MODELLING THE HEAVE OF A HEAVILY LOADED FOUNDATION

KEYWORDS: Design, Diffusivity, Heaving, Marls, Monitoring, Slabs.

ABSTRACT: A method is presented to calculate heave and rate of heave due to a change in water potential or in the applied octahedral normal stress. The dependence of swell on the variables is assumed to be of the same form as in the consolidation method. The volumetric strains depend on the logarithm of the ratio of final to initial values of both water potential and applied stress independently. The data for this model are easily measured in the laboratory. The model also involves the definition of an average crack spacing and an estimate of the lateral restraint factor, used to translate the volumetric strains into vertical movement. The usefulness of the procedure is illustrated by its application in extrapolating existing heave records of two heavily loaded slabs on a power plant site. Almost a perfect match is achieved between the model's output and the records, and the soil properties needed by the model are similar to those measured in the laboratory. The slab loads are 350 and 255 kPa. The heaves recorded to the present are 12 cm for both slabs. However, final heaves of about 45 cm are predicted for the heavier slab, and of only 20 cm for the lighter. The cause of swelling is identified to be the combination of extremely low water potentials of the "in situ" marl and the relative purity of the water solution that fills the cracks. The differential heaves observed are attributed to variations in the amount of dissolved solids observed in ground water samples from the site.

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Modelling the Heave of a Heavily Loaded Foundation

SUMMARY A method is presented to calculate the heave and the rate of heave of foundations resting on expansive soil deposits. The approach is based on a few simple laboratory tests to determine the swell properties of the soil mass and some field observations of the shrinkage crack network. The accuracy of the procedure is illustrated with an application to the cases of two heavily loaded slab foundations of a power plant, on which a long record of heave measurements was available.

1 MODEL FOR HEAVE PREDICTION

1.1 Final Heave

The volumetric strains experienced by an elemental volume of soil due to a change in water potential can be calculated (Lytton, 1977) with the following equation:

$$(\Delta v/v) = -\gamma_h \log_{10} (h_f/h_i) - \gamma_\sigma \log_{10} (\sigma_f/\sigma_i) \quad (1)$$

where:
h_i & h_f are the initial and final water potentials,
σ_f is the applied octahedral normal stress,
σ_i is the octahedral normal stress above which overburden pressure restricts volumetric expansion, and
γ_h & γ_σ are two constants characteristic of the soil.

The volumetric strains have the same sign as the first term. The contribution of the second term, which has the contrary sign, is only considered with increasing depth until the strains become zero.

The surface displacement (S_f) is calculated by dividing the active depth into a number (n) of slices (thickness Δz_i) and finding the average volumetric strain $(\Delta v/v)_i$ for each slice from equation (1). The displacement is then calculated from:

$$S_f = \sum_{1}^{n} f_i \, (\Delta v/v)_i \, \Delta z_i \quad (2)$$

where:
f_i is a factor to include the effects of the lateral confinement; it ranges from 1.0 for non-fissured soil to 0.33 for highly fissured deposits.

In summary, what is needed to use equations (1) and (2) are the soil properties γ_h and γ_σ and the initial and final water potential profiles throughout the active depth.

1.2 Heave Rate

Expansive soil deposits exhibit a characteristic shrinkage crack network, which results in a fabric of superimposed soil blocks. Groundwater movement, within the cracks, is relatively fast compared with the diffusion into the blocks. However, it is this diffusion that dictates the rate of heave.

The rate of heave can then be predicted by modelling the diffusion of water into the average block. The water diffusivity and the distance to diffuse, one half of the smallest dimension of the typical block, are the two variables that determine the rate. Assuming that the diffusivity is constant, within the range of interest of water potential, the governing partial differential equation can be reduced to a linear diffusion equation, analogous to the consolidation equation in saturated soils. A fairly good approximation to the exact solution (Lytton, 1977) can be found from:

$$S(t) = S_f \, (1 - e^{-t/T_1}) \quad (3)$$

where:
$S(t)$ indicates surface heave at time t,
T_1 is the time constant governing the heave rate and is defined as:

$$T_1 = \frac{s^2}{12\,D} \quad (4)$$

where:
s is the average crack spacing, and
D is the average diffusivity constant in the expected range of water potentials.

