Interaction of expansive soil with structures

Interaction entre un sol expansible et des constructions en béton armé

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ABSTRACT: In this paper, a floating slab and a fixed slab were constructed on a highly expansive soil that was contained in a stainless steel box. A controlled heave test was conducted to investigate the effect of heave rate on swelling pressure. Furthermore, an instrumented reinforced concrete structure and a field test site were constructed to investigate surface and deep heaves, and structure-expansive soil interaction while the site was artificially wetted through vertical sand drains. The swelling pressure obtained from the constant volume test almost coincides with the average pressure predicted from the braced fixed slab test. A major reduction in swelling pressure takes place if the expansion is not completely restrained as revealed from both the controlled heave rate test and the instrumented field test.

RESUME: Dans cette étude deux dalles, une flottante et une fixe ont été construites sur un sol très expansible, contenue dans une boite métalique inoxydable. Un essai de force dirigé a été pratiqué dans le but d'étudier l'effet de l'effort sur la pression due au gonflement. De plus, des experiences sur une construction en beton armé avec instruments de mesures et un champ de construction ont été construits afin d'étudier les efforts a la surface et au fond du sol, et l'interaction du sol pendant que le terrain était artificiellement mouillé a travers une colonne de sable. La pression due au gonflement obtenue a l'essai du volume constant coincide à peu près avec la pression moyenne prevue dans l'essai avec la dalle flottante a supports. Une réduction très nette de pression due au gonflement du sol apparaitra si la pression n'est pas arreteé, comme cela a été démontre lors des deux premières expériences.

1 INTRODUCTION

The arid climate, the geology and the severe weathering conditions in Saudi Arabia have produced a wide distribution of expansive soils (Dhowian et al. 1985; Abduljauwad 1993). Due to the realisation of the importance of soil-structure interaction caused by shrink and swell in the expansive clays, the author conducted laboratory and field research to investigate the heave and structural distress which took place due to volume change of the supporting soil. A site was selected in Al-Qatif, a city of the Eastern Province of Saudi Arabia, for the soil-structure interaction study. The geotechnical and physicochemical properties and mineralogical composition of the soil were determined. A stainless steel box was designed to obtain large undisturbed block samples and used as part of the laboratory test setup once the sample was secured. A floating and a fixed slab setup were arranged in the laboratory. Furthermore, a reinforced concrete structure was constructed on the same site to investigate ground movement, structure-expansive soil interaction and structural damage, following the flooding of the site with water through vertical sand drains which accelerate the increase in moisture content of the supporting expansive clay.

2 EXPERIMENTAL PROGRAMME

Soil samples were collected to determine geotechnical and physicochemical properties and mineralogical composition. The extraction of block samples was accomplished by the use of a stainless steel box which was later incorporated in the test setup (Abduljauwad et al. 1996). Two types of setup were arranged: the first one to simulate a floating slab on grade, and the other to simulate a slab with some end restraints.

2.1 *Floating slab*

A complete schematic drawing of the setup for the floating slab is shown in Figure 1. After placing the reinforcement (five 12 mm diameter bars in each direction), a 50 mm thick concrete slab was cast. Five steel rods, 25 mm in diameter and 350 mm in length, were used to place the tip of the dial gauges on them to monitor the movement of slab during testing. Four rods were located in the four corners of the slab and one was located in the centre of the slab. The slab was preloaded with sand applied by means of a metal sheet container open from the top and bottom. The clay was subjected to a surcharge of about 6.8 kPa due to the weight of the sand, slab and steel rods. The whole box setup was then placed in a water tank where water was introduced to the soil sample through the holes in the base plate of the box.

2.2 *Fixed slab*

Prior to testing, the box was braced on all four sides as well as the bottom by welding steel channels as shown in Figure 2, to minimize the lateral and bottom bulging in the steel box. Circular indentations were made at the top surface of the soil to embed seven soil pressure gauges. The gauges were then covered with a thin layer of clay, and the exposed top part of the soil was covered with plastic sheet to prevent any water from reaching the soil sample during casting of the concrete slab. To achieve anchorage points at the ends of the slab, a special U shape frame was constructed from a steel channel having 5 cm depth and 2.5 cm wide flanges. This frame was then fastened to the inside of the steel box at the top of the soil after covering it with the plastic sheet. The whole box setup was then placed in a water tank and water was introduced to the soil through the perforations in the base plate of the box.

