SWELLING SOIL UNPREDICTABLE BY CLASSICAL GEOTECHNICAL TESTS

SOL GONFLANT INDETECTABLE PAR LES ESSAIS GEOTECHNIQUES CLASSIQUES

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SYNOPSIS : Unpredictable swelling of aridic soil is explained on the basis of a case history : area N.E. of Riyadh, Saudi Arabia. At the design stage, classical geotechnical investigations didn't mention the possibility of volume change. Heaving and structure damages appeared several years after construction, due to progressive moisture increase of the soil from uncontrolled gardens and patios watering. This behaviour, still difficult to highlight by classical geotechnical tests has been studied from a point of view relevant to multidisciplinary soil investigations (i.e. clay mineralogy, micromorphology, ...) with the geotechnical engineer. The swelling process seems to occur within a restricted range of moisture content which can be related to capillary stresses (i.e. negative matric water potential). Consideration is to be given to these factors in swelling tests; the classical tests use gravitational water pressure which is a too high water potential (positive) to reproduce the field phenomenon.

INTRODUCTION AND HISTORY

In 1976, soil investigations were performed by international geotechnical contractors on the site of a very large building complex (500,000 square meters) erected on desert soil at about 20 km NE of Riyadh, Saudi Arabia.

At this stage (design and soil investigation), no evidence of swelling soils was discovered by the classical geotechnical tests. The presence of potentially swelling soils was unknown in this region, and still unproved in 1988 (DHOWIAN et al 1988).

The construction works started in 1977 and, in early 1981, the buildings were completed and ready for operation. It was at the end of 1985 that the first damage was observed : cracks in masonry walls and minor settlement along outer gangway. The conclusion at that time was that the damage was the result of minor, unrelated, problems of no significance.

By September, 1986, the damage in the buildings had increased in degree and in area involved. As the cause was unknown, it was decided in 1987 to monitor the situation and to perform an additional soil investigation. Over the following months the damage continued to spread and became more extensive.

DESIGN OF THE BUILDINGS

The main building is a single level reinforced concrete structure, composed of two separate bearing structures :

- a main structure supported by footings poured in the natural soil;
- a groundfloor slab poured on a backfill, independent of the main structure, and supporting the internal walls.

The ground coverage of this building is approximately six hectare, or 60,000 square meters. Included in this total are patios and courtyards which cover a combined area of slightly less than one hectare.

The columns are mounted on individually isolated footings based on a grid design system with 7.2 meter centres.

The footings have been designed to accommodate a net bearing pressure in the natural soil of 0.25 MPa. All footings were placed in the natural soil strata at depths varying from 2.7 meters to 3.5 meters below the finished floor level. The size of the footings varies from a minimum of 1.5 meter to 2.7 meter square.

The backfill was put in place and compacted to a dry density of 95 % optimum Modified Proctor, after the completion of the footings and the lower column sections.

DESIGN SOIL INVESTIGATIONS (1976-77)

Description

This geotechnical investigations made at the design stage included the following field tests :

- geophysical surveys;
- drilling program with sampling and related laboratory tests;
- borings with pressuremeter tests.

The core borings were made down to 15 m deep with sampling and SPT's. Three Menard pressuremeter borings tests have been carried out down to 13 m deep to verify the mechanical behaviour of the in situ soil.

The samples are submitted to the following classical tests :

Identification	Geomechanical	Chemical
. dry density	. shear	. pH
. water content	. oedometer	. sulphate
. Atterberg	. triaxial	. chloride
. grain-size		. carbonate...

Results And Conclusions In 1977

The bedrock is made of beige limestone more or less weathered, covered by reddish brown fine to very fine sand with some silt and fine gravel, and containing occasional lenses and thin layers of clayey sand and clay. This material is in a medium dense to very dense state of compaction. (Fig. 1.)

