

Behavior of Building Foundations in Expansive Soil: A Case Study

Musharraf Zaman and Joakim G. Laguros
School of Civil Engineering
University of Oklahoma
Norman, OK 73019, U. S. A.

1. Importance

Expansive soil can cause serious problems to structural foundations.

Foundation movements or pier uplifts due to swelling of expansive soil can cause cracking of floor slabs and distress to doors and other serviceable components of a building, cracking of the exterior and interior walls, and heaving of floor slabs.

A large number of residential and commercial buildings in the United States have experienced some type of structural damage due to expansion or swelling of the foundation soil.

Improper design of foundations and poor construction methods often contribute to the distress caused by expansive soil.

Another contributory factor is associated with the inadequate soil investigation process.

Foundation and structural damages caused by expansive soil can be very costly and dangerous.

The case study site, called Rose Rock Villa, is a four story building in Norman which is owned by the Norman Housing Authority (NHA) in Norman, Oklahoma. It consists of four interconnected units as shown in Fig. 1 and is supported on drilled shaft foundations with lengths of approximately 15 feet. The soil survey map, as well as a detailed analysis of laboratory test data from soil samples from the site, indicate a foundation consisting of a large amount of expansive soil. The building has undergone a severe heaving of floor slabs in the interior part of the building that resulted in cracks in its interior walls and windows. The problem prompted a lawsuit.

2. Objective

Conduct a case study to investigate the causes of the distress in the slabs.

3. Steps Involved in the Case Study

The first step in this study is to analyze the load-displacement behavior of the drilled shafts due to swelling soil at the site by using two different computer programs developed by O'Neil (1988) and by Johnson (1984). Four critical groups of piers or shafts are selected from the four units based on the amount of dead and live loads carried by each pier.

In the second step, the movements of the shaft and the surrounding soil are determined by the analysis of the load-displacement behavior. These values are used to determine the magnitude of soil pressure acting on the selected slabs. The amount of soil pressure, of course, depends upon the relative movement of the piers and the soil.

A computer program, SAP 80, is used in the third step to analyze the slabs under consideration for the soil pressure resulting from swelling of the soil and pier movement by using the finite element method.

In the fourth and final step, the displacement values obtained from the computer program are compared with the corresponding actual displacements. Also, a deformed shape of the slab is generated by using a graphics package that provides a qualitative assessment of the extent of distress to be expected from the slab movements. The flexural moments obtained from SAP 80 are used to calculate the bending stresses and these stresses are plotted using the package program, SURFACE.

- Oklahoma Testing Laboratories
- ⊗ Shepherd Engineering
- Terracon Consultant
- ⊙ Additional

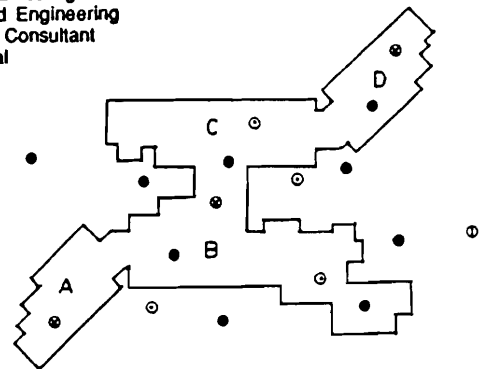


Figure 1 Layout Plan Including Borehole Location

4. Subsoil Conditions

The subsoil at the site consists essentially of 4 to 8 inches of dark brown to grayish brown porous granular soil. The lower subsoil consists of firm brown blocky clay up to 50 inches thick. The next layer consists of brown heavy noncalcareous clay, mottled with yellow and reddish brown which grades down to a depth of 72 to 96 inches into an alkaline reddish silty clay or silty shale.

