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State-of-the-Art Review of Collapsible Soils

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التربة القابلة للهبوط : مراجعة و تحليل

عامر بن علي الرواس

خلاصـة: تنتشر التربة القابلة للهبوط في المناطق الجافة وشبه الجافة. ومثل هذه التربة يحتمل أن تسبب مشكلات للبـناء وذلك لهبوطها عند تعرضها للماء. إن ظاهرة هبوط التربة تعود وبشكل رئيسي الى البنية غير المتماسكة التي تتمتع بها. ولقد قدمت الأدبيات في هذا الجانب عدة تصنيفات لهذا النوع من التربة بناء على بعض الخواص مثل المحتوى المائي والكثافة الجافة وحدود آتربرج والمحتوى الطيني وذلك كمؤشرات لاحتمال هبوط التربة. ومن الضـروري اجراء قياس مباشر لمقدار الهبوط باستخدام الفحوص المعملية والحقلية عند ظهور مؤشرات الهبوط المحتمل. ولقد أظهرت طرق المعالجة المختلفة مثل استبدال التربة والدمك والتثبيت الكيميائي انخفاضاً ملحوظاً في معـدلات الهـبوط التي تسببها هذه التربة. ويعتمد تصميم قواعد البناء على هذه التربة على عمق التربة ومقدار الهـبوط واقتصاديات التصميم. يشيع استخدام القواعد المستمرة في حالة وجود هذه التربة على أعماق سطحية ، بيـنما ينصح بقواعد الركائز و الدعامات في الحالات التي تمتد عندها التربة الى عدة أمتار تحت سطح الأرض. وتقـدم هـذه الورقة مراجعة شاملة للتربة القابلة للهبوط . وتشمل الأنواع المختلفة للتربة القابلة للهبوط ، وطرق التعرف والتصنيف ، والاختبارات المعملية والحقلية ، وطرق المعالجة ، وارشادات تصميم القواعد.

ABSTRACT: Collapsible soils are encountered in arid and semi-arid regions. Such soils cause potential construction problems due to their collapse upon wetting. The collapse phenomenon is primarily related to the open structure of the soil. Several soil collapse classifications based on parameters such as moisture content, dry density, Atterberg limits and clay content have been proposed in the literature as indicators of the soil collapse potential. Direct measurement of the magnitude of collapse, using laboratory and/or field tests, is essential once a soil showed indications of collapse potential. Treatment methods such as soil replacement, compaction control and chemical stabilization showed significant reduction in the settlement of collapsible soils. The design of foundations on collapsible soils depends on the depth of the soil, magnitude of collapse and economics of the design. Strip foundations are commonly used when collapsing soil extends to a shallow depth while piles and drilled piers are recommended in cases where the soil extends to several meters. This paper provides a comprehensive review of collapsible soils. These include the different types of collapsible soils, mechanisms of collapse, identification and classification methods, laboratory and field testing, treatment methods and guidelines for foundation design.

KEYWORDS: collapsible soils, identification, laboratory tests, field tests, stabilization, foundation design.

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1. Introduction

Collapsible soils are widely distributed in most parts of the world (e.g. United States of America, Brazil, Egypt, Kuwait, South Africa and China) particularly in arid and semi-arid regions. These soils pose potential threat to structures built on them when wetted. At their dry, natural state, they possess stiffness and high apparent shear strength; but upon wetting, they could exhibit a significant decrease in volume (collapsing, hydroconsolidation, hydrocompression). Such soils, which exhibit this phenomenon at fairly low levels of stresses are called *collapsible soils* (Tadepalli *et al*. 1992; Rollins and Rogers 1994).

The sources of water can be either natural, such as rainfall and fluctuation in ground water table, or man-made, such as excessive irrigation and leakages from water and sewer lines. Collapse may be triggered by water alone or by wetting and loads acting together. The existence of collapsible soils has long been recognized since World War II. However, the recent infrastructure developments in arid regions, accompanied by the use of large quantities of water and the associated construction problems warrant a comprehensive investigation of these soils. The collapse of soils due to wetting may result in settlements of 2 to 6 percent of their thickness (Beckwith and Hansen 1989). The settlement can be large as demonstrated by irrigation canal settlements of 4.5 m in the West Central Part of the San Joaquin Valley in California (Bull 1964). Ismael *et al*. (1987) reported construction problems associated with the collapse of compacted sands in Kuwait which include settlement and cracking of the ground floor slabs of many villas and buildings, failure and erosion of slopes, and settlement and cracking of highway pavements. Intolerable settlements can cause extensive damage to civil engineering structures and result in significant financial losses. In the United States, the estimated cost of repair to structures at a cement plant in central Utah located on collapsible soils was more than $20,000,000 (Hepworth and Langfelder 1988).

This paper presents a state-of-the-art review of collapsible soils, with special reference to: (a) the types of collapsible soils; (b) collapse mechanisms; (c) identification and classification; (d) laboratory and field tests; (e) stabilization techniques; and (f) foundation design.

The following symbols are used in this paper:

CI = indicator;
CP = collapse potential;
e = void ratio at natural moisture content;
e_o = original void ratio;
e_L = void ratio at liquid limit;
$(e_o)i$ = initial void ratio;
Δe_c = change in void ratio of the specimen upon wetting;
Δe_s = change in void ratio of the specimen caused by collapse of soil structure;
H_o = original height of the specimen;
ΔH = change in height of the specimen upon wetting;
i_c = collapse index;
I_p = plasticity index;
K, K_D, K_L = coefficients of subsidence;
m = natural moisture content;
PI = plasticity index;
PL = plastic limit;
p_o = overburden pressure;
p_c = preconsolidation pressure;
P_{pr} = proportionality limit;
Δp = pressure increment;
q_c = cone penetration tip resistance;
R = collapse ratio;
Sr = degree of saturation;
w = water content;
ε_o = bearing plate settlement value corresponding to overburden pressure;
ε_{pr} = bearing plate settlement value corresponding to proportionality limit.

2. Types of Collapsible Soils

The term *collapsible soils* encompasses a wide range of materials that exhibit the collapsing phenomenon. These materials include aeolian or wind deposits, water deposits, residual soils and colluvial deposits. The mechanisms that account for almost all naturally occurring collapsible soil deposits are debris flows and alluvial deposition, and deposition of wind-blown materials (Beckwith 1995; Derbyshire *et al*. 1995). However, Bell and Bruyn (1997) reported that the majority of naturally occurring collapsible soils are aeolian deposits. According to Jennings and Knight (1975), the soil deposits most likely to collapse are: (a) loose fills; (b) altered wind-blown sands; (c) hillwash of loose consistency; and (d) decomposed granite and other acid igneous rocks. Collapsible soils are generally characterized by their loose structure of bulky shaped grains, often in the silt to fine sand size with a small amount of clay. There may be only slight cementing agents such as calcium carbonate, salts and dried clay, with combinations being common. A brief description of the common types of collapsible soils is given in the following paragraphs.