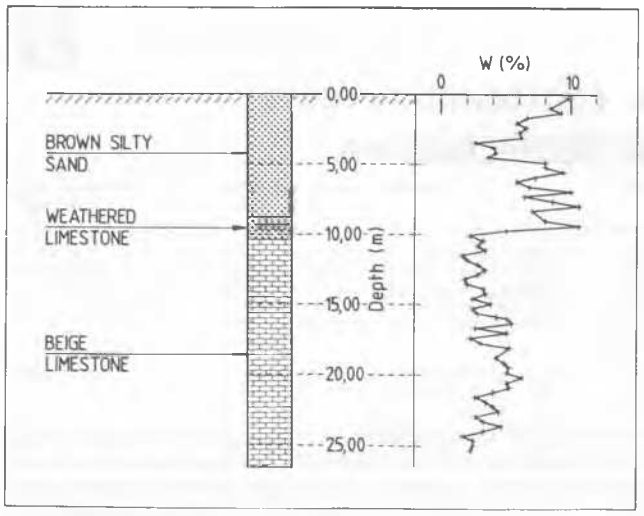

Fig. 1. Typical borelog and water content (1988)

No ground water has been found in the borings. Due to the dry nature of the materials encountered, it has been recommended by the geotechnical contractors to give special attention to draining surface water away from the edges of the buildings to avoid loosening of the foundation soil and differential settlement.

The ASTM classification has been plotted (Fig. 2.) together with the swelling potential related to the liquid limit WL and three curves showing areas with different Free Swelling (FS in %) experienced in same country (Oolo et al 1988). According to the laboratory tests, the clayey fraction of the soil is a CL type (lean clay), while the swelling clays are generally classified into CH type.

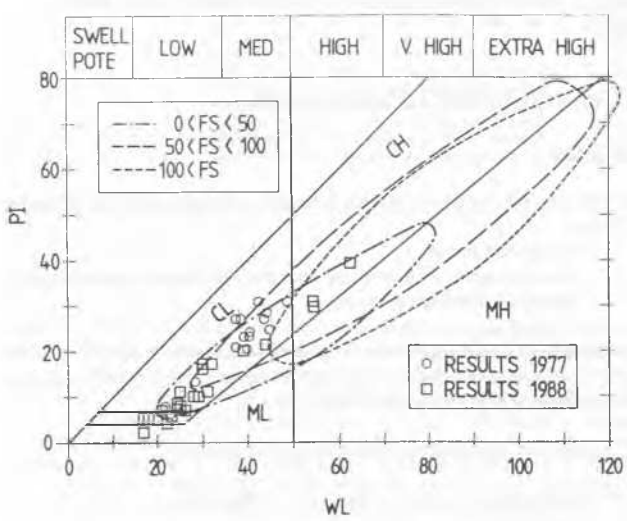

Fig. 2. Soil classification according to ASTM D2487

No possibility of presence of swelling clays had therefore been mentioned by the geotechnical contractors.

Using the SPT results, the pressuremeter values and the Meyerhoff formulas, the allowable bearing capacity was higher than 0.3 MPa for footings founded deeper than 1.5 m. At this depth, the SPT N values were higher than 50 blows, the effective friction angles above 30° and the limit pressure higher than 2 MPa with pressuremeter modulus bigger than 40 MPa.

HEAVING/TOPOGRAPHICAL OBSERVATION (1987-1990)

To understand the sudden modification of behaviour shown by the structures and the foundations, multiple topographical surveys of the roof slab, and of the ground floor slab have been performed between 1987 and 1990, using more than 400 topographical points.

The surveys of the roof slab were performed in order to identify and evaluate a possible foundation movement, from measuring points located above the columns. Topographical surveys of the ground floor slab were performed in order to evaluate the behaviour of the backfill.

By mid 1988, damages, observed from 1986 but then unexplained, were attributed to heaving of the soil. The location of the heaves, and of the abnormal openings of the roof slab joints, identified the irrigated patios as the main centres of the lifting activity. It was believed that water from excessive irrigation was reaching the underlying soil and was the cause of heaving and localized superficial soil collapses.

