CHAPTER 2

LITERATURE REVIEWS AND THEORY

2.1 LITERATURE REVIEWS

The principle of hydraulic fracturing has been utilized by the oil industry for many years to increase the reservoir permeability and the production of oil and gas (Howard and Fast, 1970). Hydraulic fracturing has also been recognized as important when considering the limitation of the maximum pressure in connection with pressure grouting (Elston, 1958), leakage from oil wells, leakage from rock and earth fill dams (Kjaernsli and Torblaa, 1968; Jaworski et al., 1981), field permeability measurements in soil (Bjerrum et al. 1972), and pile driving and driven sand drains (Massarch, 1978). Hydraulic fracturing has also been proposed as a way to measure the in situ minor principal stress in rock (Halmson, 1968) and in soil (Bjerrum and Anderson, 1972) by recording the closing pressure of a crack opened by hydraulic fracturing.

Recent ground improvement techniques; such as grouting and grout jetting, usually utilize a very high fluid pressure to transport grout material through the designated ground. The direction of applied pressure in most cases is in the horizontal. There are few reported cases where control of grout zone was lost and grout material was found outside the desired area (Ando and Makita, 1977). Natural fractures and cracks, hydraulic fracture and fine particle detachment may be the main contributor to such occurrence.

The way that hydraulic fractures grow in soil has been largely ignored. Only their initiation has been of concern, because the problem is to prevent them from happening at all. They are useful in engineering projects such as injection, grout, permeability testing, deep-well injection, core dam construction. Little is known about the physical appearance and mechanical behaviour of a hydraulic fracture in soil, or about methods of analyzing fracture growth.

In contrast, hydraulic fracturing of rock has been studied in detail because it is widely used as a method of increasing the yields of wells that produce oil, nature gas, water or stream. The problem of recovery hydrocarbons from reservoirs in petroleum engineering is analogous to the problem of recovering contaminants from aquifers in environmental engineering. It has been shown that it is feasible to create hydraulic fractures and fill them with sand at shallow depths in silty clay glacial deposit (Murdoch, 1990).

In clayey soil, the application of high fluid pressure may result in hydraulic fracture. As a consequence, sudden loss in pressure resistance may be found. Fracture orientation depends upon the direction of the applied fluid. Fig. 2.1 shows two possible planes of hydraulic fractures. The fracturing pressure, $(P_w)_F$, (Valko and Economicles, 1995) can be generally expressed as:

$$(P_w)_F = \sigma_v + t_v \quad : \textit{In case of horizontally induced fracture} \quad \ldots\ldots (2.1)$$

$$(P_w)_F = \sigma_h + t_h \quad : \textit{In case of vertically induced fracture} \quad \ldots\ldots (2.2)$$

Where σ_v, t_v and σ_h, t_h are the stresses and tensile resistances in the vertical and horizontal directions, respectively.

When fracture and/or erosion occurs, additional channels for the flow of fluid are formed and loss of hydraulic resistance may be expected. The direct measure in observing the fracture and erosion are to dyne the soil grain along the fracture plane and to detect the amount of fine content detached from the sample. Both grain dyne and amount of particle detachment are the processes for proving the occurrence of additional flow channels. Nevertheless they do not give the information on the behavior of soil. The behavior of soil under applied pressure could, therefore, be detected by continuously monitoring the flow characteristics.

Barriers with a low hydraulic conductivity are used as part of waste contaminant system to prevent ground water contamination. Sand-bentonite mixtures has been increasing interest in the use as the mineral impervious layer in both landfill liners and vertical cut-off walls, partly because they are less susceptible to frost damage and desiccation cracking than compacted clay. Sand and Bentonite Mixtures was very advantage in impervious structure projects, such as, dam core, and landfill.

A) Minimization of rain and melt-water infiltration and flow through a bed of dry waste products required an impervious layer, or of some sort of compound which absorbs water so slowly during period of rain or snowfall that it can evaporate in intermediate dry periods. Soil-type barriers may serve well but it is clear that only very fine-grained materials can be used. If such a bed is sufficiently thick it will retain a high isolateing capacity through its flexibility also if fairly large irregular displacement occur (Pusch, 1983).

The major design criteria of the top soil bed which introduced by Pusch (1984) are list as the following.

1) Sufficiently low hydraulic conductivity to prevent water from penetrating the bed.
2) Preserved sealing capacity when wetted after desiccation (cyclic).
3) Long-term chemical stability of clay minerals.
4) Frost penetration penetrated.
5) Reasonable cost.

