CHAPTER THREE

EXPANSIVE SOIL

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3.1 General

Sudan is one of the countries with a wide distribution of expansive soils. Over one-third of Sudan's, 1,886,068 kilometers square have expansive soils. Unfortunately, this area includes most of the nation's population centers and development projects. The frequent failure to recognize the potential problem has resulted in extensive damages. In some cases, the cost of repairs or replacement of damaged structures has exceeded the structure's initial value .Sudan has a tropical climate over most of the country. The climate ranges from a desert climate in the north having a short rainy season during the months of July-September, to an equatorial rainy zone in the southern border.

The term expansive soils applies to soils which have the tendency to swell when its moisture content is allowed to increase. The moisture may come from rain, flooding, leaking water or sewer lines, or from a reduction in surface evapotranspiration when an area is covered by a building or pavement.

Fig. 3.1 Distribution of expansive soils (Gazira state)

3.2 Literature review and Problem Discussion

Expansive soil is a clay of smactite type which exhibits significant swelling when absorbs water. The three most important groups of clay minerals are montmorillonite, illite, and kaolinite. Such clay minerals are formed through a complicated process from an assortment of parent materials. The magnitude of expansion depends upon the kind and amount of clay minerals present. In general, montmorillonite clays swell when the moisture content is increased, while swelling is absent or limited in illite and kaolinite(Nelson, J.D. and Miller, D.J. ,1992) . Potentially expansive soils can be recognized in the laboratory by their plastic properties. Clays of high plasticity, generally those with liquid limits exceeding 50 percent and plasticity index over 30, usually have high swelling potential. In the field, expansive clay soils can be easily recognized in the dry season by the deep cracks, in roughly polygonal patterns, in the ground surface (as shown in Figure 3:2).

Fig. 3.2 Expansive soil with polygonal patterns cracks

The problems associated with expansive soils are not widely appreciated by architects, engineers, contractors and others who have not worked in expansive soils areas. The term cracking soils is also used for these soils since they have the tendency to shrink and crack when the moisture is allowed to decrease. Soils containing the clay mineral montmorillonite (a smectite), such as the Clay Plain soils in Sudan, generally exhibit these properties.

Table 3 .1—Relation between Plasticity Index and Potential Swell

Plasticity index (1)	Potential swell (2)
0-15	low
15-35	medium
35-55	high
above 55	very high

Structures situated on these soils may be damaged if the structure cannot adequately resist the soil expansion or adequately accommodate differential vertical movement. Expansive soils also represent a problem in railway and road building because of its poor bearing capacity when wet. Several correlations have been found, which are useful in identifying potentially expansive soils. Visual indications include: Wide and deep shrinkage cracks occurring during dry periods; soil "rock-hard" when dry, but very sticky and soft when wet; and expansive soil damage to other structures in the area.

(Chen, 1979) Indicates that soils having a plasticity index in excess of 15 should be considered potentially expansive, and Table (3.1) lists the potential swell for various ranges of plasticity indexes. Sowers indicates that little swell will occur if the in situ soil moisture content exceeds the plastic limit. The initial dry density is also a well-recognized factor in determining the magnitude of potential swell. Climate is also pertinent since precipitation and evapotranspiration effect soil moisture and, thus, potential swell. For an undisturbed site where shallow water tables do not exist, a soil's moisture is controlled by precipitation and evapotranspiration. One means of classifying soil moisture is the Thorn thwaite Moisture Index (TMI), which is defined as the difference in mean annual precipitation and the potential evapotranspiration. A positive TMI indicates a net soil moisture deficit. Soils having a negative pore water pressure (soil suction) also indicate a net soil moisture deficit.

Potentially expansive soils having a soil moisture deficit are generally considered most prone to expansion. Snethen indicates that in situ soil suction (water tension) has a closer correlation to potential expansion than index properties alone because soil suction represents the potential for moisture uptake. An in situ soil suction in excess of 140 kN/m^2 should be considered as potentially expansive. As a general rule, wilting of vegetation indicates a soil suction of approximately 1,500 kN/m^2 (15 bars). If a soil has been determined to be potentially expansive, quantitative value s for swell and swell pressures are needed for foundation design along with information on field conditions. Table (3.2) indicates the potential for damage based on swell and swell pressure. For example, potential swell of 2% or more under expected surcharge conditions or potential swelling pressure above 50 kN/m^2 should be considered potentially damaging to lightweight structures and may require special foundation designs.