2.3 *Controlled rate of heave tests*

Additional tests were conducted to investigate the effect of heave rate on swelling pressures. Samples of clay were trimmed to 100 × 100 × 250 mm blocks and four sides plates were screwed around them to contain the sample. Figure 3 shows the complete setup for the controlled rate of heave test. A pressure transducer, which was impeded inside the clay sample, and a load cell, which was placed

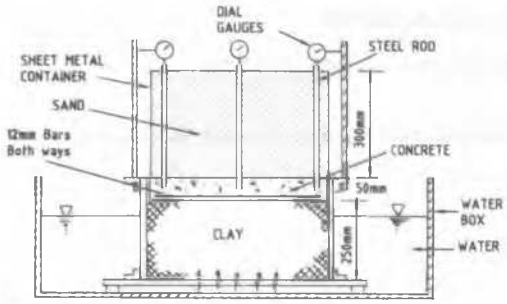

Figure 1. Setup for floating slab.

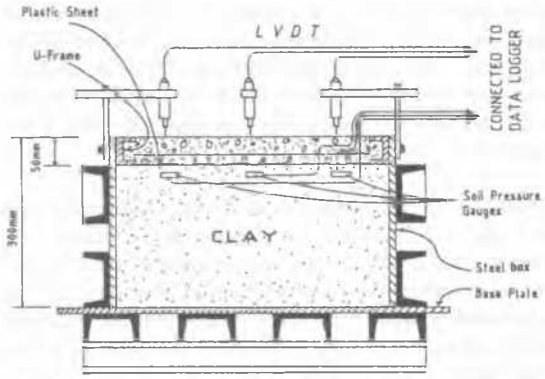

Figure 2. Section view for the edge fixed slab simulation box.

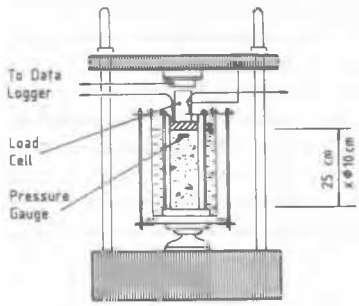

Figure 3. Controlled heave rate device.

between the top rigid steel plate and a loading frame, were used to monitor the swelling pressure. All samples were subjected initially to a surcharge of 6.8 kPa. Water was then added and the bottom plate was allowed to move down at certain rates to simulate different tests conditions. The rates were selected based on the results of slab tests and field interaction.

2.4 *Field soil-structure interaction setup*

A simple reinforced concrete structure was constructed on the same site where the block samples were obtained for the laboratory interaction studies. The structure was 4.2 m × 5.4 m in plan and 3.5 m high, and was composed of isolated square footings. The design details of the reinforced concrete structure was discussed by Abduljauwad et al. (1996).

To measure the absolute movements, two permanent bench marks were constructed (Abduljauwad et al. 1996). Surveying was used to monitor the magnitude of vertical movement of the structure and the ground surface as a result of swelling. The deformation surveys were carried out using automatic levels.

Twenty-eight sand drains 100 mm in dia. and 10.0 m deep were drilled around the structure for accelerating water intrusion to the soil. A network of pipes was used to provide water for saturation. Pipes were used with valves placed on the top of the sand drains. Figure 4 shows the complete instrumentation of the structure in the Al-Qatif area.

Swelling tests were also conducted at four locations around the structure. These tests were performed at different depths, including 0.0 m, 1.0 m, 2.0 m and 3.0 m from ground level. In order to perform tests at 1.0 m, 2.0 m and 3.0 m depths, three holes were drilled using a 25 cm auger and without using water (dry drilling). After reaching the desired depth for each hole, a 25 cm PVC pipe casing was installed and the hole was closed with a plastic plate. Later, a circular plate connected with a vertical rod was placed in each hole to estimate the heave generated in the soil as it was exposed to water. Sand was added to simulate the field stresses at the different levels.

