

A review of the engineering behaviour of soils and rocks with respect to groundwater

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ABSTRACT: The effect of groundwater on the engineering behaviour of soils and rocks is of fundamental importance, indeed to paraphrase Terzaghi — without water there would be no soil mechanics. This paper reviews these effects in terms of the variation in the properties and behaviour of soils and rocks brought about by changes in moisture content, and associated changes, notably in effective stress, and by the dissolution of parts of the rock or soil mass.

Introduction

The principal source of groundwater is meteoric water, that is, precipitation (rain, sleet, snow and hail). Part of this water enters the ground by infiltration whilst the remainder is lost either as runoff on the surface or by evaporation and transpiration to the atmosphere. The main conditions which influence the relationship between infiltration, runoff, evaporation and transpiration are climate, topography, vegetation and geology.

Two other sources of water which are occasionally of some consequence are juvenile water which is derived from magmatic sources and connate water which represents the water trapped in the pore spaces of sedimentary rocks as they were deposited.

The retention of water in a soil depends upon the capillary force and the molecular attraction of the particles. As the pores in a soil become thoroughly wetted the capillary force declines so that gravity becomes more effective. In this way downward percolation can continue after infiltration has ceased but as the soil dries, so capillarity increases in importance. No further percolation occurs after the capillary and gravity forces are balanced. Thus, water percolates into the zone of saturation when the retention capacity is satisfied.

Many engineering properties of soils and rocks are very much influenced by their water content. But the water content in the ground can change above the water table and the water table itself can fluctuate in position. Such changes may be relatively rapid and are primarily influenced by the weather and climatic conditions as well as the retentivity of the ground. Accordingly, the effect of groundwater on the behaviour of the ground can change during the period of construction of a structure. Indeed changes can be brought about as a consequence of construction as, for example, in dewatering operations. It is therefore important prior to construction, to investigate what effects changes in the groundwater regime will have

on the engineering performance of soil and rock masses. If adverse effects such as excessive ground movements due to swelling, shrinkage or settlement, are indicated as likely to occur, then precautionary measures should be taken.

Intrinsic properties

Porosity and permeability are the two most important factors governing the accumulation, migration and distribution of groundwater. However, both may change within a rock or soil mass in the course of its geological evolution. Furthermore, it is not uncommon to find changes in both porosity and permeability with depth due to variation in a number of features, including pore size distribution. The actual size of pores is significant since in narrow pores or capillaries, surface tension forces exert a control over the movement of fluids. In addition, chemical interaction may occur between, on the one hand water and dissolved gases, and on the other, certain rock or soil constituents, particularly clay — and soluble — minerals.

Porosity

Apart from particle size distribution, porosity is also affected by sorting, grain shape, fabric, degree of compaction and cementation, and mineralogical composition.

The highest porosity is commonly attained when all the grains are the same size. The addition of grains of different size to such an assemblage lowers its porosity and this is, within certain limits, directly proportional to the amount added. Irregularities in grain shape result in a larger possible range of porosity, as irregular forms may theoretically be packed either more tightly or loosely than spheres. Similarly angular grains may either cause an increase or a decrease in porosity.

After a sediment has been buried and indurated, several additional factors help determine its porosity. The chief amongst these are closer spacing of grains, deformation and crushing of grains, recrystallisation, secondary growth of minerals, cementation and, in some cases, dissolution. Hence diagenetic change undergone by a rock may either increase or decrease its original porosity.

The porosity of a deposit does not necessarily provide an indication of the amount of water that can be obtained therefrom. Even though a rock or soil may be saturated, only a certain proportion of water can be removed by drainage under gravity or by pumping, the remainder being held in place by capillary or molecular forces.

The ratio of the volume of water retained to that of the total volume expressed as a percentage, is referred to as the specific retention. The amount of water retained varies directly in accordance with the surface area of the pores and indirectly with regard to the pore space. The relationship between porosity and specific yield on the one hand and particle size distribution on the other, is shown in Fig. 1. In soils the specific yield tends to decrease as the coefficient of uniformity increases. Examples of the specific yield of some common types of soil and rock are given in Table 1 (it must be appreciated that individual values of specific yield can vary considerably from those quoted).

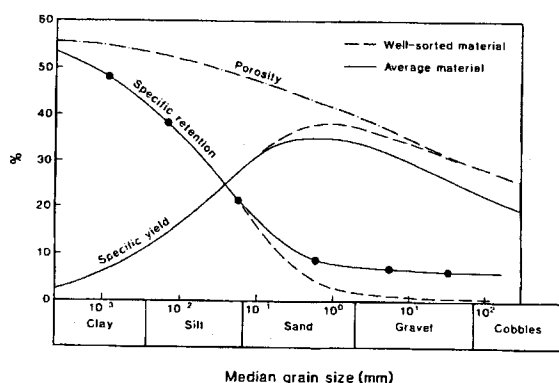


FIG. 1. Relationship between grain size, porosity, specific retention and specific yield (from Bear, J. 1979. *Hydraulics of Groundwater*, reproduced by permission of McGraw Hill, New York).

Permeability

In ordinary hydraulic usage a substance is termed permeable when it permits the passage of a measurable quantity of fluid in a finite period of time and impermeable when the rate at which it transmits that fluid is slow enough to be negligible under existing temperature-pressure conditions (Table 2).

TABLE 1. Some examples of specific yield.

Material	Specific yield (%)
Gravel	15 -30
Sand	10 -30
Dune sand	25 -35
Sand and gravel	15 -25
Loess	15 -20
Silt	5 -10
Clay	1 -5
Till (silty)	4 -7
Till (sandy)	12 -18
Sandstone	5 -25
Limestone	0.5 -10
Shale	0.5 -5

The flow through a unit cross section of material is modified by temperature, hydraulic gradient and the permeability. The latter is affected by the uniformity and range of grain size, shape of the grains, stratification, the amount of consolidation and cementation undergone and the presence and nature of discontinuities. Temperature changes affect the flow rate of a fluid by changing its viscosity. The rate of flow is commonly assumed to be directly proportional to the hydraulic gradient but this is not always so in practice.

The permeability of a clastic material is also affected by the interconnections between the pore spaces. If these are highly tortuous then the permeability is accordingly reduced. Consequently tortuosity figures importantly in permeability, influencing the extent and rate of free water saturation. It can be defined as the ratio of the total path covered by a current flowing in the pore channels between two given points to the straight line distance between them.

Stratification in a formation varies within limits both vertically and horizontally. It is frequently difficult to predict what effect stratification has on the permeability of the beds. Nevertheless, in the great majority of cases where a directional difference in permeability exists, the greater permeability is parallel to the bedding. For example, the Permo-Triassic sandstones of the Mersey and Weaver Basins are notably anisotropic as far as permeability is concerned, the flow parallel to the bedding being higher than across it. Ratios of 5:1 are not uncommon and occasionally values of 100:1 have been recorded where fine marl partings occur.

The permeability of intact rock (primary permeability) is usually several orders less than the *in situ* permeability (secondary permeability). Although the secondary permeability is affected by the frequency, continuity and openness, and amount of infilling, of discontinuities, a rough estimate of the permeability can be obtained from their frequency (Table 3). Admittedly such estimates must be treated with caution and cannot be applied to rocks which are susceptible to solution.

TABLE 2. *Relative values of permeabilities.*

Rock types	Porosity		Permeability range (m/s)						Well yields			Type of water-bearing unit
	Primary (grain)	Secondary (fracture) ^a	10 ⁰	10 ⁻²	10 ⁻⁴	10 ⁻⁶	10 ⁻⁸	10 ⁻¹⁰	High	Medium	Low	
			Very high	High	Medium	Low	Very low	Impermeable				
<i>Sediments (unconsolidated)</i>												
Gravel	30-40		—————						—————			Aquifer
Coarse sand	30-40		—————						—————			Aquifer
Medium to fine sand	25-35		—————						—————			Aquifer
Silt	40-50	Occasional	—————								—————	Aquiclude
Clay, till	45-55	Often fissured	—————								—————	Aquiclude
<i>Sediments (consolidated)</i>												
Limestones, dolostone	1-50	Solution joints, bedding planes	—————						—————			Aquifer or aquifuge
Coarse, medium sandstone	<20	Joints and bedding planes	—————						—————			Aquifer or aquiclude
Fine sandstone	<10	Joints and bedding planes	—————								—————	Aquifer or aquifuge
Shale, siltstone		Joints and bedding planes	—————								—————	Aquifuge or aquifer
Igneous rocks, Volcanic rocks e.g. basalt		Joints and bedding planes	—————						—————			Aquifer or aquifuge
Plutonic and metamorphic rocks		Weathering and joints decreasing as depth increases	—————								—————	Aquifuge or aquifer

^a Rarely exceeds 10%TABLE 3. *Estimation of secondary permeability from discontinuity frequency.*

Rock mass description	Interval (m)	Permeability	
		Term	k (m/s)
Very closely to extremely closely spaced discontinuities	Less than 0.2	Highly permeable	10 ⁻² -1
Closely to moderately widely spaced discontinuities	0.2-0.6	Moderately permeable	10 ⁻⁵ -10 ⁻²
Widely to very widely spaced discontinuities	0.6-2.0	Slightly permeable	10 ⁻⁹ -10 ⁻⁵
No discontinuities	Over 2.0	Effectively impermeable	Less than 10 ⁻⁹

Water possesses three forms of energy, namely, potential energy attributable to its height, pressure energy owing to its pressure, and kinetic energy due to its velocity. The latter can usually be discounted in any assessment of flow through the soils. Energy in water is usually expressed in terms of head. The head possessed by water in soils or rocks is manifested by

the height to which water will rise in a standpipe above a given datum. This height is usually referred to as the piezometric level and provides a measure of the total energy of the water. If at two different points within a continuous area of water there are different amounts of energy, then there will be a flow towards the point of lesser energy and the difference

in head is expended in maintaining that flow. Other things being equal, the velocity of flow between two points is directly proportion to the difference in head between them.

The basic law concerned with flow is that enunciated by Darcy (1856) which states that the rate of flow per unit area is proportional to the gradient of the potential head. Darcy's law is valid as long as a laminar flow exists. Departures from Darcy's law, therefore, occur when the flow is turbulent, such conditions arise when the velocity of flow is high. Turbulent flow may occur in very permeable media, in which the Reynolds number attains values above four. Accordingly it is usually accepted that this law can be applied to those soils which have grain sizes finer than gravels. Furthermore, Darcy's law probably does not accurately represent the flow of water through a porous medium of extremely low permeability, because of the influence of surface and ionic phenomena and the presence of gases.

Apart from an increase in the mean velocity, the other factors which cause deviations from the linear laws of flow include, firstly, the non-uniformity of pore spaces, since differing porosity gives rise to differences in the seepage rates through pore channels. A second factor is an absence of a running-in section where the velocity profile can establish a steady state parabolic distribution. Lastly, such deviations may be developed by perturbations due to jet separation from wall irregularities.