Table 3.2 —Relation between Volume Change and Swelling Pressure versus Potential Damage

Volume change percent(1)	Swelling pressure in kN/m^2 (2)	Potential damge (3)
0-1.5	0-50	low
1.5-5	50-250	medium
5-25	250-1,000	high
above 25	above 1,000	very high

3.3 Identification of Expansive Soils

Identification methods can be divided into two general groups; those used for mineralogical identification, and those used for direct physical properties. The following are listings of the methods used in each group.

3.3.1 Mineralogical Identification:

- Microscopic examination
- X-ray diffraction
- Differential thermal analysis
- Infrared analysis
- Dye adsorption analysis
- Chemical analysis

3.3.2 Physical properties:

- Free swell test
- Atterberg limits
- Colloid content determination
- Measurement of linear shrinkage
- Direct measurement of volume change by means of Mechanical apparatus.

Mineral identification methods are too time-consuming and demanding special skills and equipment. For this reason most laboratories prefer simple identification procedures based on physical properties of the soils (Ahmad Ardani, 1992).

3.4 Predicting potential volume change

Accurately predicting the potential volume change of expansive soils are requisite for the selection of treatment methods. It should be noted that there is no definite dividing line between some identification methods and methods used to predict the magnitude of volume change. In general the techniques that are used to predict volume change fall into three categories:

- Soil suction test
- Odometer swell test (consolidometer testing)
- Potential Vertical Rise (PVR)

Once an expansive soil has been identified and characterized using the above mentioned methods, measures must be taken to mitigate the anticipated volume change (Atkinson J.H, 2007).

3.5 Investigation and assessment

It is important to recognize the existence, and understand the potential problems, of expansive soils early on during site investigation and laboratory testing, to ensure that the correct design strategy is adopted before costly remedial measures are required. However, it is important that investigations determine the extent of the active zone.

Despite the proliferation of test methods for determining shrinkage or swelling properties, they are rarely employed in the course of routine site investigations in the UK. Further details of tests commonly employed around the world are given by Chen (1988) and Nelson and Miller 1992). This means that few data sets are available for data-basing the directly measured shrink–swell properties of the major clay formations, and reliance has to be placed on estimates based on index parameters, such as liquid limit, plasticity index, and density .Such empirical correlations may be based on a small data set, using a specific test method, and at only a small number of sites.

Variation of the test method would probably lead to errors in the correlation. The reason for the lack of direct shrink–swell test data is that few engineering applications have a perceived requirement for these data for design or construction.

3.6 Expansive Soil treatment methods

The following are description of treatment methods used by Colorado DOT and other transportation agencies in alleviating detrimental volume change of expansive soils (Ardani, A. and Laforce1986):

- Sub-excavation and removal of expansive soil and replacement with non-expansive soil.
- Application of heavy applied load to balance the swelling pressure.
- Preventing access of water to the soil by encapsulation.
- Stabilization by means of chemical admixtures Mechanical stabilization.
- Explosive treatment to correct swelling shale's.
- Pre-wetting the soil.
- Avoiding the expansive soil.

3.7 General design requirements (I.S.I)

CODE OF PRACTICE IS: 3370-part II, 1965)

3.7.1 Plain Concrete Structures: Plain concrete member of reinforced concrete liquid retaining structure may be designed against structural failure by allowing tension in plain concrete as per the permissible limits for tension in bending. This will automatically take care of failure due to cracking. However, nominal reinforcement shall be provided, for plain concrete structural members.

3.7.2. Permissible Stresses in Concrete.

(a) For resistance to cracking: For calculations relating to the resistance of members to cracking, the permissible stresses in tension (direct and due to bending) and shear shall confirm to the values specified in Table (3.3). The permissible tensile stresses due to bending apply to the face of the member in contact with the liquid. In members less than 225mm. thick and in contact with liquid on one side these permissible stresses in bending apply also to the face remote from the liquid.

(b) For strength calculations: In strength calculations the permissible concrete stresses shall be in accordance with Table (3.3). Where the calculated shear stress in concrete alone exceeds the permissible value, reinforcement acting in conjunction with diagonal compression in the concrete shall be provided to take the whole of the shear.