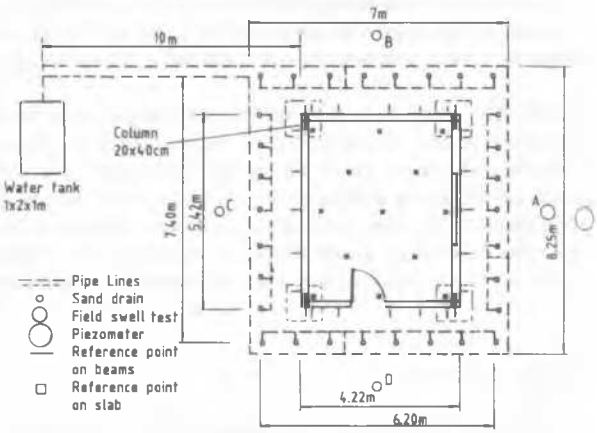

Figure 4. Plan view of structure and complete instrumentation.

3 RESULTS AND DISCUSSIONS

The investigated clay has a significantly high swell potential as indicated by the high consistency limits shown in Figure 5. This behaviour is in conformity with the high values of the percentages of swell and swelling pressures obtained using the conventional oedometer device. A typical result is shown in Figure 6. The percentage of swell was obtained by using the simple oedometer test, while the swelling pressure was obtained from both the simple oedometer and constant volume tests (Abduljauwad & Al-Sulaimani 1993; Johnson & Snethen 1978). A percentage of swell of 36% was recorded for the sample obtained from a depth of 0.5 m from the ground surface, while the swelling pressures obtained from the simple oedometer and constant volume tests were 3100 kPa and 800 kPa, respectively. Complete details of the geotechnical property and swelling characteristics of this soil were presented by Abduljauwad et al. (1996).

X-ray diffraction analysis indicated that the main soil constituents are smectite (52%), illite (23%), palygorskite (5%), dolomite (9%), gypsum (6%) and quartz (5%). The high value of the initial total suction indicated that the uppermost 1 to 3 m becomes desiccated during the summer period, during which time the drilling was conducted. The final suction was measured for soil samples obtained 4 months after flooding the site with water through the vertical sand drains. Both initial and final suctions were measured using a transistor psychrometer developed by Soil Mechanics Instrumentation (Woodburn et al. 1993).

The heave prediction shown in Figure 5 was obtained from swelling pressure measurement, using both simple oedometer and fixed volume tests. The equations presented by Johnson and Snethen (1978) were used to calculate heave.

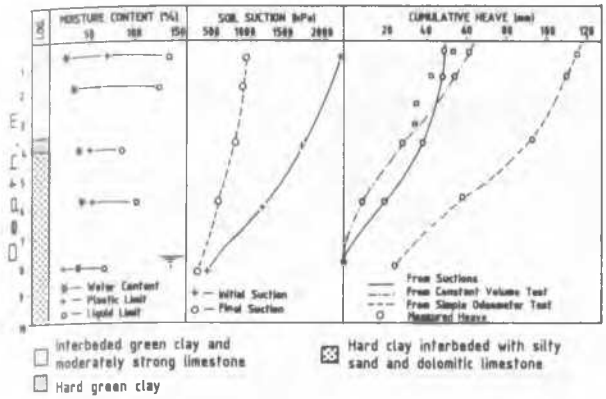

Figure 5. Soil profile and heave comparison.

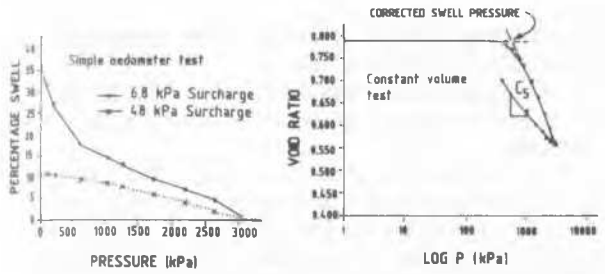

Figure 6. Percentage of swell and swelling pressure for selected soil sample.

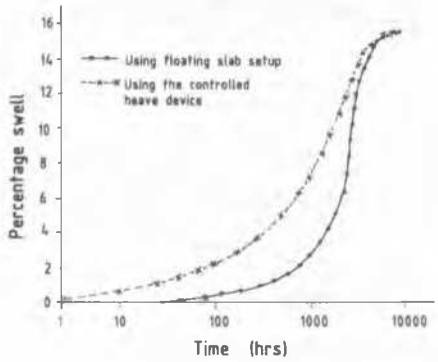

Figure 7. Percentage of swell versus time for floating slab and controlled heave test.

