Design and Construction of Embankment Dams

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1. INTRODUCTION

1.1 Preface

Dams, which are constructed of earth and rock materials, are generally referred to as embankment dams or fill-type dams. The history of construction of embankment dams is much older than that of concrete dams. It is evident that some earth dams were constructed about 3,000 years ago in the cradles of ancient cultures such as east countries.

According to the standard manual provided by the International Commission on Large Dams (ICOLD), in which about 63 member countries are now associated, dams with the height of more than 15m are referred to as "high dams". About 14,000 high dams have been registered up to the present, and more than 70 percent of them are embankment dams. A recent report on the construction of high dams has also noted that among about 1,000 of high dams constructed in recent two years, just about 20 percent are concrete dams and remaining 80 percent are embankment dams.

It is thus readily recognized that construction of embankment dams is a recent world-wide trend in place of concrete dams. Two major distinct features and advantages are noticed for the construction of embankment dams.

1. Rigorous conditions are not required for the dam foundation, while hard and sound rock foundation is necessary for concrete dams. Embankment dams can be constructed even on the alluvial deposit and pervious foundations.
2. Construction of embankment dams has an economical advantage; i.e., the dam project can be planned in the outskirts of city area because of the merit mentioned above, and construction materials are principally to be supplied near the dam site.

In this brief note, several important issues associated with the design and construction of embankment dams, which engineers often encounter in the dam project, are summarized, and some discussions are given on them by introducing recent development of design procedures and construction technology.

1.2 Types of Embankment Dams

Embankment dams are classified into two main categories by types of soil mainly used as construction materials, such as earthfill dams and rockfill dams. The latter ones further can be classified into a few groups by configurations of dam sections, as one with a centrally located core, one with an inclined core and one with a facing, as shown in Fig.1.1. The main body of rockfill dams, which should have a structural resistance against failure, consists of rockfill shell and transition zones, and core and facing zones have a role to minimize leakage through embankment. Filter zone should be provided in any type of rockfill dams to prevent loss of soil particles by erosion due to seepage flow through embankment. In earthfill dams, on the other hand, the dam body is the only one which should have both structural and seepage resistance against failure with a provided drainage facilities.

The dam type in a project is determined by considering various factors associated with topography and geology of the dam site, and quality and quantity of construction materials available. The inclined core is adopted instead of the center core, for instance, in cases where the dam foundation has a steep inclination along the river, where a blanket zone is provided in the pervious foundation to be connected with the impervious core zone, and where different construction processes are available for the placement of core and rockfill materials.

Key Words: rockfill, transition .... pervious zone, to have structural strength
core, facing .... impervious zone, to keep water tight
filter .... to prevent loss of soil particles
drain .... to pass water from upstream to downstream
(to dissipate pore water pressure)
core trench, grouting .... to keep water tight in the foundation

