JOURNAL OF MATERIALS & CONSTRUCTION

Journal of Materials and Construction
ISSN 2734-9438
Website: www. jomc.vn

Deformation characteristics of foundations on expansive soils

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KEYWORDS

Expansive soils
Deformation characteristics
Shallow foundation
Pile foundation

ABSTRACT

This paper focuses on evaluating the influence of expansive soils on shallow and pile foundations. The calculated results are analyzed to predict the magnitude of structural deformation when the subgrade beneath foundations becomes wetted, and to show that piles may be heaved during the construction stages.

1. Selected research findings on the analysis of foundation on expansive soils

Studies on expansive soils largely concentrate on determining swelling characteristics and their correlations with physical and mechanical indices of soils [1–3]. In addition, for structures constructed on expansive soils, increasing attention has been paid to foundation solutions that mitigate the adverse effects of swelling on structural performance and stability [4–8].

Field swelling strain sometimes differs from laboratory measurements because in-situ soils may not be fully saturated. The wetting factor α, reflecting the in-situ tendency to swell, depends on the degree of saturation before and after wetting [6]. Following Chen (1988), the upward displacement due to soil swelling can be estimated by the simple relation:

$$\delta_{sw} = \sum \alpha h \varepsilon_{sw} \ (1)$$

Where:

- h: layer thickness
- ε_{sw}: the swelling strain.

Computation of swelling in this context is analogous to layer-by-layer settlement analysis. The uplift of a foundation S_{sw} caused by wetting-induced swelling can be expressed as [4]:

$$S_{sw} = \sum_{i=1}^{n} \varepsilon_{swi} \times h_i \times m \ \ (2)$$

Where:

ε_{sw}: swelling strain;

h_i: thickness of layer i;

m: condition factor.

For pile foundations, analysis primarily addresses the swelling-induced uplift and the resisting capacity against uplift. Chen (1988) proposed:

$$Q_{up} = \pi d \alpha_u p_{sw} L_1 \ (3)$$

Where:

- d: pile diameter;
- α_u: the interface coefficient between soil and concrete (α_u = 0,15);
- p_{sw}: the swelling pressure.

The segment length L_1 located in the unstable (wetted) zone is contingent upon local environmental factors. Empirical observations indicate that the wetted zone is commonly bounded within approximately 150 to 450cm measured from the pile head, subject to site conditions.

An alternative approach is to relate the uplift capacity to the undrained adhesion in the same manner as shaft friction under compressive loading. Accordingly, the uplift force Q_{up} can be rewritten as:

$$Q_{up} = \pi d \alpha_s c_u L_1 \ (4)$$

Where:

α_s: adhesion factor between soil and concrete under swelling conditions;

c_u: undrained cohesion.

The segment of the pile located in the stable zone must be long enough to preclude extraction. If L_2 denotes the length of pile within the stable region, the corresponding uplift resistance Q_R equals the shaft friction developed along the pile surface in that region.

$$Q_R = \pi d L_2 \alpha c_u \ (5)$$

Where:

α: the adhesion factor under compressive loading.

Two cases should be considered:

- When excluding the dead load Q applied at the pile head, with FS = 1,2: $Q_{up} = \frac{Q_R}{1,2}$
- When including the dead load Q applied at the pile head, with FS = 2,0: $Q_{up} - Q = \frac{Q_R}{2,0}$

In general, a number of methodologies are available for quantifying the upward heave of foundation bases associated with soil swelling. Broadly, these methods can be grouped into two categories. The first comprises procedures that evaluate heave directly from laboratory-determined swelling indices-most notably the swelling strain and the swelling pressure-obtained from oedometer or related tests. The second comprises approaches that infer heave from the volumetric response of the soil within the framework of unsaturated soil mechanics, wherein changes in matric suction and effective stress govern volumetric expansion. Irrespective of the framework adopted, the computational models ultimately require the swelling pressure measured in controlled laboratory testing as a fundamental input parameter.

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Received 15/09/2025, Revised 02/10/2025, Accepted 03/10/2025
Link DOI: https://doi.org/10.54772/jomc.v15i02.1112

2. Shallow foundation analysis on expansive soil

Laboratory data indicate that stiff clays in Southern Vietnam exhibit medium to high swelling potential [9], including some soils in the Southeast and South-Central regions [1]. In design practice, however, swelling is not always explicitly treated. Heave of piles during construction has been observed, yet often lacks quantitative explanation. Cracking and tilting of small to medium structures due to wetting from water-collection acilities have also been reported.

