

# Damaged Houses on Pile Foundation in Expansive Soil

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## SYNOPSIS

A group of 176 residential houses, on soil probably expansive and pile foundation, has suffered damage in brick walls and partitions. A three-dimensional finite element method has been applied to the set soil-piles-foundation beams to find out the relationship between swelling and stresses in several structural elements. It has been shown that the reason for the cracks is having embedded the foundation beams in the soil. A neighbouring set of houses with exactly the same design, but pier foundation and the foundation beams separated from the ground has suffered no damage.

## INTRODUCTION

A group of 176 residential houses on pile foundation, in Seville, has suffered damage in brick walls (fig. 1) and partitions. The research undertaken to ascertain the cause of damage will be described below. A preliminary report on soil-structure interaction has already been published (Justo et al., 1987).

## STRUCTURE AND FOUNDATION

The houses, two storeys high (fig. 1), have a frame structure and brick cladding. There are expansion joints each three to four houses (fig. 1 and 4).

The foundation soil was supposed to be expansive and a pile foundation was chosen (fig. 2).

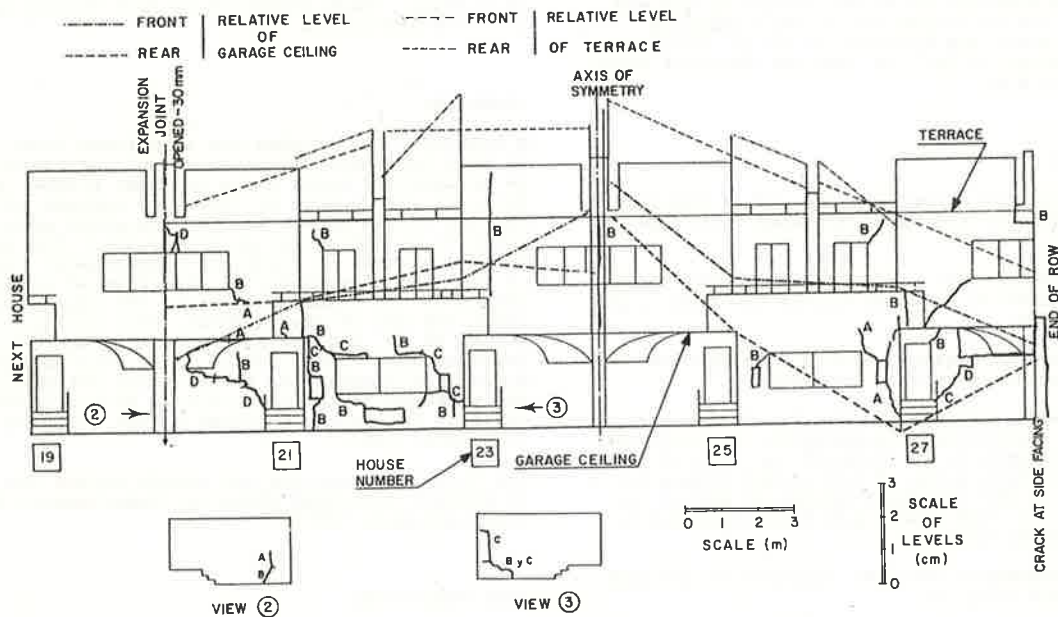


Figure 1: Cracks in façade (Roger de Lauria St.) and levels of garage ceiling and terrace

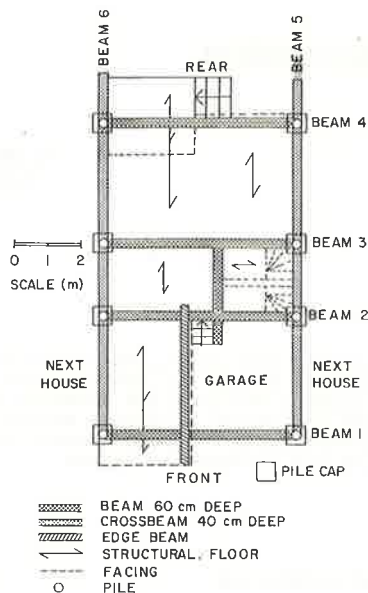


Figure 2: Foundation and ground floor structure of one house

The ground structural floor, 30 cm above the soil, is supported by beams, 60 cm deep, embedded in the ground and supported by the piles. The pile caps are braced in a direction perpendicular to these beams by crossbeams, 40 cm deep, also embedded in the ground. There is one single pile for each cap, except under expansion joints, where there are two piles under each cap. The pile diameter is 45 cm, except at the end of each row of houses and at the expansion joints, where the diameter is 35 cm. The depth of the piles is 10.5 cm, and the depth of reinforcement 8 m.

