TECHNICAL NOTE

ESTIMATING UPLIFT OF FOUNDATIONS DUE TO EXPANSION: A CASE HISTORY

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SYNOPSIS

In this study the effects of a burst water pipe beneath the footings of a structure resulting in foundation uplift were assessed. Several boreholes were drilled under the damaged area and various laboratory tests were conducted on core samples, including swell tests. The swell potential or percent was evaluated for specimens under equivalent footing stresses by two methods. The first using equations readily available in the literature and the second by conducting restrained swell tests in the laboratory. The results indicate that the amount of foundation uplift can be predicted, with reasonable accuracy, by either method provided that the active zone is well defined.

INTRODUCTION

In the field of geotechnical engineering, it has long been recognized that swelling of expansive soils caused by moisture change may result in considerable distresses and, therefore, in severe damage to overlying structures. In the past thirty years or more, various investigators have conducted extensive studies to evaluate the important factors that influence swelling and to develop methods of analysis for prediciting heave (Gromko, 1974; O'Neill & Ghazzaly, 1977; Mowafy & Baure: 1985). However, predicting soil behavior, in this case heave, in the laboratory is one thing and in the field is another. Even under well reproduced conditions in the laboratory soils may behave quite differently than in the site. One of the most influential soil properties that affects heave is moisture variation. The field moisture content of soils, in particular, its fluctuation from one season to another, is very difficult to estimate. Consequently, predicting the amount of foundation uplift due to in situ heave is rather difficult. However, careful assessment of the appropriate soil properties along with the field conditions can lead to a good estimation of uplift.

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The objective of this paper is to show how uplift of foundations can be predicted by using available equations and/or appropriate laboratory tests. To accomplish this task a case study in Irbid city in northern Jordan was considered. As part of the prediction process a series of tests to identify the soil and to determine its swelling properties were performed. In the following sections, the case study, laboratory test procedures and the swell predictive equations which were adopted for this study are presented.

SOILS OF IRBID REGION

The climate in the region is semi-arid. This is considered to be an excellent prerequisite for expansion, especially with the existing soils. Based on a recently-completed study at Jordan University of Science and Technology, JUST, (Tuncer, Basma & Taqieddin, 1989) the following can be stated about the soils in the region :

1) The existing clays are calcareous in nature with the occurrence of abundant inclusions of limestone and chert fragments, indicating that they are superficial deposits derived mainly from the weathering of marl, limestone and chert parent rock.

2) The clay is highly plastic, extensively fissured and overconsolidated with an average clay content of 65%, liquid limit of 80%, plastic limit of 40% and shrinkage limit of 16%.

3) The soils contain a large percentage (about 55%) of interlayered montmorillonite-illite, a mineral mixture known to have a high capacity for water absorption and therefore highly expansive in nature.

The problem of expansion is the most predominant in the area. Many reported incidents at different locations in and around the city in residential buildings have indicated serious damage to structures due to heave.

CASE STUDY

In 1985, the city of Irbid decided to add an additional stand for spectators in the football stadium. A thorough investigation of the soil was performed by the Geotechnical Engineering and Materials Testing Company, GEMT, in Amman (GEMT report, 1986). Based on their recommendations, an ultimate bearing capacity, q_u, of 750 kPa was to be used ($\approx$ 7.5 kg/cm^2) for a standard size footing

placed at 2 m below the surface. Using a safety factor of 3, the suggested allowable bearing capacity, q_a, was 250 kPa.

A safe factor of 3 was applied to q_a producing a design value for footings of 83.3 kPa. This resulted in footings 2 m × 2 m in dimensions for calculated column loads of 325 kN ($\approx$ 32.5 tons). Construction started in October 1985. The top 2.25 meters of the soil was first excavated. As a construction precaution a 25 cm thick flexible non-expansive selected fill material was used. This material was mostly fine to course well graded sand (SW) compacted at about 90% standard Proctor density. The replacement material was recommended to be graded in such a manner that the larger sizes are in direct contact with the natural, potentially expansive soil; to provide a relieving effect for the likely swelling expected. The reinforced concrete footings were cast in place and the excavated soil was then vibro-compacted. The entire structure was completed in April of 1986.

