Foundations on Swelling Clays

Fondations sur Argiles Gonflantes

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Summary

The general lines for the design of a foundation on swelling clay are given. These were studied to decide whether the soil is dangerous and it was concluded that test measurements of swelling pressure and free swelling under constant load are indispensable. From the curves of the swelling pressure *versus* allowed swelling the probable heaving of a foundation can be calculated, but it has to be observed that this must only be taken as an indication. Constructive measures are indicated and formulae for the calculation of the strain in the structure are given. The favourable effect of a deformable layer under the foundation is shown. Reference is also made to the variation of the volume of the soil with the temperature, without change of humidity, which it is believed has not been reported before. However, it is shown that this phenomenon has little practical importance in this case.

Sommaire

On donne des directives générales pour l'étude des fondations sur argiles gonflantes. On étudie les méthodes pour reconnaître si un sol est dangereux en concluant que l'essai de la mesure de pression de gonflement et de gonflement libre sous une pression définie est indispensable.

A partir des courbes pression de gonflement–gonflement permis on peut calculer le soulèvement probable d'un fondation, mais on fait remarquer que cela ne peut se faire qu'à titre d'indication. On indique des mesures constructives et on donne des formules pour calculer les efforts dans la structure. On démontre l'effet favorable d'une couche déformable sous la fondation. On parle également de la variation de volume du sol sous l'effet de la température, sans changement deteneur en eau, au sujet de laquelle on croit qu'il n'y a pas eu de publication antérieure. Cependant, on démontre que dans le cas qu'on étudie, cet effet n'à aucune importance.

The phenomenon of cracking of structures founded on clay with great volume changes is found in many countries, e.g. Burma (Wooltorton, 1936), South Africa (Jennings, 1950), U.S.A. (Dawson, 1953), North Africa (Delarue, Mariotti and Berthier, 1953), Cuba (Tschebotarioff, 1953; Pulido Morales, 1953) and Spain (Salas, Serratosa and Sobreviela, 1955; Salas, 1955; Vizcaino and Oliveros, 1955; De la Hoz, 1956).

It occurs more usually, or with a greater gravity, in arid climates. Kantey and Brink (1952) established that the phenomenon occurs when there is a combination of (a) a clay profile, (b) a low water table, (c) a condition of desiccation in the soil.

However, this assertion is not completely exact and it refers to a particular form of producing the phenomenon. It is also found in a somewhat different way in a wet climate like that of the United Kingdom (Cooling and Ward, 1948; Ward, 1953).

For the form of presentation of the phenomenon, as we have already indicated elsewhere (Salas, Serratosa and Sorrieviela, 1955), the relation between evaporation in a free water surface and rainfall is fundamental.

Bare earth loses humidity by evaporation to approximately the same degree as a surface of free water, but on reaching a certain point of humidity, called the 'break point' this speed of evaporation decreases greatly. The 'break point' is situated, according to Terzaghi (Altmeyer, 1955), in the proximity of the plastic limit. From this one may deduce that when meteorological conditions lead to long periods of time during which evaporation on a free surface of water is greater than the precipitation, the soil takes on levels of moisture around the plastic limit, but in this state the progress of drying is very slow.

There are two principal forms which are presented by the phenomenon: seasonal, as observed by Cooling and Ward (1948), or a steady rise of the terrain, although mixed with a seasonal effect (Jennings, 1950). The difference between the two forms may be related to the ratio between free surface evaporation and annual rainfall. When the first one is several times greater than the second one (as happens in the greater part of Spain) then there exists a permanent moisture deficiency in the ground, and the building of a structure produces a permanent swelling, which corresponds to the new state of equilibrium in which the evaporation has remained limited by the protection of the structure.

Steps for the Study of the Design of a Foundation on Swelling Clay

There are three: (a) recognition of the swelling properties of the foundation soil, (b) prediction of the volume changes which are going to be produced, (c) constructive measures.

Recognition of the swelling properties of foundation soil—In the bibliography are found some tentative criteria to determine, by means of very simple tests, whether a clay is expansive. A very high liquid limit is sometimes a sufficient condition, but not necessarily (Kantey and Brink, 1952). According to Altmeyer (1955) the shrinkage limit is a better indication, in the following form:

Shrinkage limit	10	10–12	12
Degree of volume change	Critical	Marginal	Non-critical

On the other hand, Holtz and Gibbs (1954) who also think the shrinkage limit and the plasticity index of value, also use the free swell value. Above 100 per cent the soil is dangerous. Under 50 per cent the soil very seldom exhibits appreciable volume changes, even under very light loadings.