#### CRACKING

Cracks existed already at the end of construction (June 84) and have increased with time (fig. 3).

Damage ranges from non-existent in some houses to a maximum in Roger de Lauria St. (fig. 1 and 3) of 14 mm.

The following damages related to movements in the structure have been observed:

1. Cracks in cladding, in the façade and rear, with a predominantly parabolic pattern that corresponds to a rising of the axis of symmetry of each group of 3-4 joined houses respect to the expansion joints or end of row (fig. 1). The cracks are more important in the lower floor.
2. Predominantly vertical fissures in the side facings (fig. 1).
3. Cracks in partitions and glazed tiles corresponding to the general pattern indicated under paragraph 1 above.

4. Fissures, breakings and separations of floor tiles.

5. Sticking doors and windows.

A report delivered in May 1985 indicated the existence of cracks that cross the wall in a 14% of the houses, cracks in a 49%, and at least fissures in a 97%.

Notwithstanding this, it cannot be said that the whole urbanization gives a sensation of distress. The highest damage corresponds to the houses of figure 1. The width of the cracks indicated in this figure is shown in Table I. This damage may range from appreciable to severe according to the criterion of Macleod and Littlejohn (1975), because although there are cracks up to 14 mm wide, crossing the wall, no loss of strength in beams has been detected up to now.

Table I

Width of the cracks grouped under different letters in Figure 1

Crack group	Width (mm)
A	>0.1-0.5
B	>0.5-2
C	>2-6
D	>6-15

According to the criterion of the British Department of the Environment (v. Geddes, 1984) the damage would be classified as "moderate", and according to Bozozuk (v. Driscoll, 1984) as "heavy".

Figure 4 shows the general aspect of two houses in the most damaged street. Three years after the end of construction, the house No. 21 has not yet been occupied by his owner, and there is a legal claim presented by five owners against the architects.

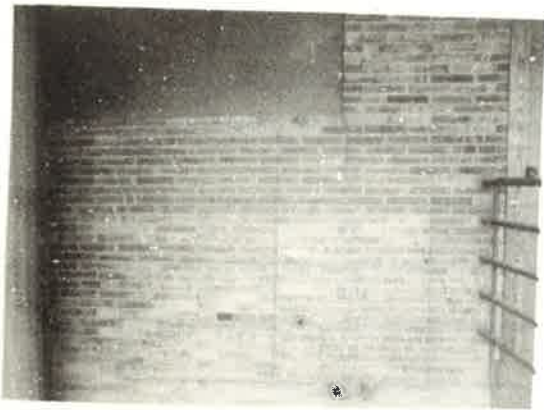
#### LEVELLING

A levelling was carried out in October 1986, more than two years after the end of construction. As no levelled bench marks had been placed before, it was decided to level the corners of the ceiling of the garage and terrace, which were supposed to be horizontal as constructed. The results are shown in figure 1. A systematic rising of the axis of symmetry of the group of joined houses of the figure respect to the expansion joint and end of row is shown. This rising is around 55 mm, and the angular distortion of the façades range from 1/100 to 1/300. The measured distortion corresponds to the crack pattern, and the measured deflection ratio (0.002 or 1/440) explains the damage suffered by the building (v. Justo, 1987).

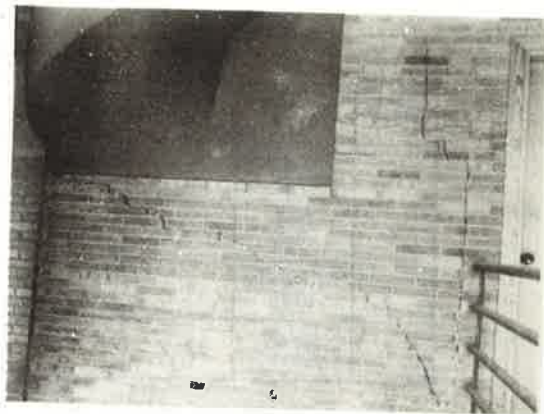
The levellings carried out inside follow the same pattern indicated above (v. Departamento de Cimentaciones, 1987).