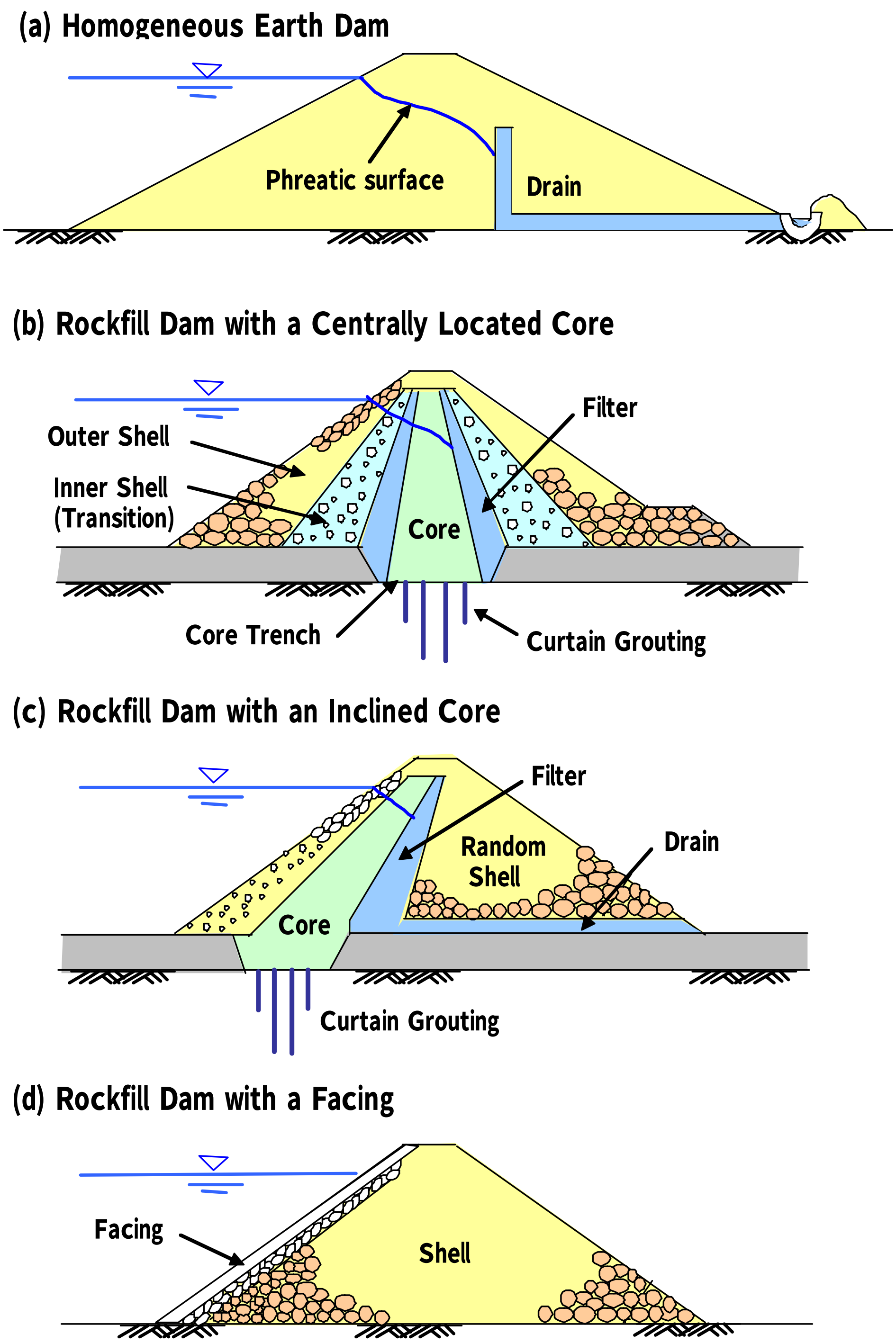


Fig.1.1 Earth and Rockfill Dams

1.3 Investigation, Design and Construction

There are three main steps of working in a dam project: i.e., investigation, design and construction. Individual works in these three steps are summarized as listed in Table.1.1 with key words associated with them.

Table.1.1 Key Words Associated with Investigation, Design and Construction

1.**Investigation**

#*Site Investigation*: Check of dam planning for appropriate purposes
/Meteorological and hydrological surveys
/Topographical and geological investigations
(landform, terrace, geological time, outcrop, lithofacies, folding, fault,
discontinuity, erosion, weathering, sedimentation, stratum)

#*Foundation Survey*: Check of required conditions for a base foundation
/Geophysical exploration (seismic prospecting, electrical prospecting, ...)
/Boring exploration (core drilling, sampling, sounding), /Test pitting
/In-situ testing (permeability, grouting, bearing capacity, compressibility)
/Rock classification

#*Fill Materials*: Check of required quality and quantity of materials
/Geological survey (stratum, volume)
/Laboratory testing (shear strength, compressibility, compaction, permeability,...)
/In-situ testing (roller compaction, density log, field permeability, sampling)

2.**Design**

#*Stability of Dam Body*
/Stability against sliding failure of embankment
(evaluation of pore-water pressure during and after construction,
shear strength and deformation characteristics of fill materials)
/Seismic stability
(seismic coefficient method, liquefaction, dynamic deformation characteristics,
dynamic response analysis, earthquake resistant design)
/Stability at the contact face of dam body and base foundation
(contact clay, compaction, relative displacement, arching, cracking)