A low hydraulic conductivity required that a clay-based layer be used as a major sealing component. It has to be protected from abrasion and erosion by a permeable drained layer cover with means that the top bed should actually be a sandwich construction, typical drawing is show in Fig.2.2. The entire series should be sloping (1:50

at minimum), permanently towards the periphery of the waste pile, where the slope is increased as mush as safe stability allows for.

There are numerous examples of clay and ballast compositions which yield very low hydraulic conductivities. Basing the composition of the ballast on the classifical Fuller curve and mixing it in an air-dry condition with finely granulated commercial Na bentonite to a weight percentage of air-dry bentonite of about 10, the hydraulic conductivity is easily reduced to 10^{-11} m/s if the mixture is effectively compacted. This is achieved by applying the mixture in 25-cm layers and compacting it air-dry by 10-15 runs of a moderately heavy vibrating roller. The mixing of bentonite and ballast is suitably performed in large concrete mixers by which a very homogeneous mass is obtained (Pusch and Nilsson, 1982).

The figure 10% is not arbitrary; it turns out to be a suitable average concentration if commercially available sodium bentonite are used and if the effective overburden pressure is about 50-100kPa (Pusch and Alstermark, 1984). Actually, a careful analysis of the required amount of such material to fill the pores of a suitably graded ballast forming a continuous grain skeleton with a homogeneous sealing clay gel, leads to slightly more than 5% of bentonite by air-dry weight.

This holds for ballast materials composed of a few selected sand and gravel fractions but since there are unavoidable variations in the blending and application processes some excess clay material is recommended and 8 – 10% is therefore a reasonably clay concentration. Provided that the pore-water chemistry remains constant, higher concentrations of expanding clay than about 5% yield significant swelling pressure which tend to lift the overburden and also may result in clay penetration into it. Such negative effects are eliminated by giving the upper, coarse-grained cover a sufficient weight and by composing its lower part so that the pores are small enough to prevent upward clay migration.

A recently tested alternative concept, by which the required amount of smectite clay and therefore also the cost is reduced to a minimum, is to use till as ballast and add a small amount of clay. Experiments run at the Swedish State Power Board using a material with the granulometry shown in Fig.2.3 and 5 % of commercial "Yellowstone Western" Na bentonite (air-dry weight), gave the evaluated hydraulic conductivity shown in Fig.2.4 (Cederstrom et al., 1980).

B) The Canadian concept for disposal of nuclear fuel waste involves placement of this material in a vault, located deep underground in plutonic rock (Rosinger and Dixon, 1982). Radionuclide migration to the biosphere will be related by the rock and also by a series of engineered barriers, each designed to be compatible with the other system components. The proposed components of the multiple barrier system in the disposal vault are shown schematically in Fig.2.5 (Dixon, Gray, and Thomas, 1985).

In the Canadian disposal concept, swelling and non-swelling clays are being examined as components of the buffer material. A Na bentonite and crushed illitic shale are being studied. Quigley (1984) has determined the mineralogical compositions of these materials. Table 2.1 presents engineering classification data for the two clays and the quartz sand being considered for mixing with the clays. The particle size distribution of the sand was specified using the Talbot-Richart and Fuller-Thompson methods, to yield a high maximum compacted density (McGill G.R.C., 1982).

C) The Stripa Project is an international project managed by the Swedish Nuclear Fuel Supply Co/Division KBS. The heat production of waste material is simulated by the heaters which produce a power of approximate 600 W per cylinder. The inner part of the site is filled with a mixture of sand-bentonite. The site is conducted at approximately at 350 m depth in the Stripa Mine. The main principle in the choice of a suitable composition for the backfill of tunnels and shafts of repositories is that these masses should have a low permeability. It is also required that they have a low compressibility in order to minimize the displacement caused by the swelling of the highly compacted bentonite. A sufficiently small average pore size is required as well to prevent the minute montmorillonite particles from migrating from the deposition holes into the pore system of the backfill. Finally, backfills must have a certain swelling potential to fill up voids that might arise in the course of the application of the material.

The Stripa Project used the 10% bentonite mixture which has an optimum water content of about 14% as concluded from the diagram in Fig.2.6. This value corresponds to an almost 90% degree of water saturation, so the water content in the field study was made lower, i.e. 10 ± 1%, to make it possible to detect the water uptake. The expected bulk density of the compacted mass was 2.0 – 2.05 t/m^3 which should increase to 2.15 – 2.2 t/m^3 when the water uptake is completed (Nilsson, 1985).