In August of the same year, cracks were observed in the outside walls in addition to several fissures in the columns and beams. Furthermore, a pond of accumulated water was observed under and around one of the footings in the vicinity of the cracked area. A team consisting of the author and Dr. Mohammed Shaiab from the Civil Engineering Department at JUST representing the geotechnical and structural engineering areas, respectively, was asked to investigate the problem. After carefully studying the problem, the concrete quality, placement and construction were found to be better than the acceptable limits. In addition the actual foundation loads were well below the maximum allowable. Consequently, structural failure was excluded as the cause of such crackers. It was conluded that soil heave was the cause because of the following reasons: 1) the soils in the area are highly expansive, 2) no bearing capacity failure was expected since the applied pressure was much smaller than q_a, and 3) the problem occurred just after the summer season in which the water content of the soil is minimum and, thus, any increase in moisture will cause swelling of the soil. The major cause of this problem is the large amount of accumulated water. Further investigation revealed that an old water pipeline, running one meter below the footings, (3 m below the surface) had cracked under the pressure induced by the footings causing a leak. This, consequently, resulted in a sudden increase in the moisture content of the soil thereby causing heave and differential uplift of footings as shown in Fig. 1. It should be noted that this water pipeline was not detected during construction.
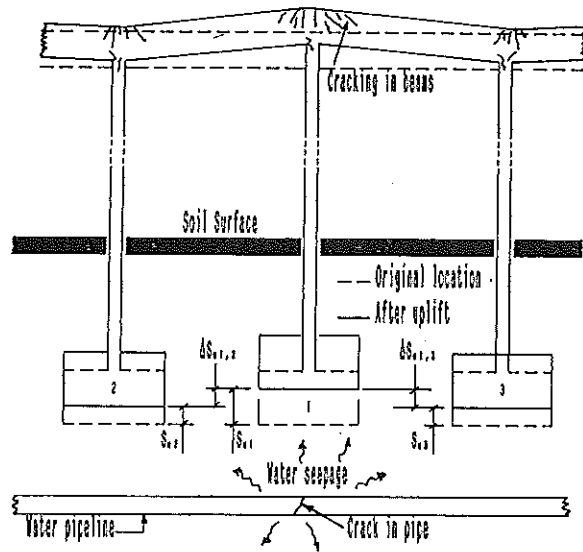
Fig. 1 Uplift of Footings near the Location of Pipe Burst.

Table 1 Grains Size Distributions and Mineralogical Composition of Soils under Footing 1, Percent of Whole*

Mineral	Depth below surface					
	1 m	2 m	3 m	4 m	5 m	6 m
Sand fraction	6.2	9.2	4.2	5.1	8.7	5.7
Silt fraction	39.7	32.9	30.8	30.9	30.3	28.3
Kaolinite	9.2	10.1	12.0	12.1	8.5	10.0
Mixed layers of Montmorillonite and Illite	44.9	47.8	53.0	51.9	52.5	56.0

* Average of five values at each depth

EXPERIMENTAL INVESTIGATION AND RESULTS

Specimens under the footing with maximum uplift (termed footing 1 in Fig. 1) were collected from a borehole drilled using a double rotary core barrel with air flush down to the bed rock. Core samples (about 8 cm in diameter) were wrapped with aluminum foils and tightly packed in labelled plastic bags then transported to the laboratory for testing. The experimental tests consisted of a) grain size distribution and mineralogical analysis, b) water content, unit weight and index properties determination, and c) swell measurements. It is important to point out that the 25 cm of selected fill materials was excluded from the experimental investigation.

a) Grain size distribution and mineralogical testing:

The grain size distribution of the soils were determined by both sieve and hydrometer analysis while the mineralogical analysis was carried out using the x-ray diffraction technique on samples taken at different depths. Table 1 gives
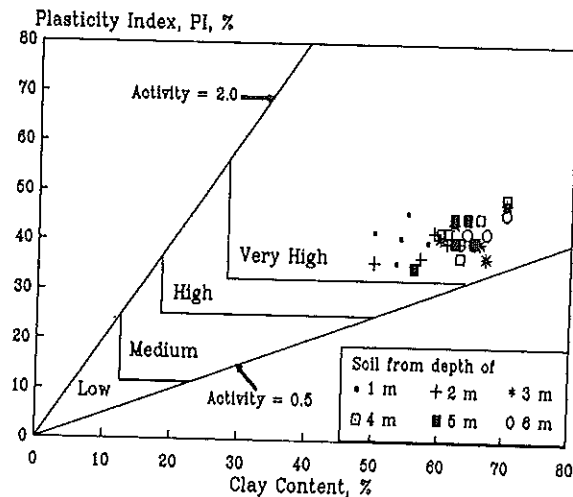