Aeolian deposits such as loess, dunes and other wind-blown deposits are encountered in different parts of the world. Clemence and Finbarr (1981) reported that loess distribution covers about 17 % of the United States, about 17 % of Europe, 15 % of Russia and Siberia, and large areas of China. Loess are also encountered in South America (i.e. Argentina and Uruguay) and southern Africa. Calcareous windblown dune sands are encountered in Kuwait and some parts of the Arabian Peninsula (Ismael *et al*. 1987). Aeolian soils have a loose open, metastructure binded by cementing agents, which upon wetting, become weak and may dissolve causing collapse. These soils are composed primarily of quartz along with feldspar and clay minerals. Bell and Bruyn (1997) reported that increasing the clay mineral content decreases the likelihood of collapse.

Water deposits include alluvial fans, mud flows and flash flood deposits. These deposits are laid down by water in a saturated state. They become hard and less compressible with relatively low density as they dry. The structure is usually open and porous, and soil grains are bonded together by cementing agents during the deposition. If these deposits are subsequently exposed to water accompanied with or without additional loading, they may collapse, thereby causing large settlement. Alluvial deposits that exhibit collapsible phenomenon are encountered in Saudi Arabia and other parts of the Middle East (El-Nimr *et al* , 1992; Fookes *et al*. 1985).

Residual soils cover a wide range of sizes, from clay size up to the gravel range. The collapsible structure is developed as a result of the leaching of the soluble and colloidal matter from the residual soil. This leaching of the soluble and fine materials results in a porous and unstable structure.

3. Collapse Mechanisms

Collapse mechanisms have been studied by several researchers (Casagrande 1932; Jennings and Knight 1957; Holtz and Hilf 1961; Burland 1965; Dudley 1970; Barden *et al.* 1973; Mitchell 1993). The collapse phenomenon is primarily related to the open structure of the soil. Casagrande (1932) has demonstrated that a portion of the fine-grained fraction of the soil exists as bonding material for the larger-grained particles and that these bonds undergo local compression in the small gaps between adjacent grains resulting in the development of strength Figure 1. At natural moisture content, these soils compress slightly as a result of increasing overburden pressures due to construction. However, the structure remains sensibly unchanged. When the loaded soil is exposed to moisture, and a certain critical moisture content is exceeded, the fine silt or clay bridges that are providing the cementation will soften, weaken and/or dissolve to some extent. Eventually, the binders reach a stage where they no longer resist deformation forces and the structure collapses as shown in Figure 1.

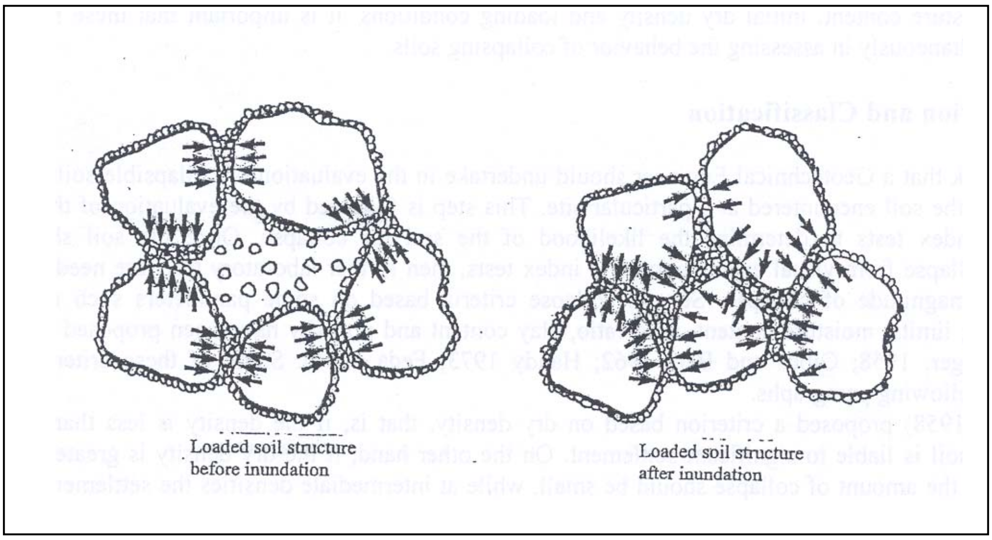

Figure 1. Loaded structure before and after inundation (after Casagrande 1932).

Aitchison and Donald (1956) reported that for uniform spherical grains in an open cubical packing, the maximum added pressure due to the capillary effect occurs at a moisture content of about 32%. For the densest packing of uniform spherical grains, the maximum tension occurs at 10% moisture content. Investigations conducted with various collapsible soils showed that the peak effective stress values occur at moisture contents less than saturation and above 10% water content (Clemence and Finbarr 1981).

Holtz and Hilf (1961) described the collapse mechanism as the result of capillary pressures approaching zero and the degree of saturation increasing to 100 %. Burland (1965) described the collapse mechanism in terms of the stability at the interparticle contact points. Due to wetting, the negative pore water pressure at the contact points decreases, causing grain slippage and distortion. Dudley (1970) explained that as the soil dries below the shrinkage limit, the remaining water at the grains contact points is placed under tension. Thus the excess water pressure becomes negative and therefore, the actual effective stress becomes larger than the total stress applied by the load. This increases the apparent strength of the soil.

Dudley (1970), Barden *et al*. (1973) and Mitchell (1993) explained the collapse phenomenon in terms of the cementing agents at the contact points of the soil grains. They identified four conditions required for the collapse to occur:

1. An open, partially unstable, partially saturated fabric.
2. A high enough total stress so that the structure is metastable.
3. A strong enough clay binder or other cementing agent to stabilize the structure when dry.
4. The addition of water to the soil which reduces the soil suction and thus produces the collapse.

