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J.L. JUSTO
Professor of Soil Mechanics & Foundations
J. SAURA
Assistant Professor of Structures
A. JARAMILLO
Assistant Professor of Soil Mechanics & Foundations
A. DELGADO
Postgraduate student
P. GENTIL
Architect

University of Seville, Spain

Prediction and Performance for Seasonal Swelling-shrinkage of Buildings on Expansive Clay

Prédictions et Comportement pour le Gonflement-retrait Staticnal des Batiments sur Arg... Expansive

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SYNOPSIS The seasonal movements of buildings and swelling-shrinking soil profile have been measured from 1976. A finite element non-linear method has been proposed to predict the stresses and movements of the foundation.

INTRODUCTION

In a village near Seville (El Arahal), ten 4--storey reinforced concrete frame blocks, built in 1965, suffered damage produced by clay shrinkage during the drought period 1973-75 (Justo, 1980). Some pillars became fisured by tension, and one block had to be evacuated for repair.
Twenty seven levelling plugs have been placed at the blocks. Eight surface levelling plugs have been placed outside the blocks on the soil.
Ten surface levellings plugs have been placed in the crawl space below the buildings. Nine subsurface heave indicators have been placed at depths of 3 and 5 m. Levelling dati have been placed at a depth of 7.5 m.
Precision levelling is being taken from September 1976.
This paper collects the results of the levellings, and the movements calculated by the finite element method described by Justo et al. (1984 b).

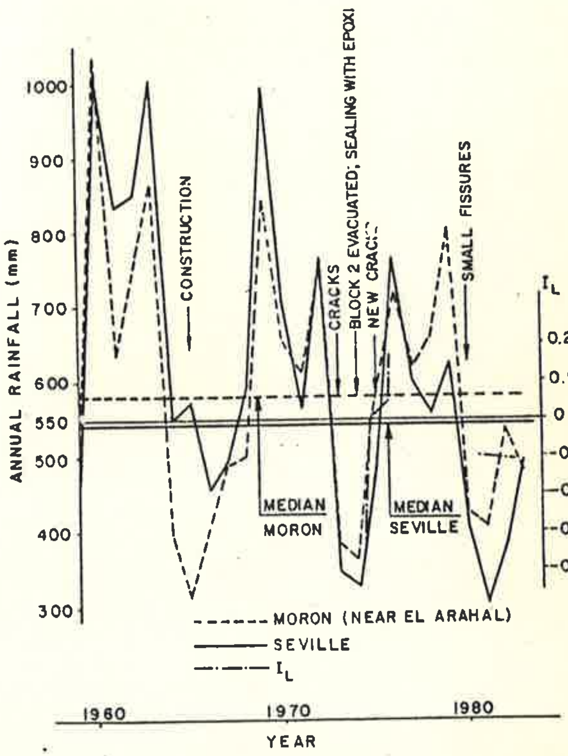

Fig.1 Rainfall from 1959 onwards and development of events

CLIMATE

Seville has a Mediterranean climate. The Thornthwaite moisture index in the villages around El Arahal has an average value of -18. The average annual rainfall in Seville from 1876 till 1983 is 546.5 mm (Justo, 1983). Near El Arahal this value may be 581.4 mm (from 1956 till 1983).
Figure 1 shows the annual development of events. Construction was carried out after the most wet known 4-years rainy period (1960-1963), but during the very dry year 1965. We have data from 1871 onwards. On the other hand damage appearead during the dry biannual period 1973--74.

SOILS

The foundation soil is a clay belonging to the Miocene (Tortonien).
Compacted and undisturbed samples, of the clay, taken at different times are being thorougly studied at the laboratory (Justo, 1980; Justo et al,, 1984 a). The range of the Atterberg limits of the samples tested is:

w_L=39-75 w_P=17-33 I_P=18-45 w_s=18-30

Samples were taken at six dates, and a summary of the results is shown in Table I.
There is a correlation between annual rainfall and liquidity index (fig.1), and a time lag between the monthly rainfall and liquidity index.
The regression between laboratory suction and liquidity index is:

$$u_a-u_w \text{ (kPa)} = 600-3500\, I_L \qquad (1)$$

TABLE I

Tests on samples taken at five dates

Sampling	Date	q_u kPa	Sample type	I_L	Depth m	Block	Suction kPa	q_t kPa	N blows/30 cm
1st	Nov. 74	350-390 (370)	tube	-0.52--0.07 (-0.32)	3-6	2		(50)*	
2nd	Jan. 75	360-390 (380)	"	-0.02-0.05 (0.00)	3-12	2		(50)*	
3rd	Feb. 76	140-2000 (1100)	block	-0.45-0.45 (-0.03)	1-3	2-3-5		20-270* (150)*	
3rd	Feb. 76	1200-2000 (1600)	"	-0.06-0.22 (0.05)	1-3	2		160*-270* (220)*	
4th	Ap. 76	10-320 (180)	tube	-0.20-0.37 (0.12)	1-7.5	2-3-5		1*-40* (25)*	21-44 (29)
4th	Ap. 76	210-320 (260)	"	0.06-0.35 (0.17)	4-7	2		28*-45* (35)*	29-44 (37)
5th	Dec. 80		block	-0.10		2			
6th	Dec. 82	73	"	0.15	0	2	15	10	
			"	-0.18--0.01 (-0.11)	2-3	2	1100-1300 (1200)	40*-80* (60)*	
			"	-0.04	1.5	2in	30	20*	
7th	Ap. 84	57-161 (97)	"		1-1.5	2			

* calculated ( ) average value q_t = tension strength

For this clay, the ratio between undisturbed compressive and tensional strength may be around 7.4. The ratio between compacted and undisturbed tensile strength, for the same dry density and water content may be 1.5. From these ciphers and the measured compressive strength in undisturbed samples or tensile strength in the Brazilian test in compacted samples, the calculated tensile strength for undisturbed soil indicated in table I has been found.
Swelling tests have been carried out in undisturbed soils corresponding to the 3rd sampling, made in February 1976 (single oedometer test with the simplification of Ralph and Nagar; v. Jennings et al., 1973; Justo, 1980), 5th and 6th (fig.2 & 3) samples tested had a lower density and liquidity index. Swelling under low pressures was higher in compacted samples, but the swelling pressure was higher in undisturbed samples.
The equation of the soaking under loading curve (v. Justo et al., 1984 a) is:

$$\varepsilon=-19.11+23.58 \log p-11.51 \log^2 p+2.09 \log^3 p \quad (2)$$

where:

p = external stress (kPa)
ε = soil strain (%). Positive for compression and negative for swelling.

The modulus of deformation obtained from plate loading tests is 44 MPa (for 1.5 and 3.0 m depth) and decreases very slightly with increase in loading.

