

A Finite Element Method to Design and Calculate Pier Foundations in Expansive-collapsing Soils

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SUMMARY A three-dimensional finite element method has been devised to calculate the movements and stresses of a pier foundation on expansive-collapsing soil isotropic, non-linear and non-homogeneous. The lack of linearity and homogeneity due to the stress state is taken into account through the dependence of the initial swelling and oedometric modulus upon the vertical stress as explained in the paper.

Stress-path has a large influence in the calculated heave and tensions in pier, and is taken care of.

A parametric study has been made. The parameters that have more influence in the problem are: swelling-collapse of the free soil profile, pier dimensions, net pressure for large base piers, an increase of Poisson's ratio above 0.3, the oedometric modulus of the soil and the placement of a soft material between pier shaft and soil.

Conventional calculation methods underestimate wetting heave for a given value of final suction.

NOTATION

C = slope of unloading after soaking line in semi-log plot.
E = modulus of elasticity
 E_{oed} = oedometric modulus
 p_h = horizontal external pressure on a vertical plane
 p_v = vertical external pressure on a horizontal plane
 p_{vo} = external vertical overburden pressure
 p_{vf} = final p_v
 p_{vm} = geometric average of initial and final p_v values
 p_{vs} = soaking p_v
 q_u = unconfined compressive strength
 ϵ_0 = isotropic volumetric strain
 ϵ_v = oedometric volumetric strain
 ϵ_{vf} = final ϵ_v
 ϵ_{vs} = ϵ_v after soaking
 σ_{3min} = minimum value of minor principal stress. Negative values correspond to tension
 ν = Poisson's ratio
 τ_{max} = maximum value of maximum shear stress
 τ_s = maximum shear stress at pier shaft

1 INTRODUCTION

The use of elastic methods in expansive-collapsing soils has been summarized by Justo and Saetersdal (1981).

An important step in this trend was the method of Poulos and Davis (1973) for the study of piles in swelling-shrinking soils. In this paper, a true elastic behaviour of the soil is assumed, and it is recommended to use an elasticity modulus corresponding to a process of loading. As shown by Justo et al. (1984) and in table II, this might lead to an overestimate of swelling. Also the method is based in Mindlin equations and would not be applicable to pier foundations.

A plane strain finite element elastic method was applied by Livneh et al. (1973) to the study of flexible pavements on swelling clay. In this case, the

elasticity modulus employed corresponds to the "wetting under loading" curve and is, thus, more correct (Justo et al., 1984).

Amir and Sokolov (1976) have developed axisymmetric or plane strain finite element methods applicable to piles and piers in expansive media, but as Poulos and Davis, they use an elasticity modulus corresponding to a process of loading.

Justo (1982) and Justo et al. (1983) have presented preliminary reports on a three-dimensional finite element method to find the stresses and strains on pier foundations in expansive-collapsing soils. In this method, for the first time, the stress-path followed by the soil elements during the loading and wetting processes are clearly taken into account. The physical basis of the method and more complete results are established in this paper, which justifies its presentation at this conference.

2 CASE RECORD AND SOIL TESTED

A school building in Camas (Seville) suffered cracking in the floor slabs during construction, following a rainy period. The foundation beams lay on a bed of boulders cut in clay with the index properties indicated by Justo et al. (1984), and:

Unconfined compression:

1 to 3 m depth, $q_u = 1200$ kPa, $E = 67$ MPa

4 m depth, $q_u = 360$ kPa, $E = 20$ MPa

The depth of the water table was from 4.5 to 5 m. The reason for cracking was the heaving pressure of the clay on the underside of the beams, transmitted through the boulders, which favoured the wetting of the clay.

It was recommended to separate the beams from the soil, but it was necessary to calculate the heaving of the free soil profile, so as to know the minimum separation and the future heaving of the pier foundations, 3 m deep, once the beams were liberated.

disturbed block samples were taken at 1, 2, 3 and 4 m depth.

oedometer tests carried out in the sample at 4 m depth are included in a companion paper (Justo et al. 1984, figure 3).