The extension of equation (3) for the case where the applied loads change with time is accomplished using superposition. Here, superposition means that the heave under a combination of loads is equal to the heave that would occur without surface loads minus the changes introduced by each load independently. This is expressed in the following formula:

$$S(t) = S_0(1-e^{-t/T_1}) + \sum_{i=1}^{N} (S_i - S_{i-1})(1-e^{-(t-t_i)/T_1}) \quad (5)$$

where:
S_0 is the final swell with the stresses induced by the soil's own weight, excluding all surface loads,
S_i is the final surface swell under all the load increments applied up to the time $t = t_i$, and
N is the number of load steps that have taken place up to the time t under consideration.

In summary, the prediction of the heave rate is accomplished with only two additional soil properties: The average crack spacing and the average diffusivity constant.

2 CASE STUDY

The above procedure was used in the extrapolation of the heave records of the foundation mats for a power plant located in a confidential site.

2.1 Site Characteristics

At this site, heaving of the subsurface materials was evidenced by the cracking of the protective mat. This was noticed a few months after the excavation and site grading had been completed. A preliminary survey, done at that time, showed heaving rates of about 1 cm per month. This led to the placement of a grid of markers whose vertical movements have been recorded since then, over a period of more than nine years.

The grid consists of more than 150 points which cover an area of approximately 9,000 m^2. The recorded heaves are not homogenous throughout the site; they range from less than one centimeter to more than six. The loads transmitted by the slabs range from 255 to 350 kPa. The differences in transmitted loads are not the cause of the differential heaves. In fact, the lowest heaves were recorded on the least heavily loaded mat, and the highest on one of the more heavily loaded mats. Therefore, the differential heaves can only be attributed to inhomogeneities of the foundation material.

The plant is founded on a marl sequence of reddish-brown strongly cemented claystone with scattered gypsiferous veins. The percentage of the clay fraction is erratic with depth and throughout the site, ranging between 5 and 25%. Besides being in the veins, gypsum and anhydrite are dispersed within the marl matrix. The percentage of anhydrite tends to increase with depth, from about 1 to 2% near the surface to more than 10% with depth.

The mechanism of swelling was, at first, partially attributed to the anhydrite. However, recent studies (Holliday, 1970) suggest that there is no volume increase upon gypsification; in fact, long term laboratory swell tests, done by others, on commercially available pure anhydrite, show volumetric strains smaller than one half of a percent. Therefore, the cause of swelling could not be attributed to the anhydrite. Mineralogical analysis of the clay fraction done at that time, showed the presence of smectites in amounts that could be responsible for the swelling observed.

In studies done by others, once the heave problem had been identified, a perched groundwater table was detected at about the same elevation as the protective mat. It was also found that this groundwater had originated in recent infiltration of rainfall, most probably after the site grading, by correlating tritium levels in the groundwater and in the rain.

Permeability tests implemented inside exploratory boreholes showed a marked decrease in permeability of the foundation marl at about 10 m below the protective mat level. The compression wave velocities measured in geophysical probing of the site, also showed an increase beyond the 10 m zone. This is attributed to a crack network that is thought to have developed because of rebound of the subsurface materials upon grading and excavation of the site.

The chemical composition of the solution filling the cracks was measured on samples obtained from previously installed piezometers. Their electrical conductivity ranged from 4 to 40 mmho cm^{-1} depending on the plan location of the piezometer. The relation of conductivity to osmotic pressure of a solution depends on the specific salt in question. However, this relation can be approximated using typical correlations (U.S.S.L.S., 1954) for solutions of saline and alkaline soils. In this manner, the above range of conductivities corresponds to osmotic pressures from 100 to 2,000 kPa. The most abundant anions were chlorine and sulfate, while the most frequent cations were sodium, calcium, and magnesium.

2.2 Laboratory Test Program

A laboratory test program was carried out to obtain those properties not already available for the application of the above model. The specimens were selected from samples obtained in the high heave areas, since the main interest was to model the behavior of the foundations that had experienced the highest movements. Nevertheless, for comparison purposes, one sample, labelled L-1, corresponding to the lower heaving area was also included.