Consider a near-surface expansive clay layer with the following properties:

- Unit weight $\gamma = 18{,}4$ kN/m^3;
- Water content w = 15,5 %;
- Void ratio $e_0 = 0{,}682$;
- Cohesion c = 20 kPa;
- Friction angle $\varphi = 17°04'$.

Laboratory swelling characterization gives $p_{sw} = 164$ kPa and $\varepsilon_{sw} = 8{,}9$ %. The compression–swelling (oedometer) curve accounting for the specimen's expansiveness is shown in Figure 1 [9], constructed in accordance with the guidance in [4].

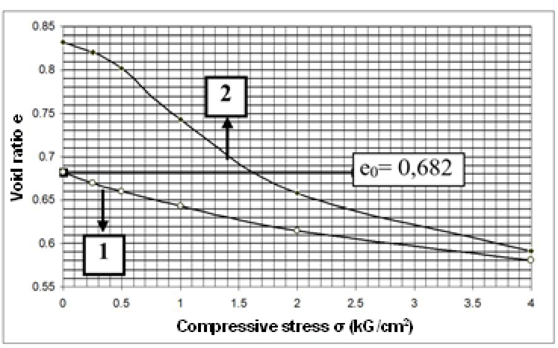

Figure 1. Compression curves in dry condition (curve 1) and after swelling then reloading (curve 2).

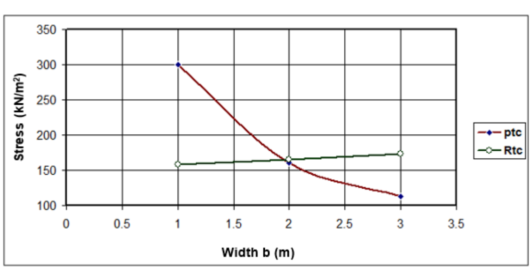

Figure 2. Relationship of R^{tc}, p^{tc} and footing width b.

A strip footing of width b is placed at depth $D_f = 1$ m; the line load is $N^{tc} = 280$ kN/m. The interaction chart of bearing capacity R^{tc}, design contact stress p^{tc}, and footing width b indicates a reasonable width b = 2m (Figure 2).

The bearing-capacity check for a footing of width b = 2 m, performed according to Terzaghi and Vesic, indicates an allowable bearing capacity of at least 200 kN/m^2 with a factor of safety *FS = 2*.

In the absence of inundation, swelling does not occur. In this case, foundation settlement can be evaluated using conventional methods. For a footing with width b = 2 m, $D_f = 1$ m, the net stress beneath the base is:

$$p_{gl} = p^{tc} - \gamma_{tb} \times D_f = 160 - 20 \times 1 = 140 \text{ kN/m}^2$$

With the stress distribution, the compressible thickness is 6,5m; layer summation gives total settlement $S = \Sigma, S_i = 7{,}5$ cm.

In the case of prolonged inundation, water infiltration into the subgrade induces soil swelling. Because the mean stress beneath the footing is approximately equal to the subgrade swelling pressure, $p_{sw} = 164$ kN/m^2, the resultant footing heave is negligible. If some settlement had occurred prior to wetting, subsequent inundation would tend to lift the footing back toward its original elevation, i.e., the pre-construction position.

In practical construction, for most projects using shallow foundations in the area, the contact stress under the footing generally does not exceed 100 kN/m^2. Consider the case where the stress under the footing is 90 kN/m^2. With a stress of 90 kN/m^2, the foundation settlement in non-submerged conditions will be less than 7,5 cm and within the allowable limit.

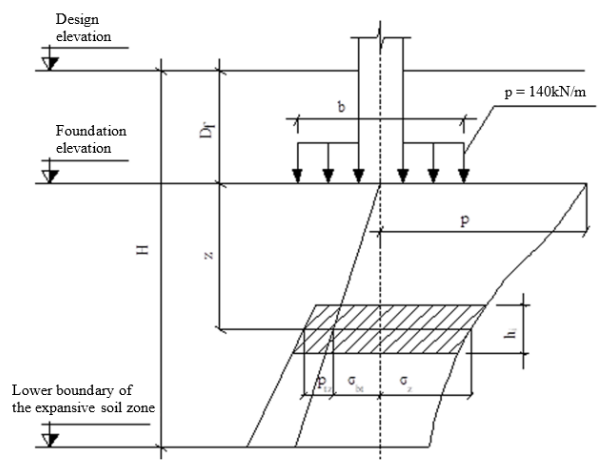

Figure 3. Calculation diagram for foundation heave in expansive soils.