#### SOIL PROPERTIES

Table II collects some average properties of the soil layers. The symbols recommended by the ISSMFE have been used whenever it was possible.



a) November 85



b) September 87

Figure 3: Comparison of cracks in brick facing at the bottom of garage in No. 21 of Roger the Lauria St., in two dates

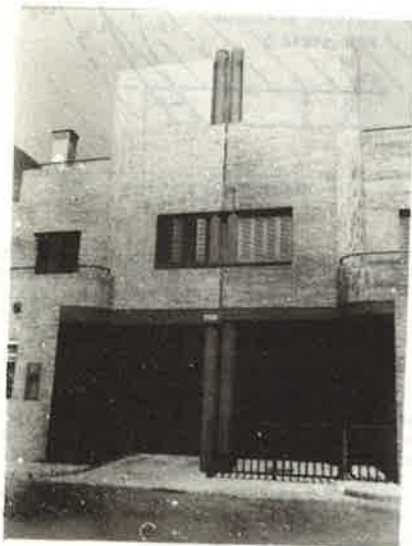


Figure 4: Houses No. 19 and 21 of Roger de Lauria St., showing cracks around expansion joint

The water table appears at a depth around 6.5 m.

A first conventional site investigation was carried out in December 81, six months before the beginning of construction. Swelling evaluation was based upon Lambe's test (Lambe, 1960), which gave swelling indexes of 300 and 120 kPa for layers 2 and 3, respectively, of table II.

From one single oedometer test carried out in an undisturbed sample of layer 2 it may be deduced that swelling pressure of this sample lies around 17 kPa, and a collapse of 2.5% under overburden pressure was obtained. The liquidity index was 0.09.

A second site investigation was carried out in the summer of 1986, when damage was well established. The water content was near the plastic limit. Many swelling-under-loading tests, on undisturbed samples, obtained near the site of figures 1, 3 and 4 were carried out. The samples were flooded at the oedometer, and volume change under the overburden pressure ranged between a swelling of 0.9% and a collapse of 4.8%; the average value for the active layer was a collapse of 1.4%. The samples were not far from saturation in its natural state, and the index  $(e_0 - e_L) / (1 + e_0)$ , where  $e_L$  is the void ratio corresponding to the liquid limit, was around -0.5. So, according to Miheev's criterion the soil would not be collapsible (v. Justo and Sae tersdal, 1981).

Wetting and drying cycles, applied at the laboratory, indicated for this soil a rather special behaviour. Each new cycle produced an increase in void ratio (fig. 5). In the figure the first wetting produces a swelling of 1%. The second wetting, after shrinkage at an approximate suction of  $pF=6$ , produces a swelling of 4.8%. The second drying produces a shrinkage of 3.4%, also larger than the first one (1.8%). The third swelling (3.7%) and shrinkage (3.3%) are somewhat smaller than the second ones, but still rather important.

We also see in the figure (2nd wetting) that flooding the sample may produce collapse followed by swelling. The reverse has been described when suction is decreased up to zero (Escarro and Saez, 1973).

Recently Alonso et al. (1987) have developed an interesting theory to explain the volume change of expansive soils. The theory is based upon the existence of three yield surfaces (fig. 6) in a pressure-suction space.

State 0 corresponds to the initial state of the sample, and inside the yield surfaces the sample exhibits an elastic behaviour. When a yield surface is reached plastic (irrecoverable) deformations begin.  $S_1$  is the yield surface for a suction increase, and seems to correspond to the highest suction,  $s_0$ , ever experienced by the



Table II  
Soil properties

Layer	Soil type	Depth m	T <sub>200</sub> %	w <sub>L</sub>	I <sub>p</sub>	ρ <sub>d</sub> kg/m <sup>3</sup>	q <sub>u</sub> kPa	N <sub>d</sub> blows/20 cm	N blows/30 cm
1	Brown clay	1.0	89	58	31	1,570	50	12	
2	Red clay	5.5-6.5	95	46	27	1,770	220	29	20
3	Gravelly clay	8.5	58	31	15			108	
4	Sandy gravel	12.5						R	

T<sub>200</sub> = percentage passing ASTM 200 sieve  
 q<sub>u</sub> = unconfined compressive strength  
 R = refusal