The unit weight of the marl averaged about 2.47 g cm^{-3}, and the moisture contents ranged from 5 to 7%. The moisture content was determined by oven drying the specimen, and, therefore includes the structural water of the gypsum that is lost by heating above the dehydration temperature.

As indicated by the low moisture contents, the water potential of the "in situ" marl is extremely low (negative), well beyond the range of psychrometers. The only available technique is to define the sorption curve in a vacuum desiccator. With this purpose, the sample is first allowed to equilibrate over a sulfuric acid solution with an osmotic pressure of 100,000 kPa. Once equilibrium is reached, what is determined by reaching constant weight, the final weight is recorded and the specimen is exposed to a solution of smaller osmotic pressure. From these data and a final moisture content determination, it is possible to define the moisture content versus water potential curve. The results for one of the specimens from this site are shown in Figure 1. The water potential corresponding to the initial moisture content is selected as the "in situ" water potential. The results for all tests are included in Table I below. The results considered to be more reliable correspond to the specimens labelled WG-1 and WG-2. The reason is because the other specimens were placed in aluminum containers and some corrosion took place during equilibration over the sulfuric acid solution. Based on these two values, the average "in situ" water potential is -70,000 kPa.

Simultaneously with the above determinations, the volumes of the specimens were measured, using a mercury displacement technique, before the test and after it had reached equilibrium over a known osmotic pressure solution. From these data, the γ_h coefficient is backfigured using Equation (1) for the case of unrestricted swelling; in other words, for $\sigma_f = \sigma_i$. The results found are presented in Table I. The accuracy of these soil properties is somewhat uncertain due to an imprecise control of the temperature during equilibration in the vacuum desiccator especially at high water potentials above -10,000 kPa (Aitchison et al, 1965), which is the case for the h_f levels in these tests.

The coefficient γ_σ is backfigured from swell pressure tests. With this purpose, Equation (1) is used for the case of no change in water potential,

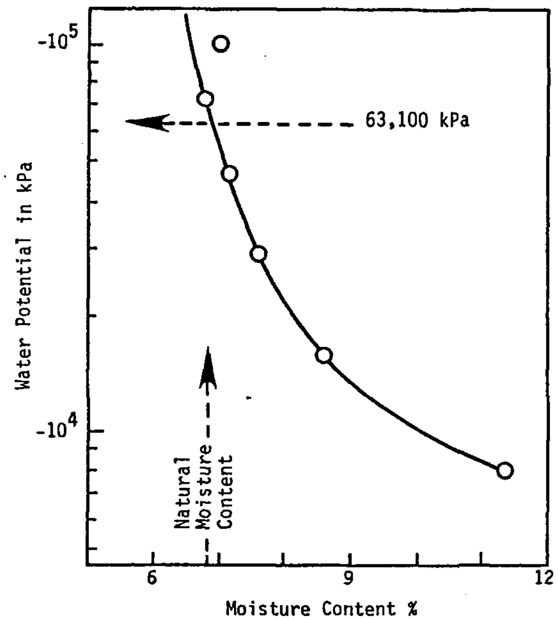

Figure 1. Sorption Curve for Specimen WG-2

which corresponds to $h_f = h_i$. The stress applied during the swell phase is considered as σ_i, and the stress needed to bring back the specimen to the original volume is σ_f. The volumetric strain is the percent swell under σ_i. The values of γ_h, backfigured from test results of previous investigations, ranged from 0.0715, for specimens of initial moisture content at about 7%, to as much as 0.115 for those at 4.7%.

The diffusivity constant was determined using a

TABLE I

LABORATORY TEST RESULTS

Specimen Number	Unit Weight g cm^{-3}	Initial Moisture Content %	Initial Water Potential kPa	γ_h
HS-1	2.414	6.30	-100,000	0.069
HS-2	2.417	5.35	-98,000	0.064
HS-3	2.348	6.70	-110,000	0.074
WG-1	--	6.25	-79,400	--
WG-2	--	6.80	-63,100	--
L-1	2.369	5.85	-68,400	0.004

procedure similar to that proposed by Mitchell (1979). A disc of about 1.5 cm was cut from a marl core. One side of this disc was enclosed with an impermeable membrane wrapped on the 1.5 cm shaft, and a psychrometer was inserted between the core and the membrane. The other face of the disc was exposed to a constant water potential of -5000 kPa.