D) At Imatra Power Company in Finland, the low and intermediate level radioactive waste will be landfill in hard crystalline rock at depth of about 50 – 100 m. The backfill materials consist of three components: crushed rock aggregate, finely ground rock aggregate and bentonite. Holopainen (1985) was investigated hydraulic conductivity of 15% of Na bentonite mixture, the approximate value is 5 x 10^{-9} m/s. And he has reported that the swelling potential for the mixture varies between 20 and 60 kPa depending mainly on the salinity of the groundwater. Fig. 2.7 shows the schematic picture of the production of backfill material and the backfilling process.

As described in case history of impervious bentonite-ballast mixture used varies of amount of bentonite are between 5 to 25%. Their applied site is very dangerous of solid waste, leaking of leachate never allow. However, nobody studied about the lateral hydraulic resistance, or hydraulic fracture in this important material. This study will show the behaviour of hydraulic fracture in sand-bentonite mixture, to be the data to prevent it from happening.

2.2 THEORY

2.2.1 Grain Size Distribution

Several organizations have attempted to develop the size limited for gravel, sand, silt, and clay based on the grain size present in soils. Table 2.2 presents the size limits recommended by the American Association of State Highway and Transportation Officials (AASHTO) and the United (Corps of Engineers, Department of the Army and Bureau of Reclamation) Soil Classification Systems. Table 2.2 shows that soil particles smaller than 0.002 mm have been classified as clay. However, clays by nature are cohesive and can be rolled into thread when moist. This property is caused by the presence of clay minerals such as kaolinite, illite, and morillonite. In contrast, some minerals such as quartz and feldspar may be present in a soil in particle sizes as small as clay minerals. Hence they are called clay-size particles, not clay particles.

In any soil mass, the size, of various soil gains vary greatly. To classify a soil properly, you must know its grain-size distribution. The grain-size distribution of coarse-grained soil is generally determined by means of sieve analysis. For a fine-grained soil, the grain-size distribution can be obtained by means of hydrometer analysis.

A sieve analysis is conducted by taking a measured amount of dry, well-pulverized soil. The soil is passed through a stack of progressively finer sieves with a pan at the bottom. The amount of soil retained on each sieve is measured, and the cumulative percentage of soil passing through each sieve is determined. This percentage is generally referrer to as percent finer. Table 2.3 contains a list of U.S. sieve numbers and the corresponding size of their hole openings. These sieves are commonly used for the analysis of soil for classification purposes.

The percent finer for each sieve determined by a sieve analysis is plotted on semilogarithmic graph paper. Note that the grain diameter, D, is plotted on the logarithmic scale, and the percent finer is plotted on the arithmetic scale.

Two parameters can be determined from the grain-size distribution curves of coarse-grained soils: (1) the uniformity coefficient (C_u) and (2) the coefficient of gradation, or coefficient of curvature (C_z). These coefficients are

$$C_u = \frac{D_{60}}{D_{10}} \qquad \text{....... (2.3)}$$

$$C_z = \frac{D_{30}^2}{D_{60} D_{10}} \qquad \text{....... (2.4)}$$

where D_{10}, D_{30}, and D_{60} are the diameters corresponding to percents finer than 10, 30, and 60%, respectively.

2.2.2 Compaction and Proctor Compaction Test

Proctor compaction test. A standard laboratory soil-compaction test was first developed by Proctor (1933), and this is usually referred to as the standard Proctor test (ASTM designation D-698, AASHTO designation T-99). The test is conducted by compaction of three layers of soil in a mold that is 944 cm^3 in volume. Each layer of soil is subjected to 25 blows by a hammer weighing 24.5 N with a 30.48 cm. drop. The dry unit weight of compaction can be determined as

$$\gamma_{dry} = \frac{\gamma_{moist}}{1 + w} \qquad \text{....... (2.5)}$$

where w = moisture content of soil.

The test can be repeated several times at various moisture contents of soil. By plotting γ_d against the corresponding moisture content, the optimum moisture content and the maximum dry unit weight can be obtained.

