Pressures Exerted by Clay Soil on Buried Conduits

Pressions de gonflement sur des conduites souterraines

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Summary

Experience in Israel on the failure of piling, buried conduits, and other underground structures in clay has emphasised the need for research on the pressures exerted by such soils in the field. It is accepted that appreciable pressures result from the swelling of fat clays when subjected to moisture changes by irrigation or seasonal variations. The prediction of these pressures can contribute to the design of stable structures on clay or other expansive soils.

In conjunction with a research on stresses induced in pipes by a swelling clay an hypothesis was evolved for the determination of the maximum vertical swelling pressure exerted on a structure by the subgrade. According to the theory advanced by the authors, the maximum swelling pressure develops at sections along the structure where maximum restraint is imposed against its movement. As a result the structure moves less than the subgrade next to it. A method is suggested for prediction of the subgrade swelling pressure by determining the amounts of movement of the structure, the adjacent soil, and the difference between the two. This method uses the results of several laboratory swelling tests of a clay sample, in which (a) the maximum swelling is obtained under no load and (b) the maximum swelling pressure develops when the sample is restrained from movement.

The authors' theory was examined in the light of results obtained in a field study, in which observations were made on the longitudinal deformations and vertical movements of two pipelines buried in clay under two different moisture-density conditions and exposed to seasonal changes as well as to irrigation. The longitudinal strains were interpreted in terms of moments, and the distribution of load along the pipes was established. A reasonable correlation was found between the theory, the loads developed in the field and the results of laboratory swelling tests.

Sommaire

L'expérience sur les constructions souterraines en Israel a démontré la nécessité d'étudier la pression des argiles gonflantes sur ces constructions. Des pressions considérables se produisent et sont dues aux changements de la teneur en eau des argiles grasses. La prédiction de ces pressions pourrait permettre d'augmenter la stabilité des constructions situées sur des sols gonflants.

On a développé une hypothèse pour la détermination des pressions de gonflement maximum. Suivant la théorie émise, la plus grande pression s'applique aux endroits de mouvement minimum.

La méthode de prédiction utilise une série d'essais de laboratoire pour déterminer :

(a) le gonflement maximum sans surcharge ;

(b) la pression maximum de gonflement de l'échantillon à volume constant.

La théorie fût controlée par la mesure in-situ des mouvements verticaux et des déformations longitudinales de deux conduites souterraines.

La répartition des moments et des contraintes fut étudiée et une relation plausible avec les résultats des essais de laboratoire fut trouvée.

Introduction

It is widely accepted that appreciable pressures result from the swelling of fat clays subjected to moisture changes from irrigation or seasonal variations. These pressures have been measured almost exclusively in the laboratory (GOLDBERG and KLEIN, 1952; HOLTZ and GIBBS, 1954; SALAS and SERRATOSA, 1957; ALPAN, 1957, etc.), whereas relatively little work has been reported in the determination of swelling pressures under actual field conditions (DI BIAGIO and BJERRUM, 1957).

Experience in Israel with failure of piling, buried conduits and other underground structures in desiccated clays has emphasised the need for research on the pressures exerted by such soils in the field (ZEITLEN and KOMORNIK, 1960; WISEMAN, 1959; KASSIFF, 1960). It was felt that the prediction of these pressures can contribute towards the design of stable structures erected on expansive clays.

The problem of predicting the swelling pressure exerted on a structure is extremely complicated. Most clay soils are not homogeneous and anisotropic so that at various points within an apparently uniform body there may be different and unpredictable pressures. In addition, the swelling pressure which develops under or adjacent to a structure depends on the extent of restraint to which the movement of the clay is subjected. The highest pressures are produced by complete restraint. The quantitative relationship between the amount of swelling under a structure in the field and the corresponding swelling pressure is usually unknown.

The authors review their research into this problem in conjunction with a study on stresses induced in pipes by a swelling clay (KASSIFF, 1960) and a hypothesis was developed for determining of the maximum vertical swelling pressure exerted on a conduit by a clay subgrade. The theory was examined in the light of field and laboratory investigations and found to be valid within certain limits.