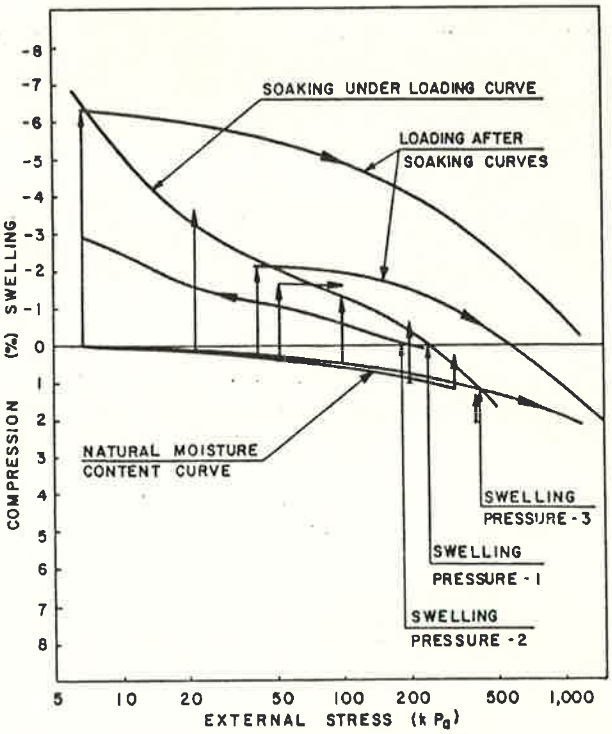

Fig.2 Soaking tests on undisturbed samples from Arahal

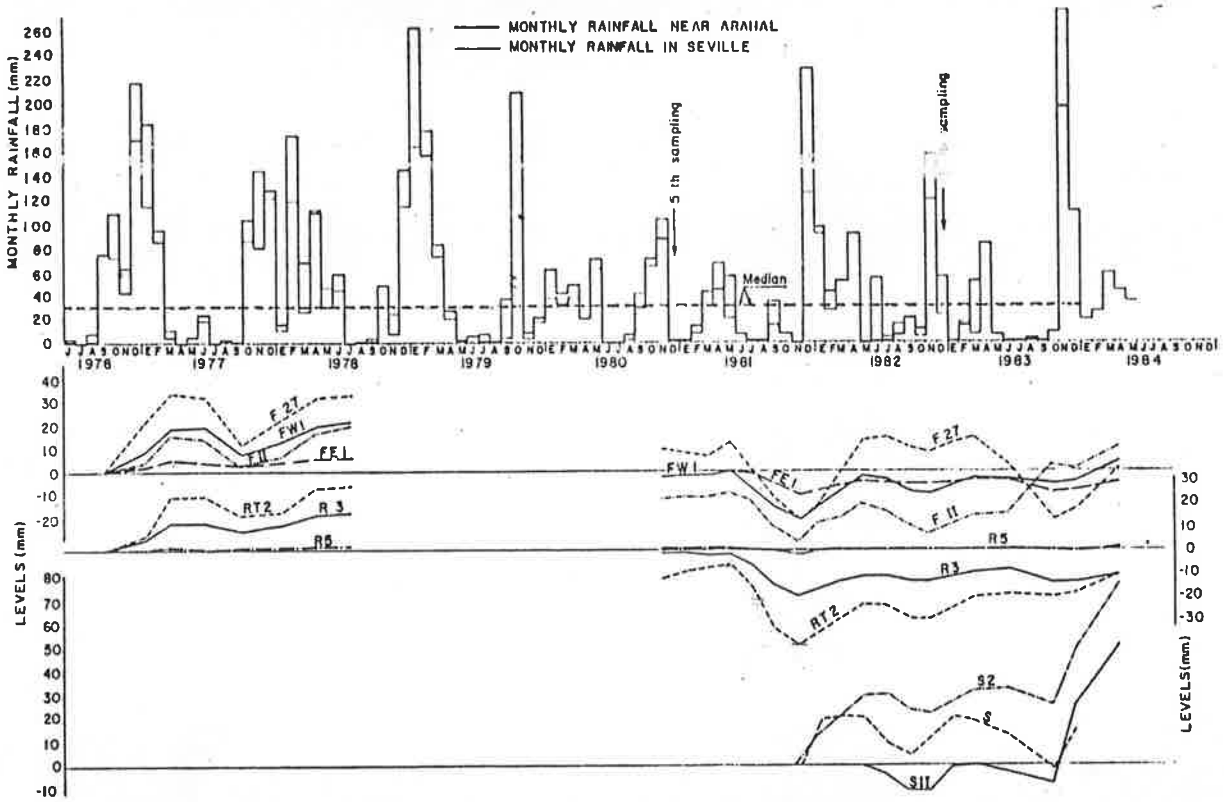

Fig.3 Monthly rainfall and variation of levels with time

FOUNDATION

The pier foundation reached a depth from 1.7 to 1.9 m. There was a crawl space separating the structural floor from the soil.

LEVELLINGS

Levelling plugs were constructed as the levelling stations designed by Cheney (1974). Subsurface heave indicators and levelling dati were both as the levelling dati designed by the same author. Placement of the signals was made as indicated by Justo (1980).
Figure 3 shows the following levellings as a function of time:

1. The average of the levels of the levelling plugs placed in façades oriented to the west corresponding to the 7 blocks with more movement (FW1). Id. for the plug with the largest oscilation (F27).
2. As 1 for façades oriented to the east (FE1).
3. Id. for the west façade of block 2 (FII).
4. Id. for surface signals (S). Id. for the surface signal with the largest oscillation (S2).
5. Id. for the surface signals near block 2 oriented west (SII).
6. The average for all signals placed at 3 m depth in the soil (R3). The levels of the signal at 3m with larger oscillation (RT2).
7. Id. for signals placed at 5 m depth (R5).