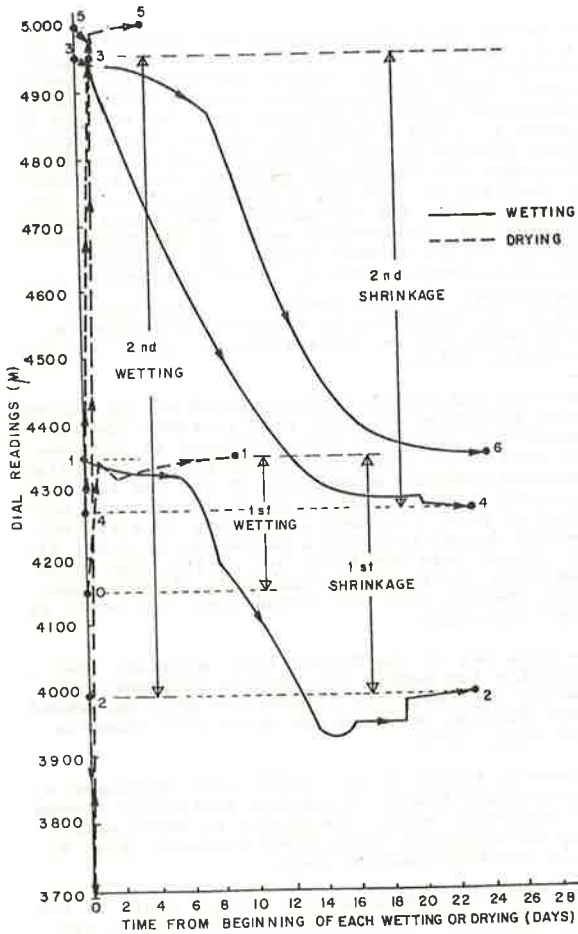


Figure 5: Wetting and drying cycles for undisturbed sample. Numbers indicate sequence of wetting and drying. Sample height - 20 mm. Normal pressure 2.6 kPa. Roger de Lauria No. 21. Depth 2 m

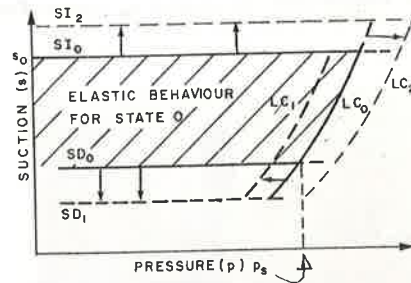


Figure 6: Yield surfaces in pressure-suction space and coupling between yield surfaces for expansive soil. Pressure is vertical stress for oedometer test or mean normal stress for triaxial test

soil. SD is the yield surface for a suction decrease; this surface is specific of expansive soils, and does not exist in less plastic soils. It accounts for the irrecoverable swelling of expansive soils when suction is decreased. LC is the yield surface for loading-collapse, and takes account of the irrecoverable settlements when suction is decreased, beyond the swelling pressure,  $p_s$ .

The figure gives also the coupling between the yield loci when a yield surface is surpassed. - When SI is surpassed, LC changes to LC<sub>2</sub> in a process of strain-hardening. When SD is surpassed, LC changes to LC<sub>1</sub> in a process of strain-softening.

This theory would imply an essentially elastic behaviour after the first cycle of wetting-drying as far as the "preconsolidation" suction, previously reached is not surpassed. Experimentally, this elastic behaviour has been checked after the first drying in soils of low plasticity (v. Alonso, 1987) and after the first wetting in plastic soils (Justo, 1982; Delgado, 1986; Alonso, 1987).

On the other hand, in the soils that we are dealing with in this paper, there is a very important accumulation of irreversible expansion, specially up to the second wetting. A slow accu

mulation of expansion has been observed in other soils plastic or moderately plastic (v. Escario and Sáez, 1986; Alonso et al., 1987).

In a sample taken in No. 25 of Roger de Lauria St. (v. fig. 1) the swelling of a sample who had previously suffered one cycle and a half of drying-wetting was, under the overburden pressure, 2.1%.

As a summary the samples tested give, in their natural state very small or no swelling, but this might be due to an increase in its moisture content respect to its state before construction. On the other hand, shrinkage is important, and swelling after shrinkage very important in many cases.

#### INVESTIGATION OF STRUCTURE

As the measured difference of level (v. fig. 1) might also be interpreted as a settlement at the expansion joint and end of row, the piles at the rear of the expansion joint were discovered (fig. 7). The strength of both the concrete of the pile and the cap was adequate (from 20 to 25 MPa).