In service, the foundation soil may become wetted locally (e.g., at manholes or soak pits). Assume the wetted patch has an aspect ratio of:

$$\frac{L}{B} = \frac{24}{12} = 2$$

Subdivide the soil beneath the footing into eight 1-m layers and compute the resultant stress at the midpoint of each layer for the swelling condition. The midpoint of the layer adjoining the footing base is at depth $z + h = 1{,}5$m. Under $\frac{z+h}{B} = \frac{1{,}5}{12} = 0{,}125$, take $m_n = 0$.

The vertical stress contributing to settlement at the footing base is:

$\sigma_z = k_0 \times p_{gl} = k_0 \times (p^{tc} - \gamma_{tb} \times D_f) = 0{,}953 \times (90 - 20 \times 1) = 66{,}71$kN/m^2

Computations follow reference [4]; results are summarized in Table 1.

At a depth 7,5 m below the footing base, the total stress is approximately equal to the soil's swelling pressure; therefore, the thickness of the swelling (active) zone is taken as 7,5 m measured from the footing base. Based on the laboratory test charts, the void ratio ei can be determined from the applied effective stress. Hence:

$$\varepsilon_{sw} = \frac{\Delta h_{sw}}{h_0} = \frac{e_i - e_0}{1 + e_0}$$

Table 1. Computation of foundation heave induced by soil swelling.

z (m)	$\frac{z + D_f}{B}$	m_n	$\sigma_z = k_0 \times p_{gl}$ (kN/m^2)	$\sigma_{bt} = \gamma z$ (kN/m^2)	p_{tz} (kN/m^2)	p_t (kN/m^2)	m
0,5	0,125	0,000	66,71	9,20	0,000	75,91	0,779
1,5	0,208	0,000	46,90	27,60	0,000	74,50	0,780
2,5	0,292	0,000	32,41	46,00	0,000	78,41	0,777
3,5	0,375	0,000	24,22	64,40	0,000	88,62	0,769
4,5	0,458	0,000	19,25	82,80	0,000	102,05	0,758
5,5	0,542	0,042	15,89	101,20	5,023	122,11	0,742
6,5	0,625	0,125	13,51	119,60	17,250	150,36	0,720
7,5	0,708	0,208	11,69	138,00	32,531	182,22	0,694

Table 2. Relationship between swelling strain ε_{sw} and staged applied stresses.

p_t (kN/m^2)	75,91	74,50	78,41	88,62	102,05	122,11	150,36	182,22
e	0,770	0,772	0,768	0,753	0,744	0,722	0,693	0,665
ε_{sw} (%)	5,23	5,35	5,12	4,22	3,69	2,38	0,66	1,01

Uplift of the foundation:

$$S_{sw} = \sum_{i=1}^{n} \varepsilon_{swi} h_i m = 21{,}2\text{cm}$$

For the strip-footing case considered above, it can be observed that under submerged conditions, if the contact stress beneath the footing is approximately equal to the swelling pressure, the settlement of the footing is negligible. However, if some footings within the project area are submerged while others are not, the foundation system may undergo rigid-body movements leading to differential settlement. In the present calculation, the vertical displacement difference between footings can reach 7,5 cm.

When the contact stress beneath the footing is lower than the swelling pressure, swelling-induced uplift may produce vertical displacements whose magnitudes can exceed the settlement (even for $\varepsilon_{sw} = 4$ %). Such behavior results in differential movements and may compromise the stability of the structure.

3. Pile foundations in expansive soils

The analysis of pile foundations in expansive soils primarily concerns determining the uplift force induced by the soil's swelling pressure. According to existing sources, at depths of 30–50 m in Ho Chi Minh City and adjacent areas, a semi-hard to hard clay stratum is commonly encountered. Laboratory testing indicates that this stratum has a thickness of 8–12 m and exhibits medium to high swelling potential [9]. Piles penetrating this layer may be affected by swelling pressure if the ground becomes submerged. In this study, we evaluate the influence of swelling on the response of a bored pile with 1,0 m diameter and 40 m length.