Collins (1978) explained that an open fabric, which is a prerequisite for collapse, may be comprised of: (a) clothed grain-grain contacts; (b) grain bridges or buttresses comprised of either clay or silt plus possibly cementitious material such as iron oxide or carbonates; and (c) clay aggregates. The rate of collapse depends on the type of bonding and this was indicated by Tadepalli *et al*. (1992) who reported that the collapse phenomenon is primarily related to the reduction of the matric suction during inundation. The rate of collapse depends to a large extend on the type of bonding as well as the other influencing factors. Clemence and Finbarr (1981) reported that collapse is more immediate in the case where soil grains are held together by capillary suction, but slow in the case of chemical cementing, and much slower in the case of clay buttresses.

Based on the above studies, it seems that the collapsing phenomenon is a complex one, which includes fabric, initial moisture content, initial dry density and loading conditions. It is important that these factors be looked at simultaneously in assessing the behavior of collapsing soils.

4. Identification and Classification

The first task that a Geotechnical Engineer should undertake in the evaluation of collapsible soils is to visually examine the soil encountered at a particular site. This step is followed by the evaluation of the soil behavior using index tests to determine the likelihood of the soil for collapse. Once the soil showed indications for collapse from visual examination and index tests, then further laboratory tests are needed for determining the magnitude of collapse. Several collapse criteria based on some parameters such as dry density, Atterberg limits, moisture content, void ratio, clay content and porosity have been proposed in the literature (Clevenger, 1958; Gibbs and Bara 1962; Handy 1973; Feda 1966). Some of these criteria are discussed in the following paragraphs.

Clevenger (1958) proposed a criterion based on dry density, that is, if the density is less than 1.28 Mg/m^3, then the soil is liable to significant settlement. On the other hand, if the dry density is greater than 1.44 Mg/m^3, then the amount of collapse should be small, while at intermediate densities the settlements are transitional. However, Jennings and Knight (1975) disputed these conclusions. It should be noted that since unstable soils have a wide range of dry densities which also apply to non-collapsible soils, the dry density alone seems to be misleading and an unreliable indicator of collapse. Furthermore, other factors such as the soil moisture content, which greatly influences the soil collapse was not taken into account.

Bell and Bruyn (1997) reported a criterion suggested by Gibbs and Bara (1962) based on natural dry density and liquid limit, which distinguishes between collapsible soils and non-collapsible soils as shown in Figure 2. The method is based on the premise that a soil, which has enough void space to hold its liquid limit moisture at saturation is susceptible to collapse on wetting. Figure 2 shows that soils above the lines are in a loose condition, and when fully saturated will have a moisture content greater than liquid limit. Handy (1973) suggested a criterion based on the ratio of liquid limit to saturation. If this ratio is less than 1, the soil is collapsible. However, if it is greater than 1, the soil is non-collapsible. Dudley (1970) reported that most of the collapsing soils have liquid limits below 45 % and plasticity indices below 25%.

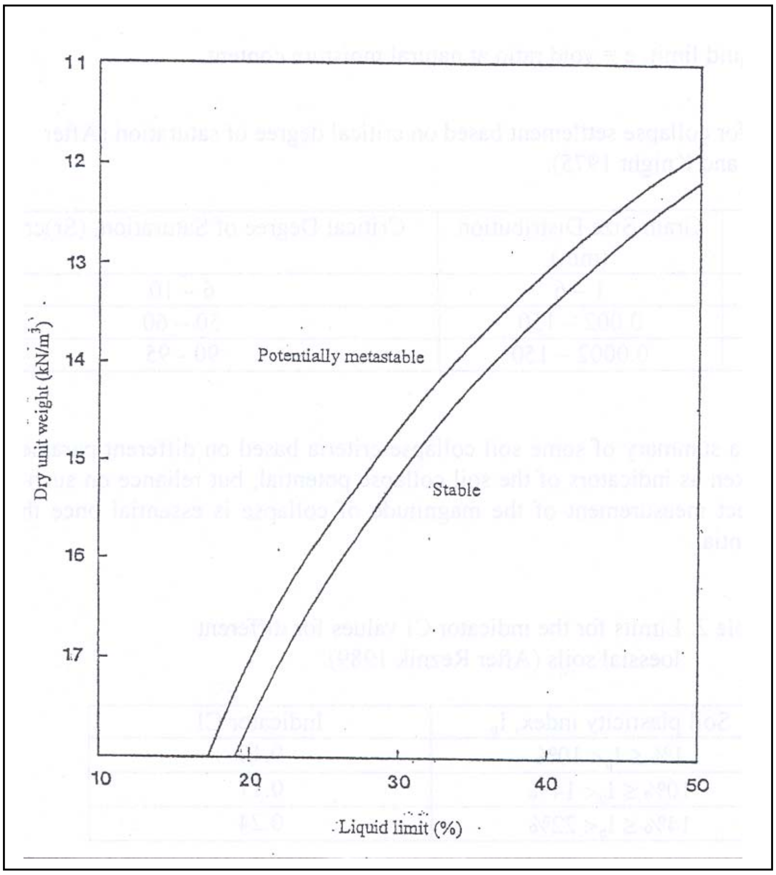

Figure 2. Classification of collapsing soils (after Gibbs and Bara 1962).

A collapse index (i_c) involving natural moisture content (m), degree of saturation (Sr), plastic limit (PL) and plasticity index (PI) was proposed by Feda (1966). The collapse index was defined in the following formula

$$i_c = \frac{m / Sr - PL}{PI} \tag{1}$$

If the collapse index was greater than 0.85, then this was indicative of collapsible soils.

One of the key prerequisites for collapse to occur is that the soil must be partially saturated; and there appears to be a critical value for the degree of saturation (Sr) below which the collapse phenomenon can occur and above which it will not occur. Jennings and Knight (1975) suggested a guideline, which relates the type of soil, grain size distribution and the critical degree of saturation as given in Table 1. According to the Soviet Building Code (Reznik 1989), soils are identified as collapsible if their degree of saturation is less than 0.8 and the values of the indicator CI are less than the ones shown in Table 2. The indicator CI is defined in the following formula

$$CI = -\frac{e_L - e}{1+e} \tag{2}$$

Where e_L = void ratio at liquid limit, e = void ratio at natural moisture content.

Table 1. A guide for collapse settlement based on critical degree of saturation (After Jennings and Knight 1975).

Soil	Grain Size Distribution (mm)	Critical Degree of Saturation, (Sr)crit. (%)
Fine gravels	1 – 6	6 – 10
Fine silty sands	0.002 – 150	50 – 60
Clayey silts	0.0002 – 150	90 - 95

Das (1995) provided a summary of some soil collapse criteria based on different parameters (Table 3). These criteria should be taken as indicators of the soil collapse potential, but reliance on such criteria can be misleading. Therefore, direct measurement of the magnitude of collapse is essential once the soil showed indications of collapse potential.