The plot of γ_d vs. Moisture content of soils is influenced by several factors, the most important of which are the type and amount of compaction effort, the grain-size distribution, and the amount and type of clay minerals present. The compactive effort of Standard Proctor compaction test is equal to 12,375 ft.lb/ft^3 (593 kJ/m^3)

2.2.3 Unconfined Compression Strength

The unconfined compression test is a special case of unconsolidated undrained triaxial test. In this case, no confining pressure to the specimen is applied (i.e., $\sigma_3 = 0$). For such conditions, for saturated clays, the pore water pressure in the specimen at the beginning of the test is negative (capillary pressure). Axial stress on the specimen is gradually increased until the specimen fails. At failure, $\sigma_3 = 0$ and so

$$\sigma_1 = \sigma_3 + \Delta\sigma_f = \Delta\sigma_f = q_u \qquad \text{.......} (2.6)$$

where q_u is the unconfined compression strength.

Theoretically, the value of a saturated clay should be the same as that obtained from unconsolidated undrained tests using similar specimens. Thus s = S_u = $q_u/2$. However, this seldom provides high-quality results.

2.2.4 Hydraulic Conductivity

The continuous void spaces in a soil permit water to flow from a point of high energy to a point of low energy. Permeability is defined as the property of a soil which allows the seepage of fluids through its interconnected void spaces.

A fundamental relation for the quality of seepage through a soil mass under given condition, such as shown in Fig. 2.8. The cross sectional area of the soil is equal to A and the rate of seepage is q.

According to Bernoulli's theorem, the total head for flow at any section in the soil can be given by

Total head = elevation head + pressure head + velocity head

The loss of head Δh between sections *A* and *B* is

$$\Delta h = (z_A + h_A) - (z_B + h_B) \qquad \text{....... (2.7)}$$

where z_A and z_B are the elevation heads, and h_A and h_B are the pressure heads.

The hydraulic gradient I can be written as

$$i = \Delta h/L \qquad \text{....... (2.8)}$$

where *L* is the distance between sections *A* and *B*.

Darcy (1856) published a simple relation between the discharge velocity and the hydraulic gradient:

$$v = ki \qquad \text{....... (2.9)}$$

where v = discharge velocity

i = hydraulic gradient

k = coefficient of permeability

Hence, the rate of seepage q can be given by

$$q = kiA \qquad \text{....... (2.10)}$$

The coefficient of permeability k has the units of velocity, such as cm/s or mm/s, and is a measure of the resistance of the soil to flow of water. When the properties of water affecting the flow are included, and can express k by the relation

$$k\ (cm/s) = K\rho g/\mu \qquad \text{....... (2.11)}$$

where K = intrinsic permeability, cm^2

ρ =mass density of the fluid, g/cm^3

g =acceleration due to gravity, cm/sec^2

μ = absolute viscosity of the fluid, poise (g/(cm·s))

The coefficient of permeability of soils is generally expressed at a temperature of 20oC. At any other temperature T, the coefficient of permeability can be obtained from

$$\frac{k_{20}}{k_T} = \frac{\rho_{20}}{\rho_T}\frac{\mu_T}{\mu_{20}} \qquad \text{....... (2.12)}$$

where; k_T, k_{20} = coefficient of permeability at T°C and 20°C, respectively

ρ_T, ρ_{20} = mass density of the fluid at T°C and 20°C, respectively

μ_T, μ_{20} = coefficient of viscosity at T°C and 20°C, respectively

Since the value of ρ_{20} ρ_T is approximately 1, then;

$$k_{20} = k_T \frac{\mu_T}{\mu_{20}} \qquad \text{....... (2.13)}$$

Table 2.4 gives some typical values of the coefficient of permeability.

Factors affecting the coefficient of permeability, the coefficient of permeability depend on several factors, most of which are list below:

1. Shape and size of the soil particles.
2. Void ratio. Permeability increase with increase of void ratio
3. Degree of saturation. Permeability increase with increase of degree of saturation.
4. Composition of soil particles. Permeability increase with decreasing thickness of the diffuse double layer.
5. Soil structure. Permeability of fine-grained soils with a flocculated structure higher than those with a dispersed structure.
6. Viscosity of the permeant.
7. Density and concentration of the permeant.

Coefficient of permeability can determine in the laboratory. There are 4 common laboratory methods for determining the coefficient of permeability of soils as list below;

1. Constant-head test.
2. Falling-head test.
3. Indirect determination from consolidation test.
4. Indirect determination by horizontal capillary test.

The general principles of two methods are given below.