Theory

A section of a conduit buried in clay in its initial location is given in Fig. 1 (a). At this stage the pressures acting on the pipe are in equilibrium. With the access of moisture to the clay near the pipe, swelling pressures develop which cause vertical heave. It has been demonstrated in the laboratory that maximum swelling pressure occurs in a clay sample when the sample is completely restrained from heaving. Conversely, if the sample is allowed to move freely, maximum swelling is obtained. Under intermediate loads, intermediate swelling occurs and the pressure-swell curve assumes a hyperbolic form as indicated in Fig. 1 (b).

Accordingly, under field conditions, the subgrade adjacent to the pipe heaves an amount, Δ_v, corresponding to the overburden pressure exerted by the fill (Fig. 1 (a) and 1 (b)). However, under the conduit itself there is a restraint to the vertical movement because of

1. the overburden and the pipe loads, p_v, and

2. an additional pressure, p_x, introduced by arching action of the backfill and longitudinal restraint of the pipe at locations of more stable soil. Hence, the subgrade directly under the conduit will heave less, by Δ_x, than in the sides, and the total heave under the pipe will be Δ_v-Δ_x. As a result, vertical swelling pressure exerted on the conduit will be greater than the overburden pressure, and will be equal to $p_s = p_v + p_x$.

The behaviour of the pipe in the field is described in Fig 1 (a) by the dotted lines. The difference in heaving between the subgrade directly below the pipe and the soil next to it starts

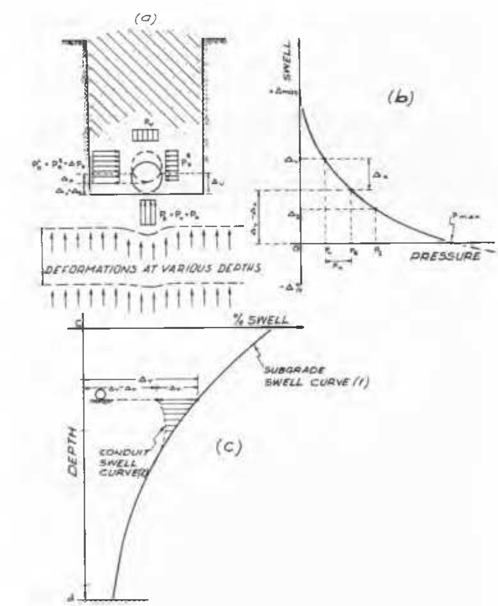


Fig. 1 (a) Pressures and movements of conduit buried in clay caused by moisture increase; (b) Pressure-swell curve; (c) Swell plotted against depth of subgrade adjacent to and directly beneath a conduit.

(a) Pressions et mouvements d'une conduite placée dans l'argile et dus à l'augmentation de la teneur en eau; (b) Courbe pression/gonflement; (c) Courbe gonflement/profondeur du sous-sol près et sous la conduite.

to take place at a certain depth, assumed equal to 4-5 times the external diameter of the pipe. The difference increases at higher elevations, reaching a maximum at the contact surface between conduit and sub grade. The problem is to predict the increment of swelling pressure, p_x, that corresponds to Δ_x.

The first step in solving the problem is to evaluate Δ_x in the field. This may be established by plotting two field curves (Fig. 1 (c)):

1. the swelling curve of the subgrade of the trench next to the conduit, in terms of percentage swell plotted against depth or the equivalent pressure, and

2. the swelling curve of the subgrade directly below the conduit as influenced by the restraint to its movement, in the same terms. The data for plotting curve (1) may be obtained either from observations in the field or from a laboratory study of the swelling characteristics of undisturbed samples taken from various depths of the swelling stratum. The integration of individual test results produces an overall swelling curve. The second curve may be constructed either by actual field measurements or by an estimate based on previous observations. The difference in swelling percentage between curve (1) and (2) is Δ_x.