#*Seepage Through Embankment and Foundation*
/Seepage analysis (discharge, pore-water pressure, leakage through foundation,
critical velocity, piping, critical hydraulic gradient, hydraulic fracture)

#*Foundation Treatment*
/Stability (counterweight fill, relief well, inspection gallery)
/Seepage (drainage facilities, grouting, blanket)

3.**Construction**

#*Planning for Construction*
/Construction equipment (roller, carrier, bulldozer, ...)
/Foundation treatment (grouting, drainage)
/Placement (execution management, field and laboratory testing)
/Observation (pore-water pressure, settlement, earth pressure, deformation)

#*Maintenance and Repair*

2. FAILURES AND DAMAGES OF EMBANKMENT DAMS

2.1 Preface

Most of catastrophic failures of embankment dams causes by overtopping of the reservoir water due to flooding or loss of free board. Despite embankment dams should not be designed to withstand erosive action of water flow over the crest, case histories reveal that inadequate capacity of spillway, that is, insufficient estimation of the amount of flooding has often led to the failure of the embankment due to overtopping. Failures of this type, however, can not be a decisive defect of embankment dams, because the accumulation of available accurate data of hydrology and the improvement of design method can readily settle the problem. This issue is therefore out of scope in this paper.

Other main factors to cause embankment failures are hydraulic erosion, high pore-water pressure, earthquake forces and so forth. More than 50 percent of embankment failures are above all due to hydraulic erosion, and remaining each several percent is caused by other respective factors. Several typical patterns of failures and damages of embankment dams and their foundations are illustrated in Table.2.1 and Fig.2.1.

Table.2.1 Failure Causes of Embankment Dams

During Construction	a) Pore water pressure built-up during construction b) Reduction of shear strength due to thixotropical property
After Construction	a) Hydraulic fracturing / Internal erosion / Piping b) Excess hydrostatic pressure due to rapid draw down c) Reduction in shear strength / Weathering, swelling of compacted soil d) Settlement and cracking e) Earthquake forces

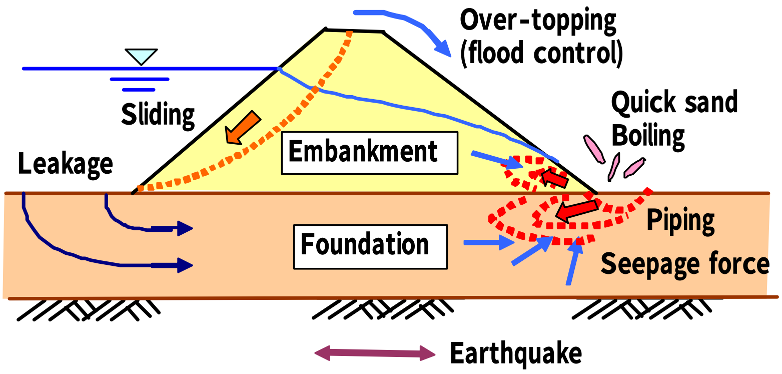


Fig.2.1 Damages in Embankment and Foundation

2.2 Sliding due to Pore-Water Pressure

An excessive and abrupt increase of pore-water pressure, such as the one built-up during construction and the residual one due to rapid drawdown of the reservoir, may cause sliding failures in embankment. In Fig.2.2, relatively high pore-water pressure was built-up in an earth dam, which led to a sliding failure during construction. In Fig.2.3, distribution of pore-water pressure at a usual stationary flow has changed during rapid drawdown, which caused a high excess pore-water pressure in the upstream part of the embankment.