Permeability also depends upon the density, and dynamic viscosity of the fluid, involved. In fact, permeability is directly proportional to the unit weight of the fluid concerned and is inversely proportional to its viscosity. The latter is very much influenced by temperature.

Ionic exchange on clay and colloid surfaces may bring about changes in mineral volume which, in turn, affect the shape and size of the pores. Moderate to high groundwater velocities are required to move colloids and clay particles. Solution and deposition may result from the pore fluids. Small changes in temperature and/or pressure may cause gas to come out of solution which may block pore spaces.

Capillarity and soil suction

Capillary movement in soil refers to the movement of moisture through the minute pores between the soil particles which act as capillaries. It takes place as a consequence of surface tension therefore moisture can rise from the water table. This movement, however, can occur in any direction, not just vertically upwards. It occurs whenever evaporation takes place from the surface of the soil, thus exerting a 'surface tension pull' on the moisture, the forces of surface tension increasing as evaporation proceeds. Accordingly capillary moisture is in hydraulic continuity with the water table and is raised against the force of gravity, the degree of saturation

decreasing from the water table upwards. Equilibrium is attained when the forces of gravity and surface tension are balanced.

The boundary separating capillary moisture from the gravitational water in the zone of saturation is, as would be expected, ill-defined and cannot be determined accurately. That zone immediately above the water table which is saturated with capillary moisture is referred to as the closed capillary fringe, whilst above this, air and capillary moisture exist together in the pores of the open capillary fringe. The depth of the capillary fringe is largely dependent upon the particle size distribution and density of the soil mass, which in turn influence pore size. In other words the smaller the pore size, the greater is the depth. For example, capillary moisture can rise to great heights in clay soils (Table 4) but the movement is very slow. In soils which are poorly graded the height of the capillary fringe generally varies whereas in uniformly textured soils it attains roughly the same height. Where the water table is at shallow depth and the maximum capillary rise is large, moisture is continually attracted from the water table, due to evaporation from the ground surface, so that the uppermost soil is near saturation. For instance, under normal conditions peat deposits may be assumed to be within the zone of capillary saturation. This means that the height to which the water can rise in peat by capillary action is greater than the depth below ground to which the water table can be reduced by drainage. The coarse fibrous type of peat, containing appreciable sphagnum, may be an exception.

TABLE 4. *Capillary rises and pressures in soils.*

Soil	Capillary rise (mm)	Capillary pressure (kPa)
Fine gravel	up to 100	up to 1.0
Coarse gravel	100– 500	1.0– 1.5
Medium sand	150– 300	1.5– 3.0
Fine sand	1000– 10000	10.0– 100.0
Clay	over 10000	over 100.0

Drainage of capillary moisture cannot be effected by the installation of a drainage system within the capillary fringe as only that moisture in excess of that retained by surface tension can be removed, but it can be lowered by lowering the water table. The capillary ascent, however, can be interrupted by the installation of impermeable membranes or layers of coarse aggregate.

At each point where moisture menisci are in contact with soil particles the forces of surface tension are responsible for the development of capillary or suction pressure (Table 4). The air and water interfaces move into the smaller pores. In so doing the radii of curvature of the interfaces decrease and the soil suction increases. Hence the drier the soil, the higher is the soil suction.

Soil suction is a negative pressure and indicates the height to which a column of water could rise due to such suction. Since this height or pressure may be very large, a logarithmic scale has been adopted to express the relationship between soil suction and moisture content, the latter is referred to as the pF value (Table 5).

Soil suction tends to force soil particles together and these compressive stresses contribute towards the strength and stability of the soil. There is a particular suction pressure for a particular moisture content in a given soil, the magnitude of which is governed by whether it is becoming wetter or drier. In fact as clay soil dries out the soil suction may increase to the order of several thousands of kilopascals. However, the strength of a soil attributable to soil suction is only temporary and is destroyed upon saturation. At that point soil suction is zero.

TABLE 5. *Values of soil suction.*

pF Value	Equivalent suction	
	(mm water)	(kPa)
0	10	0.1
1	100	1.0
2	1000	10.0
3	10000	100.0
4	100000	1000.0
5	1000000	10000.0

Moisture content in clay soils

The engineering performance of clay deposits is very much affected by the total moisture content and by the energy with which this moisture is held. For instance, the moisture content influences their density, consistency and strength. The energy with which moisture is held influences their volume change characteristics since swelling, shrinkage and consolidation are affected by permeability and moisture migration. Furthermore moisture migration may give rise to differential movement in clay soils. The gradients which generate moisture migration in clays may arise from variations in temperature, extent of saturation, and chemical composition or concentration of pore solutions.

There tends to be a general reduction in porosity, and therefore also a reduction in moisture content, with increasing age and previous overburden. Mudrocks in Britain show a range of moisture content varying from about 3 to 30% (see Cripps & Taylor 1981) depending on these factors and also present overburden and weathering state.

The Atterberg or consistency limits of cohesive soils are founded on the concept that they can exist in any of four states depending on their water content. These limits are also influenced by the amount and character of the clay mineral content. In other words a cohesive soil is solid when dry but as water is

added, it first turns to a semi-solid, then to a plastic, and finally to a liquid state. The water content at the boundaries between these states are referred to as the shrinkage limit, the plastic limit and the liquid limit respectively.

Schofield & Wroth (1968) indicated that the liquid and plastic limits for saturated remoulded soils represent critical states such that the plasticity index corresponds to an arbitrary increase in strength. Hence these limits can be regarded as being defined by water contents associated with particular strengths. Indeed the plasticity index has been redefined by Wroth & Wood (1978) as the change in water content giving rise to a one hundred-fold change in the strength of the soil.

Fully saturated clay soils often behave as incompressible materials when subjected to rapid loading. The amount of elasticity increases continuously as the water content is decreased. Elastic recovery may be immediate or may take place slowly. The linear relationship between stress and strain only applies to clays at low stresses.

Mineral hydration

Mineral hydration causes modifications to the engineering behaviour of soils and rocks by virtue of changes in volume or density, the interaction between mineral grains, and the physical properties of the materials involved.

Grim (1962) distinguished two hydration processes in clay soils, namely, intercrystalline and intracrystalline swelling. Intercrystalline swelling takes place when the uptake of moisture is restricted to the external crystal surfaces and void spaces between crystals. Such swelling may occur in all materials but it is most significant in fine grained ones, particularly clays. In relatively dry clays the particles are held together by relict water under tension from capillary forces. On wetting these forces are relaxed and the material expands. Such swelling occurs in any type of clay, irrespective of mineralogical composition, although the amount of swelling depends on a number of factors including mineral species and the type and concentration of cations present in the porewater. Intracrystalline swelling, on the other hand, is characteristic of the smectite family of clay minerals, in particular of montmorillonite. Vermiculite and some varieties of chlorite also display intracrystalline swelling behaviour. In swelling minerals the individual molecular layers of the mineral are weakly bonded so that on wetting water enters not only between the crystals, but also between the unit layers which comprise the crystals. Here the magnitude of swelling is a function of clay mineral type, the type of interlayer cations present in the mineral (see Taylor & Cripps 1984). Swelling in Na montmorillonite is the most notable and can amount to 800 to 1000 times the original volume, the clay then having formed a gel of dissociated platelets with dimensions similar to those of the unit cell (10Å).

Certain other minerals are susceptible to significant chemical hydration. Of particular importance in engineering geology are the hydration of anhydrite to gypsum and of oxides of calcium or magnesium present in some slags. These reactions and also dolomitisation are usually associated with volume increase. Volume decrease in consequence of the dehydration of gypsum may also occur in hot arid environments (see Hardie, 1967).

The hydration of anhydrite produces a theoretic swelling of up to about 63% which exerts pressures that have been variously estimated between 2 and 69 MPa. In fact Winkler & Wilhelm (1970) showed that the hydration pressure can be calculated from a consideration of the vapour pressures of the hydrated salt and atmospheric water.

When hydration occurs at shallow depths some of the gypsum formed may be removed in solution, leaving no net change in volume (see Holliday 1970). At greater depths anhydrite is effectively confined which results in a gradual build-up of pressure that may be liberated in a sudden or explosive ground movement. According to Brune (1965), in the United States such uplifts have occurred beneath reservoirs, these bodies of water providing a constant supply of hydration water. Hydration has been observed at depths as great as 152 m with an effective overburden pressure of about 2 MPa, although Brune (1965) considers that a pressure of 14 MPa would have been required to uplift and deform the overlying rockmass.

Swelling pressures attributable to the hydration of anhydrite encountered during the construction of tunnels in central Europe are similar to those given above. For example, Yuzer (1982) quoted values between 2 and 12 MPa. However, where swelling clays are associated with anhydrite it can be difficult to distinguish the cause of swelling. Serrano *et al.* (1981) attributed rapid swelling (days) to clay mineral effects and slower swelling (weeks) to the conversion of anhydrite in a gypsiferous marl formation in northern Spain.

Mineral solution and weathering

Surface and near surface environments can represent conditions in which certain rock and soil forming minerals are susceptible to chemical change within the context of weathering. The most important processes, namely, solution, oxidation and hydrolysis, may be controlled by the movement and composition of groundwater since it will act as a medium of transfer, both into and out of the reaction site, of active components and products respectively. Aspects of solutioning with respect to soluble rocks are dealt with on page 18 of this paper, but it is also pertinent that the removal of individual grains in heterogeneous materials will lead to reductions in density and strength, together with increases in porosity and permeability. Commonly occurring minerals most at

risk from solution effects include halite, carnalite and other evaporitic minerals including gypsum, anhydrite and to some extent calcite.

In dry air rocks decay very slowly. The presence of moisture hastens the rate tremendously, firstly, because water is itself an effective agent of weathering and, secondly, because it holds in solution substances which react with the component minerals of the rock. The most important of these substances are free oxygen, carbon dioxide, organic acids and acids of nitrogen.

Free oxygen is an important agent in the decay of all rocks which contain oxidisable substances, iron and sulphur being especially suspect. The rate of oxidation is quickened by the presence of water; indeed it may enter into the reaction itself, as for example, in the formation of hydrates. However, its role is chiefly that of a catalyst.

Carbonic acid is produced when carbon dioxide is dissolved in water and it may possess a pH value of about 5.7. The principal source of carbon dioxide is not the atmosphere, but the air contained in the pore spaces in soil where its content may be a hundred or so times greater than it is in the atmosphere. An abnormal concentration of carbon dioxide is released when organic material decays. Furthermore humic acids are formed by the decay of humus in soil waters; they ordinarily have pH values between 4.5 and 5.0, but occasionally they may be under 4.0. The nitrogen acids $\text{HNO} = \text{HNO}_3$ and HNO_2 , are formed by organic decay or bacterial action in soils. They play only a minor part in weathering. In volcanic regions and in the oxidised zones of sulphide deposits, the sulphur acids H_2SO_3 and H_2SO_4 , become important and in some localities their pH value may be lowered below 1.0.