The second step is to investigate the laboratory pressure-swell relationship of undisturbed clay samples which were taken from the depth of the conduit under the initial moisture-density conditions. From the test data the pressure-swell curve (Fig. 1 (b)) is plotted; Δ_v and the difference, $\Delta_v - \Delta_x$, are determined and the resulting pressures, p_x and p_s, are established.

It should be pointed out that in the case of swelling pressures exerted on conduit, in order to allow for a comparison between laboratory swelling pressures and measured field loads, it is necessary to express the load in terms of pressure. This may be accomplished by dividing the load by a certain portion of the external diameter of the conduit which is affected by the pressure.

Clay Swelling Characteristics

(a) *Index Properties:*

By observation, the clay involved in this research may be described as dark brown in colour, tough in its natural moisture content, desiccated, slickensided and highly plastic. The results of the basic classification tests are summarized in the following table:

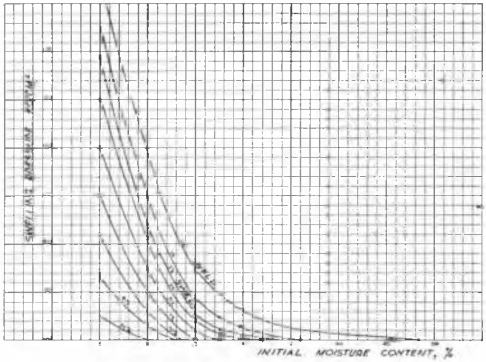
percentage smaller than 5 microns	65-70
percentage smaller than 2 microns	51-55
Liquid limit (per cent)	78-85
Plastic limit (per cent)	20-22
Plasticity index (per cent)	58-63
Shrinkage limit (per cent)	9·5-11
Activity	1·1-1·2
Free swell (per cent)	130

The clay is known to consist of high percentage (70 or more) of the montmorillonite mineral which accounts for its highly swelling properties.

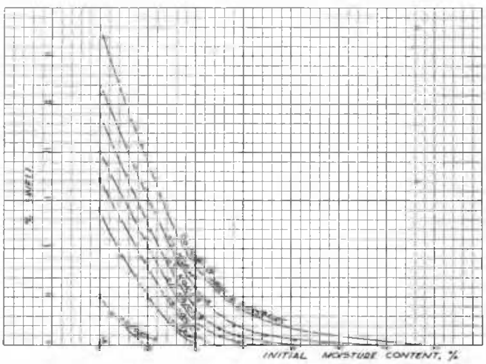
(b) *Pressure and Swell:*

Results of tests in the form of equal swelling and pressure curves are presented by plotting initial moisture content against swelling pressure (Fig. 2 (a)) and percentage swelling (Fig. 2 (b)). These results were obtained by interpolation from laboratory swelling tests of undisturbed clay samples in which

1. the maximum pressure was measured when the specimen was soaked without being allowed to move and



a



b

Fig. 2 (a) Moisture content plotted against swelling pressure of the clay for equal swelling percentages ; (b) Moisture content plotted against percentage swell of clay for equal pressures.

(a) Teneur en eau en fonction de la pression de gonflement de l'argile pour des pourcentages égaux de gonflement ; (b) Teneur en eau en fonction du pourcentage de gonflement de l'argile sous des pressions égales.

2. the amount of swelling was determined when the soil was soaked under various loads. Pressures of up to 7-8 kg per sq. cm were measured under the driest field conditions, whereas the maximum swell under zero load under the same conditions was 33 per cent.

The Field Study

(a) *Programme :*

The scope of the field study included observations on the longitudinal deformation and vertical and horizontal movements of two experimental asbestos cement pipelines buried in two different initial soil conditions and subjected to seasonal changes, as well as to irrigation. In addition, observations were made on the moisture and movement of the natural clay subgrade close to the conduits at various depths under the same moisture variations.

Measurements of longitudinal strain were made possible by installing vibrating wire strain gauges (Ward, 1955) in the longitudinal direction of the pipes, at quarter points around their circumference and at longitudinal distances of about 90 cm.