Table 2. Limits for the indicator CI values for different loessial soils (After Reznik 1989).

Soil plasticity index, I_p	Indicator CI
$1\% \le I_p < 10\%$	0.10
$10\% \le I_p < 14\%$	0.17
$14\% \le I_p < 22\%$	0.24

5. Laboratory Tests

Laboratory tests are used for obtaining quantitative estimates of collapse potential, which can be used for estimating potential settlements of structures. Ismael *et al*. (1987) carried out a study to examine the factors affecting the collapse potential of calcareous desert sands in Kuwait, which included relative density, overburden pressure, and effect of soil disturbance or remolding. They reported the following conclusions:

1. The collapse potential decreases linearly with the relative density.
2. The collapse potential increases with the applied pressure at a decreasing rate.
3. Disturbance of the ground soils results in a significantly higher collapse potential.

Direct measurement of the *collapse potential* using one-dimensional oedometer or consolidometer test, the most common test, have been conducted by several researchers as follows:

1. Double-oedometer test (Jennings and Knight 1975)
2. Single specimen collapse test (Houston *et al*. 1988) and ASTM D5333 (ASTM 1993)
3. Single point, multiple specimen test (Noorany 1992).

Table 3. Reported criteria for identification of collapsing soil[a] (After Das 1995).

Investigator	Year	Criteria
Denisov	1951	Coefficient of subsidence: K = void ratio at liquid limit / natural void ratio K = 0.5-0.75: highly collapsible K = 1.0: noncollapsible loam K = 1.5-2.0: noncollapsible soils
Clevenger	1958	If dry unit weight is less than 12.6 kN/m^3, settlement will be large; if dry unit weight is greater than 14.1 kN/m^3, settlement will be small.
Priklonski	1952	K_D = (natural moisture content – plastic limit) / (plasticity index) $K_D < 0$: highly collapsible soils $K_D > 0.5$: noncollapsible soils $K_D > 1.0$: swelling soils
Gibbs	1961	Collapse ratio, R = saturation moisture content / liquid limit This was presented graphically.
Feda	1964	$K_L = (w_o/S_r) – (PL/PI)$ Where w_o = natural water content, S_r = natural degree of saturation, PL = plastic limit, and PI = plasticity index. For $S_r < 100\%$, if $K_L > 0.85$, it is a subsident soil.
Benites	1968	A dispersion test in which 2 g of soil are dropped into 12 ml of distilled water and specimen is timed until dispersed; dispersion times of 20 to 30 s were obtained for collapsing Arizona soils.
Handy	1973	Iowa loess with clay (<0.002 mm) contents: 16%: high probability of collapse, 16-24%: probability of collapse, 24-32%: less than 50% probability of collapse, >32%: usually safe from collapse

[a] Modified after Lutenegger and Saber (1988)

Jennings and Knight (1975) suggested a procedure using the double-oedometer for determining collapse potential by taking an undisturbed specimen at natural moisture content and placing it in a consolidometer ring. The specimen is then progressively loaded up to a pressure of 200 kPa. At this pressure, the specimen is flooded with water and left for 24 hours, and the consolidation test is then carried on to its normal maximum loading limit. Figure 3 shows the stages of the collapse potential test. The collapse potential (CP) is then defined as:

$$CP = \frac{\Delta H}{H_o} = \frac{\Delta e_c}{1+e_o} \cdot 100\% \qquad (3)$$

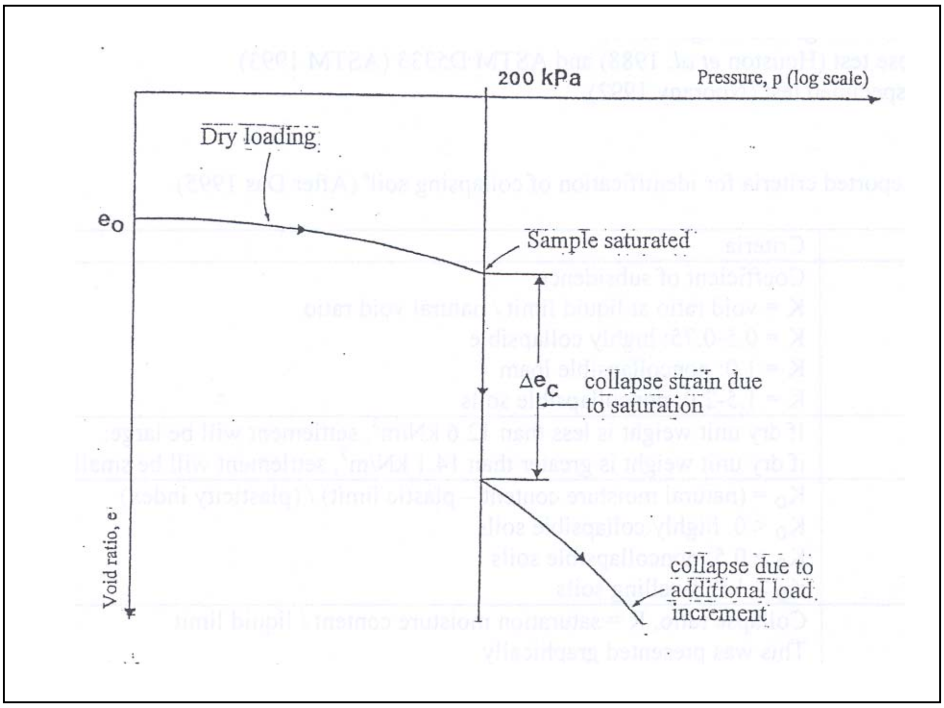

Figure 3. Typical collapse potential test result (after Clemmence and Finbarr 1981).

where ΔH is the change in height of the specimen upon flooding, H_o is the original height of the specimen, Δe_c is the change in void ratio of the specimen upon flooding and e_o is the void ratio before flooding. The collapse potential is only a guide to the collapse which may be encountered. They also suggested a classification of the potential severity of collapse based on the collapse potential as shown in Table 4, which classifies soils with collapse potential greater than 1% as metastable. However, in the United States, collapse values exceeding 2% are regarded as indicative of soils susceptible to collapse (Lutenegger and Hallberg 1988).