We ourselves make use of the criterion of the swelling pressure and the swelling under a 1 lb./sq. in. pressure (referred to in the following as *S.P.* and S_1 respectively). The value of 1 lb./sq. in. has been chosen as a term for comparison with the results of Holtz and Gibbs (1954). These tests are not very simple, but on the other hand their results give a very precise idea of the swelling properties of the ground, and, at the same time, they give very useful facts which can be employed in the successive steps for the design of the foundations.

The swelling pressure is measured in a consolidometer, the piston of which is kept in a fixed position by means of a proving ring in an arrangement very similar to the one described by Palit (1953). However, it was found that the deformation of the ring, even if very small, produced a notable error and for that reason the apparatus was put on a bench provided with a hand-operated screw (a typical strain-controlled triaxial test bench). In this way the deformation of the ring could be compensated and the piston could be kept in a fixed position with a precision of almost 1/1000 mm. Taking this precaution the height of the sample has been found to have no influence at all on the pressure, contrary to that found by PALIT (1953).

In the same way, by means of this arrangement, successive swellings of the sample can be permitted and in each case the pressure can be measured, thus obtaining the curve *S.P. versus* allowed swelling. Fig. 1 shows several typical results.

The swelling under 1 lb./sq. in. pressure (S_1) is measured simply in a consolidometer.

Interpolating or extrapolating the curves of Fig. 1 one can find the corresponding value of 1 lb./sq. in. of pressure. Nevertheless, this value does not coincide with the value of S_1

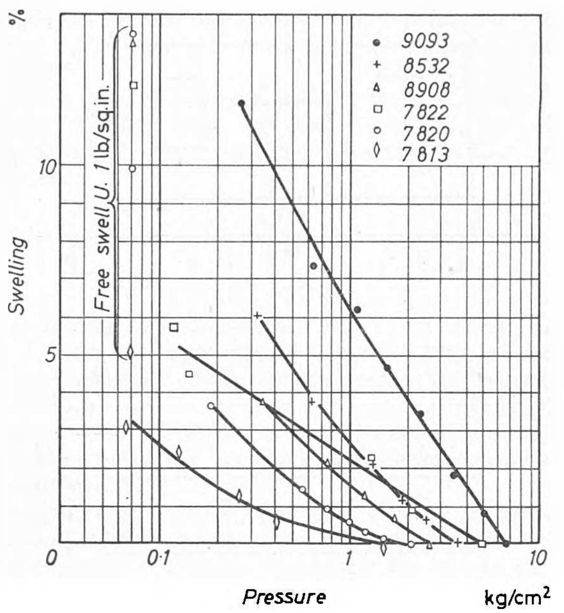

Fig. 1 Swelling pressure *versus* allowed swelling. The numbers of the samples correspond with those of Table 1

Pression de gonflement/gonflement permis. Les numéros des échantillons correspondent à ceux de la Table 1

determined directly. This may be attributed to a variation in the structure of the soil during the initial period of the controlled swelling, when the pressure is high. The relation between both values might be perhaps an indication of the sensitivity of the structure to the action of the water. Further, it has sometimes been proved, while measuring the *S.P.* for the null swelling, that the pressure falls a little if one allows several days to pass by, which also has to be attributed to a deterioration of the structure, called 'slaking' by WOOLTORTON (1954).

Even when we have said that we prefer to make use of the criterion of *S.P.* and of S_1 for judging the possible volume change of a soil, we have taken data in many cases for the use of the other criteria referred above, with the results shown in Table 1.

Table 2 gives some data on the soils of Bornos (Cádiz, Spain) where the structural damages originated by the heaving are actually very great. Nevertheless, there are no available data on the swelling pressures in this area, up to the moment.

Causes of the swelling—The swelling properties of the clay can only be explained by applying to physicochemical conceptions (SALAS, SERRATOSA and SORREVIELA, 1955; SALAS and SERRATOSA, 1953; SERRATOSA, 1953; ROSENQVIST, 1955; BOLT,