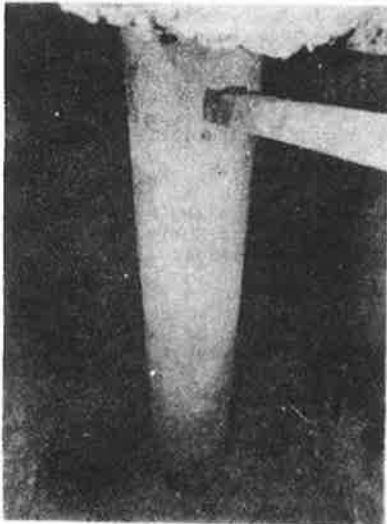


Figure 7: Piles discovered up to 3 m depth, at the expansion joint

One foundation beam was discovered in No. 21 of Roger de Lauria (v. fig. 1 to 4) and no fissures were found.

#### FINITE ELEMENT ANALYSIS

The set soil-pile-foundation beams has been analyzed by a three-dimensional FE method developed by Justo (1982), and Justo et al. (1983 & 1984). The details of the analysis have been reported by Justo et al. (1987) and will not be reproduced here.

The conclusion of the analysis is that rupture

appears first in the unreinforced zone of the pile, at 8 m depth, for a swelling of the free soil profile, (under the overburden pressure) ranging between 0.3 and 0.4%.

If the beams were free from contact with the soil, the stresses would decrease. The tensions in the pile would be around three times smaller (fig. 8). So, the problem with the piles in this building is due mainly to not having freed the beams from the soil.

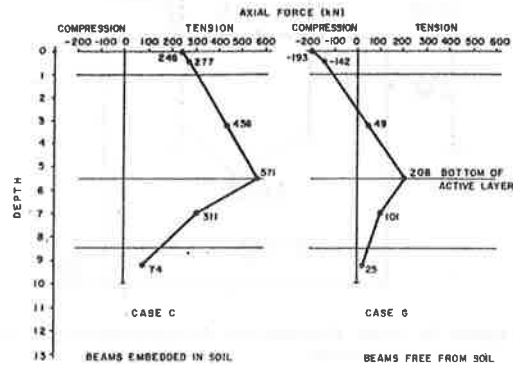


Figure 8: Axial forces in 35 cm pile for 0.37% swelling

The calculated maximum heave when the rupture of piles is produced is of the order of 3 mm. As, in this case, differential heaves of 55 mm have been measured, the piles must be broken at a depth around 8 m.

#### CLIMATE

There is some evidence to show that damage increases after rainy spells (v. Departamento de Cimentaciones, 1987).

#### PERFORMANCE OF A NEIGHBOURING SET OF HOUSES

Less than 1 km from this urbanization there is another one designed by the same architects, with the same design. The foundation soil is less plastic than the one of table II (liquid limit from 28 to 40 in the active layer). The houses have a reinforced pier foundation up to a depth of 2.5 m, with the connecting beams separated from the ground (fig. 9). The houses have suffered nearly no damage.

#### CONCLUSIONS

The paper shows that a pile foundation alone is not a safeguard against cracking in expansive soil if attention is not paid to the details. Very small swelling may produce rupture of piles and beams when these are not freed from the soil.

The finite element method is a valuable tool to resolve problems of soil-structure interaction in expansive soils. Stresses and displacements are easily found, and the influence of different

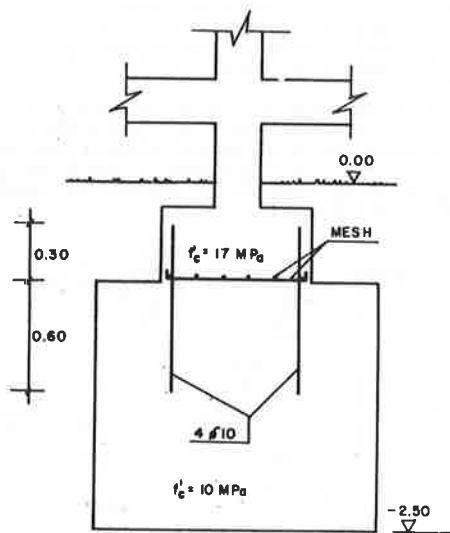


Figure 9: Pier foundation of neighbouring urbanization

parameters may be ascertained.

The behaviour of expansive soils is complex. Due attention should be paid to future suction changes from the moment of sampling. Suction-controlled tests are recommended.

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