Table 3.3 Permissible concrete stresses in calculations relating to Resistance to cracking

Grade of Concrete	**Permissible Stresses**		**Shear stress N/mm^2**
	Direct Tension N/mm^2	Tension due to bending N/mm^2	
M15	1.1	1.5	1.5
M20	1.2	1.7	1.7
M25	1.3	1.8	1.9
M30	1.5	2.0	2.2
M35	1.6	2.2	2.7
M40	1.7	2.4	2.7

3.7.3 Permissible Stresses in Steel

The stress in steel must not be allowed to exceed the following values under different positions to prevent cracking of concrete.

When steel is placed near the face of the members in contact with liquid 115 N/mm^2 for MS bars and 150 N/mm^2 for HYSD bars.

When steel is placed on face away from liquid for members less than 22 mm in thickness same as earlier.

When steel is placed on the face away from the liquid for members 225 mm or more in thickness: 125 N/ mm^2 for M.S. bars and 190 N/ mm^2 for HYSD bars.

(a) For resistance to cracking.

When steel and concrete are assumed to act together for checking the tensile stress in concrete for avoidance of crack, the tensile stress in steel will be limited by the requirement that the permissible tensile stress in the concrete is not exceeded so the tensile stress in steel shall be equal to the product of modular ratio of steel and concrete, and the corresponding allowable tensile stress in concrete.

(b) For strength calculations.

In strength calculations the permissible stress shall be as follows:

(i) Tensile stress in member in direct tension 100 Mpa.

(ii) Tensile stress in member in bending on liquid retaining face of members or face away from liquid for members less than 225mm thick 100 Mpa

(iii) On face away from liquid for members 225mm or more in thickness 125 Mpa

(iv)Tensile stress in shear reinforcement ,For members less than 225mm thickness 100 N/mm^2 For members 225mm or more in thickness 125 Mpa

(v) Compressive stress in column subjected to direct load 125 Mpa

Table 3.4 Permissible stresses in steel reinforcement for strength calculations

Types stress in steel reinforcement	Permissible stresses in N\mm^2	
	Plain round mild steel bars conforming to grade 1 of IS:482(part1)-1966	High yield strength deformed bars (HYSD) conforming to IS:1789-1966(part1)-1966
Tensile stress in members under direct tension($\boldsymbol{\delta_x}$)	115	150
Tensile stress in members under direct bending (δ_y)	115	150
a. On liquid retaining face of members.	115	150
b. On face away from liquid for members less than 225mm.	115	150
c. On face away from liquid for members 225mm or more in thickness.	125	150
Tensile stress in shear reinforcement ($\boldsymbol{\sigma_{sv}}$)		
a. For members less than 225mm thickness.	115	150
b. For members less than 225mm or more in thickness	125	175
Compressive stress in columns subjected to direct load ($\boldsymbol{\sigma_m}$)	125	175

3.7.4. Stresses due to drying Shrinkage or Temperature Change:

(i)Stresses due to drying shrinkage or temperature change may be ignored provided that:

(a) The permissible stresses specified above in (ii) and (iii) are not otherwise exceeded.

(b) Adequate precautions are taken to avoid cracking of concrete during the construction period and until the reservoir is put into use.

(c) Recommendation regarding joints for suitable lidding layer beneath the reservoir are complied with, or the reservoir is to be used only for the storage of water or aqueous liquids at or near ambient temperature and the circumstances are such that the concrete will never dry out.

(ii)Shrinkage stresses may however be required to be calculated in special case, when a shrinkage co-efficient of 300×10^{-6} may be assumed.

(iii) When the shrinkage stresses are allowed, the permissible stresses, tensile, stresses to concrete (direct and bending) as given in Table (3.4) may be increased by 33.33 percent.

3.8 Water Quantity Estimation

The quantity of water required for municipal uses for which the water supply scheme has to be designed requires following data:

Water consumption rate (Per Capita Demand in liter's per day per head)

Population to be served.

Quantity= per demand x Population

Factors affecting per capita demand:

•**Size of the city**: Per capita demand for big cities is generally large as compared to that for smaller towns as big cities have sewered houses.

•Presence of industries.

•Climatic conditions.

•Habits of economic status.

•**Quality of water**: If water is aesthetically people and their medically safe, The consumption will increase as people will not resort to private wells, etc.

•**Pressure in the distribution system.**

•Efficiency of water works administration: Leaks in water mains and services, and unauthorized use of water can be kept to a minimum by surveys.

•Cost of water.

•Policy of metering and charging method: Water tax is charged in two different ways on the basis of meter reading and on the basis of certain fixed monthly rate.