Constant-head test. This test is suitable for more permeable granular materials. The soil specimen is placed inside a cylindrical mold, and the constant head loss, h, of water flowing in a measuring cylinder and the duration of collection period. From Darcy's law, the total quantity of flow Q in time t can be given by

$$Q = qt - kiAt \qquad \text{.......} (2.14)$$

where A is the area of cross section of the specimen. But I = h/L, where Lis the length of the specimen, and so Q = k(h/L)At. Rearranging this gives

$$k = \frac{QL}{hAt} \qquad \text{.......} (2.15)$$

The values of Q, L, h, A, and t can be determined from the test, the coefficient of permeability of the soil can be calculated.

Falling-head test. This test is suitable for fine-grained soils. The soil specimen is placed inside a tube, and a standpipe is attached to the top of the specimen. Water from the standpipe flows through the specimen. The initial head difference h_1 at time $t = 0$ is recorded, and water is allowed to flow through the soil such that the final head difference at time t = t is h_2.

The rate of flow through the soil is

$$q = kiA = k\frac{h}{L}A = -a\frac{dh}{dt} \qquad \text{....... (2.16)}$$

where h = head difference at any time t

A = are of specimen

a = area of standpipe

L = length of specimen

or

$$k = 2.303\frac{aL}{At}\log\frac{h_1}{h_2} \qquad \text{....... (2.17)}$$

The values of a, L, A, t, h_1 and h_2 can be determined from the test, and then the coefficient of permeability of the soil can be calculated.

2.2.5 Hydraulic Fracture

It is generally accepted in the literature that hydraulic fracturing occurs when the minor principal effective stress, σ_3', become negative (tensile)with a magnitute exceeding the tensile strength, σ_1, of the soil, i.e., when

$$\sigma_3' + \sigma_1 \leq 0 \qquad \text{....... (2.18)}$$

A vertical crack will form if the minor effective principal stress is tangential, and a horizontal crack will form if the minor effective principal stress is vertical.

Drilling a borehole in the ground and applying a pressure in the fluid in the borehole will change the stresses around the borehole. The normal stress components are the vertical (σ_z), radial (σ_r), and the tangential (σ_θ) stresses, as illustrated in Fig 2.9.

The approaches that are proposed in the literature essentially attempt to calculate the stress changes due to the borehole fluid pressure and therefore it is fulfilled and hydraulic fracturing occurs. The various approaches that have been proposed may be grouped into the following categories: (i) elasticity theory, (ii) cavity-expansion theory, (iii) initial yielding and (iv) empirical formulas. Some comments on the various categories are given in the following.

Elasticity theory. This theory (e.g., Timoshenko and Goodiër 1982) can be used to calculate the stress changes around a bore hole in an elastic medium. The total pressure, p_f ,inside the borehole when fracturing occurs will then be

$$p_f = 2\sigma'_{ro} + u_o + \sigma_t \qquad \text{....... (2.19)}$$

where σ'_{ro} is the lateral effective stress prior to drilling of the borehole, equal in all horizontal direction; u_o is the pore pressure prior to drilling the borehole, σ_t and is the tensile strength of the soil. However, elasticity theory assumes that the soil is linearly elastic and neglects the shear-induced pore pressure.

Cavity-expansion theory. The cavity-expansion theory for expanding cylindrical and spherical cavities was first introduced by Bishop et al. (1949) for frictionless materials (metals). Gibson and Anderson (1961) developed solutions for cavity expansion in connection with the pressure meter test, and Bjerrum et al. (14972) used the theory to evaluate stresses around the piezometer pushed into the ground. Vesic (1972) gives solutions for both cohesive and frictional soils and shows how the excess pore pressure in the plastified zone can be calculated. This was used by Massarch (1978) to analyze the hydraulic fracturing around driven piles and sand drains.

The pressure, p_u , needed to expand a cylindrical cavity in an initially unstressed medium is

$$p_u = S_u\left(1 + \ln\frac{G}{S_u}\right) \qquad \text{....... (2.20)}$$

where G is the shear modulus in the elastic zone, and S_u is the shear strength of the medium and is assumed to be constant and isotropic. The cavity-expansion theory gives the excess borehole pressure when large radial displacements occur, and the fracture pressure will in most cases be lower than the cavity-expansion pressure.

Initial yielding. Initial yielding in the soil was used by Overy and Dean (1986), Mori and Tamura (1987), and Aldridge and Haland (1991) as a criterion to determine the hydraulic fracture pressure.