Based on the site investigation for the Ben Luc – Long Thanh Expressway project, the subsurface profile can be summarized as follows:

- Layer 1: Clayey mud, gray-green to dark gray, very soft, thickness 19 m.
- Layer 2: Silty clay, yellow-brown, stiff to very stiff, thickness 10,3 m.
- Layer 3: Clay, yellow-brown to reddish-brown, hard, thickness 9,4 m; natural unit weight $\gamma = 20{,}97$kN/m^3; water content $w = 16{,}0$ %; initial void ratio $e_0 = 0{,}481$; cohesion $c = 65$ kPa; internal friction angle $\varphi = 22^\circ$.
- Swelling pressure (from constrained-swell test results): $p_{sw} = 207$kN/m^2; swelling strain: $\varepsilon_{sw} = 14{,}5$ %; swelling pressure upon reloading in the wetted state: $p_{sw(wet)} = 400$kN/m^2.
- Layer 4: Medium sand, light yellow, dense structure, thickness 30 m.

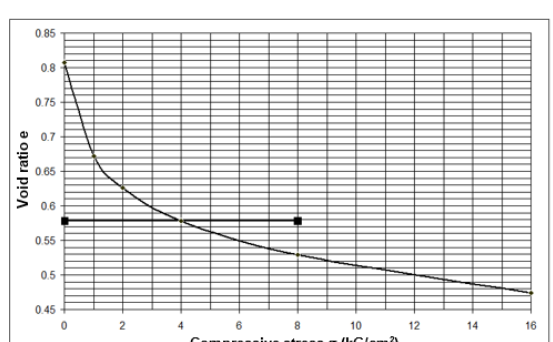

Figure 4. Compression–settlement curve for swelling and reloading.

Uplift force Q_{up}:

When the active-zone boundary coincides with the top of the expansive clay layer (case a):

$$Q_{up} = 0.$$

- When the active-zone boundary lies within the expansive clay layer (case b):

$$Q_{up} = \pi d\, \alpha_u\, p_{sw}\, L_1$$

Where:

- d: pile diameter;
- α_u: uplift adhesion factor between soil and concrete ($\alpha_u = 0{,}15$);
- p_{sw}: swelling pressure;
- L_1:pile length within the active portion of the expansive clay.

$Q_{up} = \pi d \alpha_u p_{sw} L_1 = \pi \times 1 \times 0{,}15 \times 400 \times 4{,}7 = 886$ kN.

- When the active-zone boundary coincides with the bottom of the expansive clay layer (case c):

$Q_{up} = \pi d \alpha_u p_{sw} L_2 = \pi \times 1 \times 0{,}15 \times 400 \times 9{,}4 = 1772$ kN.

Uplift resistance Q_R: $Q_R = \pi d L \alpha c_u$

Where:

- L: pile length within the soil contributing to uplift resistance;
- α: adhesion factor under compressive loading;
- c_u: undrained cohesion

$Q_{R1} = \pi \times 1 \times 19 \times 0{,}55 \times 8{,}37 = 275$ kN

$Q_{R2} = \pi \times 1 \times 10{,}3 \times 0{,}55 \times 24{,}3 = 432$ kN.

$Q_{R3} = \pi \times 1 \times 1{,}3 \times 0{,}55 \times 0{,}008 \approx 0$

Total uplift resistance: $Q_R = Q_{R1} + Q_{R2} + Q_{R3} = 275 + 432 = 707$ kN.

Without applied load on the pile: $FS = \frac{Q_R}{Q_{up}} = \frac{707}{1772} = 0{,}4 < 1{,}2$, the total uplift resistance is smaller than the swelling-induced uplift force.

Therefore, in the absence of applied load – during construction, for example – the pile may be jacked upward (heave) due to water infiltration along the pile shaft causing soil swelling.

With applied load on the pile: Assuming the axial load $N^{tc} = 6000$ kN exceeds the swelling-induced uplift $Q_{up} = 1772$ kN, the pile will not be heaved (i.e., no upward displacement occurs).