Experimental pipeline I was placed at a depth of 0·9 m, under conditions simulating an installation of a pipeline under initial wet soil conditions. The trench and backfill were wetted and backfill compaction was done by hand. Pipeline II was installed at the same depth but in a trench that had been exposed to drying and then backfilled by tamping dried clay in lumps. Both pipelines were of 4 in. diameter and were made up from individual pipe lengths of 4 m. The total length of each pipeline was about 20 m, but only the two central pipe lengths were equipped with instruments.

Only vertical movements of soil and pipes and the longitudinal deformations in the vertical direction of the pipe section and the resulting pressures are reported by the authors. The lateral pressure on the pipes and the corresponding horizontal movements were influenced by different factors requiring analysis beyond the scope of this paper.

(b) *Moment Distribution in Vertical Plane :*

The zero readings on the vibrating-wire strain gauges were taken before backfilling at the end of the summer of 1958. After completion of the backfilling operation another set of readings was taken and thereafter periodical observations were made. It was found that both pipelines were operating under slight bending after backfilling, which varied slightly following moisture increases in the clay subgrade due to the rains of the winter of 1958-59. The maximum variation in the vertical moment distributions of both pipelines, however, was established after a concentrated irrigation at the beginning of April 1959, although some major variations took place during accidental flooding of portions of the experimental site at the end of October 1958. The moment distribution variation with time along pipeline I is presented in Fig. 3, and that of pipeline II in Fig. 4.

Longitudinal bending was established in the pipes under transverse swelling pressures developing in the clay subgrade as a result of increased moisture content. In pipeline I the variation of the moments is smaller than in pipeline II, probably as a result of the wetter and more uniform initial conditions of trench and backfill of pipeline I.

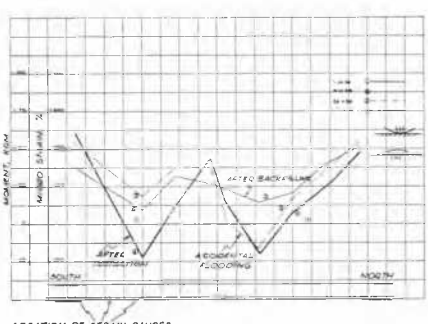


Fig. 3 Vertical moment distribution along Pipeline I with time.

Distribution des mouvements verticaux avec le temps le long de la conduite I.

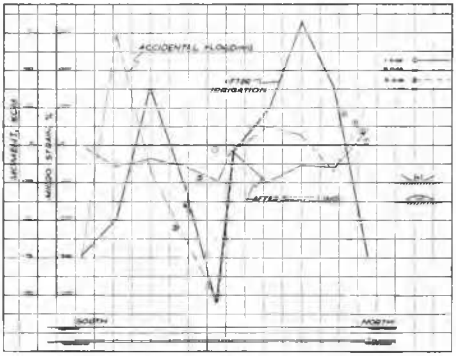


Fig. 4 Vertical moment distribution along Pipeline II with time.
Distribution des mouvements verticaux avec le temps le long de la conduite II.

(c) *Vertical Loads and Movements :*

The moments were interpreted in terms of loads, assuming that the moment distribution is curved and continuous along the two central pipes of each pipeline. The distribution of the loads obtained are presented in Figs. 5 and 6 for the two pipelines.

Below the load distribution of each pipeline its vertical movement is given, in terms of computed elastic line with time. Points obtained by independent measurement of the vertical movement of the pipelines are marked on the elastic lines. The close agreement between the shape of the computed elastic lines and the measured movements is emphasised by the authors.

The maximum load which developed on pipeline I was

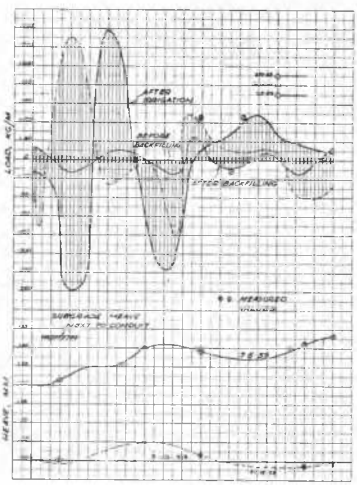


Fig. 5 Vertical load distribution and movements along Pipeline I with Time.
Distribution des charges verticales et des mouvements avec le temps le long de la conduite I.