CALCULATED HEAVING USING CONVENTIONAL METHODS

The conventional methods of calculation have been used: the double oedometer test, the single oedometer test with the simplification of Ralph & Nagar and the direct method (Jennings et al., 1973). The results are summarized in table I. The smallest pier (1x1 m) was calculated.

With the conventional methods it is not possible to account for skin friction. The total pressure of 300 kPa was reduced to 150 kPa to account somehow for this.

TABLE I

HEAVING CALCULATED BY CONVENTIONAL METHODS

	Heaving (cm)		
	Double oedometer	Single oedometer	Direct method
Soil profile	14.2	12.5	6.1
Pier	1.5	-0.7	0.3

STRESS-PATH AND CALCULATION SEQUENCE (FIG. 1)

We are only interested in the strains produced by the net load on the pier and the wetting of the soil. The zero state for strains corresponds to the overburden pressure p_{vo} .

metric conditions:

1. The soil is subjected to the net load on the pier under "natural moisture" conditions. This gives the strains and settlements produced by gravity loads.

2. The soil is assumed to wet. The pier friction induces stresses in the soil around to counteract the tendency to volume change. In the present case, the suction decrease is coupled with an increase in total stress above the base of the pier, and generally with a decrease in total stress below the base of the pier (fig. 4). According to the preliminary results reached by Justo et al. (1984), when suction is decreased as the sample is loaded or unloaded, the final swelling reached is little dependent upon stress-path, and depends mainly on the final suction and total stress, reaching finally the "wetting under loading" line (fig. 1) for the calculated final suction.

4.1 Calculation stress-path

As the stress change produced by the pier friction is unknown beforehand, the in situ stress-path may be changed by stress-path 012f.

1. The soil is subjected to the net load on the pier under "natural moisture" conditions (state 0 to state 1).

2. The overburden pressure is added to the stresses obtained above, to calculate p_{v1} .

3. The volume changes corresponding to the p_{v1} values at every point are calculated from the wetting under loading curves, assuming that there is no change in the vertical stresses (state 1 to state 2).

If the soil is supposed linear-elastic and isotropic (v. Justo et al., 1983), under oedometric conditions, from state 1 to 2 $\Delta p_v = 0$, and then:

$$\Delta \epsilon_0 = \Delta \epsilon_v \frac{1 - u}{1 + u} \quad (1)$$

$$\Delta p_h = -E \frac{\Delta \epsilon_v}{1 + u} \quad (2)$$

The wetting isotropic volume change is calculated from equation 1.

4. The foundation is calculated with the soil subjected to $\Delta \epsilon_0$, as initial isotropic volumetric strains, with no external loads, and with an "equivalent secant oedometric modulus" corresponding to the "wetting under loading" curve (state 2 to state f).

This calculation gives the wetting strains and movements.

5. To find the final stresses (state f), the stresses calculated in points 2 and 4 above are added.

4.2 Simplified stress-path

The simplified stress-path 03f may be followed:

1. The volume changes $\Delta \epsilon_v$ corresponding to the overburden pressure at every point are calculated (state 0 to state 3).

2. The foundation is calculated with the soil subjected to $\Delta \epsilon_0$ (eq. 1), to the net load of the foundation and with the equivalent oedometric modulus of the wetting under loading curve (from state