Under the Unified Classification System (UCS), the soil at the site can be classified as CH, which is considered a highly plastic clay. The liquid limit was found to range from 26 to 54 percent and the plasticity index was in the range of 23 to 37 percent (Table 1). The hydrometer analysis indicates that the samples were composed of 41 percent clay, 52 percent silt and 7 percent sand. The one-dimensional swell test reveals that the percent swell increases with depth. Between the depth of 6 to 7 feet, the percent swell was found to vary from 2.02

to 2.40 percent with a maximum swelling pressure of 1,940 psf. The ground water table in the area has been found to be fluctuating between 5.9 and 14 feet.

5. Numerical Results

A detailed foundation plan was drawn for each unit of the building (e.g., see Fig. 2) and a total of four critical piers were selected for analysis of the load-displacement response. Two of the four piers are selected based on the maximum design loads, while the other two are based on the minimum design loads. The length of each pier was taken as 15 ft. Some relevant properties are listed in Table 2 and further details are given by Abdul Ghani (1989).

6. Comparison of Johnson and O'Neil Approaches

A summary of the pier and soil movements for the

critical piers is shown in Table 3. The results for all the four cases (not shown here) demonstrate that the shaft movements predicted by the Johnson and O'Neil models are substantially different: the O'Neil model predicts substantially larger movements than the Johnson model. The difference is maximum at the top and bottom of the pier. These differences can be attributed to various factors. For example, a factor f is used to convert the volumetric swell to vertical swell whose value ranges from 0.33 to 1.00. The amount of swell can, thus, significantly vary depending upon the value of f chosen. Experience of the user of the code plays an important role in selecting the appropriate value.

Second, the coefficient of swell that relates to changes in suction and used by O'Neil (1988) is only an approximation.

The input data for the Johnson model are straight-

Table 1 Summary of Soil Test Result

Test Performed By	Depth (ft)	LL	PL	PI	Dry Density (psf)	Moisture Content (%)	Unconfined Compressive Strength (psf)	Swell Pressure (psf)	Vertical Swell (%)
Oklahoma Testing Laboratories (1979)	5	47	-	29	-	22.0	11687		-
	10	39	-	24	-	20.7	14800		-
	20	42	-	27	-	20.1	10584		-
Shephard Engineering Testing (1986)	1.0 - 2.5	28	17	11	101	22.9	-	245	1.1
	2.5 - 3.0	26	17	9	99	20.4	-	350	0.3
	3.9 - 4.8	47	18	29	104	21.2	-	430	0.2
	5.5 - 6.0	47	17	30	104	19.5	-	640	1.0
	6.5 - 7.0	47	17	30	106	17.6	-	745	2.02
	7.0 - 7.8	46	16	39	-	-	-	-	-
Terracon Consultant Inc. (1987)	0.7 - 1.9	51	14	37	106	21.1	5500	1050	1.6
	4.1 - 5.0	47	17	30	110	19.4	9000	1485	1.5
	6.0 - 7.6	37	14	23	107	20.7	7000	1940	2.4

Table 2 Detail of Critical Piers for the Case Studies

CASE	PIER NO.	PIER LOCATION	SHAFT DIAM. (in)	BELL DIAM. (in)	AREA STEEL (in ²)	MIN. WORKING (kip)	MAX. WORKING (kip)
1	F2	C-4: UNIT A	24	60	4.52	-	129.42
2	F8	J1a-7a: UNIT C	20	-	2.141	13.04	-
3	F1	E-7: UNIT B	30	48	7.06	13.621	-
4			18	-	2.54	-	68.52

forward and easier than the O'Neil model.

7. Analysis of Slab Displacement

The vertical pressure exerted on the floor slab by the heaving of soil can cause an upward movement of the slab. The upward movement of the floor slab can be predicted by using the finite element method. The process involved in analyzing these critical slabs is discussed in detail by Abdul Ghani (1989).