The analysis for a 40 m long precast reinforced-concrete driven pile with a 0,4 m × 0,4 m cross-section yields results similar to those for the bored-pile case. Accordingly, the resistance factor against swelling – induced uplift does not depend significantly on the pile diameter; rather, it depends primarily on the length of the pile segment traversing the expansive clay layer. The calculations indicate that, in the absence of applied load, the pile may be pushed upward due to subsoil swelling – an outcome that is also frequently observed in local pile-construction practice.

4. Embankment subgrades using expansive materials

Laboratory studies by Nguyễn Văn Thơ and Trần Thị Thanh indicate that the embankment materials in several areas exhibit medium swelling potential. According to the same source [1], the swelling strain may reach 17 % to over 30 %, whereas the swelling pressure is relatively low, about 0,1 – 0,5 kG/cm², with isolated cases up to 1 kG/cm².

For the purposes of calculating and assessing the swelling susceptibility of the embankment foundation, we employ the geotechnical parameters reported in [1]. The experimental results characterizing the swelling behavior of this soil layer (from the tests on pp. 149 and 152 of [1]) indicate that:

- Swelling pressure: $p_{sw} = 25$ kN/m²;
- Swelling strain: $\varepsilon_{sw} = 10$ %.

Based on the available one-dimensional compression test data, the compression–settlement curve of the embankment soil specimen was re-plotted on a semi-logarithmic scale, as shown in Figure 5.

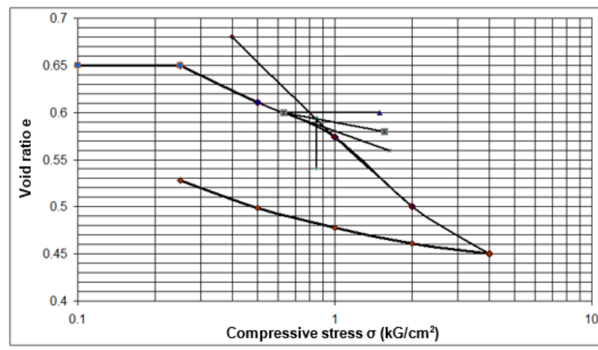

Figure 5. Semi-log compression curve of embankment.

The swelling strain of the elemental soil layer can be determined from the following expression:

$$\Delta h_i = \frac{C_s}{1 + e_{0i}} h_i \log \frac{p_{fi}}{p_{0i}}$$

Divide the 2m thickness embankment layer into four elemental sublayers, each 0.5m thickness

$$\Delta h_1 = \frac{0{,}05}{1+0{,}65} \times 500 \times \log \frac{4{,}9}{25} = -10{,}7 \text{ (mm)}$$

$$\Delta h_2 = \frac{0{,}05}{1+0{,}65} \times 500 \times \log \frac{14{,}6}{25} = -3{,}5 \text{ (mm)}$$

$$\Delta h_3 = \frac{0{,}05}{1+0{,}65} \times 500 \times \log \frac{24{,}4}{25} = -0{,}2 \text{ (mm)}$$

$$\Delta h_4 = \frac{0{,}05}{1+0{,}65} \times 500 \times \log \frac{34{,}1}{25} = 2 \text{ (mm)}$$

Total uplift: $S_{sw} = 10{,}7 + 3{,}5 + 0{,}2 - 2 = 12{,}4$ mm.

Thus, for embankments constructed with expansive materials, submergence can lead to localized heave. In addition to the magnitude of swelling strain, the swelling pressure can significantly influence both the heave magnitude and the extent of the affected zone. In this case, under the action of self-weight, no swelling occurs below a depth of 2 m.

5. Conclusions

From synthesis of prior studies and the present computations for shallow foundations, piles, and embankments, the following conclusions are drawn:

1) Utilizing the bearing capacity of the subgrade such that the base stress is approximately equal to the swelling pressure can ensure acceptable performance.

2) If base stress is designed overly conservatively (e.g., FS ≥ 3), swelling pressure may induce significant uplift exceeding serviceability limits (e.g., heave up to ≈ 20 cm while settlement is only 7,5 cm).

3) In the Ho Chi Minh City region and vicinity, swelling pressures in stiff clays can heave piles during construction before structural loads are applied.

4) Owing to the relatively low swelling pressures in certain local embankment materials [1], the heave-affected zone is shallow (≈ 1,5–2,0 m), heave magnitudes are modest, and manifestations are often localized.

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