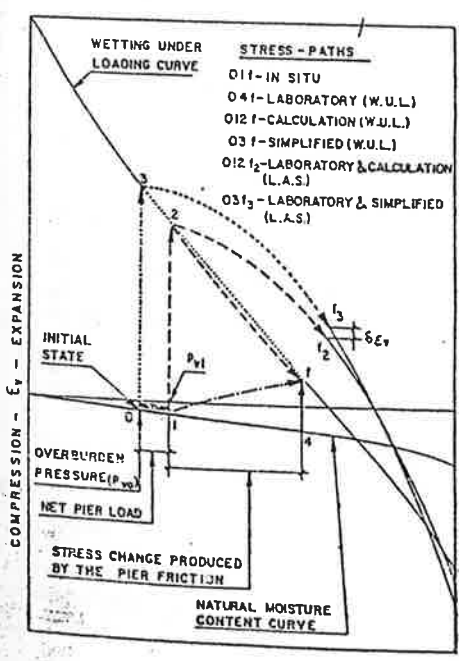


Figure 1 Stress-paths for a soil element above the base of the pier. W.U.L. = wetting under loading L.A.S. = loading after soaking shall follow the usual vertical stress-strain corresponding to a soil element, assuming oedometric conditions:

3 to state f). This calculation gives the total movements. Adding the overburden pressures we have the total stresses.

3. If we are only interested in the wetting movements, the strains corresponding to the net load under natural moisture conditions must be subtracted.

4.3 Comparison of the two calculation stress-path

Both methods lead to similar results, because, as shown with the calculations made by the finite element method, the p_v values produced by the net load are small compared with the overburden pressure.

One theoretical disadvantage of the first method is the assumption that the initial volume change depends only upon p_v , something that is only strictly true for oedometric conditions. Owing to the reasons stated above, this error is quite small.

The simplified method is simpler, as in a stratified soil the initial volumetric increment only changes in the vertical direction, and is, for all these reasons, generally preferable.

4.4 Stress-path using the "loading after wetting" curves

Although, as stated above, it seems that the "wetting under loading" curves should be used, this has not been, as yet, completely proved.

In situ, wetting and stressing are simultaneous. This stress-path must be somewhere between the stress-path of "wetting under loading" and of "loading after wetting".

In this case the two methods indicated above (012f₂ & 03f₃) would lead to a difference in volume change ($\delta\epsilon_v$), and the laboratory tests would be much longer than when using only the "soaking under loading" curve.

If a soil element suffers unloading, E_{oed} will be:

$$E_{oed} = \frac{p_{vs} - p_{vf}}{\epsilon_{vs} - \epsilon_{vf}} \frac{p_{vs} - p_{vf}}{C \log_{10} p_{vs}/p_{vf}} \quad (3)$$

5 POISSON'S RATIO

As it will be shown later, Poisson's ratio has a large influence in the results reached with the finite element method.

Poisson's ratio may be calculated from oedometer tests in which the horizontal stress is measured:

$$\nu = \frac{\Delta p_h / \Delta p_v}{1 + \Delta p_h / \Delta p_v} \quad (4)$$

Using the laboratory tests from Komornik and Zeitlen (1965), corresponding to "soaking under loading" curves, ν values from 0.495 for low p_v values to 0.1 for high p_v values (collapse zone) have been calculated.

From other tests results, ν ranges from 0.28 for final suction zero, up to 0.13 for final suction 1000 kPa, and seems little dependent upon the p_v value.

According to Livneh et al. (1973) ν may range, in expansive soils, from 0.25 to 0.45.

Justo et al. (1983) explain the finite element method employed.

The simplified stress-path has been used in the calculations.

Although the initial swellings should be obtained from the "wetting under loading" curve as indicated in paragraph 4, two other stress-path, profusely employed, have been tried, corresponding to the double oedometer test and to the single oedometer with the simplifications of Ralph and Nagar. In the case of Camas soaking conditions have been assumed. Figure 2 shows the initial swellings.