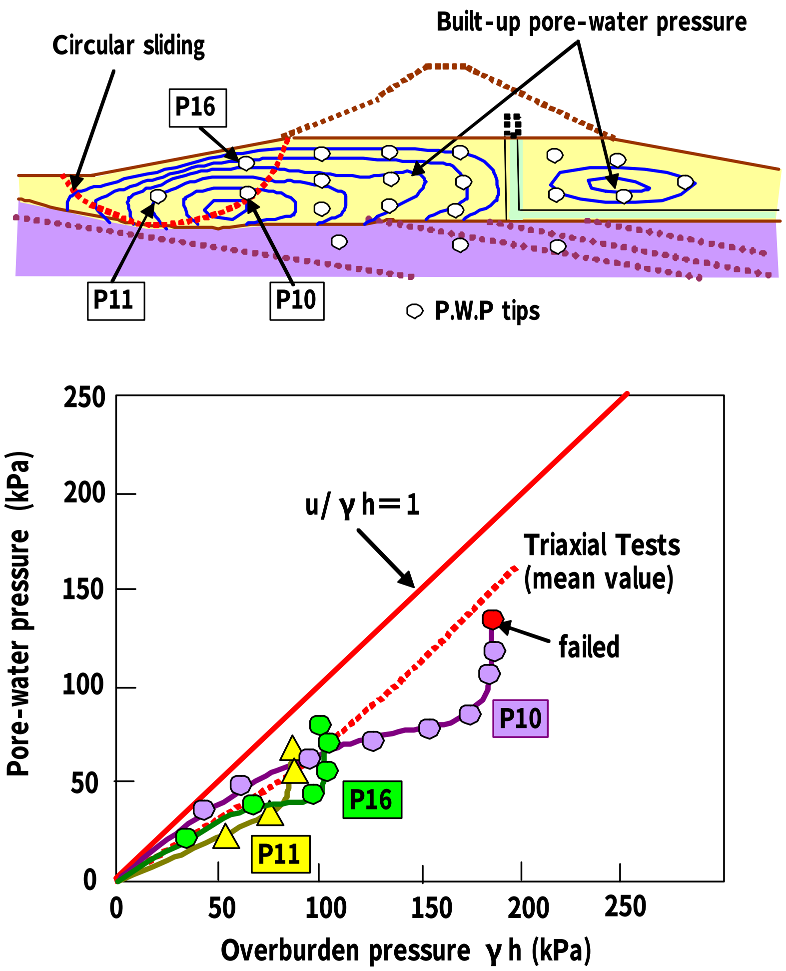


Fig.2.2 Sliding Failure During Construction

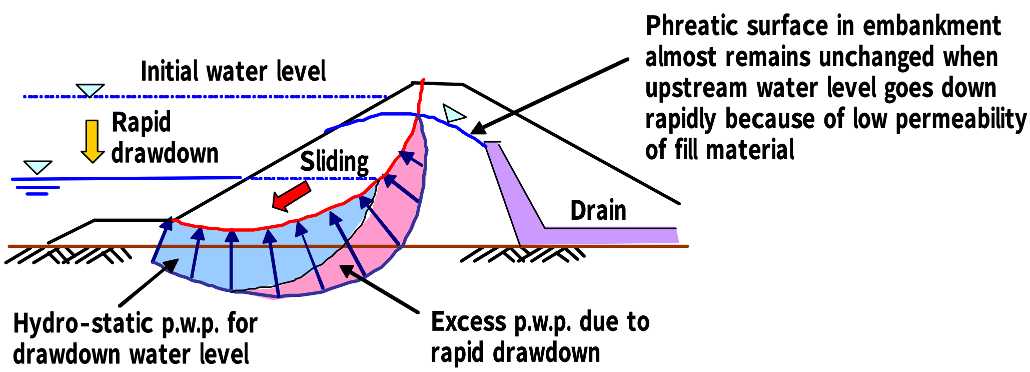


Fig.2.3 Excee Pore-Water Pressure due to Rapid Drawdown

2.3 Seepage Failure (Hydraulic Fracture)

When water flows passing through soil in an embankment and foundation, seepage forces act on soil particles due to its viscosity. If seepage forces acting in the soil are large enough as compared to the resisting forces based on the effective earth pressure, erosion by quick sand takes place by washing soil particles away from the surface, and piping successively develops as erosion gradually progresses.

In Fig.2.4(a), one of actions of seepage through pervious foundation is demonstrated, in which the uplift pressure acting on the impervious foundation causes heaving near the toe of the embankment. Hydraulic fracturing, quick sand and piping, can readily occur around the downstream toe when the hydraulic gradient increases with the concentration of flow lines, and the reduction in effective stresses is inevitable in the ground due to the action of the upward seepage forces, as illustrated in Fig.2.4(b).