Because of the hydrolysis of the dissolved metal ion, solutions forming from the oxidation of sulphides are acidic. For instance, when pyrite is initially oxidised, ferrous sulphate and sulphuric acid are formed. Further oxidation, assisted by autotrophic bacteria, leads to the formation of ferric sulphate. Very insoluble ferric oxide or hydrated oxide is formed if highly acidic conditions are produced.

Experience with Canadian Ordovician black shales (see Gillot *et al.* 1974) and British examples (see Nixon, 1978) indicate that pyritiferous mudrocks can be subject to appreciable ground movements due to pyrite oxidation. Reaction of the sulphuric acid liberated during oxidation with soil and rock components or construction materials results in increased porosity and furthermore the products may be removed and precipitated elsewhere. These processes can also make a significant contribution to rock or soil degradation due to the dissolution of cement, creation of void space and crystallisation pressures that they entail.

Weathering of silicate minerals, including feldspar, augite, hornblende, mica and olivine, is primarily a process of hydrolysis. Much of the silica which is released by weathering forms silicic acid but where it

is liberated in large quantities some of it may form colloidal or amorphous silica. Mafic silicates usually decay more rapidly than felsic silicates and in the process they release magnesium, iron and lesser amounts of calcium and alkalis. The process whereby feldspars are decomposed to form clay minerals is effected by the hydrolysing action of weakly carbonated water. The alkalis are removed in solution as carbonates and bicarbonates; some silica is hydrolysed to form silicic acid.

Clays are hydrated aluminium silicates and when they are subjected to severe chemical weathering in humid tropical regimes they break down to form laterite. The process involves the removal of siliceous material and this is again brought about by the action of carbonated waters. Intensive leaching of soluble mineral matter from surface rocks takes place during the wet season. During the subsequent dry season groundwater is drawn to the surface by capillary action and minerals are precipitated there as the water evaporates. The minerals generally consist of hydrated peroxides or iron, sometimes of aluminium, and very occasionally of manganese. The precipitation of these insoluble hydroxides gives rise to an impermeable lateritic soil which itself inhibits leaching to the extent ultimately that the formation of laterite ceases. As a consequence lateritic deposits are usually less than 7 m thick.

Effects of changes in moisture content

The amount of moisture contained within a soil or rock is a function of its porosity, the availability of water and the mineralogy of the material. The mechanical behaviour of some soils and rocks is very sensitive to pore space moisture such that significant changes of volume, porosity, strength and other parameters accompany wetting and drying. Moisture content changes occur for a variety of reasons; some of which are a consequence of deliberate drainage or recharge of groundwater, whilst others are either accidental or peripheral to other activities. Leakage from water supply or drainage systems and bodies of water is liable to result in an unintentional rise in moisture content. On the other hand, some engineering works have important side-effects with respect to groundwater regimes. The construction of dams and reservoirs usually causes a general rise in water levels, whereas tunnelling or mining operations may have the opposite effect. Significant changes in moisture content frequently relate to seasonal or longer term climatic variations. The nature of the actual ground surface also exerts a control by virtue of its influence over evaporation and transpiration of groundwater and the amount of run-off and infiltration which occurs during precipitation. Although the presence of vegetation generally increases infiltration at the expense of direct run-off; plants, particularly trees, with a high water demand

cause a decrease in soil or rock moisture. A reduction may also result due to the effects of inadequately insulated hot process foundations or conduits.

Swelling and shrinkage of clay soils and rocks

One of the most notable characteristics of clays from an engineering point of view is their susceptibility to slow volume changes which can occur independently of loading. Differences in the period and magnitude of precipitation and evapo-transpiration, poor surface drainage, leakage and other causes of moisture content change effect swelling and shrinkage behaviour. The density of a clay soil also influences the amount of swelling likely to occur. Expansive clay minerals adsorb water into their lattice structure, tending to expand into adjacent zones of looser soil before volume increase of the soil mass takes place. Once any void space has been filled and in densely packed soils, the soil mass has to swell to accommodate the volume change of the expansive clay particles.

Holtz & Gibbs (1952) showed that expansive clays can be recognised by their plasticity characteristics. Subsequently in a review of the various methods which have been used to determine the amount of swelling that an expansive clay is likely to undergo, O'Neill and Poormoayed (1980) supported the use of the United States Army Engineers Waterways Experimental Station (USAEWES) classification of potential swell. It can be seen from Table 6 that this also makes use of the plasticity characteristics of clay soils, as well as initial soil suction. Van der Merwe (1964) used the activity of a clay as a means of predicting its swell potential (see Fig. 2). As far as the maximum movement due to swelling beneath a building is concerned, Brackley (1980) suggested that this could be obtained from:

$$\text{Swell (\%)} = \frac{(PI - 10)}{10} \log_{10} S/Q_{of}$$

where PI is the plasticity index, S is the soil suction at the time of construction (in kPa) and Q_{of} is the overburden pressure plus foundation pressure (kPa) acting on each layer of soil.

In the initial and residual stages of shrinkage the reduction in soil volume is less than the volume of water lost. Only during the normal shrinkage stage are the two equal. This can give rise to differential movement. Williams & Jennings (1977) found that soil structure has a major influence on the pattern of the shrinkage process, as well as on the total amount of shrinkage. They indicated that in clay soils with the same initial moisture content and density, the more random the particle arrangement, the less the total shrinkage.

Swelling of rocks is associated with weathering: shales, mudstones and marls being particularly prone although small amounts of swelling have been recorded in some sandstones. The clay mineral content plays a significant role in these rocks. For example, kaolinite is not expansive whilst montmorillonite is, Na montmorillonite being able to expand to many times its original volume. Since swelling is principally due to the ingress of water, the rock must be porous or fractured. If a rock has an intact unconfined compressive strength exceeding 40 MPa, it is not subject to swelling.

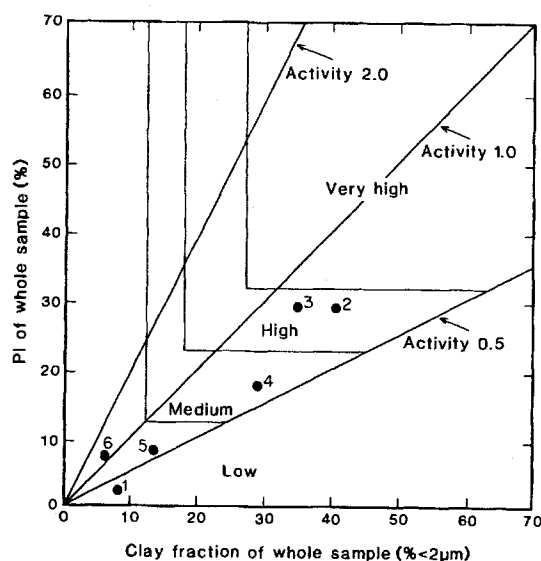


FIG. 2. Determination of potential expansiveness of soils (after Van der Merwe, 1965).

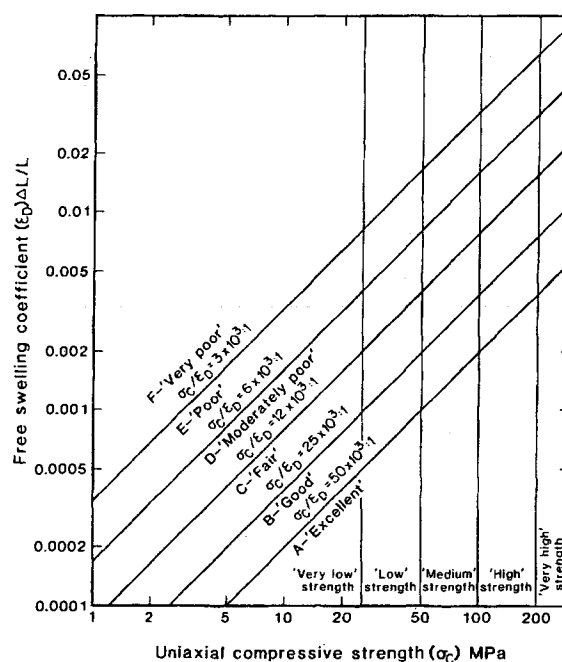


FIG. 3. Geodurability classification of the intact rock material (after Olivier, 1979).

- Note:
1. ϵ_D determined from oven-dried (105°C) to 24-hour saturation condition.
 2. ϵ_D plotted as the range and mean of the test results.
 3. Strength ratings according to Deere and Miller (1966), as modified by Bieniawski (1973).

TABLE 6. USAEWES classification of swell potential (from O'Neill and Poormoayed, 1980; reproduced by permission of American Society of Civil Engineers).

Liquid limit (%)	Plastic limit (%)	Initial <i>in situ</i> suction (kPa)	Potential swell (%)	Classification
Less than 50	Less than 25	Less than 145	Less than 0.5	Low
50 to 60	25 to 35	145 to 385	0.5 to 1.5	Marginal
Over 60	Over 35	Over 385	Over 1.5	High

Failure of consolidated and poorly cemented rocks occurs during saturation when the swelling pressure (or internal saturation swelling stress, σ_s) developed by capillary suction pressures exceeds their tensile strength. An estimate of σ_s can be obtained from the modulus of deformation (E):

$$E = \sigma_s / \epsilon_D$$

where ϵ_D is the free-swelling coefficient (i.e. the ratio

of the change in length after swelling to initial length). Olivier (1979) proposed the geodurability classification (Fig. 3) which is based on the free-swelling coefficient and uniaxial compressive strength. This classification was developed primarily to assess the durability of mudrocks and poorly cemented sandstones during tunnelling operations, since the tendency of such rocks to deteriorate after exposure governs the stand-up time in tunnels.

Desiccation of clay soils

The volume change brought about by evaporation from a clay soil can be conservatively predicted by assuming the lower limit of the soil moisture content to be the shrinkage limit. Desiccation beyond this value cannot bring about further volume change. However, the effects of evaporation in semi-arid regions such as the highveld of South Africa may be reduced significantly by the presence of an inactive layer at the surface or by vegetation offering shade to the surface of the soil. On the other hand, transpiration from vegetative cover is a major cause of water loss from soils in semi-arid regions. Indeed the distribution of soil suction in the soil is primarily controlled by transpiration from vegetation. Very high values of soil suction have been recorded, for example, Williams & Pidgcon (1983) referred to a pF at a depth of 6 m being 4.8, whilst that at 15 m was 34.8.