3.8 Fluctuations in Rate of Demand:

Average Daily per Capita Demand

= Quantity Required in 12 Months/ (365 x Population)

If this average demand is supplied at all the times, it will not be sufficient to meet the fluctuations.

•Seasonal variation: The demand peaks during summer. Firebreak outs are Generally more in summer, increasing demand. So, there is seasonal variation.

•Daily variation depends on the activity. People draw out more water on Sunday sand Festival days, thus increasing demand on these days.

•**Hourly variations** are very important as they have a wide range. During active household working hours i.e. from six to ten in the morning and four to eight in the evening, the bulk of the daily requirement is taken. During other hours the requirement is negligible. Moreover, if a fire breaks out, a huge quantity of water is required to be supplied during short duration, necessitating the need for a maximum rate of hourly supply.

So, an adequate quantity of water must be available to meet the peak demand. To meet all the fluctuations, the supply pipes, service reservoirs and distribution pipes must be properly proportioned. The water is supplied by pumping directly and the pumps and distribution system must be designed to meet the peak demand. The effect of monthly variation influences the design of storage reservoirs and the hourly variations influences the design of pumps and service reservoirs. As the population decreases, the fluctuation rate increases.

Maximum daily demand = 1.8 x average daily demand

Maximum hourly demand of maximum day i.e. Peak demand

= 1.5 * average hourly demand

= 1.5 * Maximum daily demand/24

= 1.5 * (1.8 x average daily demand)/24

= 2.7 * average daily demand/24

= 2.7 * annual average hourly demand

3.9 Design requirement of concrete (I. S. I)

In water retaining structure a dense impermeable concrete is required Therefore, proportion of fine and course aggregates to cement should be such as to give high quality concrete. Concrete mix weaker than M20 is not used. The minimum quantity of cement in the concrete mix shall be not less than 30 kN/m^3.The design of the concrete mix shall be such that the resultant concrete issue efficiently impervious. Efficient compaction preferably by vibration is essential. The permeability of the thoroughly compacted concrete is dependent on water cement ratio. Increase in water cement ratio increases permeability, while concrete with low water cement ratio is difficult to compact. Other causes of leakage in concrete are defects such as segregation and honey combing.

All joints should be made water-tight as these are potential sources of leakage. Design of liquid retaining structure is different from ordinary R.C.C, structures as it requires that concrete should not crack and hence tensile stresses in concrete should be with in permissible limits. A reinforced concrete member of liquid retaining structure is designed on the usual principles ignoring tensile resistance of concrete in bending. Additionally it should be ensured that tensile stress on the liquid retaining ace of the equivalent concrete section does not exceed the permissible tensile strength of concrete as given in table (3.4) For calculation purposes the cover is also taken into concrete area.

Cracking may be cause due to restraint to shrinkage, expansion and contraction of concrete due to temperature or shrinkage and swelling due to moisture effects. Such restraint may be caused by:

(i) The interaction between reinforcement and concrete during shrinkage due to drying.

(ii) The boundary conditions.

(iii) The differential conditions prevailing through the large thickness of massive concrete use of small size bars placed properly, leads to closer cracks but of smaller width. The risk of cracking due to temperature and shrinkage effects may be minimized by limiting the changes in moisture content and temperature to which the structure as a whole is subjected. The risk of cracking can also be minimized by reducing the restraint on the free expansion of the structure with long walls or slab founded at or below ground level, restraint can be minimized by the provision of a sliding layer. This can be provided by founding the structure on a flat layer of concrete with interposition of some material to break the bond and facilitate movement. In case length of structure is large it should be subdivided into suitable lengths separated by movement joints, especially where sections are changed the movement joints should be provided .Where structures have to store hot liquids, stresses caused by difference in temperature between inside and outside of the reservoir should be taken into account.

The coefficient of expansion due to temperature change is taken as $11*10^{-6}$ /C^{o} and coefficient of shrinkage may be taken as $450*10^{-6}$ for initial shrinkage and $200*10^{-6}$ for drying shrinkage.

3.10 Minimum reinforcement

(a) The minimum reinforcement in walls, floors and roofs in each of two directions at right angles shall have an area of 0.3 per cent of the concrete section in that direction for sections up to 100mm, thickness. For sections of thickness greater than 100mm, and less than 450mm the minimum reinforcement in each of the two directions shall be linearly reduced from 0.3 percent for 100mm thick section to 0.2 percent for 450mm, thick sections. For sections of thickness greater than 450mm, minimum reinforcement in each of the two directions shall be kept at 0.2 per cent. In concrete sections of thickness 225mm or greater, two layers of reinforcement steel shall be placed one near each face of the section to make up the minimum reinforcement.