Table 4. Potential severity of collapse (After Jennings and Knight 1975).

Collapse Potential (%)	Severity of Problem
0 – 1	No problem
1 – 5	Moderate trouble
5 – 10	Trouble
10 – 20	Severe trouble
> 20	Very severe trouble

Houston and Houston (1997) compared the three common laboratory tests (single specimen test, double oedometer test, and single point, multiple specimen test) based on three case studies and reported that:

1. The advantage of the single specimen test is that more useful data is obtained from the specimen. However, it is not recommended to characterize a soil with just one test specimen.
2. There is no need to test a dry sample in the double oedometer test.
3. The single point, multiple specimen test follows the actual path postulated for the field.

6. Field Tests

Field tests are frequently used to identify collapsible soils. A very simple field test is the "sausage" test (Clemence and Finbarr 1981). A block of soil of about 500 cm^3 is taken from the test trial pit and broken into two pieces, and each is trimmed until they are approximately equal in volume. One specimen is then wetted and molded in the hands to form a damp ball. The volume of this ball is then compared with the volume of the undisturbed specimen. If the wetted ball is obviously smaller, then collapse may be suspected. This test is only a guide as to whether or not a soil can collapse.

Plate load tests are the most common field tests for the evaluation of allowable pressures under foundations. These tests are normally conducted near the ground surface. In this test, the water is introduced to the loaded soil and the resultant displacement due to wetting is recorded. The bearing plate settlement values for the same load intensity and soil conditions depend on their dimensions as indicated in the literature. It was reported that the former USSR Building Codes (Reznik 1992) specify that field test loading may be performed in open pits on circular rigid plates with areas of 600, 2500, and 5000 cm^2. The results of bearing plate tests are shown in the form of plate load-settlement curve (Figure 4) where the proportionality limit (Ppr) on this curve is accepted by the soviet engineers as the safe bearing capacity for foundations.

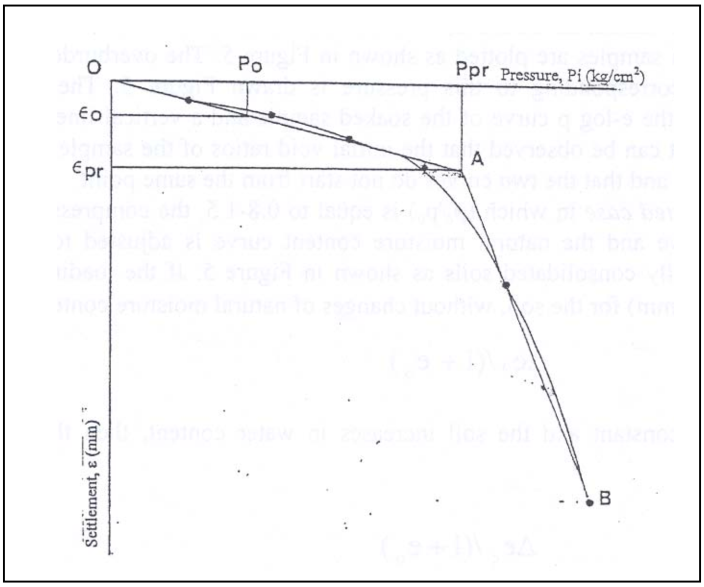

Figure 4. Diagram illustrating commonly used graphical construction for selecting a proportionality limit using a load intensity-bearing plate settlement curve. P_o and P_{pr} = values of overburden pressure and proportionality limit; ε_o and ε_{pr} = corresponding bearing plate settlements.

The advantages of plate load test include the minimization of soil sample disturbance, larger volume of soil being tested, and the test followed the actual field situation. However, difficulties in extrapolating the plate load test to prototype foundations normally preclude the estimation of settlement from field plate load test on collapsible soil (Houston and Houston 1997). Recent advances in field tests have eliminated the shortcomings of plate load test (Houston *et al*. 1995).

Collapsible soils may also be determined in the field using the cone penetration test (CPT) results (Reznik 1989). Rollins *et al*. (1998) carried out CPT at six field locations in Nephi, Utah (USA) according to the ASTM D-3441-86. They found that the tip resistance (q_c) of the soil at its natural moisture content (w = 7 % to 10 %) was typically between 3000 and 5000 kPa, but decreased to between 1,000 and 2,000 kPa for the wetter soil profile.

7. Estimation of Collapse Settlement

Collapsible soils exhibit differential settlements due to differential wetting and their heterogeneity in nature, which results in structural damage and distress to structures. The procedure for estimating collapse settlement based on laboratory results is given below.

Laboratory tests results of soil response to wetting are necessary for estimating the collapse settlement at a particular site. Jennings and Knight (1975) proposed a method for calculating collapse settlement of a soil for design purposes using the results of a double oedometer test. In this test, two identical undisturbed samples prepared with minimum disturbance (normally block samples cut by hand) are used. The two samples are fitted into the oedometer rings and placed in the oedometer under a light 1 kPa seating load for 24 hours. After 24 hours, one of the samples is saturated by flooding with water, while keeping the other sample at its natural moisture content. Both samples are left for a further 24 hours. If the soaked sample has swelled it is possible that either it is of an expansive type or that a rebound effect is experienced. Most collapsing soils of a sandy character will show little or no change during this 24-hour soaking period. The test is then carried out in the same manner as the standard consolidation test.

The e-log p graphs for both samples are plotted as shown in Figure 5. The overburden pressure (p_o) is calculated and a vertical line corresponding to this pressure is drawn Figure 5. The preconsolidation pressure (p_c) is determined from the e-log p curve of the soaked sample and a vertical line corresponding to this pressure is drawn Figure 5. It can be observed that the initial void ratios of the samples are not identical after the first 24 hours of loading, and that the two curves do not start from the same point.

For *the normally consolidated case* in which (p_c/p_o) is equal to 0.8-1.5, the compression is considered to take place on the virgin curve and the natural moisture content curve is adjusted to the ($e_o p_o$) point ordinarily associated with normally consolidated soils as shown in Figure 5. If the loading is increased to Δp, then the unit settlement (mm/mm) for the soil, without changes of natural moisture content will be

$$\Delta e_s/(1+e_o) \tag{4}$$

If the applied loading remains constant and the soil increases in water content, then the unit additional settlement will be

$$\Delta e_c/(1+e_o) \tag{5}$$

If these values are used with various selected layers in the profile, the total settlement under both conditions may be calculated. It should be noted that if Δp is small, then the accuracy of the estimates will be less.