The maximum soil suction which can be developed is governed by the ability of vegetation to extract moisture from the soil, that is, the water demand of the plants concerned. The field capacity of a soil refers to the point of desiccation, which defined in terms of soil suction is a pF value of approximately 2. The level at which moisture is no longer available to plants is termed the permanent wilting point and this corresponds to a pF value of about 4.2. In semi-arid regions the complete depth of active clay profiles usually does not become fully saturated during the wet season. For example, in the highveld even during the summer rainfall season the soil suction can still approach the wilting point where the soil is covered with vegetation. Nonetheless, the pF may fall to 3.2 during the wet season and changes in soil suction may be expected down to a depth of approximately 2m between the wet and dry seasons.

The moisture characteristic (moisture content v. soil suction; Fig. 4) of a soil provides valuable data concerning the moisture content corresponding to the field capacity, and the permanent wilting point as well

as the rate at which changes in soil suction take place with variations in moisture content. This enables an assessment to be made of the range of soil suction and moisture content values which are likely to occur in the zone affected by seasonal changes of climate. The extent to which vegetation is able to increase the soil suction to the level associated with the shrinkage limit is obviously important. In fact the moisture content at the wilting point exceeds that of the shrinkage limit in soils with a high content of clay and is less in those possessing low clay contents. This explains why settlement resulting from the desiccating effects of trees is more notable in low to moderately expansive soils than in the more expansive ones.

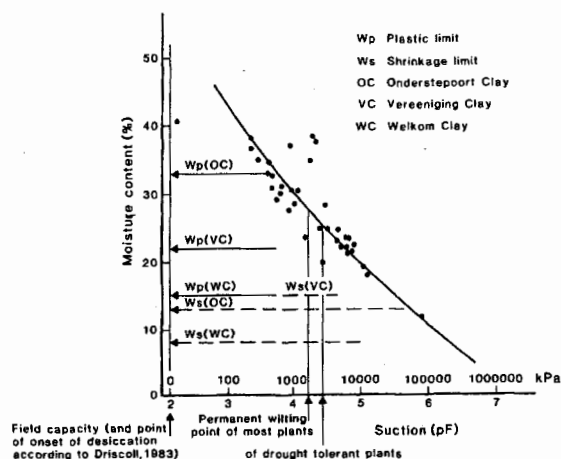


FIG. 4. Moisture content - soil suction relationships (the moisture characteristic) for some South African clays (modified after Williams & Pidgcon 1983).

TABLE 7. Relative strength of various sandstones (Price, 1960).

Rock type	Porosity (percent of volume)	Air dry pore water content as percentage of pore volume	Relative strengths for various pore water contents expressed as percentages of 'completely' dry rock.		
			Completely dry (σ_d)	Air dry (% of σ_d)	Saturated (% of σ_d)
Markham Sandstone (No. 21)	6.0	22 ± 5	100%	57	Not determined
Parkgate Rock (No. 23)	10.0	9 ± 1.5	100%	68	45
Pennant Sandstone (No. 26)	2.5	42 ± 3	100%	51	45
Darley Dale Sandstone (No. 27)	19.5	3 ± 1	100%	80	45

When vegetation is cleared from a site, its desiccating effect is also removed and the subsequent regain of moisture by clay soils can lead to swelling. Williams & Pidgeon (1983) noted that swelling movements on expansive clays in many areas of South Africa, associated with the removal of vegetation and the subsequent erection of buildings, amounted to about 150 mm. The largest upward movement recorded so far is 374 mm. Similar swelling movement was reported by O'Neill and Poormoayed (1980) from building sites in Texas which were located on areas of expansive clay cleared of vegetation. Uncleared areas did not swell to the same extent. In the latter areas surface movement was cyclical but irregular with an amplitude of up to 60 mm. Popescu (1979) noted that maximum seasonal changes in moisture content of the expansive clays of Rumania were around 20% at 0.5 m depth, 10% at 1.2 m and less than 5% at 1.8 m. The corresponding cyclic movements of the ground surface are about 100 to 120 mm. Desiccation of clays during the dry season leads to the soil cracking. The cracks can gape up to 150 mm and frequently extend to 2 m in depth. Popescu stated that the depth of the active zone in expansive clays in Rumania varies from about 2.0 to 2.5 m. In Australia, India, South Africa and the United States it may extend to 3 m depth, whereas in Israel depths of up to 6 m have been recorded.

The suction pressure associated with the onset of cracking is approximately pF 4.6. The presence of desiccation cracks enhances evaporation from the soil. Mahar & O'Neill (1983) assumed that such cracks led to a variable development of suction pressure, the highest suction occurring nearest the cracks. This, in turn, affects the preconsolidation pressure. Indeed it has been claimed that the effect of desiccation on clay soils is similar to that of heavy overconsolidation. For example Williams & Pidgeon (1983) suggested that if the pF of a soil rose to 4.5, then the state of stress therein approximates to a preconsolidation pressure of 3.1 MPa.

Sridharan & Allam (1982) found that repeated wetting and drying of clay soils in arid and semi-arid regions can bring about aggregation of soil particles due to cementation by Ca, Mg, Al and Fe compounds. This enhances the permeability of the clays and increases their shear strength. Such interparticle desiccation can reduce the swelling capacity of an expansive clay soil or may even render it non-expansive.

Many clay soils in Britain, especially in south-east England, possess a large potential for volume change. However, apart from periods of extreme drought, any significant deficits in soil moisture developed during the summer are confined to the upper 1.0 to 1.5 m of the soil and these are made good during the winter. Even so, deeper permanent deficits can be brought about by large trees. With this in mind, Driscoll (1983) suggested that desiccation could be regarded as commencing when the rate of change in moisture

content (and therefore volume) with increasing soil suction, increases significantly. He proposed that this point approximates to a suction of pF about 2 (10 kPa). Similarly, notable suction could be assumed to have taken place if, on its disappearance, a low-rise building was uplifted due to the soil swelling. Driscoll maintained that this suction would have a pF value of about 3 (100 kPa). He went on to state that the moisture content (m%) of the soil at these two values of suction is approximately related to the liquid limit (LL) as follows:

- (a) at pF = 2, $m = 0.5 \text{ LL}$
- (b) at pF = 3, $m = 0.4 \text{ LL}$.

Hence these values of liquid limit provide crude estimates of the moisture contents at the beginning of desiccation and when it becomes significant.

Breakdown and slaking of argillaceous rocks

The natural moisture content of shales varies from less than 5% increasing to as high as 35% for some clayey shales. When the natural moisture content of shales exceeds 20% they frequently are suspect as they tend to develop potentially high pore water pressures. Usually the moisture content in the weathered zone is higher than in the unweathered shale beneath. According to Olivier (1980) the degradation processes in shales subjected to changes in the moisture content are related to partially irreversible anisotropic expansion and shrinkage of the rock as a result of capillary action and drying respectively.

Morgenstern & Eigenbrod (1974) carried out a series of compression softening tests on argillaceous materials. They found that the rate of softening of these materials when immersed in water largely depends upon their degree of induration. Poorly indurated materials soften very quickly and they may undergo a loss of up to 90% of their original strength within a few hours. Mudstones, at their natural water content, remain intact when immersed in water. However, they swell slowly, hence decreasing in bulk density and strength. This time-dependent loss in strength is a very significant engineering property of mudstones. A good correlation exists between their initial compressive strength and the amount of strength loss during softening.

Depending upon the relative humidity, many shales slake almost immediately when exposed to air (see Kennard *et al.* 1967). Desiccation of shale, following exposure, leads to the creation of negative pore water pressure and consequent tensile failure of the weak intercrystalline bonds. Hence shale particles of coarse sand and gravel size are produced. Alternate wetting and drying causes a rapid breakdown of compaction shales and low-grade varieties, in particular, undergo complete disintegration after a few cycles of drying and wetting.

On the other hand, well cemented shales are fairly resistant to slaking. The slaking behaviour of mudstones is often dominated by a tendency to breakage along irregular fracture patterns, which when well developed, can mean that these rocks disintegrate within one or two cycles of wetting and drying.

Russell (1984) examined the durability of two major shale formations in Ontario by means of the slake durability test. Although the shales appeared similar, one had a lower durability than the other. This was attributed to the difference in the nature of the microcracks and to the degree of cementation. In the less durable shale, the microcracks were more curved and it was also poorly cemented. Microcracks allow the penetration of water which leads to slaking and the more curved they are, the more they tend to meet on slaking thereby causing breakdown. Russell also noted that the durability of compaction shales was influenced by the amount of clay minerals present in the clastic fraction whereas this was not the case in cemented shales. Slaking in argillaceous sediments can bring about an increase in their plasticity index and augment their ability to swell. The air pressure in the pore spaces helps the development of the swell potential under cyclic wetting and drying conditions. On wetting the pore air pressure in a dry clay increases and it can become large enough to cause breakdown, which at times can be virtually explosive. The rate of wetting is important, slow wetting allows the air to diffuse through the pore water so that the pressure does not become large enough to cause disruption of the material.

Argillaceous materials are capable of undergoing appreciable suction before pore water is removed, drainage commencing when the necessary air-entry suction is achieved (about $pF\ 2$). Under increasing suction pressure the incoming air drives out water from a shale and some shrinkage takes place in the fabric before air can offer support. Generally as the natural moisture content and liquid limit increase so the effectiveness of soil suction declines.

Morgenstern & Eigenbrod (1974) used a water absorption test to assess the amount of slaking undergone by argillaceous material. This test measures the increase of water content in relation to the number of drying and wetting cycles undergone. They found that the maximum slaking water content increased linearly with increasing liquid limit and that during slaking all materials eventually reached a final water content equal to their liquid limit. Materials with medium to high liquid limits exhibited very substantial volume changes during each wetting stage, which caused large differential strains, resulting in complete destruction of the original structure. Thus materials characterised by high liquid limits are more severely weakened during slaking than materials with low liquid limits.

After a study of the disintegration of shales in water, Badger *et al.* (1956) concluded that this was brought about by two main processes, namely, air

breakage and the dispersion of colloid material. It was noted that the former process only occurred in those shales which were mechanically weak whilst the latter appeared to be a general cause of disintegration. They also observed that the degree of disintegration of a shale when it was immersed in different liquids was governed by the manner in which those liquids affected air breakage and ionic dispersion forces. For example, in a liquid with a low dielectric constant little disintegration took place as a result of ionic forces because of the suppression of ionic dissociation from the shale colloids. It was found that the variation in disintegration of different shales in water was not usually connected with their total amount of clay colloid or the variation in the types of clay minerals present. It was rather controlled by the type of exchangeable cations attached to the clay particles and the accessibility of the latter to attack by water which, in turn, depended on the porosity of the shale. Air breakage could assist this process by presenting new surfaces of shale to water.