In an actual dam design, adequate drainage facilities such as filter zones and drains are provided in the interior of the embankment, and piping failures as stated above would not be expected to occur in ordinary situations. One of unusual situations to be considered is the generation of interior cracks in the impervious zone and foundation, which is mainly caused by differential settlements during and after construction, as described in the following.

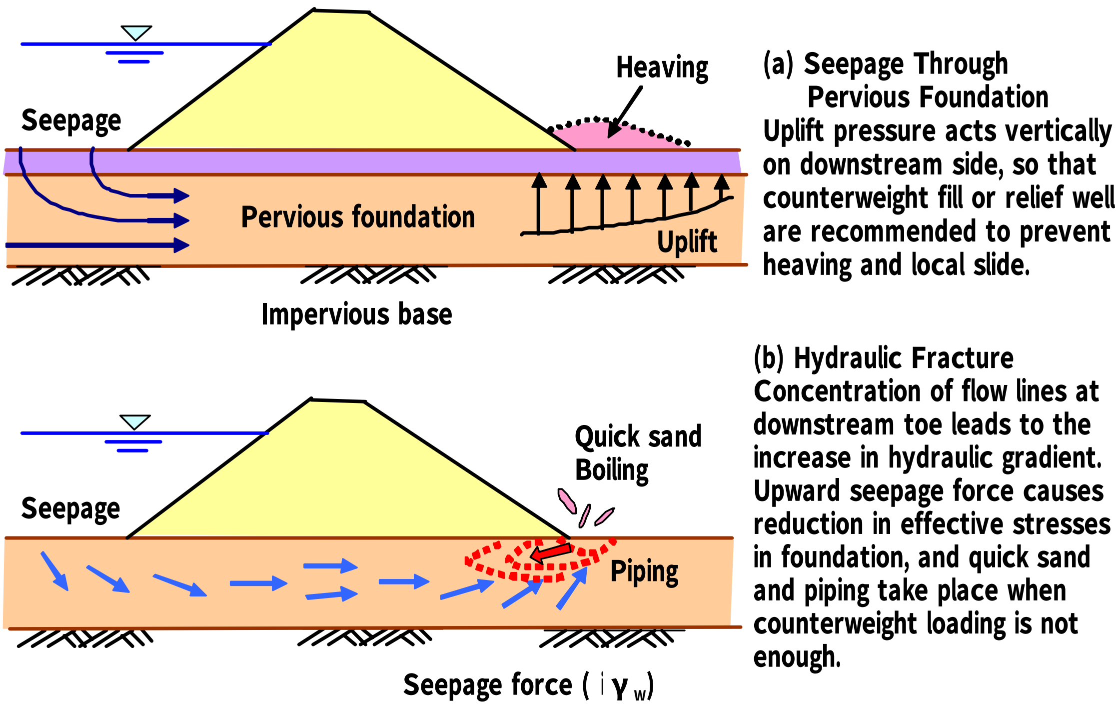


Fig.2.4 Seepage Through Foundation and Hydraulic Fracturing

2.4 Differential Settlement, Deformation and Cracking

Many types of differential settlement and associated severe deformation such as open cracks appear in both dam body and base foundation, due to compressibility of fill materials and foundation soils and/or their relative rigidity. Fig.2.5 shows several patterns of differential settlement and open cracks which dam engineers often encounter in the field.