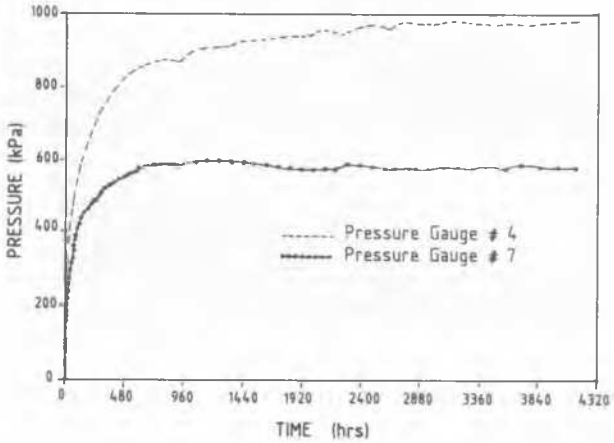

Figure 8. Variation of pressure with time for fixed slab.

3.1 *Floating slab*

The plot in Figure 7 depicts the percentage of swell versus time for the floating slab setup using a logarithmic time scale. The swell seems to stabilize at about 16% after a time interval of about 10000 hours (a 14 month period). This long period is due to the low permeability of the clay under investigation ($K = 10^{-8}$ cm/s).

The 16% final swell from the floating slab test compares to the 36% swell obtained from the simple oedometer test, the latter being 2.4 times greater. The difference in result can be attributed in part to the complete lateral rigid confinement in the oedometer. Also, the presence of open fissures will consume part of the swell in the slab test.

3.2 *Fixed slab*

Figure 8 shows the variation of pressure with time for selected pressure gauges. Pressure gauge No. 4, which is located at the corner, gave a maximum swell pressure of 950 kPa. As expected, the minimum pressure was recorded by pressure gauge No. 7 located under the centre of the slab. The average swell pressure for all gauges was 780 kPa.

A comparison with the constant volume test on clay with similar initial moisture content shows the measured swelling pressures to be almost identical. In contrast, the simple oedometer test yielded a swelling pressure about 4 times greater than the one measured from the fixed-slab setup. This is because in the simple oedometer test the soil specimen passed through two phases of deformation and compression.

3.3 *Controlled heave rate test*

Figure 7 shows the effect of sample size on the percentage of swell. The percentage of swell was measured in the controlled heave device, while the sample is free to move upward under a surcharge of 6.8 kPa. It is very clear that the rate of heave for the small sample (100 mm × 100 mm in cross section) is higher than the large sample (500 mm × 500 mm in cross section), which was tested in the floating slab setup. The thickness and boundary conditions are the same for both samples. However, the maximum percentage of swell for both tests are identical (16%).

Figure 9 shows the variation of pressure with time, while no vertical movement was allowed. A maximum swell pressure of 1000 kPa was recorded. This value is similar to swell pressure recorded by pressure gauge No. 4 under the corner of the fixed slab. The rate of heave during the primary swelling was 3.5×10^{-3} mm/hrs under a pressure of 6.8 kPa as depicted in Figure 7. The test was repeated twice and the bottom plate was allowed to move downward with the rates of 3.5×10^{-2} and 3.5×10^{-1} mm/hrs, respectively. The movement of the plate was allowed after 10 hrs of sample inundation. The tests were terminated when a reduction in pressures was observed. Figure 9 indicates that an increase of 50 kPa in pressure was obtained in the sample in which the rate of heave was 0.35 mm/hr compared to an increase of about 700 kPa in the sample in which the vertical movement was not allowed. The reduction in pressure was about 93%.

3.4 *Field soil-structure interaction*

Figure 10 schematically depicts the heave of the base of the structure for 50, 100 and 300 hour time intervals. The figure shows that, after 50 hours, the structure as a whole has already heaved about 12 mm. Even though the soil profile around the structure was quite uniform, a differential heave was observed. After about 50 hrs of water migration into the soil, the corner located between points 1 and 24 heaved more and faster than the other corners. This corner was the closer one to the water tank. Even though the soil profile

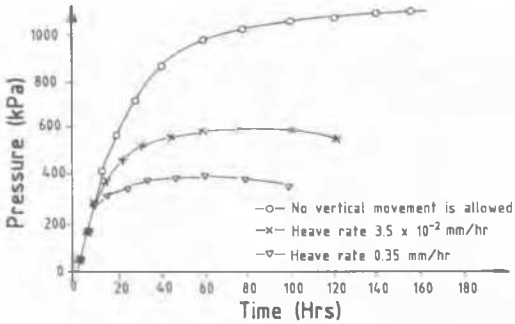

Figure 9. The effect of heave rate on swelling pressure.