Nakano (1967) found that although some mudstones from Japan, when immersed in water swelled slowly and underwent a consequent decrease in bulk density and strength, they did not disintegrate even after immersion for a lengthy period of time. However, if they were dried and then wetted they disintegrated rapidly into small pieces. After conducting a series of slaking tests in vacuum, as well as in air, Nakano concluded that air breakage was not a significant mechanism in the breakdown of mudstones since he noticed no difference in the results of the slaking tests. He attributed the weakening of mudstone to chemical dissolution by hydrogen bonding of originally absorbed water molecules around clay particles with newly absorbed ones. He assumed that part of the free energy, which was evolved by water molecules when absorbed around clay particles, acted as the destructive force in slaking. It was observed that mudstones started to slake when drying in a relative humidity of 98%. In another instance, the relative humidity was 94%. In each case the drier the mudstone, compared with its natural state, the greater was the intensity of disintegration in water. This means that such mudstones (the clay fraction in these mudstones consisted of montmorillonite) may deteriorate readily in the zone of fluctuating water table or water vapour pressure. This does not mean that beneath these zones mudstones will not be affected, they can be softened, especially if heavily fractured.

After an investigation of the breakdown of Coal Measures rocks, Taylor & Spears (1970) found that some seatearths, notably the Brooch and Park seatearths, after desiccation, disintegrated very rapidly in water (the former was literally 'explosive' and the latter broke down in less than 30 min). Although the expandable clay content in these two rocks is high, and leads to intraparticle swelling, the authors maintained that this alone was not

responsible for their rapid disintegration. In contrast to the work done by Nakano (1967), Taylor and Spears found that breakdown could be arrested by the removal of air from the samples under vacuum. Thus they concluded that air breakage was a principal disintegration mechanism in the weaker rocks. They suggested that during dry periods evaporation from the surfaces of rock fragments gives rise to high suction pressures which result in increased shearing resistance. With extreme desiccation most of the voids are filled with air, which, on immersion in water, becomes pressurised by the capillary pressures developed in the outer pore spaces. The mineral fabric may then fail along its weakest plane exposing an increased surface area to the same process. Taylor and Spears did not dismiss the physico-chemical ideas of Badger *et al.* (1956) or those of Nakano (1967), but suggested that such processes became progressively more important with time.

Strength of rocks

Much of the work on the effect of moisture on the compressive strength of rocks has concentrated on the variation of the uniaxial compressive strength between the dry and saturated state. Price (1960) tested a number of Coal Measure sandstones and found that the compressive strength for air-dried samples was between 51% and 80% of that for 'completely' dry samples and for saturated samples was 45% of the 'completely' dry strength. The loss of strength between the 'completely' dry and the air-dry state would appear to relate to the degree of saturation and the porosity (Table 7). In the semi-saturated state water tends to form a film on grain surfaces. As the ratio of grain surface area to pore volume is higher in low porosity rocks, the pore water content (i.e. the degree of saturation is higher for rocks of low porosity. It would seem, therefore, that for sandstones of low porosity in particular, much of the loss of strength on wetting takes place at very low degrees of saturation.

From his work on air-dry and saturated Fell Sandstone, Bell (1978) stated that the influence of moisture saturation on strength appears to become less effective as the strength of sandstones increases. This is only true when comparing the air-dry with the saturated strength, because, as the porosity decreases (and the strength increases), more of the strength loss on wetting takes place between the 'completely' dry and the air-dry states.

West (1979) demonstrated a loss of compressive strengths of 38% between the air-dry and saturated states for Bunter Sandstones from Warrington. From the results he derived a linear relationship between the uniaxial compressive strength and moisture content.

Colback and Wiid (1965) worked on quartzitic sandstones and shales from South Africa. A number of compression test were carried out at eight different moisture contents. These tests indicate that for both

rock types the saturated strength is about half the 'completely' dry strength and that the relationship between strength and moisture content is only linear over the 'wetter' part of the curve (Fig. 5). Work on other rock types such as schist and gneiss by Feda (1966) and basalts, granite, gneiss and limestone by Ruiz (1966) shows similar loss of strength on saturation.

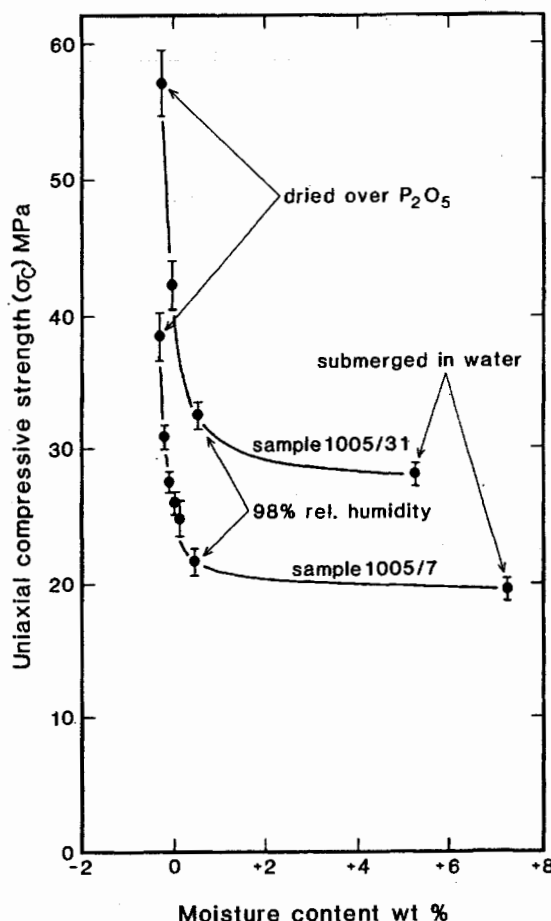


FIG. 5. Relationship between uniaxial compressive strength and moisture content for quartzitic sandstone specimens (after Colback & Wiid, 1965). Note: Moisture content zero datum defined as 50% relative humidity.

Colback & Wiid (1965) plotted the Mohr envelope for quartzite shale at two different moisture contents. From Fig. 6 it will be noted that the slopes of the Mohr envelopes are not sensibly different, indicating that the coefficient of internal friction is not significantly affected by changes in moisture content. Colback & Wiid therefore tentatively concluded that the reduction in strength witnessed with increasing moisture content, was primarily due to a lowering of the tensile strength, which is a function of the molecular cohesive strength of the material. Tests of

specimens of quartzitic sandstone showed that their uniaxial compressive strength was inversely proportional to the surface tension of the different liquids into which they were placed. As the surface free energy of a solid submerged in a liquid is a function of the surface tension of the liquid, and since the uniaxial compressive strength is directly related to the uniaxial tensile strength, and this is to the molecular cohesive strength, it was postulated that the influence of the immersion liquid was to reduce the surface free energy of the rock and hence its strength. The authors therefore concluded that the reduction in strength from the dry to the saturated condition of predominantly quartzitic rocks was a constant which was governed by the reduction of the surface free energy of quartz due to the presence of any given liquid.

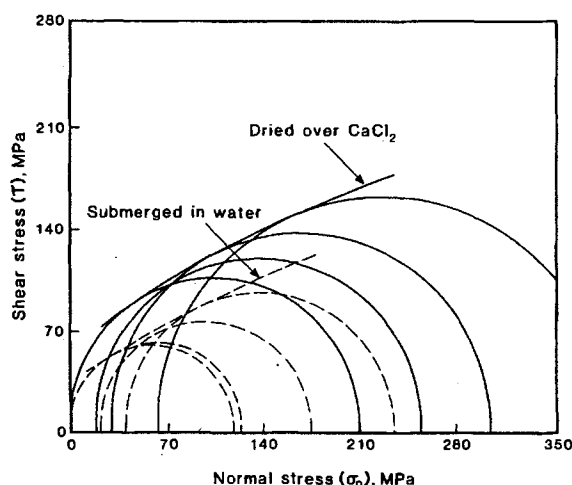


FIG. 6. Mohr envelope for quartzitic shale at two moisture contents (after Colback & Wiid 1965).

Volume change in loess, organic soils and fills

Instances of dramatic subsidence have been attributed to increases of moisture content of loess, a type of continental wind blown silt deposit, certain alluvial fan deposits formed under arid or semi-arid conditions and loosely compacted fills. In a dry state these materials may be sufficiently strong to resist void closure due to the overburden present. However, wetting weakens interparticle bonding to the extent that compaction takes place. Subsidence may also occur as a consequence of the drying out of organic soils.

Extensive damage to buildings, roads and services in the San Joaquin Valley of California, has been caused by hydrocompaction processes following an increase in moisture content of the underlying ground. Bull (1964) explained that materials susceptible to this type of behaviour comprise mudflow and other soils intermediate between waterlain and mudflow

deposits. In the area in question, these contain about 20% clay (less than 0.004 mm) size particles and are in a desiccated condition with water contents between wilting and hygroscopic values. Under these circumstances the clay which coats the silt sized particles, develops high suction pressures that impart high strength to the material. Although many of these deposits also contain gypsum, it would appear that this mineral is not leached, so the principal cause of hydrocompaction is the loss of strength which accompanies a reduction in suction forces when the material is wetted. The problem is not prevalent in water lain deposits since in these the clay size fraction is concentrated into seams rather than it filling the interparticle void space or coating the sand and silt sized particles. Neither are deposits with small amounts of clay susceptible to hydrocompaction since they are insufficiently strong to resist void space closure as the sedimentary load increases. It is significant that in the alluvial fan deposits of the San Joaquin Valley, the principal clay mineral is montmorillonite. In deposits with high clay size fractions little or no structural collapse occurs due to wetting due to the clay swelling, in fact volume increase may occur.

Besides compositional factors and the moisture condition of the material, the depth of overburden also directly controls the amount of subsidence consequent upon hydrocompaction. In the San Joaquin Valley most settlement occurs within the upper 60 m of deposits. Subsidence commonly attains 1.0 to 1.6 m with up to about 3.2 m in some small areas. The rate of compaction depends on the progress of the wetting front, but where continuous seepage of surface water has occurred, initial rates of about 75 mm per day have been recorded.

Ireland *et al.* (1984) describe pre-treatment carried out in connection with the California Aqueduct which crosses a number of areas susceptible to hydrocompaction. In these, ponds about 135 m wide were created along the alignment and in addition, infiltration wells were drilled on a 33 m grid over the central two-thirds of the ponds. During the 12 to 18 months period in which water depths of 0.5 to 0.6 m were maintained in the ponds, subsidence of the order of 0.8 to 1.6 m occurred.

Loess is a material which is liable to undergo appreciable volume reduction if wetted. Again collapse is attributed to the softening of clay bonding so that the soil settles under its own weight. Larionov (1965) recognised three micro-structures in loess namely, granular, where a filmy distribution of the fine dispersed fraction predominates; aggregate, consisting mainly of aggregates; and granular-aggregate, having an intermediate character. He suggested that generally loess soils with granular micro-structure are more likely to collapse on wetting than the aggregate types.