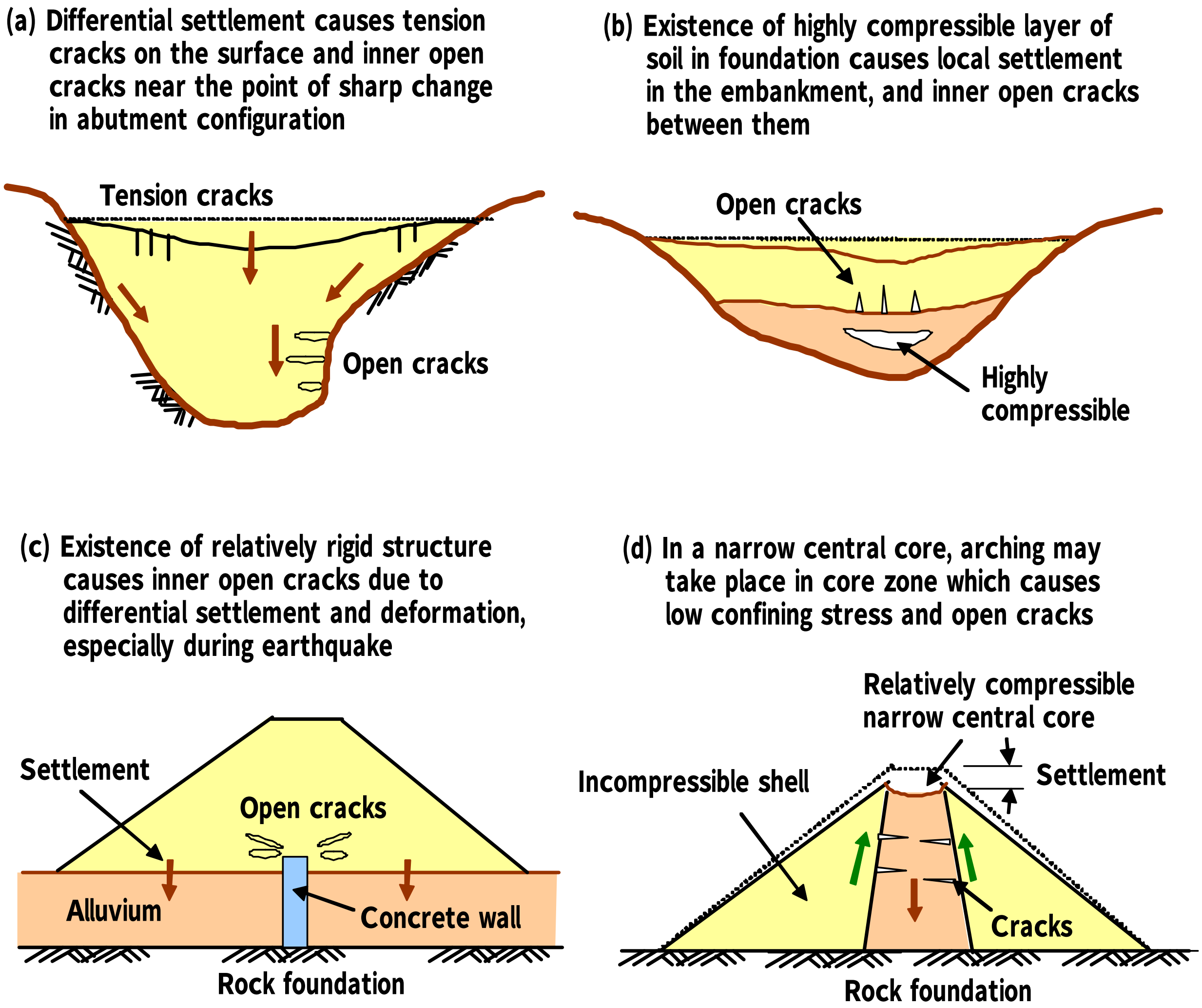


Fig.2.5 Differential Settlement and Cracking

2.5 Earthquake Damage

Embankment failures due to earthquake excitation can be classified into two groups. One is damages caused by liquefaction or softening of sand foundation and the other is sliding and cracking of embankment body resting on hard foundation. In the former case, high excess pore-water pressure is generated during earthquake by the application of cyclic shear stresses, and large deformation as well as vertical displacement develops in the foundation. These deformations generally lead to catastrophic damages due to overtopping, as shown in Fig.2.6.

According to the investigation reports on earthquake damages of actual embankment dams and also to the experimental studies through large scale shaking table tests on the dynamic response of earth and rockfill dams, embankment failures caused by strong excitation are classified into several patterns in their mechanism. Three distinct patterns of embankment failures due to earthquake excitation are schematically illustrated in Fig.2.7, for different types of embankment configuration.

