of loading on the shear strength and it was seen that increased rates of loading gave rise to increased strength. It has been shown in an extensive investigation by A. Casagrande and W. L. Shannon (1948) that increased rate of loading gives rise to increased modulus of deformation. When dealing with equation (2) attention must be paid to the rates of strain.

In this investigation several rates of strain have been used and in the following expressions these have been designated as subscripts giving approximate time to failure.

Using the shear strength and CBR data available, C_1 was obtained from Eq. 2 for times to failure of one minute and 10 minutes and the corresponding values of $\frac{E}{s}$ calculated.

Independently of the above data, $\frac{E\,(1\text{ min})}{s\ (1\text{ min})}$ has been evaluated from unconfined compression tests on undisturbed samples. The shear strength was taken as one half of the unconfined compressive strength and the modulus of elasticity was calculated at $2\frac{1}{2}$ per cent strain (corresponding to 0·1″ penetration of a two-inch diameter footing, Skempton (1951)).

The following conclusions may be drawn from these studies :

(a) The ratio of the modulus of elasticity to the shear strength, both determined by unconfined compression tests in which the time to failure is one minute, is approximately constant and equal to about 40

$$\frac{E\,(1\text{ min})}{s\ (1\text{ min})} = 40.$$

(b) The value of the CBR divided by the shear strength as measured by either the static penetration test or the vane test is equal to 4·0 sq. cm per kg. Both vane test and static penetration tests were carried out to failure in approximately 1 minute. Hence,

$$C_1 = \frac{\text{CBR (10 min)}}{s\,(1\text{ min})} = 4{\cdot}0 \text{ sq. cm per kg (Fig. 2)}$$

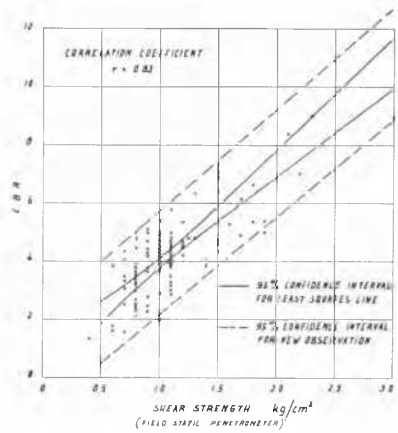

Fig. 2 California bearing ratio plotted against shear strength.
Rapport entre le sondage de Californie et la contrainte.

(c) The value of the CBR divided by the shear strength as determined by the CBR at 0·5 in penetration is equal to 5·4 kg sq. cm

$$C_1 = \frac{\text{CBR (10 min)}}{s\,(10\text{ min})} = 5{\cdot}4 \text{ sq. cm per kg (Fig. 3)}$$

Substituting the above values of C_1 in the equation (3) we obtain

$$\frac{E\,(10\text{ min})}{s\,(1\text{ min})} = 31$$

$$\frac{E\,(10\text{ min})}{s\ (10\text{ min})} = 43$$

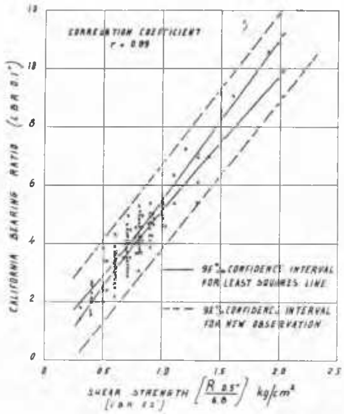

Fig. 3 California bearing ratio plotted against shear strength.
Rapport entre le sondage de Californie et la résistance au cisaillement.

which we see are consistent with the influence of rate of strain, and fulfill the logical conditions :

$$\frac{E\,(10\text{ min})}{s\,(10\text{ min})} = 43 > \frac{E\,(10\text{ min})}{s\,(1\text{ min})} = 31$$

$$\frac{E\,(10\text{ min})}{s\,(1\text{ min})} = 31 < \frac{E\,(1\text{ min})}{s\,(1\text{ min})} = 40$$

$$\frac{E\,(1\text{ min})}{s\,(1\text{ min})} = 40 \approx \frac{E\,(10\text{ min})}{s\,(10\text{ min})} = 43$$

For the clays tested, the use of a coefficient, C_1, for particular rates of strain, is justified analytically, as well as experimentally.