Several collapse criteria have been proposed which depend upon the void ratios at the liquid limit (e_l) and the plastic limit (e_p). According to Audric &

Bouquier (1976) collapse is probable when the natural void ratio is higher than a critical void ratio (e_c) which depends on e_i and e_p . They quoted the Denisov and Feda criteria as providing fairly good estimates of the likelihood of collapse:

$$e_c = e_i \text{ (Denisov)}$$

$$e_c = 0.85e_i + 0.15e_p \text{ (Feda).}$$

The main feature of the collapsible soil is the soil structure, which is formed by soil domains with a low number of grain contacts. Clevenger (1958) noted that on wetting, large settlements and low shearing resistance are encountered when the density of loess is below 1.28 Mg/m^3 , whereas if the density exceeds 1.44 Mg/m^3 , then settlement is small and shearing resistance fairly high. Moreover, the results of plate load tests have indicated that the bearing capacity of low-density loess may exceed 540 kPa when dry, and fall to as low as 27 kPa when wetted. The supporting capacity is reduced notably in low-density loess when the moisture content exceeds 15%. Indeed laboratory tests have shown that samples of low-density loess consolidate between 15 and 20% when prewetted (little consolidation occurs if there is no prewetting). It is not surprising therefore, that large settlements have occurred beneath footings in loess after it has been wetted.

Increased rates of settlement or a sudden resumption of settlement may occur due to the increased moisture content of domestic refuse, opencast mining and other fills. The problem has also been observed in compacted natural soils and rock fills, particularly if these materials are compacted dry of their optimum water contents.

Jennings & Burland (1962) referred to collapse phenomena due to inundation in laboratory tests on air dried and partly saturated silts. They attributed collapse which occurred even for small loads, to the loss of the bond effect produced by surface tension forces acting in water menisci at inter-particle points of contact. An alternative explanation is provided by Sowers *et al.* (1965) following laboratory tests. These demonstrated the significance of a reduction of rock strength at points of contact due to the entry of water into microfissures in the highly stressed contact zones.

The amount of settlement depends on many factors, including the depth, type and density of the fill. Often the exact nature of fill can be difficult to ascertain and the density depends on the method of emplacement. Leigh & Rainbow (1979) gave details of several backfill operations following opencast mining in which increased settlement has been attributed to a rise in groundwater levels. In one particular case in which the backfill consisted predominantly of shale excavated by scraper and drag line equipment, a large increase in the rate of settlement coincided with the cessation of groundwater pumping from an adjacent site. During the nine months follow-

ing the completion of backfilling and prior to the cessation of pumping, the 21.8 m thickness of fill settled 24 mm. The accelerated settlement then gave rise to a total settlement of 168 mm in the succeeding two years.

Charles *et al.* (1977) described an opencast site at Horsley, Northumberland, in which 63.1 m of sandstone and mudstone fill was placed. During the initial 8-months during which a low water table was maintained, surface settlement at one particular measuring station amounted to 8 mm. Again a gradual rise in watertable was accompanied by an increased rate of settlement such that in the succeeding 32 months the total surface settlement amounted to 303 mm. The vertical strain measured depth ranges of 6 m within the fill was very variable. A maximum compression of 1.4% was recorded whereas extensions occurred in some places.

Although rising groundwater levels associated with the cessation of mining operations are commonly the cause of problems, ingress of water from the surface may also be responsible for the wetting. Charles *et al.* (1978) described a case in which surface water entered via trenches dug in the course of construction work. Many materials may be susceptible to this type of behaviour, for example, domestic refuse (see Sowers 1973), rock fill (see Sowers *et al.* 1965) and natural soils (see Clayton & Simons 1981). Settlement may occur even when a considerable time has elapsed between backfilling and wetting. Leigh & Rainbow (1978) describe a case in which structural damage to a building took place 24 years after backfilling. The cessation of pumping in the area 10 years before the damage occurred apparently caused partial inundation of the 10 to 18 m thickness of backfill which settled 1 to 2% as a result.

All organic soils subside when drained due to shrinkage upon drying out and consolidation due to the loss of the buoyant force of groundwater. For instance, the organic soils in the Everglades of Florida undergo an average subsidence of 25 mm per year; in the Sacramento-San Joaquin delta of California the average rate reported is 760 mm per year; and for the organic soils of Michigan the average rate was 91.6 mm per year. Stephens & Speir (1969) maintained that the subsidence rates for organic soils are positively correlated with the depth of groundwater -the higher the water table, the lower the rate of subsidence.

Peat is the most compressible of materials and is highly porous so that its water content may range up to 2000%. A classic example of subsidence of peat due to drainage is provided in the Fenlands. There drainage of the peat has taken place over the last 400 years. In some parts of the Fenlands the thickness of peat has almost been halved as a result. For example, in the period between 1848 and 1932 a total subsidence of 2.7 m was recorded by the Holme Post, the original thickness of peat being 6.7 m. Subsidence due to the reclamation of peat land has also taken place in the New Orleans area (see Traugher *et al.* 1979).

Effects of changes in effective stress

Below the water table the water contained in the pores of soil or rock is under normal hydrostatic load, the prewater pressure increasing with depth. Because these pressures exceed atmospheric pressure they are designated positive pressures. On the other hand the pressures existing in the capillary zone are less than atmospheric and so are termed negative pressure. Thus the water table is usually regarded as a datum of zero pressure between the positive pore water pressure below and the negative above.

Such water pressures have a significant influence on the engineering behaviour of most rock and soil masses, and their variations are responsible for changes in the stresses in these masses, which affect their deformation characteristics and failure.

The efficiency of a soil in supporting a structure is influenced by the effective or intergranular pressure, that is, the pressure between the particles of the soil which develops resistance to applied load. Because the moisture in the pores offers no resistance to shear, it is ineffective or neutral. Since the pore water or neutral pressure plus the effective pressure equals the total pressure, reduction in pore water pressure increases the effective pressure. Reduction of the porewater pressure by drainage consequently renders soil stronger and affords better conditions for carrying a proposed structure.

The presence of water in the voids of a granular soil does not usually produce significant changes in the value of the angle of shearing resistance. However, if stresses develop in the pore water they may bring about changes in the effective stresses between the particles whereupon the shear strength and the stress-strain relationships may be radically altered. Whether or not pore water pressures develop depends upon the drainage characteristics of the soil mass and its tendency to undergo volume changes when subjected to stress. If the pore water can readily drain from the soil mass during the application of stress, then the granular material behaves in a similar manner as it does when dry. On the other hand if loading takes place rapidly, particularly in fine grained sands which do not drain as easily, then pore water pressures are not dissipated. Since the water cannot escape readily from the voids of loosely packed, fine grained sands, no volume decrease can occur so the pressure increases in the pore water. If the sample is loose enough, almost the entire stress difference may be carried by the pore water so that very little increase occurs in the effective stress. In dense sands if the stress and drainage conditions are such that the water cannot flow into the sand as it is stressed, then the usual volume increase characteristic of dense dry sand does not occur and a negative pore water pressure develops.

There is some evidence which suggests that the law of effective stress as used in soil mechanics, in which the pore water pressure is subtracted from all direct

stress components, holds true for some rocks. Those with low porosity may, at times, prove the exception. However, Serafim (1968) suggested that pore water pressures have no influence on brittle rocks. This is probably because the strength of such rocks is mainly attributable to the strength of the bonds between the component crystals or grains.

Pore water pressures also depend on the overall hydrological regime in which the soil or rock is situated. The movement of water through the ground is governed by a complex interaction between the permeability of the ground and the gravitational potential of sources and sinks within the system. Generally, unconsolidated or poorly cemented arenaceous or rudaceous materials or jointed rocks constitute the aquifers whilst aquicludes consist of fine-grained mudrocks or well-cemented or crystalline unjointed rocks. Water gaining access to an aquifer at a high level may become confined by an overlying aquiclude where the buried surface of the aquifer is lower than the water table in the recharge area. Under these circumstances there is an excess of pressure necessary to raise the water above the base of the confining layer. Where this pressure is sufficient to raise the water level above the ground level the conditions is described as artesian; otherwise it is known as sub-artesian and the water rises to a lower level. Artesian and sub-artesian conditions most commonly occur in the eroded cores of synclinal structures in systems of permeable and impermeable beds. They may also arise where impermeable igneous intrusions or impermeable fault gouge produce barriers that inhibit the movement of water through the ground.

Critical hydraulic gradient, quick conditions and hydraulic uplift

As water flows through the soil and loses head, its energy is transferred to the particles past which it is moving. This in turn creates a drag effect on the particles. If the drag effect is in the same direction as the force of gravity, then the effective pressure is increased and the soil is stable. Indeed the soil tends to become more dense. Conversely if water flows towards the ground surface, then the drag effect is counter to gravity thereby reducing the effective pressure between particles. In certain soils if the velocity of upward flow is sufficient it can buoy up the particles so that the effective pressure is reduced to zero. This represents a critical condition where the weight of the submerged soil is balanced by the upward acting seepage force. A critical condition sometimes occurs in silts and sands. If the upward velocity of flow increases beyond the critical hydraulic gradient a quick condition develops.

Quicksands, if subjected to deformation or disturbance, can undergo a spontaneous loss of strength. This loss of strength causes them to flow like viscous liquids. Terzaghi (1925) explained the quicksand phenomenon in the following terms.

Firstly, the sand or silt concerned must be saturated and loosely packed. Secondly, on disturbance the constituent grains become more closely packed which leads to an increase in pore water pressure, reducing the forces acting between the grains. This brings about a reduction in strength. If the pore water can escape very rapidly the loss in strength is momentary. Hence the third condition requires that pore water cannot escape readily. This is fulfilled if the sand has a low permeability and/or the seepage path is long. Casagrande (1936) demonstrated that a critical porosity existed above which a quick condition could be developed. He maintained that many coarse-grained sands, even when loosely packed, have porosities approximately equal to the critical condition whilst medium- and fine-grained sands, especially if uniformly graded, exist well above the critical porosity when loosely packed. Accordingly fine sands tend to be potentially more unstable than coarse grained varieties. It must also be remembered that the finer sands generally have lower permeabilities.

As the velocity of the upward seepage force increases further from the critical gradient the soil begins to boil more and more violently. At such a point there is a complete loss of bearing capacity so that any supported structure sinks. Liquefaction of potential quicksands may be caused by sudden shocks such as the action of heavy machinery (notably pile driving), blasting and earthquakes. Such shocks increase the stress carried by the water, the neutral stress, and give rise to a decrease in the effective stress and shear strength of the soil. There is also a possibility of a quick condition developing in a layered soil sequence where the individual beds have different permeabilities. Hydraulic conditions are particularly favourable where water initially flows through a very permeable horizon with little loss of head, which means that flow takes place under a great hydraulic gradient.