The changes in water potential at the protected face are measured with psychrometers as soon as the potential drops within the range appropriate for them. The results of one test are presented in Figure 2 where the change of the water potential measurement with time is shown, together with the predicted dependence by the linear diffusion equation for several values of the diffusivity.

These solutions were found assuming that all boundaries are impermeable, except the exposed face, which is assumed at a constant water potential of -5000 kPa. The initial conditions are assumed homogeneous within the disc, at a water potential of -70,000 kPa. The remarkable agreement between both trends indicates that the assumption of a constant

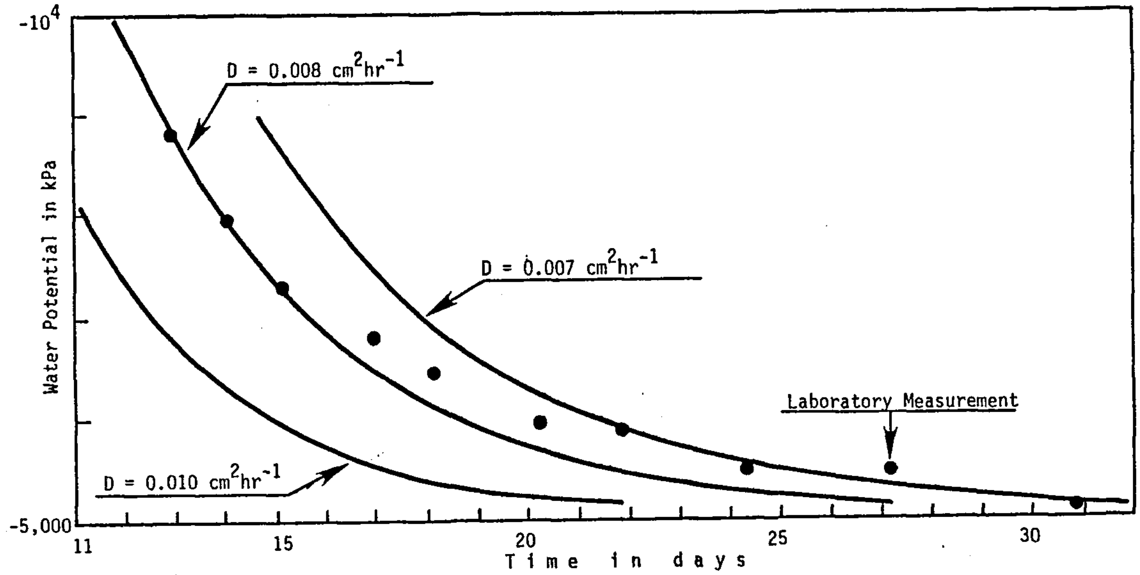

Figure 2. Determination of the Diffusivity Constant for Specimen D-1

diffusivity is quite appropriate, at least in this case. An average value of two determinations is 0.009 $cm^2\ hr^{-1}$.

2.3 Model Predictions

Application of the model requires the average soil properties and representative profiles of the initial and final states of water potential throughout the active depth. The laboratory test program implemented is considered insufficient to completely characterize the variation of material properties with depth, due to the high degree of inhomogeneity evidenced by the high differential heaves. Inhomogeneities of the marl are one cause, but a no less important cause is the difference in osmotic pressures (Morgenstern et al, 1980) of the fluid filling the cracks in the marl.

Because of these uncertainties about the average marl properties and due to the availability of swell records, a mixed approach making use of the latter was adopted. Those soil properties that are known reliably are fixed and the rest are varied to fit the output of the model to the swell records.

For this purpose, two heavily loaded slabs were selected. Since the first year of heave was not monitored, the records had to be extended back somewhat arbitrarily. The time origin was set as the date that site grading was completed. The heave was estimated by fitting a simple exponential to the first part of the records.