Also eliminated as a cause of the damage were the normal temperature variations which have been continuously present. The expansion joints have been operating satisfactorily.

ADDITIONAL SOIL INVESTIGATION (1988-1989)

Description

Together with the topographical observation, extensive additional geotechnical and mineralogical investigations have been performed (Lousberg et al 1988-89).

The field investigations consisted of static cone penetration tests (CPT's), dynamic cone penetration tests (DPT's), core and auger borings, trenches and plate loading tests. Disturbed and undisturbed samples have been taken. All the tests were conducted according to the applicable ASTM, British Standards (BS) or internationally accepted standards.

Identification tests made on disturbed samples were :

- grain size analysis and sedimentometry (ASTM D422),
- Atterberg limits (ASTM D4318) or sand equivalent (ASTM D2419),
- shrinkage limits (ASTM D427).

The moisture contents have been also measured (ASTM D2210).

On selected samples from the backfill modified proctor tests (ASTM D1557) and CBR tests (ASTM D1883) have been made to determine the compaction characteristics of the soil.

Chemical analyses, i.e. pH, and the determination of sulphate, chloride and carbonate contents, have been made according to BS 1377.

Swelling tests according to ASTM D4546 method A, B and C, together with density measurements, moisture contents and identification tests, were carried out on undisturbed samples to measure their swelling pressures and percentage of heave.

Interpretation Of The Tests

Effect of water

Both the CPT's and the DPT's have provided high values in the non irrigated areas, in both the backfill and the natural soil. In the patios, the penetration tests have all been stopped on a whitish hard layer at four meters deep.

The effect of water was obvious. The higher the moisture content, the lower the soil resistance.

The evolution of water content measured in borings also indicated the penetration

of the water from the ground surface (Fig. 1). The water content measured outside of the building area showed a general very low water content, almost dry (W = 2 to 4%).

No swelling clays according to identification tests

The natural soil can be directly classified from the results of the identification tests. According to the ASTM classification system, the clayey fraction of the natural soil is mainly «lean clay» of the CL-type (Fig. 2).

The classical geotechnical identification characteristics did not indicate the possibility of swelling (now in evidence).

There was a high quantity of fine crystals of calcite (micrite) in the soil, mixed with clay particles, which agglomerate into greater size particles. Where the apparent sedimentometric clay content was low, a decarbonatation of some samples (according to the ASTM method using boiling acid) revealed a larger quantity of clay. For the «active» clay particles smaller than two microns, the quantities were sometimes increased by more than ten percent after the acidic treatment. The clay contents found in the soil were not that high but it is well known that low clay content can also induce swelling.

Swelling tests

It's only after cautious and extensive tests that swelling pressures have been found in samples from the site, with a maximum pressure of about 70 KPa at five meter depth although those samples have been contaminated by water in the past.

But some samples produced no swelling and sometimes collapsed during the ASTM swelling tests when inundated.

The distribution of some significant swelling tests results have been produced on figure 3, showing that no real swelling pressure able to heave all the foundations and the surrounding soil has been discovered.

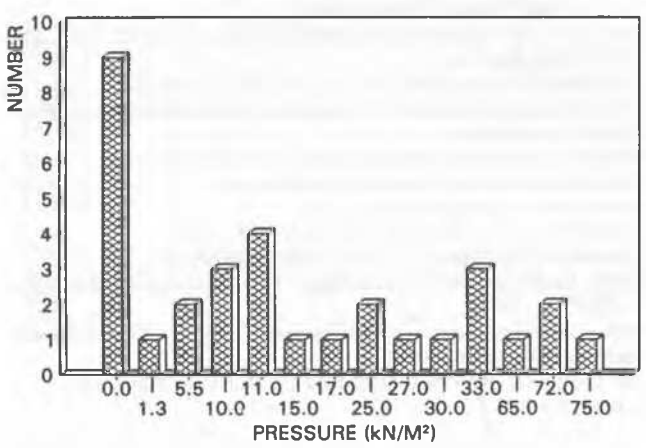

Fig. 3. Distribution of swelling pressures in the first 5 m deep

MINERALOGY AND MICROMORPHOLOGY (1988-1992)

After using soft decarbonatation treatment, the presence of smectite (swelling mineral) amongst palygorskite, illite and kaolinite as clay fraction (smaller than 0.002 mm) in a quartz and calcite matrix has been detected on XRD patterns.