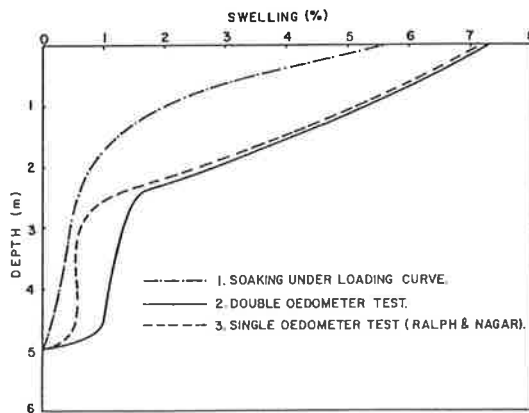


Figure 2 Free soil profile swelling (Camas)

Figure 3 shows the corresponding initial oedometric moduli. Comparing with the E values from unconfined compression (paragraph 2), the values of figure 3 for "natural moisture" are too small. If we are interested in the true settlements under "natural moisture" conditions, they will give too large settlements, but if they are used to add to the total heave, to find the wetting heave, the oedometric values should be used.

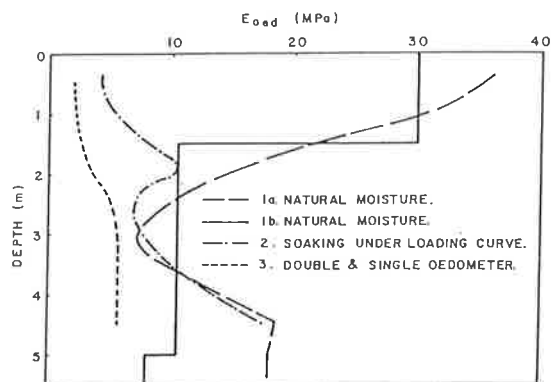


Figure 3 Oedometric moduli (Camas)

As the stresses produced by the net weight are negligible compared with the overburden pressure, and the variation of the "natural moisture" modulus in this range is small (Justo et al., 1984) the tangent modulus corresponding to this pressure may be taken.

In the "soaking under loading", and double and single oedometer, the oedometric moduli for the samples at 1 and 2 m correspond to an "average" in space and pressure range. For the samples at 3 and 4 m an average between the tangent modulus corresponding to

the overburden pressure and the modulus corresponding to the stress increment given by the methods of paragraph 3 was taken.

The modulus of elasticity was calculated from the formula:

$$E = E_{oed} \frac{1 - \nu - 2 \nu^2}{1 - \nu} \quad (5)$$

6.1 Parametric study

Based upon 62 cases calculated, the following conclusions have been reached:

The parameters that have a strong influence in the problem are the free soil profile swelling, the pier dimensions, the net load on the pier for the large pier (4.5 x 2.2 m), the increase of Poisson's ratio above 0.3, the variation of the oedometric modulus of soil and the presence of a soft material between the pier wall and the soil.

Of no practical influence are the variation of Poisson's ratio between 0 and 0.3, the variation of the elasticity modulus of concrete within allowable limits, the allowance or not of tensions between the pier base and the soil and the net load in small piers (1 x 1 m).

The allowance of horizontal movements has a moderate influence (reduces concrete tensions) for $\nu = 0.3$. On the other hand, for $\nu = 0.495$, this allowance may change the sense of movements.

This fact is of great importance, as it makes allowable the calculation method indicated in paragraph 4.1 for $\nu = 0.3$. This is because when we do not allow horizontal movements in the finite element method, we are adopting oedometric conditions, and the volume change depends only upon p_v .

A summary of the results is shown in table II. The maximum tension in soil and in concrete, τ_{max} & τ_s