When water percolates through heterogeneous soil masses it moves preferentially through the most permeable zones and it issues from the ground as springs. Piping refers to the erosive action of some such springs, where sediments are removed by seepage forces, so forming subsurface cavities and tunnels. In order that erosion tunnels may form, the soil must have some cohesion, the greater the cohesion, the wider the tunnel. In fact fine sands and silts are most susceptible to piping failures. Obviously the danger of piping occurs when the hydraulic gradient is high, that is, when there is a rapid loss of head over a short distance. As the pipe develops by backward erosion it nears the source of water supply so that eventually the water breaks into and rushes through the pipe. Ultimately, the hole, so produced, collapses from lack of support.

Hydraulic uplift phenomena occur under a wide range of geological conditions and at differing scales. For example, Moore & Longworth (1970) recorded a failure in a brick pit, 29 m in depth, excavated in

Oxford Clay. The failure was brought about by a build-up of hydrostatic pressure in an underlying aquifer (either the Cornbrash or Blisworth Limestone located at depths of 6 and 11 m respectively below the surface of the pit). This initially gave rise to a heave of some 1500 mm, and then ruptured the surface clay, thereby allowing the rapid escape of approximately 7000 m³ of water. The floor of the pit then settled up to 100 mm. Structures founded below ground water level are acted upon by uplift pressures. If the structure is weak this pressure can break it and, for example, cause a blow-out of a basement floor or collapse of a basement wall. If the structure is strong, but light, it may be lifted, that is, subjected to heave.

Consolidation and subsidence

As indicated in Table 8, subsidence attributable to the withdrawal of groundwater has occurred in numerous regions throughout the world. It has developed with most effect in those groundwater basins where there was, or is, intensive abstraction (see Poland, 1984). Such subsidence is attributed to the consolidation of sedimentary deposits as a result of increasing effective stress. The total overburden pressure of partially saturated or saturated deposits is borne by their granular structure and the pore water. When groundwater abstraction leads to a reduction in pore water pressure by draining water from the pores, there is a gradual transfer of stress from the pore water to the granular structure. For instance, if the water table is lowered by 1 m, then this gives rise to a corresponding increase in average effective overburden pressure of 10 kPa. As a result of having to carry this increased load the granular structure may deform in order to adjust to the new stress condition.

Consolidation may be elastic or non-elastic depending upon the character of the deposits involved and the range of stresses induced by a decline in the water level. In elastic deformation, stress and strain are proportional, and consolidation is independent of time and reversible. Non-elastic consolidation occurs when the granular structure of a deposit is rearranged to give a decrease in volume, that decrease being permanent. Generally recoverable consolidation represents compression in the pre-consolidation stress range, while irrecoverable consolidation represents compression due to stresses greater than the pre-consolidation pressure.

The amount of subsidence which occurs is governed by the increase in effective pressure, the thickness and compressibility of the deposits concerned, the length of time over which the increased loading is applied, and possibly the rate and type of stress applied. For example Deflache (1979) reported that the most noticeable subsidence in the Houston-Galveston region of Texas has occurred where the declines in head have been largest and where the thickness of clay in the aquifer system is greatest. Furthermore, the ratio between maximum subsidence and ground-

TABLE 8: *Areas of major land subsidence due to groundwater overdraft (from Poland 1972).*

Location	Depositional environment and age	Depth range of compacting beds (m)	Maximum subsidence (m)	Area of subsidence (km ²)	Time of principal occurrence	Remedial or protective measures taken
Japan, Osaka	Alluvial and shallow marine; Quarternary	9-400	3	192	1928-68	Reduced groundwater pumpage; built dikes, drainage pumping plants
Tokyo	As above	9-305	4.3	192	1920-70+	Reduced groundwater pumpage; built dikes, drainage pumps; pumping stations.
Mexico Mexico City	Alluvial and lacustrine; Late Cenozoic	9- 49	8.6	128	1938-70+	Reduced pumpage; imported water; built recharge wells.
Taiwan, Taipei Basin	Alluvial and lacustrine; Quaternary	9-244	1.3	128	1961-69+	Groundwater management code adopted; recharge planned.
United States, Arizona Central	Alluvial and lacustrine; late Cenozoic	91-550	2.3	640	1948-67	
California, Santa Clara Valley	Alluvial and shallow marine; late Cenozoic	55-305	4	640	1920-70	Built detention dams; increased local recharge; built dikes; imported water.
San Joaquin Valley (three subareas)	Alluvial and lacustrine; late Cenozoic	61-915	2.9-8.5	10750 (>0.3 m)	1935-70+	Built reservoirs and imported water to reduce groundwater pumpage.
Lancaster Area	Alluvial and lacustrine;	61-305	0.9	384	1955-67+	
Nevada, Las Vegas	Alluvial; late Cenozoic	61-305	0.9	512	1935-63	Moved well field away from fine-grained deposits; imported Colorado River water.
Texas, Houston Galveston area	Fluvial and shallow marine; late Cenozoic	61-610	0.9-1.5	6784 (>0.15 m)	1943-64+	Plans for surface-water imports under way
Louisiana, Baton Rouge	Fluvial and shallow marine; Miocene to Holocene	46-610	0.3	640	1934-65+	

water reservoir consolidation is related to the ratio between depth of burial and the lateral extent of the reservoir. In other words, small reservoirs which are deeply buried do not give rise to noticeable subsidence, even if subjected to considerable consolidation. By contrast, extremely large underground reservoirs may develop appreciable subsidence.

The rate at which consolidation occurs depends on the rate at which the pore water can drain from the system which in turn, is governed by its permeability.

For instance, the low permeability and high specific storage of aquitards and aquicludes under virgin stress conditions means that the escape of water and resultant adjustment of pore water pressures is slow and time-dependent. Consequently, in fine-grained beds the increase in stress which accompanies the decline in head becomes effective only as rapidly as the pore water pressures are lowered towards equilibrium with the pressure in adjacent aquifers. The time required to reach this stage varies directly

according to the specific storage and the square of the thickness of the zone from which drainage is occurring and inversely according to the vertical permeability of the aquitard. In fact, it may take months or years for fine-grained beds to adjust to increases in stress. Moreover, the rate of consolidation of slow-draining aquitards reduces with time and is usually small after a few years of loading.



FIG. 7. Historical faulting induced by groundwater level decline in Fremont Valley in southern California. Offset has occurred along a pre-existing geological fault (by courtesy of T.L. Holzer).

In addition to being the most prominent effect in subsiding groundwater basins, surface fissuring and faulting (Fig 7) may develop suddenly and therefore pose a greater potential threat to surface structures. In the USA such fissuring and faulting has occurred especially in the San Joaquin Valley, the Houston-Galveston region and in central Arizona. These fissures, and more particularly the faults, frequently occur along the periphery of the basin. The faults are high angled, normal faults, with the downthrow on the side towards the centre of the basin. Generally, displacements along the faults are not great, less than a metre, but movements may continue over a period of years. Holzer *et al.* (1979) argued that such faults did not extend beneath the zone where stresses due to lowering of the groundwater level occur. Hence their depth is shallow.

It is not only falling or low groundwater levels that cause problems, a rising or high water table can be equally troublesome. Since the mid 1960s, the rate of abstraction from the Chalk below London has decreased significantly so that groundwater levels are now increasing by as much as 1 m/year in some places (see Hurst & Wilkinson 1986). It must be borne in mind that natural groundwater levels in these areas had been lowered appreciably in the nineteenth century and first half of the twentieth by excessive abstraction.

A number of problems arise as a result of a rising water table. Obviously tunnels and deep basements are likely to be affected by leakage of groundwater into them, a problem now being faced in certain areas of Birmingham and Liverpool. One of the major changes is that in the pore water pressure regime. As the water table rises, saturation of the pore spaces in the soil or rock reduces the effective pressure, in the case of some soils by up to 50% or more. In turn, this lowers their strength and bearing capacity. Structures with deep foundations could be adversely affected in the future because of the decline in bearing capacity. The design for new structures will need to take these changing conditions into account.

In Louisville, Kentucky, increasing groundwater levels are causing concern over the possibility of structural settlement, damage to basement floors and the disruption of utility conduits. The control of groundwater levels has, therefore, an importance which extends beyond water supply considerations.

Effects of solution

The movement of groundwater within rock masses composed of a significant proportion of soluble minerals exerts a major control over their removal in solution. The actual site of dissolution is often controlled by the presence of rock mass discontinuities and other variations in mass permeability, including low permeability clay seams. However, besides removal from surfaces, reductions in bulk density or cavity formation may also be prevalent. The amount of mineral removed from the rockmass depends mainly on the rate of flow, quantity, temperature and chemistry of the water passing through it. Also of major significance is the solubility characteristics of the rock forming mineral. Natural processes liable to enhance the aggressiveness of water are considered in the context of weathering activity on page 6 of this paper. The effects of solution will be considered with reference to the commonly occurring rock formations most at risk from solution activity, namely calcareous rocks, gypsum and anhydrite and saliferous deposits.

Calcareous rocks

Limestones are chiefly composed of calcium carbonate and they are susceptible to acid attack because CO_2 readily combines with H to form the stable bicarbonate HCO_3^- .

The degree of aggressiveness of water to limestone can be assessed on the basis of the relationship between the dissolved carbonate content, the pH value and the temperature of the water. At any given pH value, the cooler the water the more aggressive it is. If solution continues its rate slackens and it eventually ceases when saturation is reached. Hence solution is greatest when the bicarbonate saturation is low. This occurs when water is circulating so that

fresh supplies with low lime saturation are continually made available. As James & Kirkpatrick (1980) noted, a material dissolves at a rate and in a manner which is influenced by its solubility (C_s) and specific solution rate constant (K). The form a particular mineral adopts does not influence its solution rate. Not only is the rate of flow significant but the area of material exposed to flowing water is also important. Non-saline water can dissolve up to 400 ppm of calcium carbonate.

In water with a temperature of 25°C the solubility of calcium carbonate ranges from 0.01-0.05 g/l, depending upon the degree of saturation with carbon dioxide. Dolostone is somewhat less soluble than limestone. The solution of limestone is a very slow process. For instance, Kennard & Knill (1968) quoted mean rates of surface lowering of limestone areas in the British Isles which ranged from 0.041-0.099 mm annually. They also quoted experiments carried out in flowing non-saline water which produced an average solution rate of approximately 1 mm per year. Nevertheless solution may be accelerated by man-made changes in the groundwater conditions or by a change in the character of the surface water that drains into limestone. For instance, James & Kirkpatrick (1980) wrote that if dry fissured rocks are subjected to substantial hydraulic gradients then they undergo dissolution along the fissures, hence leading to rapidly accelerating seepage rates. From experimental work which they carried out on Portland Limestone they found that the values of solution rate constant (K) increased appreciably at flow velocities which corresponded with a transitional flow regime. They showed that such a flow regime occurred in fissures about 2.5 mm in width which experienced a hydraulic gradient of 0.2. According to these two authors solution takes place along a small fissure by retreat of the inlet face due to removal of soluble material. Dissolution of larger fissures gives rise to long tapered enlargements, that enable seepage rates to increase rapidly and runaway situations to develop.