Fig. 2 Potential expansiveness of soils under Footing 1.

the percentages of the clay minerals along with the percentages of sand and silt. The sand and silt fractions proved to be mostly calcium carbonate and quartz. It is evident from the table that the existing clay contains a large percentage of interlayered montmorillonite-illite. As was mentioned earlier, this mixture has a tendency to absorb water and, thus, is expected to swell.

b) Water content, unit weight and index properties:

Table 2 lists the values of water contents for various periods before and after the uplift problem along with the most important index properties. The variation of the water content, w, is presented in Fig. 3. This figure, in particular, Fig. 3 (a) indicates an increase in w immediately under footing 1 and a decrease as one moves away in both directions. Fig. 3 (b) shows the variation of water content
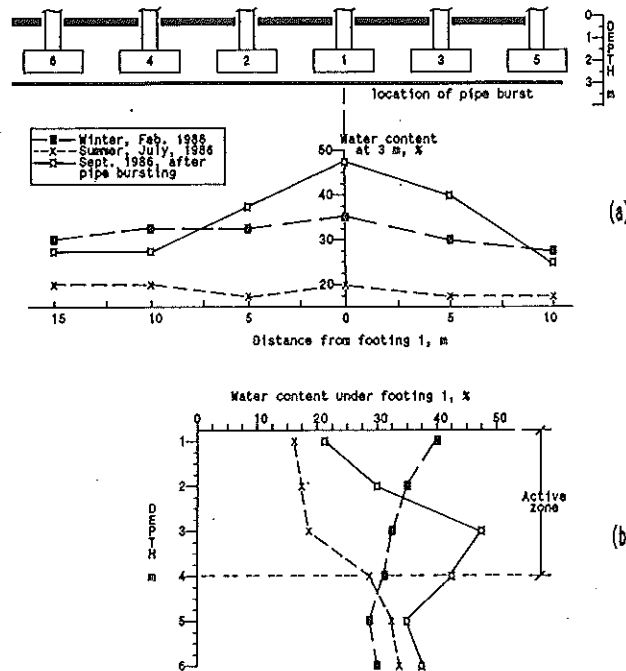
Fig. 3 Variation of Water Content (a) 3 m below Surface near Footing 1 and (b) under Footing 1.

with depth under footing 1 to define the "active zone". The active zone is the depth up to which seasonal changes of water content occur (Das, 1990). For this particular situation the active zone is 4 m, as can be seen from Fig. 3 (b).

Table 2 Soil Properties[1] under Footing 1.

property	Depth below surface					
	1 m	2 m	3 m	4 m	5 m	6 m
Water content, w, %						
Winter, 2/1986	39	35	33	29	27	30
Summer, 7/1986	17	18	20	28	29	32
After pipe burst, 9/1986	21	29	46	40	31	33
Dry unit weight, kN/m^3	14.7	14.3	15.7	14.6	15.2	15.8
Consistency limits, Liquid limit, LL, %	78	78	80	82	81	80
Plastic limit, PL, %	37	39	38	39	40	38
Plasticity index, PI, %	41	39	42	43	41	42
Activity, A	0.76	0.67	0.65	0.67	0.67	0.64
Modified free swell index	25.2	21.6	22.1	26.4	24.3	23.7
Swell potential[2], SP_o, %	9.6	8.2	12.3	10.6	10.6	6.2
Swell pressure[3], p_s, kPa	205	197	302	256	248	117

[1] 'Average of five specimens.

[2] Specimen tested at summer water content and unit weight with 7 kPa initial seating load wetted and allowed to swell.

[3] Specimen tested at summer water content and unit weight with zero swell test to determine swell pressure.