For *the over-consolidated case* ($p_c/p_o > 1.5$), the adjustment to the curve largely follows the ordinary settlement computation practices as shown in Figure 6. The only difference between the two cases lies in the determination of the point ($e_o p_o$). Jennings and Knight (1975) examined several practical cases where collapse settlement had been recorded. They found that the agreement between the observed and predicted collapse settlements was good.

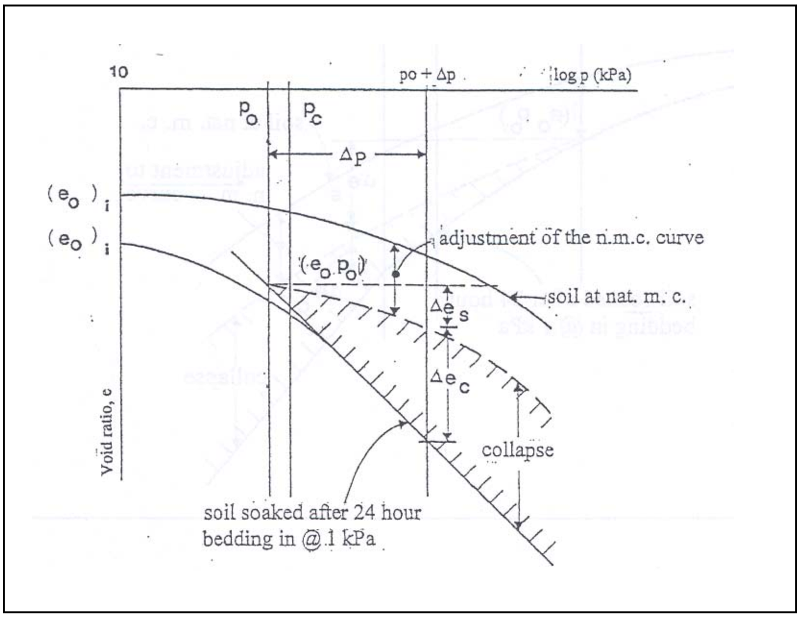

Figure 5. Double oedometer curves and adjustments for a normally consolidated soil (after Jennings and Knight 1975).

Clemence and Finbarr (1981) recommended the performance of field tests whenever possible due to the variety of mechanism supporting collapsible soils, and to check the laboratory analysis and the efficiency of the wetting technique used. However, field tests are expensive and time consuming. Estimated collapse settlement based on laboratory results may not be realized in the field. For example, Zhang and Zhang (1995) reported that in China, the actual field collapse settlements are only 1/7 of the estimated full-wetting collapse. The most likely explanation for this is that when a soil is only partially wetted, only a portion of the full collapse potential is realized (Houston 1996).

8. Stabilization of Collapsible Soils

There are several methods that can be used to minimize or eliminate the collapse of a particular soil. The choice of the appropriate method depends on the depth of the collapsing soil, type of structure to be constructed, and the cost and practicality of the method. These methods include

1. Soil replacement (Anayev and Volyanick 1986)
2. Prewetting (Houston and Houston 1997; Hansen *et al.* 1989)
3. Controlled wetting (Bally and Oltullesen 1980)
4. Moisture control (Mackenchinie 1980)
5. Compaction control (Rollins and Rogers 1994; Cintra *et al.* 1986)
6. Chemical stabilization or grouting (Clemence and Finbarr 1981; Pengelly *et al.* 1997)
7. Heat treatment (Bell and Bruyn 1997)

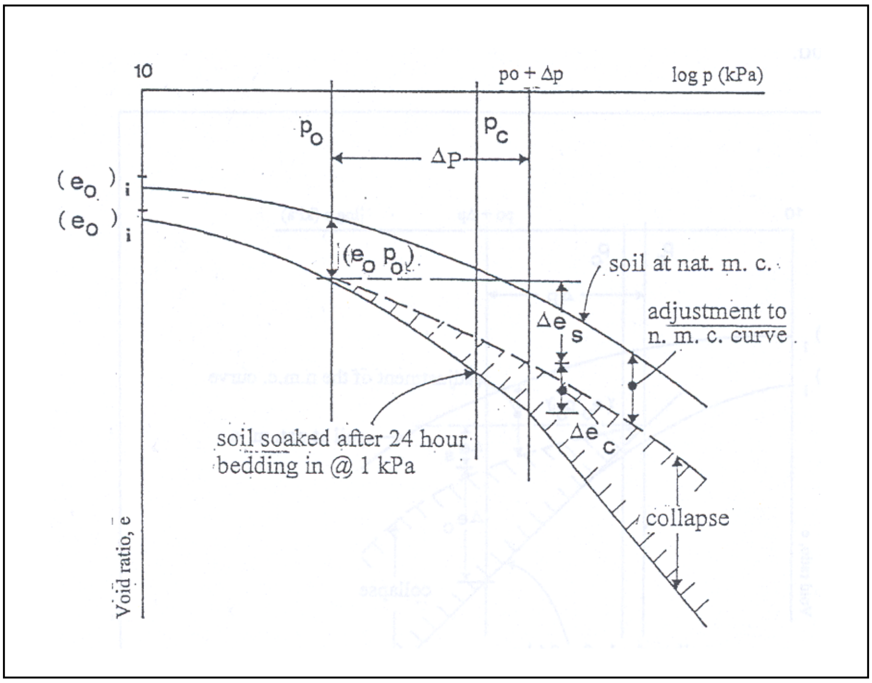

Figure 6. Double oedometer curves and adjustments for an overconsolidated soil (after Jennings and Knight 1975).

These methods are briefly described in the following paragraphs:

8.1 Soil Replacement

A simple solution is to excavate to the required depth and remove the collapsible soil. The removed soil can be compacted and used as the foundation soil. Such technique is commonly used particularly when collapsible soil occurs at shallow depth (Anayev and Volyanick 1986). The replaced soil should be compacted to a density of 95-100% (AASHTO Specifications). Jennings and Knight (1975) suggested that the soil should always be compacted at a moisture greater than 2% less than the optimum.

8.2 Prewetting

Prewetting means flooding or wetting the soil which is expected to exhibit collapse upon saturation *before* the structure is built, so that soil collapse will be minimized after the structure is built (Houston and Houston 1997; Gibbs and Bara 1967; and Hansen *et al*. 1989). Wetting the soil can be achieved through ponding or trenches and boreholes. Although prewetting is useful for canals and roadways where the induced loads are small, prewetting without preloading is not sufficient to prevent future foundation settlement. Prewetting causes the soil to collapse under its existing overburden pressure. Therefore, additional loads imposed by the foundation are not compensated for and will result in additional settlement (Rollins and Rogers 1994).