These results have been presented in a former paper (MARCOEN, et al, 1992), where the swelling behaviour of that soil has been clearly demonstrated by the complementary micromorphological approach.

Because, not only the quantity and the nature of the clayey materials but also their organization govern the hydration and the swelling during the moisturizing process, the «energetic history» of the materials must also be taken into account : it is obvious that the climatic condition, here the extreme aridity during very long periods, is a major element of the clay movement mechanism.

The following elements must be highlighted :

- the water potential gradients developed during the remoistening;
- the maximum constraints, climatic or geostatic, on the material at a certain time of its «history».

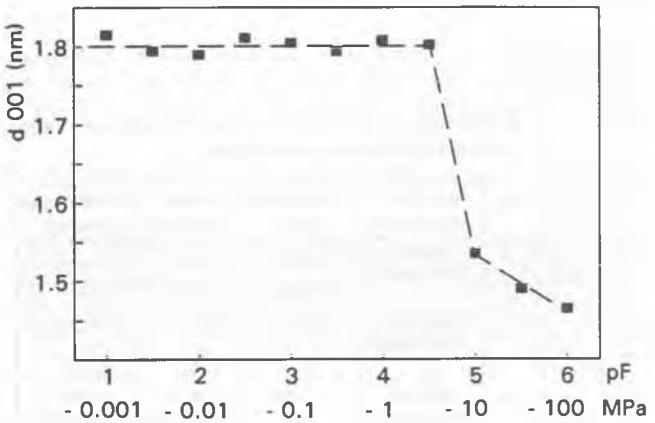

Fig. 4. Ca-Smectite interlayer spacing d001 related to water potential

Most of the researches have been limited to the study of the swelling processes of smectite clay on the scale of interlayer spacing, d 001 in MILLER notations. However, as seen on figure 4, the smectite has interlayer spacing at maximum and is very stable for water potential higher than -5 MPa. In fact, for a comprehensive understanding of the behaviour of clay with respect to the presence of water it is necessary to take into account not only the interlayer spacing variations but also the microstructure of the clay-water system.

Most volume changes observed under such aridic field conditions occur at high negative pore pressure (matric potential), soil swelling is thus not only related to layer distance change but rather to changes in interparticle distances and pores (Fig. 5 after TESSIER, 1991), which induces macroscopic behaviour.

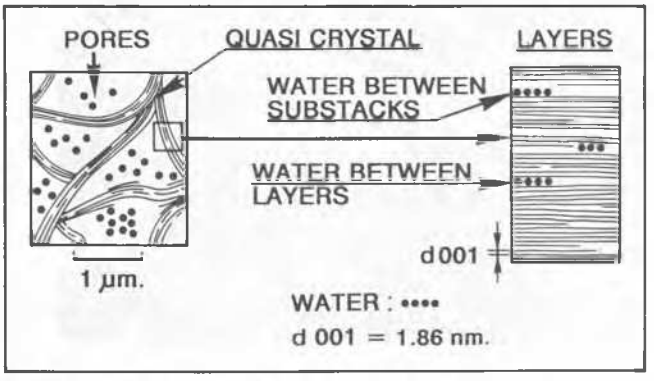

Fig. 5. Network of quasicrystals in hydrated Ca-Smectite

Information on soil micro-organization can only be obtained by electron microscopy observation carried out so as to preserve the microstructure of the clay at defined water potentials.