The shear stresses in the soil around the borehole were calculated with different assumptions regarding the change in tangential total stress. Aldrige and Haland used linear elastic solutions that give a reduction in the tangential normal stress, $\Delta\sigma_\theta$, equal to the increase in radial stress, , i.e.,

$$\Delta\sigma_\theta = -\Delta\sigma_r \qquad \text{....... (2.21)}$$

Overy and Dean and Mori and Tamura assumed that a change in the radial stress dose not cause any change in the tangential stress i.e.,

$$\Delta\sigma_\theta = 0 \qquad \text{....... (2.22)}$$

The fracture pressure was set equal to the borehole pressure when initial yielding occurs. However, it is difficult to understand why hydraulic fracturing should occur because of yielding in the soil. Fracturing is believed to be due to a tensile effective stress exceeding the tensile strength, rather than a shear failure.

Empirical formulas. The empirical formula which proposed by Jaworski et al. (1981) is

$$p_f = m\sigma_3 + \sigma_1 \qquad \text{....... (2.23)}$$

where m is an empirical factor,

Jaworski et al. (1981) suggested that σ_1 can vary between zero and the ultimate pressure from cavity-expansion theory, and the m varies between one and two. In their tests, they found m to be between 1.5 and 1.8 and σ_1 to be small.

The value of m is likely to be influenced by factors like the shear strength of the soil and whether there is flow of water into the soil. The fracture pressure depends on several factors, including soil strength, over consolidation ratio, borehole boundary condition, and flow conditions.

Fracture pressure. The fracture pressure can be established by determining the increase in borehole pressure that will give an effective stress change equal to the sum of the initial effective stress and the tensile strength, $\Delta p_m = (p_f - \sigma'_{ro})$i.e., the value of excess borehole pressure when one of the following two equations is fulfilled:

$$\Delta\sigma'_\theta = -(K_o\sigma'_{vo} + \sigma_t) \qquad \text{(vertical fracture)} \qquad \text{.......} (2.24)$$

$$\Delta\sigma'_z = -(\sigma'_{vo} + \sigma_t) \qquad \text{(horizontal fracture)} \qquad \text{.......} (2.25)$$

where σ'_{vo} is the vertical effective overburden stress, and $K_o\sigma'_{vo}$ is the horizontal effective stress prior to drilling the borehole, where K_o is the coefficient of lateral earth pressure at rest.

2.2.6 Detachment

Several studies in the literature documented theoretical formulations for detachment and subsequent transport of fine particles within a soil medium (Khilar et al., 1985; Hunt et al., 987; Rege and Fogler, 1988; Govindaraju et al., 1995; among others). It is common in most of these studies to assume that the rate of erosion, r, from a pore wall is proportional to the shear stress in excess of critical shear stress, an therefore it may be expressed as

$$r = \alpha(\tau_w - \tau_c) \qquad \text{.......} (2.26)$$

where α is the rate of change of erosion rate; τ_w is the applied shear stress on pore wall; and τ_c is the critical shear stress.

Physically, above equation indicates that in order to initiate erosion, the shear stress on the pore wall as a result of flow velocity should be greater than the critical shear stress. Implicit in this formulation is the assumption that the critical shear stress is an indicator of the strength of interparticle bonds binding the particles to the pore wall

Assuming that the shear stress on the pore wall is proportional to the seepage velocity and that the soil sample may be represented by a bundle of Poiseulle's pore tubes each of constant diameter, Khilar et al.(1985) arrived at an expression for the critical pressure gradient, J , expressed as force per unit volume, required to initiate erosion:

$$J = \frac{\tau_c}{2.828}\left(\frac{n_0}{K_0}\right)^{1/2} \qquad \text{....... (2.27)}$$

where n_o is the porosity prior to erosion; and K_o is the initial permeability (intrinsic) of the soil.

Clay Particle Detachment. The detachment or dispersion of clay particles within a soil mass is the first step in a process leading to either piping or clogging. The clays may detach themselves from pore walls or surfaces where they occur as a coating or they may break loose from clay aggregates that partially fill intergranular voids as shown in Fig. 2.10.

Clay particles, in general, are negatively charged primarily as a result of isomorphic substitution within their crystal lattice. Balancing cations or counterions are attracted to the basal surfaces of clay particles to neutralize this excess charge. An electric field is also associated with negatively charged clay particles and their cloud of counterions. The negatively charged surface of a clay colloid and its corresponding diffuse layer of conterions are known collectiveely as the "electric double layer". The strength of the electric field and its rate of decay with distance away from the surface plus the distribution or concentration of counterions at any point in the field are described by the Poisson-Boltzman equation.