Note: Some of the experimental data were obtained from GEMT report.

c) *Swell measurements :*

The clay content and the plasticity index (see Fig. 2), indicates that the clay is highly expansive (Williams, 1958). Additionally, a more recent test method suggested by Sivapullaiah, Sitharam & Rao (1987) to classify expansive clays was utilized. This test consists of obtaining 10 grams of oven-dried soil which has been well pulverized. The soil is placed in a 100 cc graduated jar containing distilled water. After 24 hours, the swollen soil sediment volume is measured and the modified free swell index, MFSI, is then evaluated as follows,

$$\text{MFSI} \quad = \quad \frac{V - V_s}{V_s} \tag{1}$$

where V = Volume of the swollen soil and V_s is the volume of soil solids. Table 2 lists the values of MFSI. Since these values are greater than 20, the potential expansiveness of the soils at all depths is defined as very high (Sivapullaiah, Sitharam & Rao, 1987).

The magnitude of swell was obtained by conducting oedometer swell tests. These tests consisted of determining both the swell potential (percent) and the swell pressure. The former was determined by the unrestrained swell test whereas the latter was evaluated by the zero swell test. For this reason two undisturbed specimens were first air dried allowing them to revert to their summer water contents (see Table 2 for water contents). The samples were then trimmed to fit in the consolidation ring (7.6 cm in diameter and 2 cm in height). Both samples were placed in the consolidation frame and a small initial seating pressure of 7 kPa (1 psi) was applied with deformation readings recorded. After the deformations ceased (within one to two minutes) the samples were fully saturated. In the first specimen, expansion readings were recorded at elapsed times of $\frac{1}{4}$, $\frac{1}{2}$, 1, 2, 4, 8, 15, 30, 60, 120, 240 minutes, 24, 48, and 72 hours. The test was continued until expansion ceased. The swell potential was calculated, in percent, as the ratio of the maximum expansion to the initial height of the specimen (2 cm).

To determine the swell pressure, incremental loads were applied to the second sample. As the specimen expanded, additional loads were added to prevent swelling. This process was continued until expansion stopped. The final load divided by the area of the specimen was used to define the swell pressure. The obtained values of the swell potential and swell pressure, at different depths, are listed in Table 2.

UPLIFT CALCULATIONS

To determine the uplift due to swelling, several calculations were performed. Table 3 summarizes the results of these calculations for footing 1 (Fig. 1). The footing stress corresponds to the stress increase under the center line of footing 1. The uplift was evaluated in the following manner:

1) The soil below the footing was divided into several layers. Each layer was one half meter thick except layer 1 immediately under the selected fill which was 25 cm thick. The geotechnical properties of each layer was represented by the average value of five samples tested (see Table 1 and 2).

2) The stress increase, Δp, due to the footing load at the top of each layer was calculated using Westegard's theory for evaluating soil pressure. The influence value for the top of the first layer, for example, is 0.95.

3) The stress due to the footing load at the top of layer 1 was calculated as the footing bearing pressure of 83.3 kPa multiplied by the influence factor of 0.95 which yields 79.1 kPa. This value was then added to the existing overburden stress, σ_o', of 38.3 kPa to obtain a sum of 117.4 kPa. The calculated values of σ_o' and p, for each layer, are plotted in Fig. 4a. Fig. 4b shows the difference between the swell pressure, p_s (from Table 2), and p. Positive values i.e. $p_s > p$ indicates that uplift is to be expected.

4) The swell potential at a stress of 117.4 kPa was evaluated by one of the following methods:

a) Using the swell potential at zero stress and the following equation (Bowles, 1989)

$$SP_p \quad = \quad SP_o (1 - 0.0735 \sqrt{p}) \tag{2}$$

where SP_p is the swell potential under a stress of $p = 117.4$ kPa, SP_o is the swell potential under zero stress, 8.2% (obtained from Table 2). Fig. 4c shows the variation of SP_o and SP_p with depth as calculated by Equation 1.

b) Performing an oedometer restrained swell test. This experiment was conducted in the following manner: For the first layer an undisturbed sample was loaded by incremental pressures starting with 12.5 kPa. The sample was allowed to fully settle under this pressure then the pressure was doubled. The process is repeated until the applied pressure is just over the field stress (200 kPa in the case of layer 1). Curve A in Fig. 5 shows the stress path for the loading stage. At a stress of 200 kPa the sample was saturated and allowed to swell and reach