Groundwater solution creates caves underground by secondary enlargement of primary fractures, and closed depressions with internal drainage are formed at the outcrop. These depressions are referred to as dolines, or, more commonly, sinkholes.

Caves and sinkholes are diagnostic features of the karst landscape found in many of the areas underlain by limestone. Sinkholes can range in size from 0.5 m to 500 m in both depth and radius, and can be classified genetically into solution, collapse, subsidence (or alluvial) and buried (or filled) sinkholes (Culshaw & Waltham 1986). Many have complex origins.

Solution sinkholes are the normal result of surface lowering in a karst landscape, their rate of formation being measured on a geological time scale. The existence of these features is dependent on drainage through caves and microcaves in the underlying bedrock. Like all sinkholes their location may be

influenced by geological structure and they occur most commonly along the outcrop edge of impermeable rocks.

Collapse sinkholes are formed by surface collapse into caves. Because of the inherent strength of most cavernous limestones, and the relative scarcity of large caves they are rarer than is commonly thought. Progressive collapse of a heavily fissured zone of bedrock is more usual than catastrophic collapse of a cave roof. The collapse process is often contemporaneous with solution erosion, creating sinkholes of multiple origin.

Subsidence sinkholes form by the subsidence of a cover rock or soil into cavities in an underlying limestone. They are commonest in unconsolidated drift, which gives rise to their alternative name of alluvial sinkholes. They are also known as ravelling sinks (Beck 1984). There are two basic types of subsidence sinkhole in unconsolidated deposits. In sand the cover sediment is progressively sapped down into the limestone fissures. No large cavities can form and the result is slow surface subsidence. A clay cover is, however, capable of bridging a void in the sediment as the lower material is flushed into the limestone. The cavity enlarges by upward stoping until the surface fails suddenly in what is known as a drop-out. The locations of incipient dropouts are difficult to predict. Experience in Florida has shown that they form most commonly where the cover is less than 20 m thick (Steward, 1966) whilst in Missouri they frequently occur where soil thickness ranges from 12 to 30 m (Williams & Vineyard, 1976). A number of conditions accelerate the development of cavities in the soil and initiate collapse. Rapid changes in moisture content lead to aggravated slabbing or roofing in clays and flow in cohesionless sands. Lowering the water table increases the downward seepage gradient and accelerates downward erosion; reduces capillary attraction in sand and increases instability of flow through narrow openings. This also gives rise to shrinkage cracks in highly plastic clays which weakens the mass in dry weather and produces concentrated seepage during rains. Increased infiltration often initiates failure, particularly when it follows a period when the water table has been lowered.

Buried sinkholes occur in a limestone rockhead and are filled or buried by overlying sediment, commonly with no surface expression. They may form by subsurface solution, or as normal subaerial sinkholes subsequently filled by sediment as a result of environmental change. There may be associated cavities in either the soil cover or bedrock.

In chalk, solution sinkholes of significant size are rare and collapse sinkholes almost non-existent. Large subsidence sinkholes formed over geological lengths of time are known but active ones are on a small scale and are limited in distribution. In Britain they mostly occur in covers of unconsolidated sediment 1 to 10 m thick. Burial sinkholes are a more common feature in

chalk and are chiefly found along the marginal outcrop close to the boundary with Tertiary deposits. They are mostly small features with diameters from 1 to 20 m and depths rarely more than 10 m.

An important effect of solution in some limestones is enlargement of the pores, which enhances water circulation thereby encouraging further solution. This brings about an increase in stress within the remaining rock framework which reduces the strength of the rock mass and leads to increasing stress corrosion. On loading the volume of the voids is reduced by fracture of the weakened cement between the particles and by the reorientation of the intact aggregations of rock that become separated by loss of bonding. Most of the resultant settlement takes place rapidly within a few days of the application of load. Sowers (1975) related a case of settlement of two- and three-storey reinforced concrete buildings founded on soft porous oolitic limestone in Florida, which had been subjected to dissolution. The limestone had been leached and was highly porous with little induration remaining 2.1 m below the rock surface. One of the buildings was surrounded by a crack in the more intact surface crust and settlement exceeding 100 mm was recorded.

Gypsum and anhydrite

Gypsum is more readily soluble than limestone, 2100 ppm can be dissolved in non-saline waters as compared with 400 ppm. Sinkholes and caverns can therefore develop in thick beds of gypsum (see Eck & Redfield, 1965) more rapidly than they can in limestone. Indeed in the United States such features have been known to form within a few years where beds of gypsum are located beneath dams. Extensive surface cracking and subsidence has occurred in certain areas of Oklahoma and New Mexico due to the collapse of cavernous gypsum (see Brune 1965). The problem is accentuated by the fact that gypsum is weaker than limestone and therefore collapses more readily. Kendal & Wroot (1924) quoted vivid, but highly exaggerated, accounts of subsidence due to the solution of gypsum which occurred in the Ripon district of Yorkshire in the eighteenth and nineteenth centuries. They wrote that wherever beds of gypsum approach the surface their presence can be traced by broad funnel-shaped craters formed by the collapse of overlying marl into areas from which gypsum has been removed by solution. Apparently these craters only took a matter of minutes to appear at the surface. However, where gypsum is effectively sealed from the ingress of water by overlying impermeable strata such as marl, dissolution does not occur (see Redfield 1968).

The solution rate of gypsum or anhydrite is principally controlled by the area of their surface in contact with water and the flow velocity of water associated with a unit area of the material. Hence the amount of fissuring in a rock mass, and whether it is enclosed by permeable or impermeable beds, is most

important. Solution also depends on the sub-saturation concentration of calcium sulphate in solution. According to James & Lupton (1978) the concentration dependence for gypsum is linear whilst that for anhydrite is a square law. The salinity of the water also is influential. For example, the rates of solution of gypsum and anhydrite are increased by the presence of sodium chloride, carbonate and carbon dioxide in solution. It is therefore important to know the chemical composition of the groundwater.

Massive deposits of gypsum are usually less dangerous than those of anhydrite because gypsum tends to dissolve in a steady manner forming caverns or causing progressive settlements. For instance, if small fissures occur at less than 1 m intervals solution usually takes place by removal of gypsum as a front moving 'downstream' at less than 0.01 m/year. However, James & Lupton (1978) showed that if the following conditions were met:

- i. the rock temperature was 10°C,
- ii. the water involved contained no dissolved salts, and
- iii. a hydraulic gradient of 0.2 was imposed,

then a fissure 0.2 mm in width and 100 m in length, in massive gypsum, would, in 100 years have widened by solution so that a block 1 m³ in size could be accommodated in the entrance to the fissure. In other words a cavern would be formed. If the initial width of the fissure exceeds 0.6 mm, large caverns would form and a runaway situation would develop in a very short time. In long fissures the hydraulic gradient is low and the rate of flow is reduced so that solutions become saturated and little or no material is removed. Indeed James & Lupton implied that a flow rate of 10⁻³ m/s was rather critical in that if it was exceeded, extensive solution of gypsum could take place. Solution of massive gypsum is not likely to give rise to an accelerating deterioration in a foundation if precautions such as grouting are taken to keep seepage velocities low.

Massive anhydrite can be dissolved to produce uncontrollable runaway situations in which seepage flow rates increase in a rapidly accelerating manner. Even small fissures in massive anhydrite can prove dangerous. If anhydrite is taken in the above example not only is a cavern formed, but the fissure is enlarged as a long tapering section. Within about 13 years the flow rate increases to a runaway situation. However, if the fissure is 0.1 mm in width then the solution becomes supersaturated with calcium sulphate and gypsum is precipitated. This seals the outlet from the fissure and from that moment any anhydrite in contact with the water is hydrated to form gypsum. Accordingly 0.1 mm width seems to be a critical fissure size in anhydrite.

Anhydrite is less likely to undergo catastrophic solution in a fragmented or particulate form than gypsum. Conversion to gypsum is more characteristic of extensive deposits of permeable granular anhydrite.

Another point which should be borne in mind, and this particularly applies to conglomerate cemented with soluble material, is that when this is removed by solution the rock is reduced greatly in strength. A classic example of this is associated with the failure of the St. Francis Dam in California in 1928. One of its abutments was founded in conglomerate cemented with gypsum which was gradually dissolved, the rock losing strength, and ultimately the abutment failed.

Saliferous deposits

Salt is even more soluble than gypsum and the evidence of slumping, brecciation and collapse structures in rocks which overlie saliferous strata bear witness to the fact that salt has gone into solution in past geological times.

Considerable natural solution of salt in Cheshire means that it does not outcrop at the surface, it usually terminates in a solution surface at a depth ranging from 70-150 m. The regions beneath such surfaces, which are subjected to circulating groundwater and solution, are referred to as wet rock head. Where beds of rock salt are beyond the reach of circulating groundwater their original thickness is preserved and these regions are referred to as dry rock head. Natural subsidence in wet rock head areas are believed to have been responsible for the formation of the Cheshire Meres but generally speaking this type of subsidence, although it operates over large areas, takes place extremely slowly, it is perhaps imperceptible within the context of historic time. Some of the Meres are floored by peat deposits and it has been suggested that they may be pre-Pleistocene in age. Keuper Marl which generally overlies the saliferous beds is incompetent which means that large solution cavities have not been able to form since the collapse of the marl was probably contemporaneous with salt removal. These collapses produced linear subsidence depressions at the surface. Where a brine run occurs beneath glacial deposits cavities have formed in the latter. These then collapse to produce rather circular shaped, small hollows at the surface. They make their appearance almost instantaneously but fortunately, they only occur infrequently. It would seem that as the sand collapses it chokes the brine run, thereby preventing further subsidence.

It is generally believed, however, that in areas underlain by saliferous beds, measurable surface subsidence is unlikely to occur except where salt is being extracted (see Bell, 1975). Perhaps this is because equilibrium has been attained between the supply of unsaturated groundwater and the salt available for solution. Exceptionally cases have been recorded of rapid subsidence, such as the 'Meade salt sink' in Kansas. Johnson (1901) explained its formation as due to solution of beds of salt located at depth. This area of water, about 60 m in diameter, formed as a result of rapid subsidence in March 1879. At the same time, 64 km to the south-west, the

railway station at Roscl and several buildings disappeared due to the sudden appearance of a sinkhole.

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