According to recent methods, Scanning Electron Micrography (SEM) and Transmission Electron Microscopy (TEM) equipped with an High Energy Electron Diffraction (HEED) were achieved.

SEM observations of a representative sample hydrated at -1KPa (swelled) have shown clay particles systematically surrounding skeleton grains, and mainly oriented among large fracture planes crossing the microstructure. (Fig. 6.) The TEM and HEED studies confirmed the nature of the planar crystals as smectite (swelling clay) closely interbedded with the large fibres of palygorskite.

Because the clay was mainly located between skeleton grains of quartz and calcite, the best conditions were met for a maximum swelling, followed by a loss of cohesion of the material when water potential reached atmospheric pressure.

Table 1. Water energy levels in the field compared to those used during swelling tests (between A and B lines)

	APPLICATION DOMAIN	WATER POTENTIAL	PRESSURE (MPa)	pF (log cm H2O)	PORE DIAM. (micron)
A	SATURATED SOIL	Gravity (free water)	> 0	# 0	> 150
	UNSATURATED SOIL	SUCTION	- 0.001	1	150
B			- 0.01	2	15
		PRESSURE = MATRIC POTENTIAL	- 0.1	3	1.5
			- 1	4	0.15
			- 1.58	4.2	0.095
	ARIDIC SOIL		- 10	5	0.015
			- 50	5.7	0.0030
			- 100	6	0.0015
	105 °C			# 7	

A : Saturation Capacity B: Retention Capacity

When water potential increases, indeed, clay swells more, but cohesion forces become very weak, resulting in a mechanical failure of the material. As a consequence, millimetric pores collapse. Because volume reduction due to the disappearance of these pores is higher than volume increase associated with clay swelling, high water content results in slaking of the aggregates.

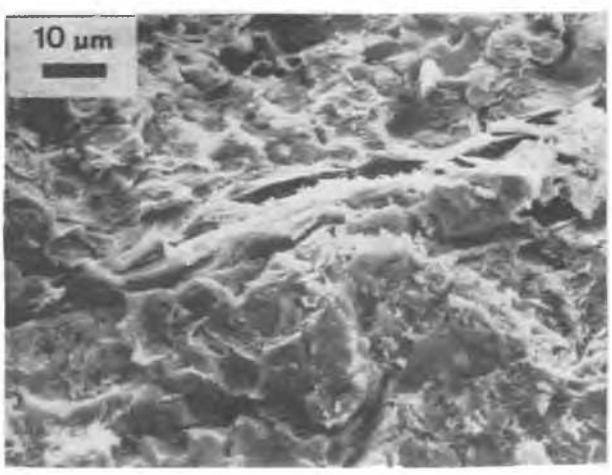

Fig. 6. SEM of a fracture surface of the bulk material.

Classical tests (oedometer type) failed to explain the swelling stage of the soil because, in these tests, the samples are put in contact with free water before applying the mechanical loads. (Table 1) In table 1, the equivalent pore diameters filled with water are connected to their requested water potentials. Water with pressure lower than - 0.01 MPa should be used during the swelling tests.

CONCLUSION

This paper shows the interest of studying soil microstructure on hydrated samples for a better understanding of soil geotechnical properties. In desert soil, submitted to very dry conditions, the soil behaviour must take into account the wetting process. When interstitial pressure remains negative the soil material can swell without collapse of the pores.

Classical geotechnical tests have to be adapted for this purpose. For a better prediction of such soil behaviour it is thus necessary to apply mechanical compression on unsaturated materials, i.e., when samples are prepared at negative water pressures as they are in situ. If so, the swelling stage of the soil can be demonstrated.

Clay studies made according to geotechnical and micromorphological methods have contributed to a better understanding of soil behaviour. A cross fertilization can result from linking different scientific approaches.

ACKNOWLEDGEMENTS

We express our sincere gratitude to Dr. D. TESSIER (INRA Versailles, France) for his helpful contribution.

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