Table 3 Heave Calculations

| Depth (m) below | | Sample | γ, kN/m³ | Influence factor | Footing stress Δp, kPa | σ_0', kPa | p, kPa | SP_0, % | SP_p, % | |
Surface	Footing								Eq. 1	measured*
2.25	0.25	A1–A5	16.9	0.95	79.1	38.3	117.4	8.2	1.76	2.00
2.5	0.5	B1–B5	16.9	0.75	62.5	42.6	105.1	10.8	2.67	2.40
3.0	1.0	C1–C5	18.8	0.45	37.5	52.0	89.5	12.3	3.74	3.60
3.5	1.5	D1–D5	18.4	0.27	22.5	61.2	83.7	12.0	3.93	4.30
4.0	2.0	E1–E5	18.7	0.13	10.8	70.6	81.4	10.6	3.57	4.40

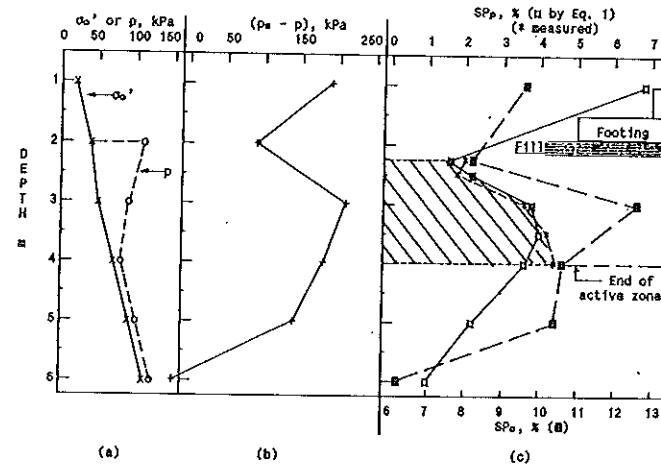* Average of five tested specimens.

Fig. 4 (a) Variation of σ_0' and p with Depth, (b) Variation of $(p_s - p)$ with Depth and (c) Variation of SP_p and SP_0 with Depth.
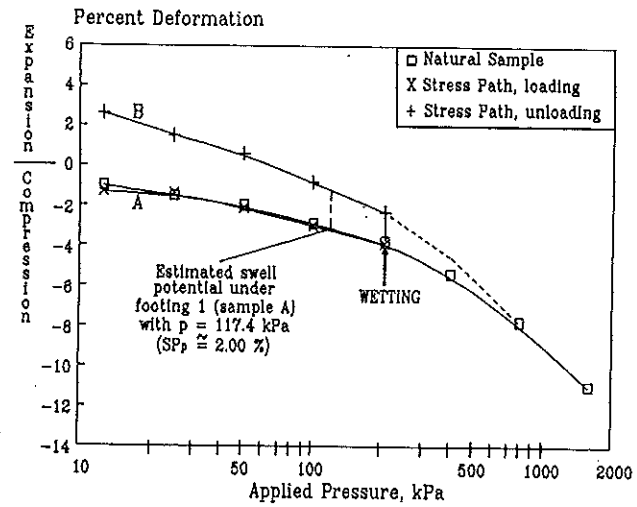

Fig. 5 Determination of Swell Potential, SP_p, for Layer 1 under Footing 1.

equilibrium. The loads were then removed gradually allowing the specimen to fully expand at each pressure level until all the loads were removed. Curve B of Fig. 5 shows the unloading stress path. The vertical difference between Curve A and B at a pressure of 117.4 kPa is used to difine the swell potential under such a field stress. The same test was conducted for soils at various levels under their corresponding field stress. Table 3 lists the measured values of SP_p for all levels. Additionally, these values are plotted in Fig. 4c.

5) The surface heave or uplift, ΔS_u, is then calculated for each level and then summed up over all layers as follows (O'Neill and Poormoayed, 1980),

$$\Delta S_u = \sum_{i=1}^{n} [SP_p \%] (Hi) (1/100\%) \qquad (3-a)$$

or

$$\Delta S_u = \sum_{i=1}^{n} [A_i] (0.01) \qquad (3-b)$$

where H_i is the thickness of layer i, n is the total number of subdivided layers beneath the footing down to the active zone, and A_i is area i beneath the footing down to the active zone on the SP_p versus depth plot (shaded in Fig. 4c).