Table 1

Sample No.	*Site*	S_1%	*S.P.* kg/cm²	*S.L.*	*L.L.*	*P.I.*	*Free swell value* %
9·093	Ecija	22·6	6·80	10·4	66·6	40·6	—
8·542	Ecija	—	5·24	9·8	53·1	23·2	—
8·532	Ecija	13·5	3·80	10·2	53·9	30·2	—
8·908	Ecija	13·2	2·87	7·9	58·6	37·9	—
7·822	Morón	12·1	5·01	12·1	49·1	28·1	80
7·820	Morón	9·9	2·02	11·2	53·4	33·4	70
7·819	Morón	6·3	0·48	12·2	46·7	26·4	60
7·813	Morón	5·7	1·60	10·5	42·0	23·2	70
9·150	Heliópolis	5·7	1·53	—	58·0	38·6	—
7·815	Morón	4·4	0·96	12·0	40·5	28·5	60
10·266	Oropesa	—	0·81	12·8	—	—	—
8·919	Ecija	3·4	—	—	30·3	15·9	—
7·823	Morón	2·5	0·55	10·4	42·4	21·0	70
7·814	Morón	2·3	0·25	11·5	37·5	23·4	60
7·817	Morón	1·8	0·55	14·4	30·9	15·4	50
9·322	Ecija	1·0	0·16	12·2	67·9	41·3	—
6·512	S. Pablo	0·0	0·0	11·5	42·0	21·7	—
5·692	Cornatel	0·0	0·0	—	26·9	11·0	—

Table 2

Sample No.	*S.L.*	*L.L.*	*P.I.*
10·102	—	70·0	45·3
10·103	19·1	102·8	60·5
10·104	13·0	57·0	32·7
10·108	12·4	65·4	45·0

1956). The purer the clay, the clearer this is shown. According to BOLT (1956) the very pure clay shows a swelling index C_1 very near to the compression index C_s, which corresponds to the physicochemical explanation of the phenomenon, but the values are every time going to be more different in clays which contain increasing proportions of coarse particles.

The phenomenon of swelling is exhibited not only by clay soils, but also by soils which contain a large quantity of organic matter (BICZOK, 1955) in a similar form to the clayey soils.

We must add that some observed phenomena of swelling may be due to phenomena of a chemical character, such as the transformation of anhydrite into gypsum (VIZCAINO and OLIVEROS, 1955).

In spite of this, however, it seems established that the swelling corresponds perfectly in most cases to the change of the moisture content in the soil, a change which alters the equilibrium between the physicochemical forces which tend to repel the particles and the capillarity forces which tend to attract them.

Swelling due to the cooling of the foundation soil—We cite, however, an effect which as far as we know has not been previously published. A change of temperature in the soil changes the vapour pressure in its pores and by that its capillary potential (EDLEFSEN and ANDERSON, 1943; GARDNER, 1955). This change produces a volume variation which has been proved experimentally by us. The experimental result coincides in order of magnitude with that calculated from the theoretical change of the capillary potential and the compressibility of the clay. However, the magnitude of this effect is not important in practice. The clay with which this experiment has been made dilated 9×10^{-3} per cent of its height, for every one degree reduction of the temperature. The conclusion of our testing was, that it is not absolutely necessary to take this effect into account in the cases we study.

Prediction of the volume changes—We divide this study into two steps. The first is the calculation of the swelling which would be produced if the site was inundated in a permanent way, which we call 'swelling under flooded conditions'.

This value can be estimated by calculating first the distribution in the soil of the pressure produced by the foundation plus those due to the overburden, and then using curves like those of Fig. 1.

By means of trial and error, and varying the dimensions of the foundation footings, it is possible to design a foundation which has no swelling. This foundation will be the most convenient one, but, it has not finished with the problem. In effect, the variation of the swelling pressures in an area is quite big, due to the minor differences of the moisture content from one point to another. They also vary from one season to the other, and therefore you have to take care to effect the sampling during the dry season.

However, there is no guarantee that the foundations which have been calculated as indicated in the preceding paragraph do not suffer movement, but, evidently, they will be much smaller than if no calculations had been made. Therefore, it is impossible to disregard the application of constructive measures, such as we are going to detail later.

For the design of normal foundation the consideration of the case of the 'flooded condition' is sufficient, but there exist other types, such as pavements, sidewalks, etc., in which it is frequently necessary to tolerate certain movements. To adopt the 'flooded condition' in this case would be too pessimistic. It is necessary to predict first the change of humidity that the soil is probably going to suffer. In wet climates the works of the Road Research Laboratory (Croney, 1952) give us a sure guide for that, but in dry climates, as we have detailed elsewhere (Salas, Serratosa and Sobreviela, 1955), the estimation is more difficult and it is necessary to apply the concept of the local equilibrium.

Regarding the change of humidity, which, in dry climates, can only be estimated very roughly, it is necessary to consider the variation of volume which this change represents. For that we have used up to now the shrinkage curves of the soil, even when perhaps better results could be obtained with the apparatus designed by de Bruijn (1955).