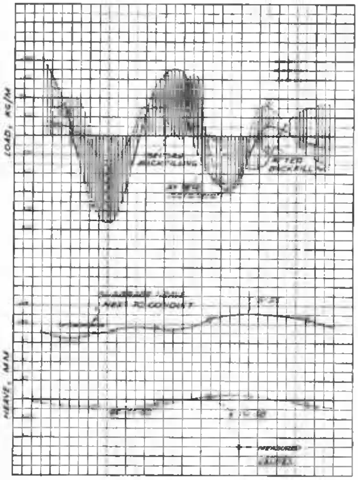


Fig. 6 Vertical load distribution and movements along Pipeline II.
Distribution des charges verticales et des mouvements avec le temps le long de la conduite II.

95 kg/m and that on pipeline II was 280 kg/m ; both of them were established at locations of minimum heave along the sections observed.

Correlation between Theory and Experimental Results

(a) *Subgrade Swelling, Pipeline I :*

The subgrade swelling curve near pipeline I was plotted in Fig. 7, in the form of percentage swelling plotted against depth by two methods :

1. based on laboratory results and

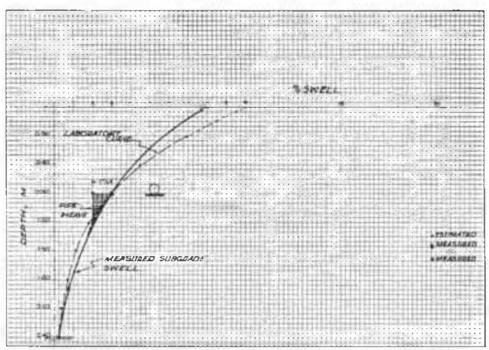


Fig. 7 Swelling curves of subgrade and conduit, Pipeline I.
Courbes de gonflement du sol et conduite I.

2. from actual field measurements. The data for the curve obtained by laboratory results were reproduced from Fig. 2 (a) and arranged in the following table, assuming unit weight of clay 1·6 tons per cub. m .

Depth m	*Equivalent pressure* kg per sq. cm	*Initial moisture* per cent	*Laboratory swelling* per cent
2·40 *	0·37	32	0·15
2·10	0·32	32	0·35
1·80	0·27	32	0·6
1·50	0·22	30	1·2
1·20	0·17	29	1·6
0·90	0·13	27·5	3·0
0·60	0·09	27	4·0
0·30	0·04	24	7·0
0·00	0·00	12	10·0**

* The soil profile at the site consisted of clay to a depth of 2·40 m overlying weathered basalt and basaltic rock.
** Value arbitrarily assumed. Results of laboratory swelling tests on air dried specimens are believed to give excessively high values at loads which are nearly zero, owing to a lateral restriction which does not operate in the field.

The data for constructing the same curve on the basis of field measurements are presented in the following table :

Depth m	*Layer thickness* mm	*Estimated average swelling* per cent	*Estimated swelling* mm	*Estimated total swelling* mm	*Measured swell* mm
2·4 - 2·1	300	0·35	1·0		
2·1 - 1·9	200	0·35	1·1		
1·9				2·1	2·1
1·9 - 1·5	400	1·3	1·3		
1·5 - 1·2	300	1·7	1·7		
1·2 - 0·9	300	2·6	2·6		
0·9				19·9	19·6
0·9 - 0·6	300	3·5	11		
0·6 - 0·3	300	5·0	18		
0·3 - 0·0	300	7·0	21		
0·0				70	70

The close agreement between these two curves indicates that for practical prediction of the amount of swelling of a clay layer in the field it is valid to use the results of laboratry swelling tests.