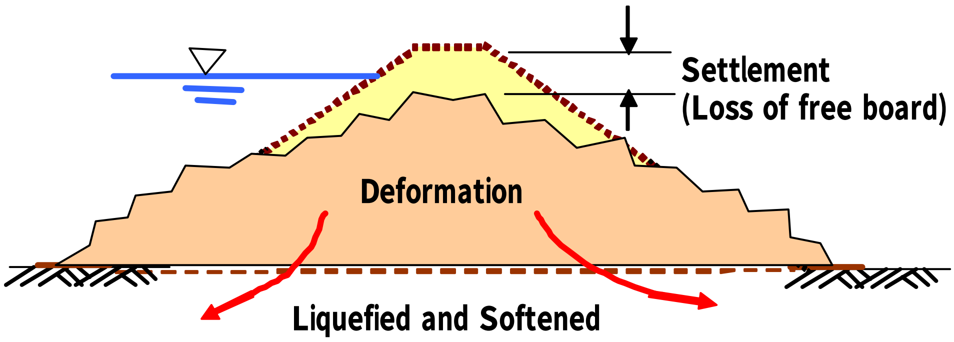
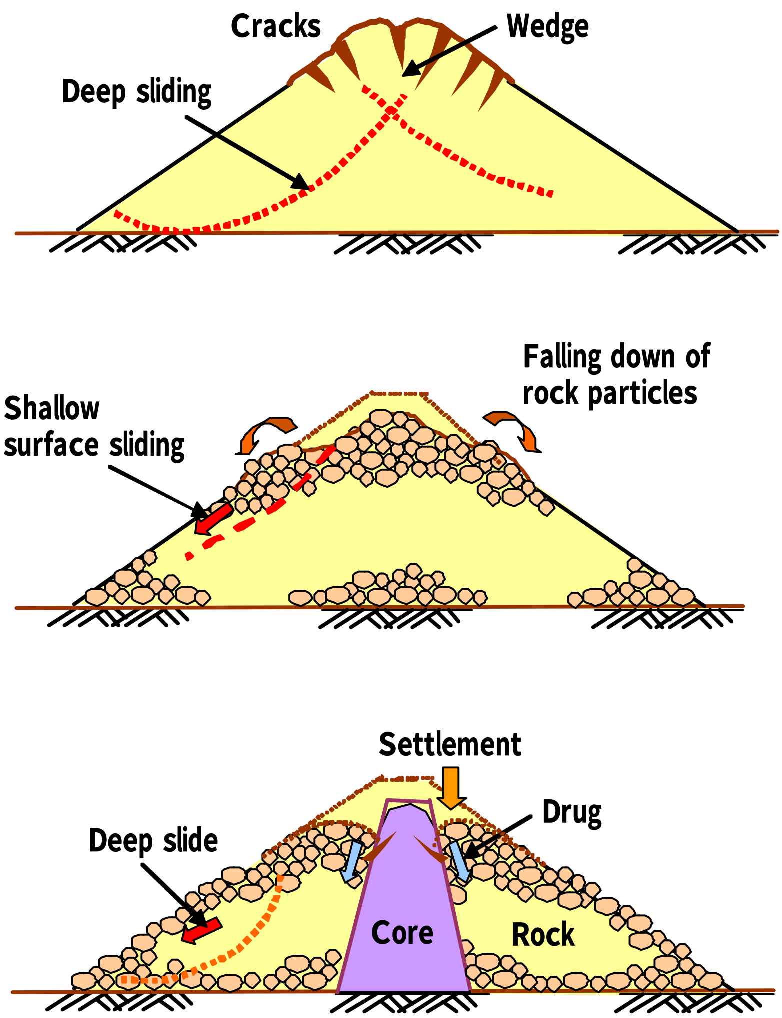


Fig.2.6   Embankment Failure due to Liquefaction of Foundation



(a) Homogeneous Earthfill Dam
A wedge type soil block is formed in the crest, and it grows up gradually as the excitation increases, causing deep circular sliding

(b) Homogeneous Rockfill Dam
The extent of falling down of rock particles along slope moves down from crest to toe, leading to a flat circular crest and lowering of the fill. Shallow surface sliding also appears during this process.