Constructive measures—According to Jennings (1950) in areas where there is the danger of movements due to the swelling of the foundation, there are three solutions: (a) rigid buildings, independent of movements; (b) flexible buildings, tolerating the movements; (c) rigid buildings, tolerating the movements. We add to this a fourth class, which is: rigid buildings on a deformable foundation.

Class (a) consists in its typical form of buildings with a foundation on piers, belled-out piles, etc., with suspended floors. The solution is secure, but expensive. We are not going to deal with this here.

Class (b) consists principally of different systems of dry-wall construction. The third class (c) is very interesting in the case of small and light buildings, which are more intensely affected by this problem. It consists in converting the building into a resistant box, which can be lifted by the soil without cracking. A complete investigation has been made about this matter in South Africa (Rigby and Dekena, 1951), studying buildings which were all erected with reinforced concrete and also with reinforced brick. This second solution seems to be particularly interesting, and, basing it on the South African experiments, we intend to use it in Spain, but at the moment we have to state that in Spain the buildings have been made, for a long time of bricks or stone-work, with two cinctures made of reinforced concrete. One of these cinctures is simply the beam of the strip foundation. The other one is in the upper part of the walls, in the supporting line of the roof. This has had generally a good result, although it is certain that the solution is not so perfect as the one proposed by Rigby and Dekena (1951) as it has no reinforcements around the voids of the doors and the windows, where cracks sometimes appear through the concentration of stress.

Reinforcement calculation—A fundamental point is the reinforcement calculation. The South African authors consider the wall of a building as a beam subjected to a bending moment:

$$M = \frac{KWl^2}{2} \qquad \ldots . (1)$$

where W is the weight of the wall, plus all the loads which rest on it, by the unit of length, l is half of the length of the wall (Fig. 2a) and K an empirical coefficient.

Evidently, case $K = 1$ corresponds to the extreme case represented in Fig. 2a. The values which are normally applied in South Africa range from $K = 0{\cdot}8$ to $K = 0{\cdot}5$, according to the sites.

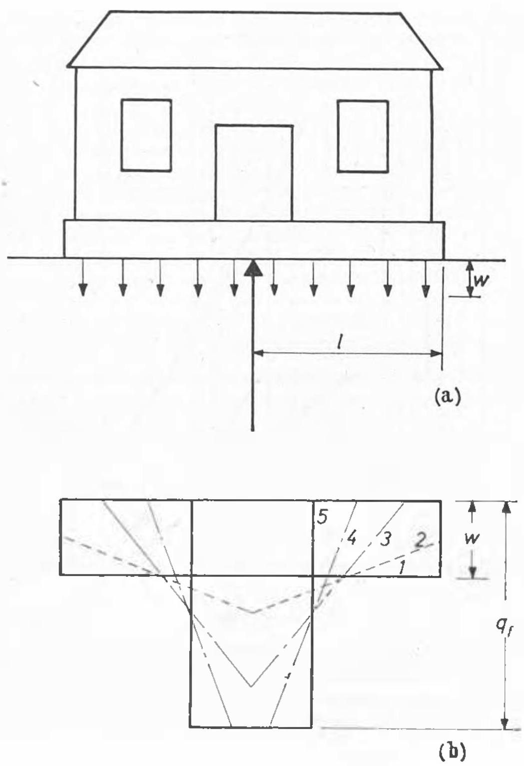

Fig. 2 (a) The central isolated sustaining force that is in the origin of the South African formula. (b) Evolution of pressures under the foundation during heaving. The pattern is determined by the ultimate load q_f.

(a) La force portante insolée qui est à la base de la formule Sud-Africaine. (b) Evolution des pressions sous la fondation au cours du soulèvement. L'allure des pressions est déterminée par la charge de rupture.

Although this is a very sound approach to the problem, let us try now to form a clearer picture of the bending moment at the wall.

Initially after construction, the pressure on the ground is uniformly distributed, and equals $w = W/b$, b being the width of the foundation (Fig. 2b).

With the beginning of the heaving, which is greater in the centre than in the corners, the pressure in the centre will increase, following roughly the patterns of Fig. 2. In the last three it is supposed that the heaving in the centre is so great that the corners separate themselves from the ground.