The following conclusions have been obtained:

a) Façade west moves much more than façade east, although this difference is not so clear in the signals placed on or within the soil. The swelling of samples taken near the west façade is not larger than the same value for the east façade.

b) Relative maxima and minima nearly coincide in time for all signals, except, perhaps, the inner surface levelling plugs, affected by leaks from broken sewers. Surface outer plugs are more influenced by inmediate raining.
Maxima coincide with the end of the rainy season, and minima with the end of the dry season. The annual oscillation is of the same order from year to year, but the drought period 80-83 produces a decrease in the average levels.

c) Maxima oscillations are produced at surface (77 mm in 2 1/2 years outside the buildings, and 61 mm in 1 1/2 years inside), followed by 3 m subsurface heave indicators (69 mm in 7 years; 29 mm in 3 years) and façade plugs (55 mm in 7 years; 36 mm in 3 years), and finally 5 m subsurface heave indicators (6mm in 7 years). Movement at 7 m is negligible.

d) Figure 4 shows the average of the movements of the soil profile with depth, starting at the end of November of the very dry year 1981.

As a summary, the response of the soil surface to rain and drought is very quick and the response of the soil at 3 m depth is more slow, with a time lag as indicated under b) above.
The movements at 3 m are very important (v. also

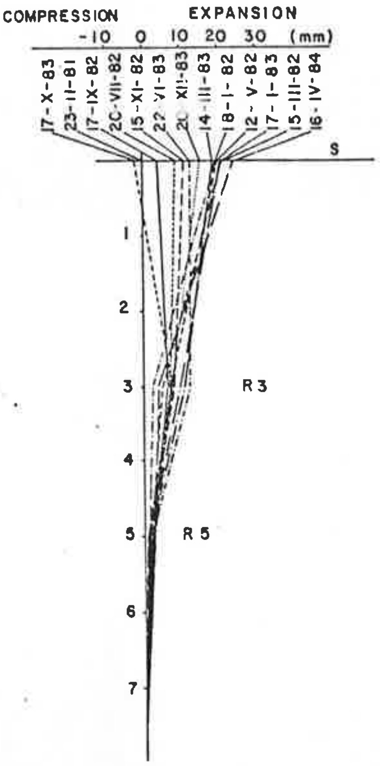

Fig.4 Movement of the soil profile from September 1976 till April 1984

figure 3), even at 5 m are significant, and at 7.5 m are negligible. So, the so called "active layer" ranges between 5 and 7.5 m.
It is interesting to state that the maximum swelling measured at a depth of 4 m is 1.3%, and the corresponding swelling calculated from figure 2 is 2%.

APPLICATION OF THE FINITE ELEMENT METHOD TO THE STUDY OF MOVEMENTS OF BUILDINGS

The fundamentals of the method have been stated by Justo et al. (1983) and Justo et al. (1984 b). The simplified stress-path indicated in this last paper has been followed.
In this case the input is the volume change corresponding to the overburden pressure, that is not calculated, as indicated in that paper, but measured.
Up to now the finite element method has been applied to block 2, the best instrumented one, and the block from whose foundation more samples have been taken.
The movements from September 1976 up to July 1978, corresponding to the maximum heave of this block have been studied.
In this period no surface levelling plugs had, as yet, been placed. The movements of the subsurface heave indicators at 3 and 5 m were interpreted as if there was a linear variation of strain between the bottom of the active layer and the surface. This gave a depth of the active layer of 5.8 m, and unit strains at surface of 4.1% at the SW corner and 3% at the NE corner. A linear variation of heave between these points was assumed.
The exact plan of the foundation had to be slightly changed so as to diminish the number of elements and reach symmetry.
From the samples taken at February 1976 only single oedometer tests with the simplification of Ralph & Nagar had been made.
The oedometric moduli (E_{oed}) were calculated from these curves using two hypothesis of loading:

1. The tangent modulus at the overburden pressure.
2. The secant modulus between the overburden pressure and the approximate foundation pressure.