The properties fixed for the application of the model include: 1) water potentials that are constant with depth at -70,000 kPa initially and -100 kPa at the end; 2) a constant unit weight of 2.472 g cm^{-3}, a reference pressure $\sigma_{1,2} = 1$ kPa, and a constant diffusivity of 0.009 $cm^2 hr^{-1}$; and 3) the surface loads are included with influence coefficients of one through the active depth, and the lateral restraint coefficient is used to multiply the vertical stress to estimate the octahedral normal stress. The only variables that may be changed to fit the heave records are γ_h, γ_σ, and s.

Heave records were fitted by trial and error. The results for Slab 1 are presented in Figure 3, with the loading sequence defined according to the construction schedule. The model predicts a final heave of 45 cm, and that the slab will appreciably heave throughout its entire service life; 90% of the total heave will only be reached after more than 80 years. Up to the present, Slab 2 has heaved about the same as the first one. However, the model predicts a final heave of only 20 cm. The comparison of the soil properties required to match the records, and those measured in the laboratory are presented in Table II.

TABLE II

COMPARISON OF MATERIAL PROPERTIES

Property	From Laboratory Tests	From Model Slab 1	From Model Slab 2
γ_h	0.065/0.075	0.100	0.100
γ_σ	0.071/0.115	0.145	0.164
s in cm	---	189	151

In all of the above analysis, a final water potential of -100 kPa was assumed, which corresponds to