When the clay particles come into contract with a liquid they tend to interact strongly with one another. If the liquid is a polar solvent such as water with a low electrolyte concentration, strong repulsive forced between particles come into play. This repulsion occurs as a result of overlapping or interference between the electric fields and diffuse cloud of counterions associated with each particle.

The magnitude and direction of these interactions, known as double layer interaction, depend on many parameters which characterize the clay and porous medium (solid phase). The interaction also depends on variables characterizing the permeant of fluid phase. When the interactions lead to strong repulsion the clay particles disperse or become detached.

Clay mineralogy. Some clay are highly dispersive and easily detached whereas others are less dispersive or more prone to swelling. Clay minerals such as kaolinite and illite tend to be dispersive while montmorillonite is characterized by its swelling behavior (Van Olphen, 1963; Rowell, 1963). Illite and kaolinite tended to exhibit dispersion or body slaking in contrast to monmorillonite which exhibited swelling slaking. Highly swelling clays, in piping studies, such as montmorillonite can also disperse, however, particle mobility is greatly impeded by gelling. Montmorillonite in a sample during a pinhole test (Acciardi, 1982) may actually require repunching the hole during the test because of closure resulting from clay swelling. In other words, any dispersieve tendency of the clay is counteracted by swelling which restricts both permeant flow and clay particle movement.

Exchange ions. The type of exchange ions on the surface of clay particles governs the degree of dispersion to a large extent. When clays are in contract with an aqueous solution the makeup of the exchange ion population depends on both the composition and ionic strength of the solution. Several predictive relationships (Gapon, 1933; USDA, 1954) can be used to determine the exchange site population as functions of the ionic composition of the aqueous solution in equilibrium with a clay.

Initial permeability and porosity. The initial permeability (K_o) and porosity (n_o) control the interstitial or seepage flow velocity through a porous medium. The tractive stress exerted by a seepage stream on clay particles in turn is determined by the magnitude of the velocity. Under conditions of high double layer repulsion, high seepage velocity can tip the interparticle force balance in favor of particle detachment and dispersion. The interstitial or seepage velocity can be computed from Darcy's law. Darcy's law may be written as follows for the case of single phase, one-dimensional, laminar flow:

$$\frac{q}{A} = \left(\frac{K_o}{\mu}\right)\left(\frac{\Delta P}{\Delta L}\right) \qquad \text{.......} \ (2.28)$$

where: q = flow rate, cm^3/sec

A = flow cross sectional area, cm^2

μ = viscosity, poise

K_o = initial permeability, cm^2

$\frac{\Delta P}{\Delta L}$ = pressure gradient, $dynes/cm^2/cm$

The seepage velocity (v_i) is obtained by dividing the superficial or approach velocity (q/A) by the porosity (n_o). Thus,

$$v_i = \left(\frac{K_o}{\mu n_o}\right)\left(\frac{\Delta P}{\Delta L}\right) \qquad \ldots\ldots (2.29)$$

Arulanandan et al. (1975) have shown that there exists a critical tractive stress (τ_c) above wich clays will setach and disperse in a seepage stream. Consequently, in order to initiate erosion the initial permeability (K_o) and porosity (n_o) must be such that for expected hydraulic gradients, the resulting seepage velocities will be high enough to produce a tractive stress (τ) greater than the critical tractive stress (τ_c). For tractive stresses greater than τ_c, the rate of erosion or particle detachment is proportional to the effective tractive stress $(\tau - \tau_c)$.

The critical tractive stress itself depends upon a numbe of variables related to both the solid and liquid (permeant) phasees (Arulanandan et al., 1975, 1976). Solid phase variables include the mineralogy and exchange ion population of the clay; liquid phase variables, the pH, temperature, dielectric constant, and electrolyte composition.

Once clay particles within the interstices of a porous medium are detached or dispersed as a result of colloidal and hydrodynamic forces they become entrained in the seepage stream. Several possible actions or outcomes are then possible, viz.,

1. Clay particles migrate in suspension out of the system without noticeable holdup.
2. Clay particles filter out or bridge at pore constrictions thereby restricting further flow.
3. Clay and other dislodged fines sediment out of suspension eventually plugging the pores.

The first action leads to internal erosion and piping. If the porous medium is an earthen structure which is not adequately protected by filter, this piping can culminate in total collapse or failure of the structure.