(b) In special circumstances floor slabs may be constructed with percentage of reinforcement less than specified above. In no case the percentage of reinforcement in any member be less than 0.15% of gross sectional area of the member.

3.11 Minimum Cover to Reinforcement.

(a)For liquid faces of parts of members either in contact with the liquid (such as inner faces or roof slab) the minimum cover to all reinforcement should be 25mm or the diameter of the main bar whichever is greater. In the presence of the sea water and soils and water of corrosive characters the cover should be increased by 12mm but this additional cover shall not be taken in to account for design calculations.

(b)For faces away from liquid and for parts of the structure neither in contact with the liquid on any face, nor enclosing the space above the liquid, the cover shall be as ordinary concrete member.

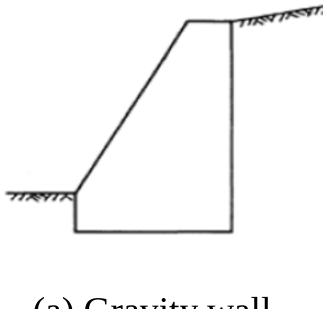
3.12 Types of retaining wall

Retaining walls are usually built to hold back soil mass to retail soil which is unable to stand vertically by themselves. However, retaining walls can also be constructed for aesthetic landscaping purposes. They are also provided to maintain the grounds at two different levels. Retaining walls shall be designed to withstand lateral earth and water pressures, the effects of surcharge loads, the self-weight of the wall.

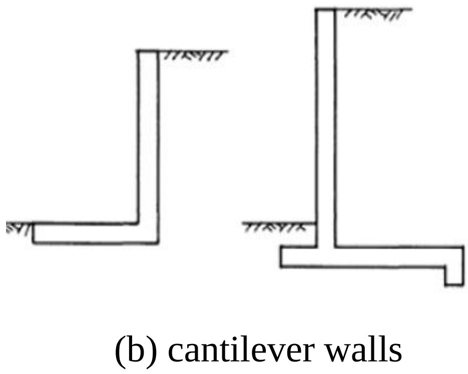
Retaining walls are structures used to retain earth which would not be able to stand vertically unsupported. The wall is subjected to overturning due to pressure of the retained material.

The types of retaining wall are as follows:

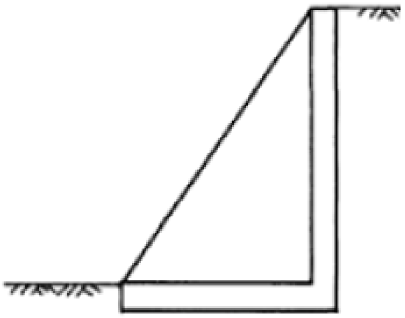
1. In a **gravity wall** stability is provided by the weight of concrete in the wall.
2. In a **cantilever wall** the wall slab acts as a vertical cantilever. Stability is provided by the weight of structure and earth on an inner base or the weight of the structure only when the base is constructed externally; Cantilever retaining walls are constructed of reinforced concrete. They consist of a relatively thin stem and a base slab. The base is also divided into two parts, the heel and toe. The heel is the part of the base under the backfill. The toe is the other part of the base.
3. In **counterfort and buttress walls** the slab is supported on three sides by the base and counterforts or buttresses. Stability is provided by the weight of the structure in the case of the buttress wall and by the weight of the structure and earth on the base in the counterfort wall. Examples of retaining walls are shown in Fig. 4.1. Designs are given for cantilever and counterfort retaining walls.



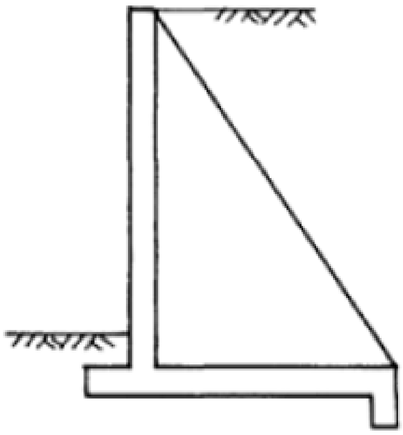
(a) Gravity wall



(b) cantilever walls



(c) Buttress wall



(d) counterfort wall

Fig 3.3 Types of retaining wall