The variation of E_{oed} with loading and depth was relatively small and so an average value of 12000 kPa was taken for the SW quarter, 8800 kPa for the NE quarter and 10400 kPa for the two other quarters.
A more detailed account of the results obtained will be given by Justo et al. (1985).
Figure 5 shows the heave of the soil surface and foundation.
Table II shows a summary of the results.
In July 1978 the average heave was 2 cm for the west façade and 0.9 cm for the east façade. Comparing with the values of table II it is clear that the order of magnitude of the movements has been well predicted, but the calculated movements are larger than the measured ones. One of the reasons may be the assumption made about a linear variation of unit heave between the bottom of the active layer and the soil surface.
When heaves of the surface points have been available, it has been realized that, actually, a hypothesis of constant unit heave in the active layer would have been more reallistic (v. fig.4).
In table II one case has been included with a unit heave of 4.3% at surface and 0 at the bottom of the active layer, uniform in plan, and with oedometric moduli obtained from the soaking under loading curve (fig.2). This case is indicated as "swelling 2" in table II. The initial oedometric moduli, taken as tangent under the overburden pressure varied between 300 kPa for the upper elements of the active layer and 7900 kPa for the bottom elements.
Pier A of figure 5 has also been calculated alone (indicated as single pier in table I).
The following comments may be made to table II:

1. A variation of Poisson's ratio between 0 and 0.3 has a negligible influence in the results.
2. The pier heave is nearly inversely proportional to pier depth.
3. A very rough discretization (2 and 3) gives an acceptable prediction of pier heave. But when wide elements are placed near the foundation shaft, the tensions in soil and concrete are seriously overestimated.
4. In some cases it may be convenient to study some piers of the foundation as singles piers, so as to have a finer discretization, mainly to predict tensions in soil and concrete.
5. Tensions in concrete must be checked, and the piers reinforced when necessary.
6. Comparing with the values of table I, tensions