Clay Particle Migration The second and the third actions cause plugging and a reduction in apparent permeability. This outcome leads to the problem of formation damage or water sensitivity that has been reported occasionally during drilling, workover, or water flooding operations in clay bearing sandstones (Monaghan et al., 1959; Gray and Rex, 1966). The likelihood of one outcome as opposed to another is also of great importance in the operation of filters designed to prevent loss of fines from earthen structures (Vaughan and Soares, 1982).

Several variables related to either the solid or fluid phase affect the outcome or consequence of clay dispersion. These variables and their influence on clay particle migration, mainly control by hydrodynamic.

Particle/pore size ratio. The ratio of average particle size to average pore size is an extremely important parameter which determines whether piping or plugging will occur. If the particle size is equal to or greater than the size of the pore, then plugging will inevitably occur even if only a few particles are dispersed. This situation would be similar to straining or sieve filtration.

An estimate of the maximum particle size that can be transported in suspension through the pores of a porous medium can be obtained from studies of filter performance (Vaughan and Soares, 1982). Both theoretical and semi-empirical relationships have been proposed between permeability, pore size, and particle size for granular and filters (Scheiddeger, 1974). In the simplest case these relationships are generally of the form:

$$k = Ad^x \qquad \text{....... (2.30)}$$

where: k = hydraulic conductivity or permeability

d = representative pore diameter

A = a geometric or tortuosity factor

Clay content and mineralogy. Both the amount and type of clay present in a porouse medium will affect the outcome or consequence of clay dispersion. The greater the amount of swelling type clay greater th likelihood of flow reduction as a result particle gelling in the pores and "squeezing" at pore constrictions. The behavior of montmorillonite in slaking tests in contrast to illite and kaolinite (Mitchell & Moriwaki, 1976) support this mechanism as do results of pinhole erosion tests on soils containing swelling type clay (Acciardi, 1982). In the latter case, the expansion rate exceeds the erosion rate, and the pinhole leak is sealed. This mechanism also explains why bentonite or sodium montmorillonite is so effective in reducing or blocking flow through soils when added in relatively small amounts (i.e., less than 5% by weight). The effectiveness of clay slurry, cutoff-walls (D'Appolonia & Ryan, 1980) depends upon this fact.

The amount of clay present is important for a number of reasons. In general, increasing the amount of clay also increases the likelihood that clay dispersion will result in plugging or clogging. In the case of flow through pore volume defects a certain minimum clay content is required regardless of salinity of the eroding fluid or permeant, simply to provide sufficient cohesion (shear strength) to maintain the integrity of the conduit.

Particle release rate (Erosion rate). An important parameter governing particle concentration in the pores is the particle release rate or erosion rate. Arulanananda et al. (1975) have shown on the basis of rotating cylinder tests that, in general, erosion takes place only when shear stress on the surface exceeds the critical tractive stress.

Furthermore, the rate of erosion was shown to be proportional to the effective shear stress. An example of this behavior and relationship is shown in Fig. 2.11. Accordingly, the rate of erosion may be expressed by

$$r_r = \alpha(\tau - \tau_c) \qquad \text{....... (2.31)}$$

where: r_r = particle release rate, gms/cm^2/min

τ_c = critical tractive stress, dynes/cm^2

α = rate of change of erosion rate, (gms/dyne-min)10^{-4}

Both α and τ_c are functions of the chemistry of the pore water and the mineralogy of the solids.

Seepage velocity. The seepage velocity significantly affects the outcomes of particle migration within a porous medium. Seepage velocity is determined by the hydraulic gradient, permeability, and porosity. The seepage velocity must enough at the outset to produce a wall shear stress greater than critical shear stress (τ_c) in order to detach the clay particles.

Once particles begin eroding and are entrained in the seepage stream their fate is governed in large part by the seepage velocity. Seepage velocity affects the outcome of particle migration via its influence on particle concentration in the pores. High particle concentrations lead to plugging (permeability decrease). High rate of particle release (detachment) also promote rapid buildups in particle concentration. However, release rates are relatively insensitive to seepage velocity, and instead are governed more by physico-chemical variables such as salinity of the permeant, SAR, and mineralogy of the clay.

Khilar et al. (1983) have shown that approximately the same total amount of clay particles are released irrespective of the flow rate and rate of salinity decrease. Consequently, clay particle concentrations in the effluent will be inversely proportional to the amount of fluid sent through the porous medium. Thus, if more fluid is passed through a porous medium per unit time, then particle concentrations will be lower. This means that high flow rates (or gradients) tend to promote piping (permeability increase) by flushing particles out of the system and limiting particle concentration buildups in the pores.