TABLE II
PIER HEAVING, MAXIMA TENSIONS AND SHEAR STRESSES

Case	Pier dimensions m	Swelling Fig. 2	ν	Net load kPa	E_{oed} Fig. 3	Discretiz.	Horizontal strain	Total pier heave cm	Wetting heave cm	σ_{3min} (kPa)		τ_{max} kPa	τ_s kPa
										Soil	Concrete		
1	1x1x3	zero	0	120	1a	1a	no	-0.09		-4.0	23.0	22.0	11.0
2	"	"	"	"	1a	2a	"	-0.19		-3.0	23.0	21.0	10.0
3	"	"	"	"	1b	1a	"	-0.09		-2.7	24.0	19.7	9.9
4	"	"	0-0.3	"	1b	2'a	"	-0.29		-2.8	24.0	19.8	10.0
5	"	"	0.3	"	1a	1a	"	-0.13		-4.0	23.0	16.5	12.0
6	"	"	0.45	"	1a	1a	"	-0.27		-2.8	24.0	20.8	10.0
7	"	1	0.3	"	2	1a	yes	1.14	1.23	8.1	-206.0	176.9	92.0
8	"	"	0.3-0.2	"	2	2a	no	1.01	1.20	10.6	-216.0	188.1	93.0
9	"	"	0.3	"	s.u.l.1	1a	no	0.80	0.89	5.4	-69.0	90.8	45.0
10	"	"	"	"	s.u.l.1	2'a	yes	0.60	0.89	5.4	-69.0	90.8	45.0
11	"	"	"	"	s.u.l.2	2'a	no	0.51	0.80	7.2	-35.0	59.7	30.0
12	"	"	"	"	s.u.l.2	2'a	yes	0.57	0.86	7.3	-13.0	53.3	27.0
13	"	"	"	"	l.a.s.	1a	no	1.64	1.73	-70.0	-756.0	495.0	250.0
14	"	"	"	"	l.a.s.	2'a	no	1.40	1.69	-50.0	-756.0	498.0	250.0
15	"	"	0.45	"	2	1a	"	0.71	0.80	7.0	-41.0	83.0	43.0
16	"	"	0.45-0.2	"	2	2a	"	0.62	0.81	5.0	-44.0	82.4	44.0
17	"	"	0.45	"	s.u.l.1	1a	"	0.36	0.45	7.0	13.2	44.0	22.0
18	"	"	0.45-0.3	"	s.u.l.1	2'a	"	0.13	0.42	7.0	13.2	45.0	22.0
19	"	"	0.45	"	l.a.s.	1a	"	1.29	1.38	-9.0	-286.0	215.0	110.0
20	"	"	0.45-0.3	"	l.a.s.	2'a	"	1.03	1.32	5.0	-286.0	217.0	110.0
21	"	2	0.3-0.2	"	3	2a	"	3.85	4.04	0.4	-256.0	203.4	110.0
22	"	"	0.45	"	"	1a	"	2.98	3.07	7.3	-76.0	105.0	53.0
23	"	"	0.45-0.2	"	"	2a	"	3.78	3.97	6.3	-86.0	107.0	55.0
24	"	3	0.3	"	"	1b	"	2.00	2.09	1.1	-291.5	181.9	96.0
25	"	"	"	"	"	1c	yes	2.11	2.20	5.9	-200.0	215.9	110.0
26	2.2x4.5x3	zero	0-0.3	200	16	2'a	no	-1.14		-41.0	21.0	96.0	48.0
27	"	1	0.3-0.2	120	2	2a	no	-0.06		-1.6	-29.0	198.6	100.0
28	"	3	0.3	0	3	1c	"	1.93		-17.0	-110.0	242.0	120.0
29	"	"	"	200	"	1c	"	-0.59		-28.0	-4.8	263.0	130.0
30	"	"	0.495	0	"	1c	"	1.25		4.5	-10.0	26.0	12.0
31	"	"	"	200	"	1c	"	-4.52		4.4	38.0	31.0	15.0

ν : when included, the second value corresponds to the non-active soil

E_{oed} : s.u.l. = from "soaking under loading" curve (equation 6) 1 = 1st iteration
l.a.s. = from "loading after soaking" curve (equation 7) 2 = 2nd iteration

Discretization: Number 1 corresponds to cases in which the non-active soil is considered rigid

Number 2 corresponds to cases in which an elastic non-active layer, 15 m deep, has been considered

In discretization 2'a (fig. 4) 6 layers have been used. In the other cases only 5 layers

a) 1st layer 0.70 m deep (fig. 4), b) 1st layer 0.95 m deep, c) 1st layer 0.50 m deep

are allowable. So we see that there is no need to recur to plastic calculations.