8.3 Controlled Wetting

Controlled wetting is similar to prewetting except that it is performed after the structure is in place. The quantities of water should be approximately measured and added in increments. This method is also used once a structure exhibit some damage or tilt due to differential settlement. The added water should be introduced in a carefully-monitored manner to correct the tilt (Bally and Oltullesen 1980).

8.4 Moisture Control

This method purports to prevent water ingress into the ground. The measures that can be used to control wetting include: (a) controlling water irrigation, (b) placing landscaping in watertight planter boxes, (c) restricting landscape vegetation adjacent to structure, (d) placing pavement or buried geomembranes around the perimeter of the structures, (e) placing effective surface and buried drainage systems, (f) informing occupants of buildings of the problems associated with collapsible soils.

8.5 Compaction Control

One of the most practical and effective methods of minimizing soil collapse is by the use of compaction. Compaction has been used for both shallow and deep collapsible soils. Rollins and Rogers (1994) pointed out that this method: (a) decreases the amount of collapsible soil in the zone of significant stress; (b) increases the depth to which water must percolate before it reaches collapsible materials; and (c) decreases the induced stress to which the collapsible soil is subjected. All of the above improve the soil properties and its engineering performance.

In southern California (USA), collapsible soil deposits of 6 and up to 10 m deep are frequently removed and recompacted (Houston and Houston 1997). However, the compaction process is expected to be effective only up to about 5 m depth, with the greatest improvement in the upper 3 m (Rollins and Rogers 1994). If sufficiently large weights are dropped from sufficiently great heights, then the effectiveness can extend some what deeper (Pengelly *et al.*1997).Compaction can be achieved by use of *rollers*, *displacement piles*, *heavy tamping* (*dynamic compaction*), and *vibration* (*vibroflotation* or *deep blasting*). These methods of compaction are briefly discussed as follows.

(a) Rollers – rollers are used to compact soils with or without water (Jones and van Alphen 1980 and Cintra *et al*. 1986). In this method, the soil is removed to the required depth, stockpiled, and then compacted in place in layers.

(b) Displacement piles - compaction by displacement piles involves the driving of displacement piles (i.e. steel pile or precast concrete) and then withdrawing the piles and backfilling the holes with soil. Densification of the surrounding soils can occur during the driving and backfilling (Abelev 1975; and Bally and Culitza 1987).

(c) Heavy tamping (dynamic compaction) - compaction by heavy tamping has been used to densify the collapsible soils by dropping very heavy weights, up to 30 tons, from great heights, up to 40 meters, into the soil (Lutenegger 1986; Rollins and Kim 1994; and Pengelly, *et al*. 1997). The weight upon hitting the ground surface imparts its energy into the soil, creating a densifying effect immediately around and to a depth below the weight. It has also been used to treat subgrade profiles consisting of collapsible alluvial materials for highway projects in New Mexico (Lovelace *et al*. 1982). Bell and Bruyn (1997) indicated that if loess contains a relatively high carbonate content, it may be difficult to achieve the desired results with dynamic compaction.

(d) Vibration (vibroflotation or deep blasting) - Compaction also has been achieved by vibration either by *vibroflotation* or *deep blasting*. Vibroflotation involves jetting a vibrating probe into place and gravel or sand is dumped alongside the probe as it is withdrawn (Lovelace *et al*. 1982). Deep blasting is used to break down the structure of the soil, so it will densify thoroughly under its own weight (Minkov *et al*. 1981). In both vibrating techniques, soil has been wetted beforehand.

8.6 Chemical Stabilization or Grouting

Chemical stabilization by additives such as sodium silicate and calcium chloride has been tried for many years with various degrees of success. The method develops cementation within the soil structure and thus it resists collapse when wetted. Penetration of chemical solutions into the desired depth is essential for the success of the operation. The method is most applicable to fine sand deposits. The advantage of grouting is that it can be used after a structure is already in place. Houston and Houston (1997) pointed out that grouting provides soil improvement by one or more of the three following mechanisms:

(i) If the grout viscosity is low enough and the soil permeability is high enough, the grout simply permeats into the soil and greatly strengthens and stiffens it.

(ii) If the grout viscosity is high and the soil permeability is low, the grout bulb compresses and densifies the surrounding soil. This process is called *compaction grouting*.

(iii) The third mechanism can be called soil reinforcement. If enough grout is put into the ground at enough locations and depths, then the stiff grouted zones will tend to carry the overburden and structural loads while loose zones will be unloaded to some extent.

Silicates stabilization is generally costly. However, it has been used successfully in the United States and other countries (Pengelly *et al*. 1977). The injection of sodium silicate solution has been used extensively in the former Soviet Union and Bulgaria. Field and laboratory tests conducted in the former Soviet Union indicated that prewetting with a 2% sodium silicate solution can significantly decrease the compressibility and increase the strength of collapsible loessial soil deposits (Sokolovski and Semkin 1984). This method is used for both dry and wet collapsible soils that are expected to subside under the added weight of the structure to be built. This method consists of three steps:

(i) Injection of carbon dioxide for removal of any water present and preliminary activation of the soil.

(ii) Injection of sodium silicate grout.

(iii) Injection of carbon dioxide to neutralize the alkali.

Field tests on noncarbonate-type sandy soils pretreated with carbon dioxide have shown strength increase of 20-25% (Clemence and Finbarr 1981). The injection of ammonia alone on wet soils has been used. However, the effectiveness of ammonia is much less than that of sodium silicates. Moreover, ammonia is also hazardous to use. The use of cement to reduce the collapse potential was successfully attempted. Ismael *et al*. (1987) showed that the use of cement as an additive in small quantities of 5% resulted in a significant decrease in the collapse potential which did not exceed 0.5%. Lime and bitumen emulsions have been used to stabilize loess soils, particularly in relation to road construction (Bell 1993). Phosphoric acid has been used in New Zealand for stabilizing loess soil (Evans and Bell 1981).

8.7 Heat Treatment

Bell and Bruyn (1997) reported on the use of heat treatment of loess in south east Europe and Russia by burning gas and fuel oil in pressurized boreholes. The boreholes are closely spaced and temperatures are generated up to 1000 °C , producing a stabilized soil column with a diameter of 1.5 - 2 m.