The slab (Unit A) displacements for case 1a (Johnson's approach) are shown in Fig. 3. The maximum displacement of 2.7 inches is located at the center of the slab. The heaving of the slab is maximum in the central part of the slab. On the slab boundary, heaving increases as it moves away from the drilled piers. Similar results for case 1b are shown in Fig. 4. The maximum slab displacement for this case is found to be 7.7 inches at the center of the slab.

8. Distribution of Bending Stresses

The bending stresses in the slab are calculated from the slab displacements. It is observed (results are not presented here due to space limitation) that high concentration of stress is located at the corners of the slab. These corners represent the locations of the drilled piers. The distribution of the stresses for some cases are approximately symmetrical about the x and y axes.

9. Concluding Remarks

The critical piers analyzed here showed relatively small movement and are considered stable. These results agree with the site observation where no distress is detected at the pier locations.

The analysis of the four critical slabs showed that at locations where the slabs are connected to the grade beams, very small displacements are detected. At locations where no grade beams are attached, displacements of the order of 1 to 3 inches are detected. Slab displacements get larger in sections of the slabs away from the location of the pier foundation.

High concentrations of stresses are observed at the locations of pier foundations compared to other areas of the slab.

The finite element approach presented here can be modified by using an appropriate iterative technique to account for reduction in soil pressure due to slab heaving.

Table 3 Net Movement and Pressure for Critical Piers

Case	Pier No.	Shaft Movement (in)	Soil Movement (in)	Net Movement (in)	Pressure (psi)
1a	F2*	+0.012	-0.76	0.77	6.467
1b	F2**	+0.046	-2.14	-2.18	18.333
2	F8	-0.091	-0.76	0.66	5.603
3	F1	+0.002	-0.76	-0.76	6.366
4	F9	+0.011	-0.76	+0.77	6.458

*By Johnson's approach (1984)

**By O'Neil's approach (1988)

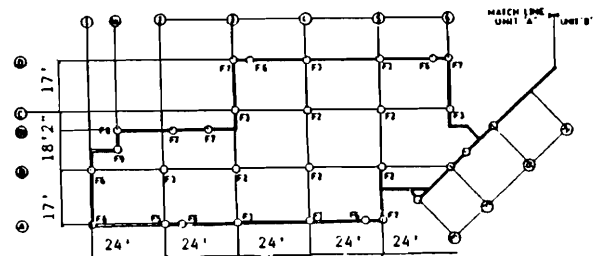


Figure 2 Foundation Plan of Unit A

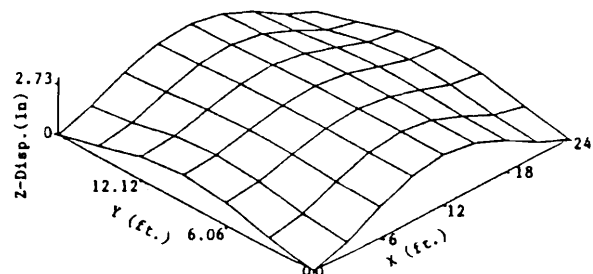


Figure 3 3-D Plot of Slab Displacements for Case 1a

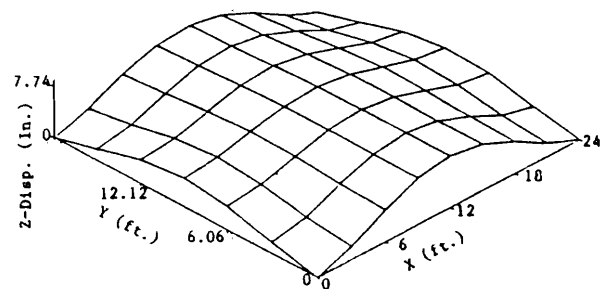


Figure 4 3-D Plot of Slab Displacements for Case 1b

10. Acknowledgement

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11. Selected References

Abdul Ghani, J. (1989), "Analysis of Drilled Piers and Its Application to a Case Study of a Damaged Building on Expansive Soil", M. S. Thesis, Univ. of Oklahoma, Norman.

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