Using this approach the calculated value of heave was 5.94 cm when using values of SP_p obtained by Equation 1 and 6.20 cm when using the laboratory data. These values compare well with the field measured heave which was 6.27 cm. 3 months after the uplift problem was detected. It is unfortunate that no periodic survey of the foundation movement was conducted. Nevertheless, this period (3 months after the pipe bursting) was assumed to be sufficient for full saturation of the soil within the active zone. Consequently, the measured uplift was considered to be a maximum.

REMEDIAL SOLUTIONS TO THE UPLIFT PROBLEM

As was previously mentioned the uplift due to the expansion in the soil caused major cracks in the outside walls and in the beams and columns. Fig. 6 shows the extent of these cracks at the top of the wall. Fig. 7 shows a series of cracks in the main girder just over footing 1.

To minimize any further heave damage to the structure, it was recommended that use of the pipeline should be discontinued. The accumulated water was pumped out while the moisture residing in the soil was allowed to evaporate and/or seep away. This eventually reduced the moisture content of

the swollen soil allowing it to shrink back to its original height. Two years after the water pipeline was rerouted, most of the cracks closed up and the wall was patched. Additionally, the fissures in the beams and columns were carefully sealed and repaired. Fig. 8 shows the condition of the wall after patching. Clearly, no evidence of cracking is observed.

CONCLUSIONS

This paper presented an approach by which the uplift of footings can be reasonably estimated knowing the soil properties and the prevailing conditions in the field. In this approach the active zone is first defined and the swelling

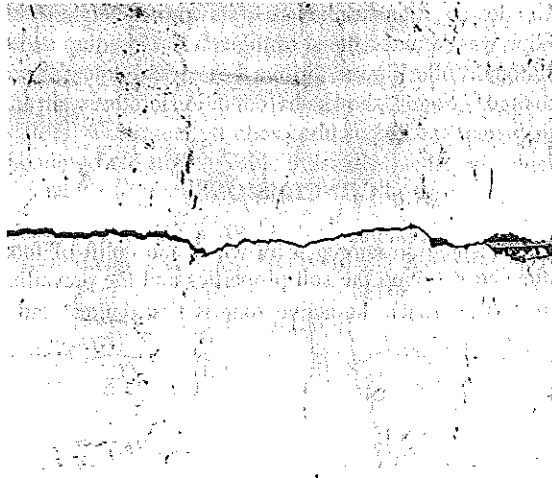**Fig. 6 View of the Cracks on the Top of the Wall due to Uplift.**

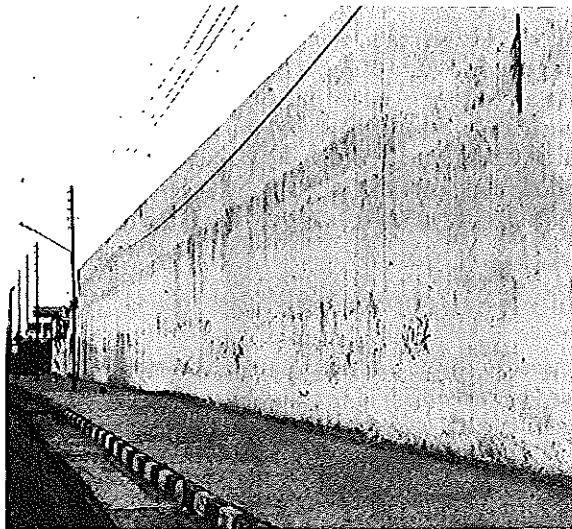**Fig. 7 Major Cracks in the Main Girder over Footing 1 due to Uplift.**

Fig. 8 View of the Outside Wall after Repair.

properties for the soil at incremental depths are determined experimentally and plotted as shown in the paper. The uplift is calculated by using available equations in the literature coupled with data obtained by appropriate laboratory tests. The procedure outlined here is found to produce a prediction of uplift close to that found in the field. The greatest source of error in the prediction is considered to be uncertainty relative to the extent of water content variation i.e. depth of active zone. Care must be taken to reduce and/or maintain such variations in swell susceptible soils.

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