8.8 Evaluation of Treatment Methods

Comparative studies on the effectiveness and economics of various treatment methods were reported in the literature. For example, Rollins and Rogers (1994) conducted a comparative study at a site located in Nephi, Utah (U.S.A.) to evaluate the cost and effectiveness of various treatment methods under field conditions using six full-scale tests on 1.5 m square footings. Treatment methods included: (1) prewetting with water; (2) prewetting with a 2% sodium silicate solution; and (3) partial excavation and replacement with compacted granular fill; (4) dynamic compaction on dry soil; and (5) dynamic compaction on prewet soil. Soil improvement was evaluated using double oedometer testing on undisturbed samples along with cone penetration tests and pressuremeter tests.

The soil profile was generally composed of clayey sandy silt (CL-ML). The natural water content was generally between 7% and 10% while the liquid limit was about 22% and the plasticity index was about 5%. The grain size distribution of the soil typically consisted of 30% sand, 60% silt and 10% clay. The settlement predicted by oedometer testing and the measured settlement for various methods is summarized in Table 5. It was found that the sodium silicate and dynamic compaction methods were the most effective methods in reducing the settlement of collapsible soils from more than 250 mm to less than 25 mm (Table 5). But they were more expensive than the other methods.

Table 5: Comparison of predicted and measured collapse settlement for various treatment methods (After Rollins and Rogers 1994).

Test cell	Treatment method	Predicted collapse settlement after loading		Measured settlement after loading	
		Before treatment (mm)	After treatment (mm)	Collapse (mm)	Creep (mm)
1	No treatment	267	N/A	282	12
2	Prewetting with water	270	300	243	12
3	Prewetting with sodium silicate	270	32	27	9
4	Partial excavation and replacement with fill	267	183	114	9
5	Dynamic compaction at natural moisture	254	31[a] 125[b]	3	14
6	Dynamic compaction after prewetting	396	15	11	18

[a] = below drop, [b] = between drop.

Rollins and Rogers (1994) also presented Table 6, which gives the advantages and limitations of various treatment methods. Similar results on the success of using compaction were reported by Souza *et al.* (1995) who showed that compaction can reduce collapse settlement of about 87% and increase allowable load of 110% based on their field plate load tests in Brazil. It should be noted that more than one method can be used in a particular situation such as prewetting and compaction.

9. Foundation Design

The choice of a particular foundation design depends primarily on the depth of the collapsible soil encountered, the magnitude of collapse, and the economics of the design method. In cases where the collapsible soil layer extends to a shallow depth (approximately 1.5 to 2 m), the use of continuous strip footings may provide a more economical and safer foundation than isolated footings. If the footing area becomes greater than 50% of the entire area of the building, then a mat foundation should be constructed for the entire foundation (Clemence and Finbarr 1981). The soil may be moistened and recompacted by heavy rollers before placing the foundations. This process is effective in eliminating settlements due to overburden pressure and additional loads.

Table 6. Comparison of advantages and limitations of various treatment methods (After Rollins and Rogers 1994).

Advantages	Limitations
(a) Prewetting with water (Test Cell 2)	
Low cost Ease of application	Excessive settlement without preloading overexcavation Failure to densify surface layers Differential settlement likely
(b) Prewetting with sodium silicate (Test Cell 3)	
Dramatic reduction in collapse settlement Development of permanent cementation Reduction in hydraulic conductivity Significant in creep settlement Potential for use as a remedial measure	Higher cost ($8-$12/cu yd) Limited experience base Treatment depth limited to less than 2 m
(c) Partial excavation and replacement with fill (Test Cell 4)	
Relatively low cost ($4-$8/cu yd) Ease of application Extensive contractor experience with the method Reduction of induced stress on collapsible soil Minimum settlement for small volumes of water Minimization of differential settlement	Treatment of surface zones only Excessive settlement following wetting of deep zones.
(d) Dynamic compaction at natural moisture (Test Cell 5)	
Dramatic reduction in collapse settlement Decrease in hydraulic conductivity Improvement to significant depths (>5m)	Higher cost ($8-$10/cu yd) Potential for damage due to vibrations Nonuniformity of treatment Less contractor experience with method
(e) Dynamic compaction after prewetting (Test Cell 6)	
Significant decrease in collapse settlement Increased compaction efficiency prior to liquefaction Reduction in level of vibrations Greater uniformity of densification Decrease in hydraulic conductivity Improvement to significant depths (>5m)	Higher cost ($9-$11/cu yd) Increase in creep (long-term) settlement Potential for liquifaction when water content is high Difficult to withdraw weight after drop Drying time following treatment may be excessive Less contractor experience with method Difficult to measure improvement

In cases where the soil layer susceptible to wetting extends to several meters (deeper than 3 to 5 m), deep foundations (piles and drilled piers) may be required to transmit foundation loads to a suitable firm strata below the collapsible soil layer. The design of piles and drilled piers must take into consideration the effect of negative skin friction resulting from the collapse of the soil structure and the associated settlement of the zone of subsequent wetting (Das 1995). Precollapsing techniques may be used to cause collapse before construction of foundations. Such techniques include vibroflotation and ponding (flooding). However, it should be noted that even with such precollapsing techniques additional settlement may occur due to the difficulty of saturating all the collapsing soil encountered.

10. Summary and Conclusions

The following points were drawn from this study

1. Collapsing soils undergo a reduction in volume due to wetting alone or wetting and loading acting together. Aeolian deposits are the most naturally occurring collapsing soil. Collapsing soils are generally characterized by their loose and open structure binded by cementing agents, which upon wetting, become weak and may dissolve causing collapse.
2. Collapse criteria based on index parameters such as dry density and degree of saturation were proposed in the literature for identification and classification purposes. Such criteria should be used only as indicators since reliance on them can be misleading.
3. Double oedometer test is widely used for estimating the collapse potential, which can be used to calculate soil collapse settlement. Plate load test is the most common field test for the evaluation of allowable pressure under foundations.
4. Several treatment methods were proposed for minimizing the collapse of soils such as soil replacement, prewetting, compaction control and chemical stabilization or grouting. The choice of such methods depends on the ground conditions, type of structure to be constructed, practicality and economics of the method. Compaction control was the most practical and effective treatment method.
5. Shallow foundations (continuous strip footings and mat foundation) are recommended when collapsing soil extends to shallow depth (1.5 to 2 m). In cases where collapsing soil extends to several meters (deeper than 3 to 5 m), deep foundations (piles and drilled piers) are recommended.

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