An important fact is that the shape of the distribution is related to the ultimate bearing capacity of the ground. That is why the triangular distribution is supposed to evolve into a rectangular one. In this case the bending moment is:

$$M = \frac{wl^2}{2}\left(1 - \frac{w}{q_f}\right) \qquad \ldots . \ (2)$$

From these results, the value of K in the South African formula can be calculated, and it is equal to

$$K = 1 - \frac{w}{q_f} \qquad \ldots . \ (3)$$

Knowing the value of the ultimate load of the foundation soil, we can estimate the maximum value of K. Nevertheless, we have to keep in mind that it is not easy to know the value of q_f, not only because of the uncertainty which exists regarding the calculation of the ultimate load, but also because of the fact that the humidity of the clay increases to a degree which we do not know exactly.

The problem of the resistance variation of the clay with the humidity is also very complex, but we have here no room to consider this.

It follows from the formula that the value of K depends on the factor of safety which we wish to use with the ultimate bearing capacity. From the values of K used in South Africa we obtained:

$K =$	0·8	0·5
$w/q_f =$	0·2	0·5
Factor of safety $=$	5	2

Rigid buildings on deformable foundation—As has been proved in Fig. 1, when swelling is permitted, even slightly, the pressure will fall considerably. This suggests the idea that the coefficient K in the South African formula could be much reduced if it were possible to introduce between the foundation and the ground some elastic or plastic element, which would permit some deformation.

In Andalusia in the areas of swelling clay the practice exists of introducing between the soil and the strip foundation a layer of lime concrete or mortar. Experience has proved that this practice is effective, but not completely so. Its origin is not known, and it is used empirically. Probably its efficacy is due to the fact that lime is very slow setting, and conserves a certain plasticity during months or years. It has also been thought that it might have some beneficial effect from ion exchange, but this is not possible, as almost all the swelling clays in Andalusia are calcium saturated.

The deformable layer can also be made with cinders, rubber, cork, asphalt, etc., but up to now we have not tested these materials.

We can estimate, however, the reduction in the value of K which would be produced by a layer of this material.

At first it would be necessary to calculate, as before, the probable swelling of the surface of the soil in contact with the foundation, in conjunction with the applied pressure, on the basis of the curves of the type of those in Fig. 1. The curve obtained can be represented, in the field of high pressures, by a logarithmic curve. For that we can write:

$$\rho = C \log_{10} \frac{S.P.}{p_1} \qquad \ldots . \ (4)$$

ρ being the swelling, p_1 the pressure on the foundation area and C a coefficient which is deduced empirically from the calculated curve.

When the foundation is laid, the pressure on the terrain will be initially uniform and equal to w. After some time, let us suppose that the corners have not shown any tendency to swelling (an assumption which falls on the safe side) but only the soil situated below the centre of the wall. The distribution of the pressure will have changed and it will be p_1 in the centre and p_2 below the corners.

In the centre there will have been a swelling ρ which will have been produced by the deformability of the elastic layer in between.

The deformation of this layer, supposing that the law of Hooke is followed, might be written:

$$s = \kappa p \qquad \ldots . \ (5)$$

s being the deformation of the layer, p the pressure on the same and κ a coefficient which depends on the thickness and the material out of which this layer has been made.

Then we have:

$$\rho = s_1 - s_2 = \kappa(p_1 - p_2) = C \log_{10} \frac{S.P.}{p_1}$$

When we suppose that the lineal distribution of the pressure, is as in Fig. 2b (formulae for other distributions have been developed; they are more complicated and, until other researches are made, there is no guarantee that they are more exact), we have also, for equilibrium:

$$\frac{p_1 + p_2}{2} = w$$

From which

$$C \log_{10} \frac{S.P.}{p_1} = 2\kappa(p_1 - w) \qquad \ldots . \ (6)$$

the equation from which p_1 can be calculated, and later, p_2.

Knowing p_1 the bending moment M in the centre of the wall can be calculated, and is:

$$M = \frac{(p_1 - w)l^2}{6} \qquad \ldots . \ (7)$$

From which it follows that the value of K corresponding to this case, in the South African formula, is:

$$K = \frac{p_1 - w}{3w} \qquad \ldots . \ (8)$$

In order to give an idea of the great reduction in the bending moment which can be produced by the presence of a deformable layer, we give some values:

$C = 0{\cdot}1505$ cm (observed); $\kappa = 0{\cdot}2$ kg/cm^3 (corresponding to a layer of cork agglomerate of 10 cm thick and $E = 50$ kg/cm^2).

$S.P. = 4$ kg/cm^2 (observed); $w = 2$ kg/cm^2.

In these conditions the value of K is shown to be only 0·063.

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