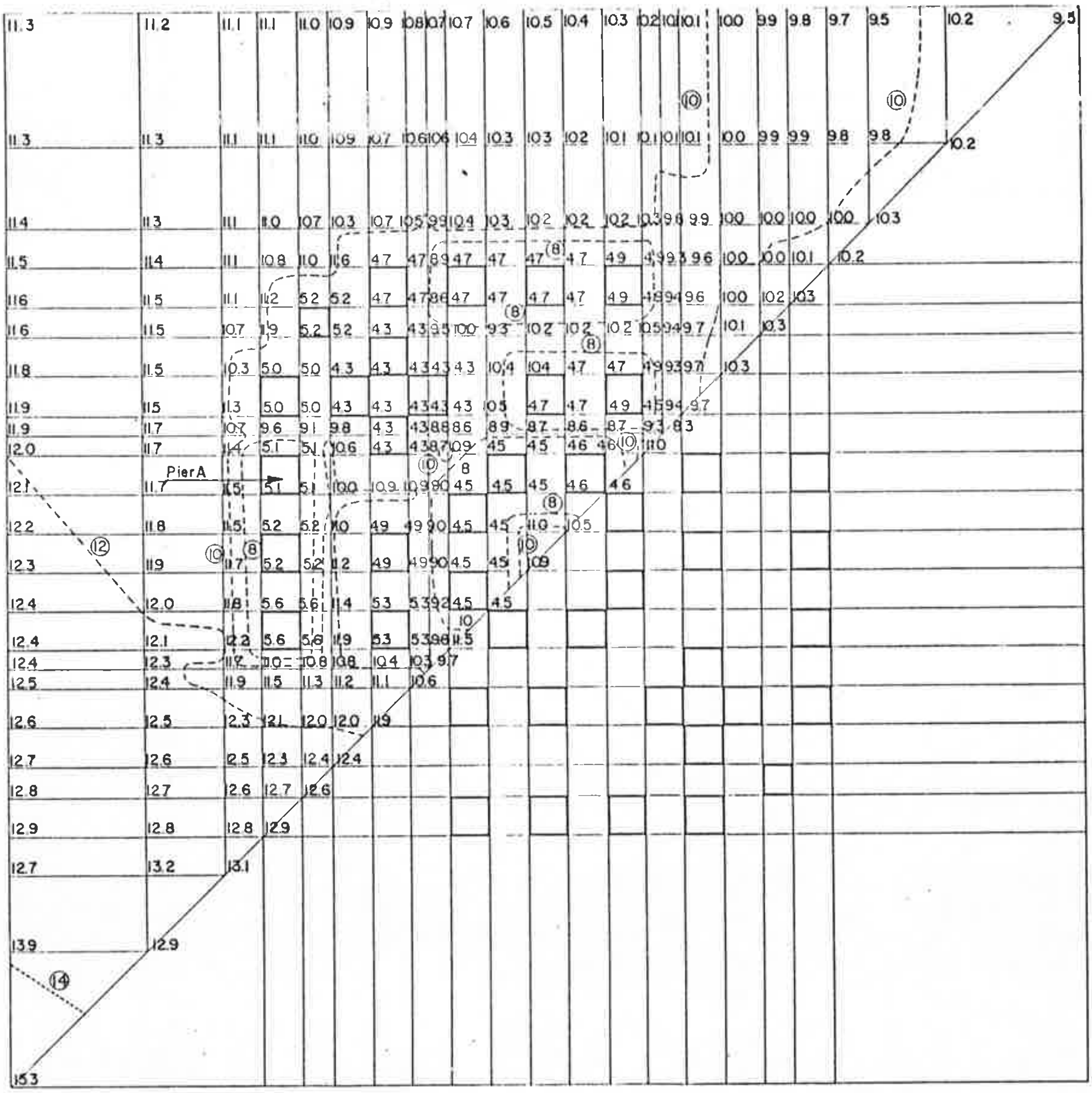

Fig.5 Vertical heaves of foundation and soil surface (cm), symmetric plan, discretization 5

and shear stresses in soil may be allowable. It must be taken into account that the stresses in table II are increments corresponding to a heave period, and not absolute stresses in the soil.

CONCLUSIONS

The finite element method proposed in the paper seems to be an adquate tool to predict movements and stresses of a foundation in swelling-shrinking soil, although more research is needed to fit it.

REFERENCES

Cheney, J.E.(1974). Techniques and equipment using the surveyors level for accurate measurement of building movement. Field Instrumentation in Geotechnical Engineering. London, Butterworths.

Jennings, J.E., Firth, R.A., Ralph, T.K., and Nagar, N. (1973). An improved method for predicting heave using oedometer test. Proc. 3rd Int. Conf. Expansive Soils, (2), 149-154, Haifa.

Justo, J.L. (1980). Estudio geotécnico sobre los daños ocasionados por los movimientos del terreno al grupo de viviendas La Paz, El Arahal, Sevilla. Unpublished report, Seville.

Justo, J.L. (1983). Diario ABC Sevilla, 18th May, pp 4-9.

Justo, J.L., Saura, J., and Delgado, A. (1983). Aplicación del método de E.F. al estudio de cimentaciones sobre arcillas expansivas. Anales de Ingeniería Mecánica, 1,1, 104-109, Madrid.

Justo, J.L., Delgado, A. and Ruiz, J. (1984 a). The influence of stress-path in the collapse-swelling of soils at the laboratory. Proc. 5th Int. Conf. Expansive Soils, 67--71, Adelaide.

Justo, J.L., Saura, J., Rodríguez, J.E., Delgado, A. and Jaramillo, A. (1984 b). A finite element method to design and calculate pier foundations in expansive-collapsing soils. Proc. 5th Int. Conf. Expansive Soils, 119-123, Adelaide.

Justo, J.L., Saura, J., Jaramillo, A., and Gentil, P. (1985). Interpretation of field measurements of seasonal swelling-shrinkage of 10 blocks on expansive clay by a finite element method. 5th Int. Conf. Numerical Methods in Geomechanics, Nagoya (Japan). Accepted for publication.

TABLE II

Pier heave, maxima tensions and shear stresses

Case	Pier depth m	Swelling	υ	Discretization	Max. pier heave cm	σ_{min} (kPa)		τ_{max} kPa	τ_s kPa
						soil	concrete		
Single pier	3	1	0	1	4.7	-180	-870	480	380
"	3	1	0	2	3.8	-75	-2900	220	
"	3	1	0.3	1	4.5	-140	-830	440	360
"	3	1	0.3	2	3.7	-110	-3400	180	
"	2	1	0	2	5.7	-150	-3500	210	
"	2	1	0.3	2	5.4	-100	-3400	170	
Whole foundation	2	1	0	3	5.0(4.3)	-25(38)	-1900(-1900)	180	
"	2	1	0.3	3	5.1(4.4)	-60(35)	-2300(-2300)	150	
Symmetric plan	2	1	0.3	4	5.5(4.9)	-150(36)	-2200(-2200)	160	
"	2	1	0.3	5	5.6(5.1)	-600(-16)	-1600(-770)	180	
"	2	2	0.3	5	3.9(3)	9.2(20)	-170(26.7)	23	

In brackets heave of pier A

τ_{max} = maximum shear stress in soil

τ_s = " " " at pier shaft