6. Relationship between CBR required and Maximum Shear Stress Induced

G. Wilson and G. M. J. Williams (1950) compared the shear stresses in the subsoil beneath a flexible pavement given by the Boussinesq theory with those given by the theory of layered systems. They found that at $h_t/a \geqslant 2{\cdot}5$ (a = radius of loaded area, h_t = total thickness of pavement) both the Boussinesq and the layered system theory gave the same results, and that at $h_t/a = 1{\cdot}5$, the theory of layered systems gave shear stresses which were 90 per cent of those given by the Boussinesq theory. They therefore concluded that "flexible pavements behave very much according to the Boussinesq theory".

By combining the CRB equation given by Turnbull and Ahlvin (1957) for (CBR values less than 12) with the equation for maximum shear stress (τ, kg per sq. cm) given by Boussinesq for the same loadings the following equation is obtained :

$$C_2 = \frac{\text{CBR}}{\tau_{\max}} = 7{\cdot}63 \times \left[1 + \left(\frac{a}{ht}\right)^2\right]^{1/2} \left(\frac{\text{sq. cm}}{\text{kg}}\right) \quad (4)$$

and is plotted in Fig. 4 for various h_t/a values. Since for clays h_t/a will usually be more than two, it may be noted that C_2 is approximately equal to 8, and hence the CBR required is numerically equal to 8 times the maximum shear stress.

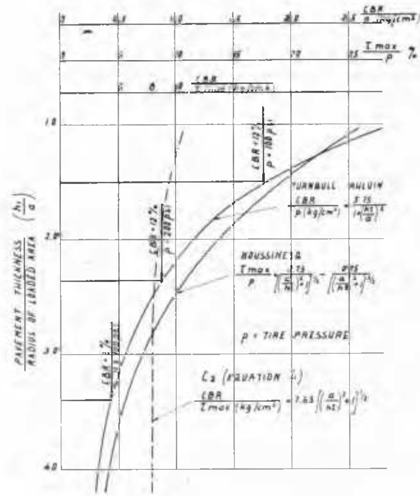

Fig. 4 CBR and shear stress in relation to pavement thickness.
Le CBR et la résistance au cisaillement pour le calcul des chaussées.

7. Overstress Ratio

Let the Overstress Ratio be define as the ratio of the maximum shear stress induced in the subsoil to its shear strength.

$$\text{Overstress Ratio} = \frac{\tau_{\max}}{s} \quad (5)$$

We may therefore write for the clays tested

$$\text{Overstress Ratio} = \frac{\dfrac{\text{CBR}}{C_2}}{\dfrac{\text{CBR}}{C_1}} = \frac{C_1}{C_2} = \frac{4}{8} = 0{\cdot}5$$

The CBR required, which was used in evaluating C_2, was for full capacity operations. For a given CBR the full design thickness (h_t) of flexible pavement may be reduced as the total number of coverages for which the pavement is designed becomes less than that associated with full capacity operation. The Overstress Ratio may be shown as a function of the pavement thickness, expressed as the percentage of the thickness determined by the CBR expression for full capacity operation. This percentage is plotted in Fig. 5, as a function of the Overstress Ratio for values of C_1 = 3, 4, 6 and 8 sq. cm per kg.

In interpreting the Overstress Ratio the following considerations must be kept in mind :

(*i*) The shear strength was determined *in situ* with a time to failure of approximately one minute. The

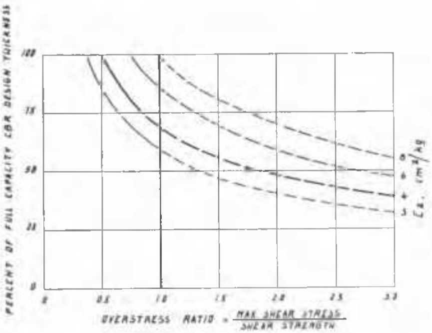

Fig. 5 Pavement thickness related to overstress ratio.
Rapport entre le calcul des chaussées et l'excédent de contrainte.

conditions of loading of airfield pavements range from transient, during landing and take off, to static long time loading at parking areas. For transient loadings the shear strength tends to be higher, whereas for long time loading the shear strength tends to be lower, than the shear strength as determined above.