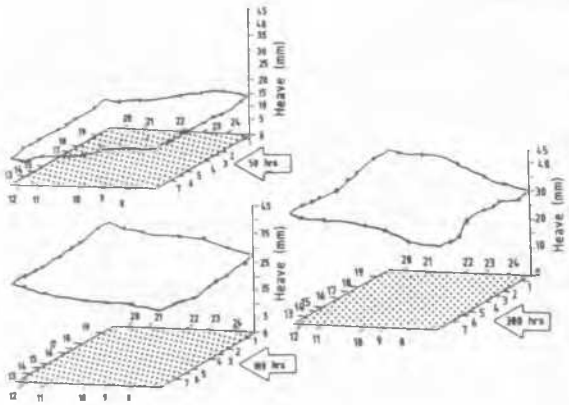

Figure 10. Heave of structure with time.

around the structure was quite uniform, more fissures were observed near this corner. The presence of fissures led to a higher permeability and more moisture migration under this part of the structure. However, later after about 100 hrs of moisture migration the other corners, in particular the one located between points 19 and 20, moved more. This is because some of the swell close to the corner located between points 1 and 24 has been consumed by the filling of fissures present in the clay.

The surface heave and the three deep heaves at 1 m, 2 m and 3 m depths are shown in Figure 11. The high surface heave of 55 mm compared to the heave of 37 mm at a depth of 1 m and 35 mm at 2 m depth indicates that the uppermost 1 m becomes highly desiccated in the summer months. There is only about 4 mm difference between 1 m and 3 m depths, even though Figure 6 indicated high desiccation in this zone. This is because the circular plate of the rod-settlement gauge at a depth of 2 m is located in a zone with a large number of open fissures which absorbed a large part of the movement. For the 3 m depth, the circular plate was located in a zone which was subjected to a higher value of an overburden pressure. Once the primary movement has started, the rate of change of degree of heave is fairly uniform, irrespective of depth. The primary phase was completed only after 100 hrs for all depths.

A comparison of measured heave at the different depths 0, 1, 2 and 3 m with the calculated heaves (Figure 5) indicated that the soil suction and constant volume test gave a much better prediction for heave than the swelling pressure measured from the simple oedometer test.

4 CONCLUSIONS

1. The simple oedometer test overpredicts the percentage of swell and swelling pressure due to the complete lateral rigid confinement which is influenced by the rigidity of the oedometer

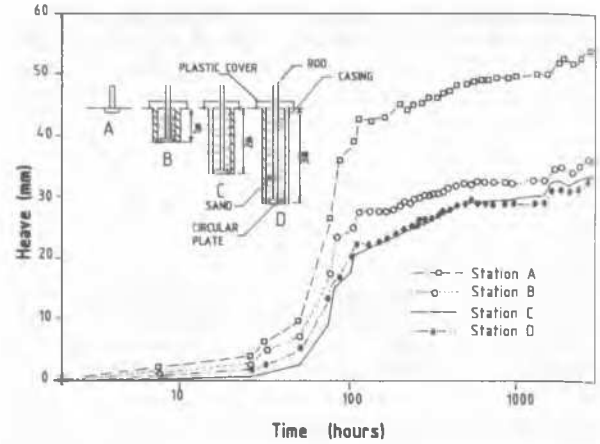

Figure 11. Surface and deep heave around the structure.

used. The floating slab setup and size of the specimen are more representative of the field conditions.

2. The average pressure between the slab and soil from the fixed slab test was in agreement with the result obtained from the constant volume test.

3. The controlled heave rate test indicated that a very slight vertical movement will have a significant effect on swelling pressure.

4. The presence of nonuniform distribution of open fissures in expansive soil will lead to differential heave and a reduction in swelling pressure because part of the swell will be consumed by fissure voids.

ACKNOWLEDGEMENTS

The research described in this paper was supported by King Abdulaziz City for Science and Technology (KACST) under grant AR-10-061 which provided financial support for this study. The support provided by King Fahd University of Petroleum and Minerals for providing the laboratory facility is gratefully acknowledged.

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