(b) *Conduit Heave, Pipeline I :*

According to the theory suggested it is now required to determine the difference in heave between the pipe and the adjacent clay subgrade. For this purpose a curve of the vertical movement of the subgrade directly below pipeline I at the zone of minimum heave (Fig. 5) was plotted in Fig. 7. The data and method of constructing this curve are presented in the following table :

Depth m	*Layer thickness* mm	*Estimated heave* per cent	*Estimated heave* mm	*Measured field heave* mm
2·4 - 2·1	300	0·35	1·0	—
2·1 - 1·9	200	0·55	1·1	—
1·9 - 1·5	400	1·3	5·0	—
1·5 - 1·2	300	1·5*	4·5	—
1·2 - 0·9	300	2·0	6·0	—
Total			17·6	17·5

* was assumed that the restraint to the pipe heave started at the depth of 1·5 m., i.e. 5 times the outside diameter below the pipe.

The field swelling, Δ_v-Δ_x, therefore, is 2 per cent as may be seen in Fig. 7. The difference in swelling, Δ_x, is therefore 1 per cent.

(c) *Prediction of the Maximum Swelling Pressure.*

The maximum swelling pressure that acted on pipeline I may be found now from the swelling curves (Fig. 2 (b)). At the initial moisture content of the clay, i.e. 27·5 per cent, the swelling under the overburden pressure (0·13 kg per sq. cm), Δ_v, was 3 per cent. If Δ_x is taken as 1 per cent, Δ_v-Δ_x is then 2 per cent, and p_s, the pressure acting on the conduit may be found as 0·25 kg per sq. cm.

In order to compare now the maximum load measured with the maximum swelling pressure predicted, it is necessary to add the overburden loas to that computed from the moments. Assuming unit weight of fill 1·4 ton per cu.m. the overburden load is :

$$p_0 = 1\,400 \times 0{\cdot}8 \times 0{\cdot}12 = 135 \text{ kg per m}$$

The total load is then 135 + 95 = 230 kg per m = 2·3 kg per cm. This load acts on a portion of the conduit diameter, which may be found from : $d' = \dfrac{2{\cdot}30}{0{\cdot}25} \approx 9$ cm, which means that the conventional assumption of loads acting on the major portion of the external diameter of a conduit applies also to swelling pressures.

However, in order to examine the validity of the theory and the results, the pressure developed on pipeline II will be predicted in a reversed direction, as follows : The maximum load computed for pipeline II was 280 kg/m. The addition of the overburden load of 135 kg/m gives 415 kg/m. The corresponding swelling pressure, therefore, is

$$p'_s = \frac{4{\cdot}15}{9} \approx 0{\cdot}5 \text{ kg per sq. cm}$$

The same curves that were drawn for pipeline I are presented in Fig. 8 for pipeline II. The swelling Δ_x is 6 per cent.

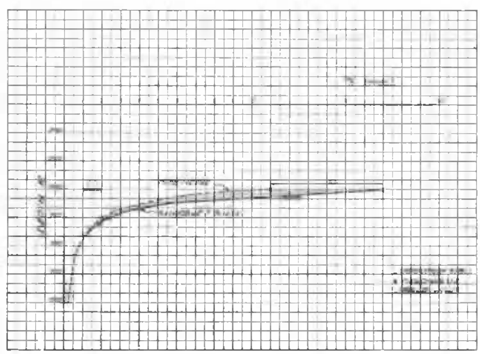


Fig. 8 Swelling curves of subgrade and conduit, Pipeline II.
Courbes de gonflement du sol et conduite II.

If this amount of swelling is deducted from the amount of swelling corresponding to overburden pressure of 0·13 kg/cm^2 for the relevant initial moisture content of 15 per cent, a pressure, p_s, equal to 0·5 kg per sq. cm is obtained, which is in close agreement with the measured value.

Conclusions

It is evident that certain assumptions had to be made in order to use the approach suggested by the authors. Also, the hypothesis was proved only for one type of structure, i.e. conduits buried in a swelling clay. However, it is believed that with accumulation of further field observations on structures erected on expansive soils, it will be possible to apply the theory to any structure, including footings, piles, runways, etc.

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