(*ii*) We have until now considered only one load application. However, airfield pavements are subjected to repetitive loading, which is usually associated with changes in strength and deformation characteristics for most clays.

(*iii*) The Overstress Ratio as used here is for local overstressing in shear and when this ratio is unity the factor of safety against rupture is still considerable. It can be shown that for an overstress ratio of unity the factor of safety against rupture is a function of $\left(\frac{h_t}{a}\right)$. Results of calculations are shown in Table No. 2.

Table 2

Factor of Safety against rupture as related to Pavement Thickness

$\frac{h_t}{a}$	1·5	2·0	3·0	4·0	5·0
Factor of safety (rupture)	4·1	3·7	3·1	2·7	2·4

From an examination of Table 2, it can be seen that for the thinner pavement structures, when the overstress ratio for local overstressing in shear is unity the factor of safety against rupture is higher than it is for the thicker pavements.

(*iv*) Overstress ratios greater than unity cease to have precise meaning since for maximum local shear stresses greater than the shear strength a redistribution of shear stresses must take place and the theory of elasticity which was used for calculating the maximum shear stress is no longer applicable. The zone of plastic equilibrium increases as the amount by which the overstress ratio exceeds unity increases, with a corresponding decrease in the factor of safety against rupture.

Referring to Fig. 5 for the clays tested (i.e. $C_1 = 4{\cdot}0$ sq. cm per kg) if the pavement is designed for full capacity operation by the CBR method, the overstress ratio is equal to a half. It may be noted that the shear strength may be reduced to as little as a half (say, under repetitive loading) and the resulting value of $C_1 = 8$ sq. cm per kg would still show an overstress ratio of unity. For intensitive clays studied by the authors such a drop in shear strength appears to be unlikely and it is therefore indicated that pavement thickness may be reduced to 70 per cent of that given by the design curves. Hence, a more logical way of approaching thickness requirements would be to start from considerations of rupture under a single load application and increase the thickness required, according to the loss of strength expected under repetitive loading for clays of varying degrees of sensitivity. It is considered logical to design for an overstress ratio of one, on the basis that excessive deformations would not occur if there were no local overstressing in shear.

If the pavement is designed for emergency use by the CBR method, the U.S. Corps of Engineers allows a 50 per cent reduction in pavement thickness. This corresponds, for $C_1 = 4$ sq. cm per kg, to an overstress ratio of two and probably corresponds to a factor of safety against rupture of from 1·2 to 2 for values of $\frac{h_t}{a}$ of from 5 to 1·5.

It is appreciated that, since the value of C_1 and the degree of sensitivity is not available for the subsoil clays studied during the development of the CBR design curves, the value of unity for the overstress ratio as suggested above needs to be confirmed by further research.

References

[1] GLOSSOP, R. and GOLDER, H. Q. (1948). The shear strength method of the determination of pavement thickness. *2nd Int. Conf. S. M., IV*, p. 164.

[2] SCALA, A. J. (1956). The use of cone penetrometers in determining the bearing capacity of soils. *Proc. 2nd Australian-New-Zealand Conf. S. M.*, p. 73.

[3] ROBINSON, P. J. M. and LEWIS, T. (1958). A rapid method of determining in-situ CBR values. *Geotechnique*, VIII, June, p. 72.

[4] TURNBULL, W. J. and AHLVIN, R. G. (1957). Mathematic expression of the CBR relations. *Proc. 4th Int. Conf. S. M.*, p. 178.

[5] WISEMAN, G. (1959). A study of expansive clay subgrade under airfield pavements. *Doctor of Science Thesis*, submitted at the Israel Institute of Technology.

[6] SKEMPTON, A. W. (1951). The bearing capacity of clays. *Building Research Congress*.

[7] CASAGRANDE, A. and SHANNON, W. L. (1948). Research on stress deformation and strength characteristics of soils and soft rocks under transient loading. *Harvard University, Soil Mech.*, series, No. 31.

[8] WILSON, G. and WILLIAMS, G. M. L. (1950). Pavement bearing capacity computed by theory of layered systems. *Proc. A.S.C.E.*, 76, Separate No. 16, p. 15.