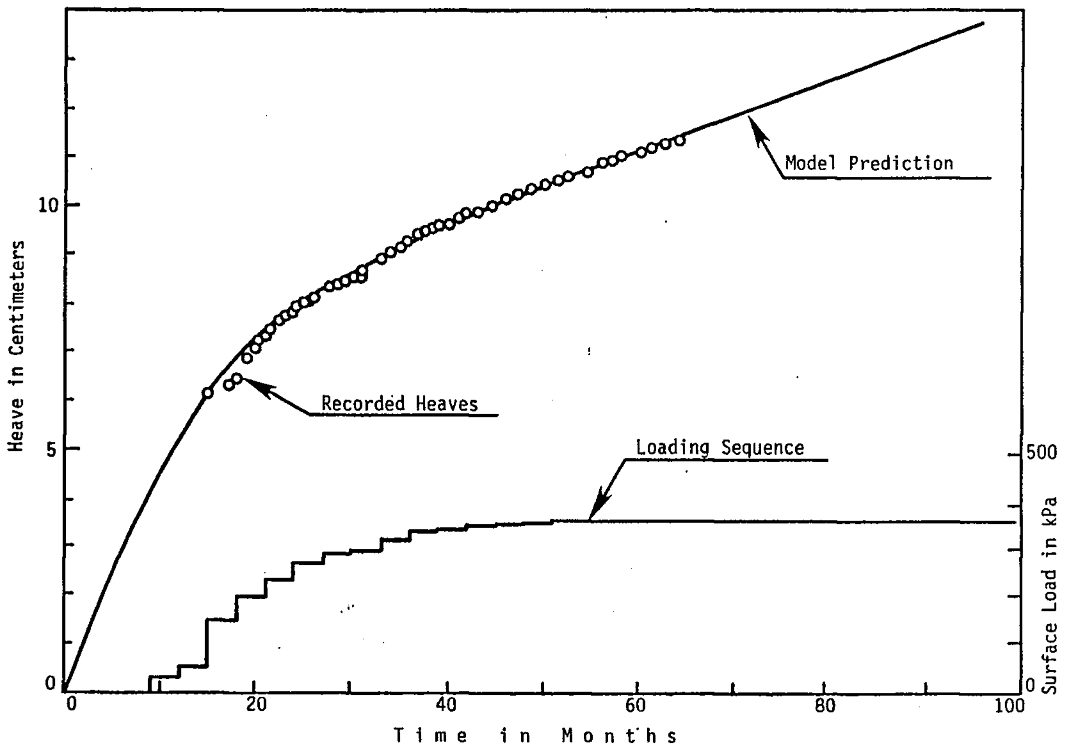

Figure 3. Comparison of Predicted and Measured Heaves for Slab No. 1

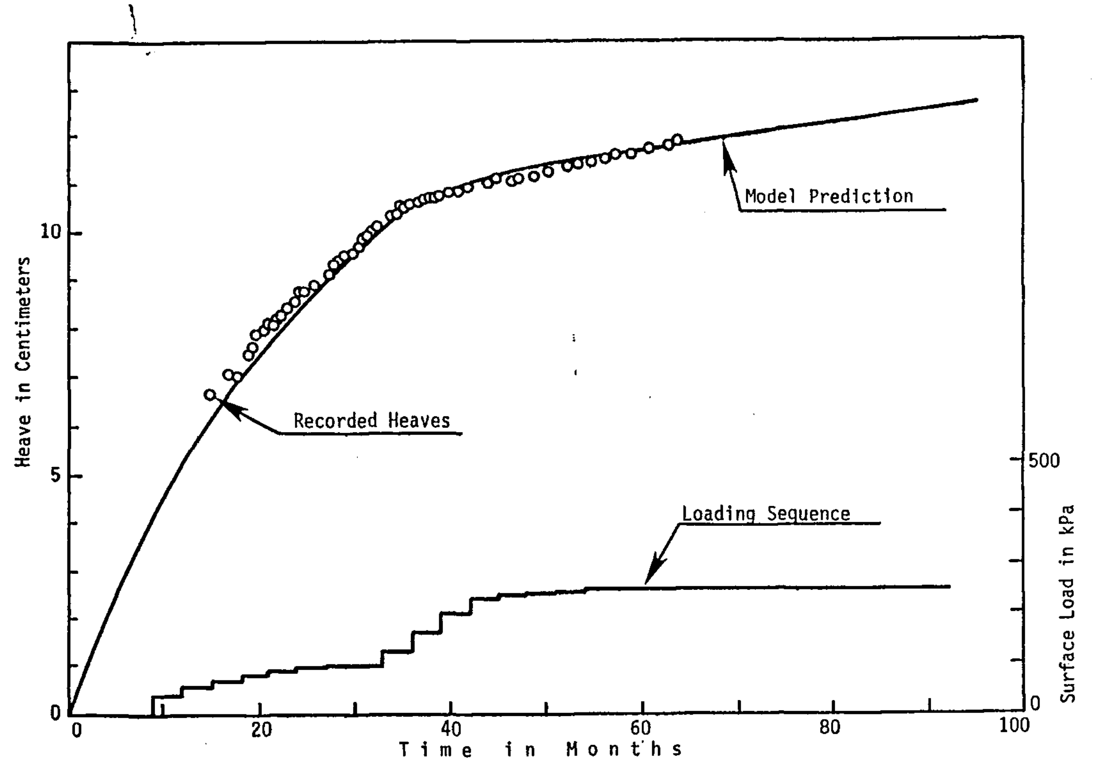

Figure 4. Comparison of Predicted and Measured Heaves for Slab No. 2

the minimum conductivities measured on water from the site. The results are drastically different if higher potentials are considered; in fact, if the maximum observed values of -2000 kPa are used, the model predicts zero heave in the long run.

3 CONCLUSIONS AND RECOMMENDATIONS

The application of the simple model proposed has been shown to be extremely versatile, because of the remarkably good fits of heave records for two widely different loading sequences.

The data necessary to apply the model can be easily and inexpensively measured in the laboratory. In most of the cases of foundations on expansive soil deposits, the average spacing between cracks can be selected by logging a few test pits scattered throughout the site. This should include observations of the distribution of crack spacing with depth; from this, a reasonable estimate can be found as a weighted average.

Nevertheless, even in the cases where the inspection of the crack spacing with depth is significantly more cumbersome, as in the case study presented, the model provides an excellent extrapolation procedure which has a physical basis. Additionally, when the procedure is used in the reverse sense, the soil mass properties can be inferred from a record of heaves; therefore, allowing a systematic evaluation and comparison of records at several locations.

The case study presented shows that even the most heavily loaded foundation can experience significant heaves, provided that the right conditions exist. In this particular case, it is the combination of an extremely low "in situ" water potential and the relatively pure solution that fills the cracks. This brings out the fact, already observed by others that the purity of the solution in the cracks is a very significant aspect on the possible heave at any site. In this case under consideration, it appears that the only feasible solution to stop, or even reduce the heaves that have already occurred, is to increase and maintain a high osmotic pressure in the solution that fills the cracks.

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