6.2 Discretization

The details of the discretization used are indicated at the foot of table II.

The consideration of an elastic layer below the active layer has generally a negligible influence in the wetting pier movement and total stresses. To find the wetting strains, the same discretization, and probably ν , should be employed under "natural moisture" and "total heave" conditions.

6.3 Iterative procedure

As indicated by Justo et al. (1984), the oedometric moduli depend upon the state of stress.

For the soils from Camas, the "equivalent oedometric moduli" obtained from the "soaking under loading" curve are given by the regression:

$$\log E_{oed} \text{ (MPa)} = -1.43 + 1.2 \log p_{vm} \text{ (kPa)} \quad (6)$$

And for the "loading after soaking" curves:

$$\log E_{oed} \text{ (MPa)} = 1.3 - 0.4 \log p_{vf}/p_{vs} \quad (7)$$

- The following iterative procedure should be used:
1. Starting with the oedometric moduli of figure 3, the p_{vf} values are calculated.
 2. The p_{vm} values at each element, from p_{vo} to p_{vf} are calculated. The corresponding E_{oed} are calculated from equation 6, and shown in figure 4.
 3. With these moduli the new total heave and new p_{vf} values are calculated (fig. 4), and the cycle is reinitiated from point 2. Convergence is very rapid, as the variation of p_{vf} is relatively small.

A similar procedure may be used with the "loading after soaking" curves.

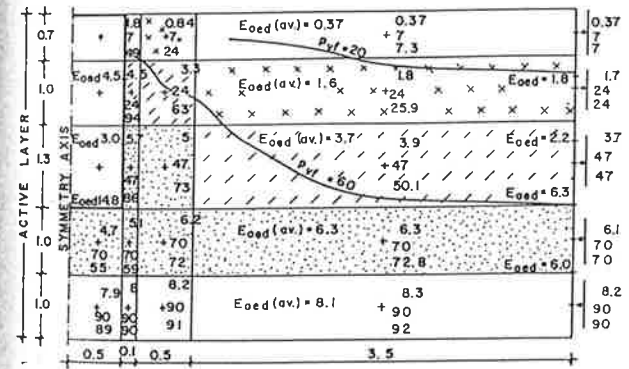


Figure 4 Discretization 2'a (the non-active layer is not included). The three values included at each point are E_{oed} (MPa) corresponding to the average p_{vf} for cases 7 & 14 (table II), p_{vo} and p_{vf} (kPa) for case 10.

7 COMPARISON OF FINITE ELEMENT AND CONVENTIONAL METHODS

In table II the best estimate of the wetting heave for the double oedometer test may be 4 cm, for the single oedometer (Ralph & Nagar) 2.2 cm, and for the direct method between 0.86 and 0.42 cm according to

Poisson's ratio. Comparing with the values of table I we see that the conventional methods underestimate the wetting heave.

In both cases we have assumed soaking conditions. Really final suction would not be zero and the heave less than calculated here.

8 CONCLUSIONS AND RECOMMENDATIONS

Conventional methods of pier wetting heave calculation underestimate heave.

Stress-path has a large influence in the calculated heave and tensions in pier.

If we are only interested in the wetting heave, the soil below the active layer may be considered rigid.

Usually the influence of considering or not horizontal strains in the finite element method is quite small, and, so, the oedometric moduli, may be taken from oedometer tests, as a function of p_v only.

Maximum tension in soil is usually allowable. But, in any case, the influence of assuming in calculation that once the maximum tension is reached the tension strength decreases to zero is negligible.

It is recommended to measure horizontal stresses during the test to find the important ν value.

Plastic flow is not reached at any point in the soil.

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