(c) Centally Located Core Type Rockfill Dam
Core zone is exposed at the crest due to settlement and drug of surrounding shells, causing decrease in lateral constraint and opening of cracks

Fig.2.7   Failure Patterns of Embankments on Hard Foundation

3. SHEAR STRENGTH OF FILL MATERIALS

3.1 Preface

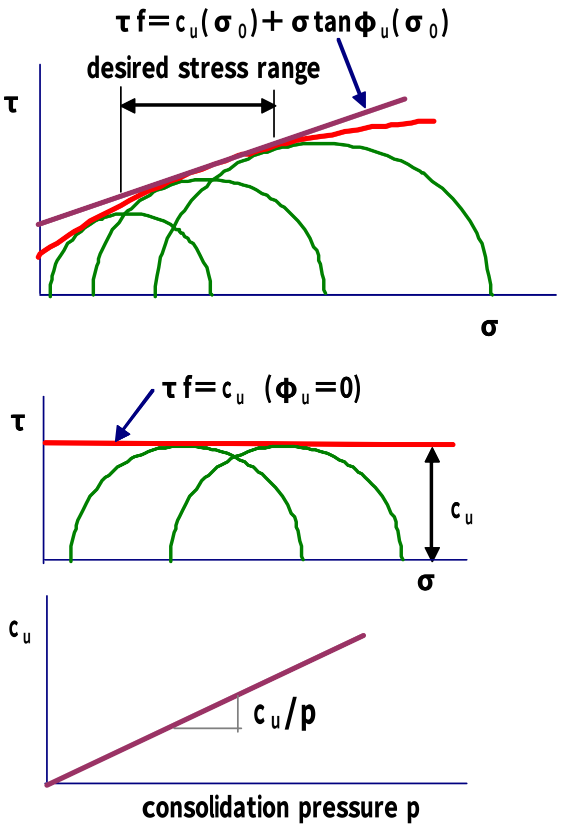
In the design of embankment dams, stability analysis of slopes is usually conducted for the following four typical situations.

1) analysis of the up- and downstream slopes immediately after the end of construction
2) analysis of the upstream slope at the first filling of the reservoir, for water level at about half of the full water level
3) analysis of the upstream slope during rapid drawdown of the reservoir water
4) analysis of the up- and downstream slopes when earthquake takes place at the full and an intermediate level of the reservoir water

In general, two types of stability analysis are performed according to the situation under consideration. One is the total stress analysis which is useful for the cases 1) and 2), and the other is the effective stress analysis which is applicable for the cases 3) and 4) mentioned above.

3.2 Total Stress Analysis

In the total stress analysis, undrained shear strength parameters c_u and ϕ_u are generally used for the failure criterion. For partially saturated soils, because the degree of saturation of soil specimen changes due to the change in the confining pressure σ_0 applied in the tests, both parameters c_u and ϕ_u are expressed as functions of σ_0, as shown in Fig.3.1. The shear strength τ_f is therefore given by the following equation.



Partially Saturated Soils:

- **Gradient of strength envelop decreases as confining pressure increases because of the increase in the degree of saturation of soil sample**
- **Because strength envelop has a curvature, c and φ are determined for an appropriate confining pressure σ_0 in the desired stress range**

Saturated Soils:

- **Undrained shear strength becomes constant for saturated soils**
- **Total stress analysis is then called as ϕ_u=0 analysis**
- **The value of c_u varies depending on the consolidation pressure p applied on the specimen**

Fig.3.1 Shear Strength in Total Stress Analysis

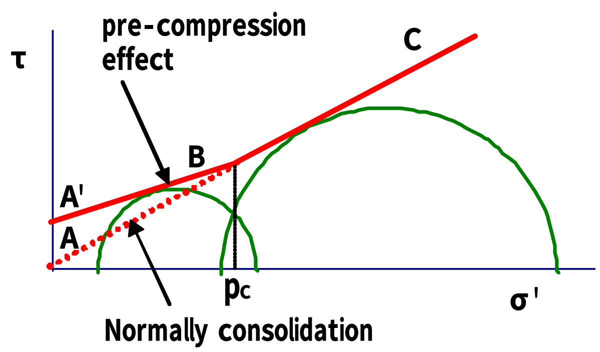
$$\tau_f = c_u(\sigma_0) + \sigma \tan\phi_u(\sigma_0) \qquad ......(3.1)$$

Parameters c_u and ϕ_u to be used in the design are usually determined for a desired stress range of σ_0 in the construction field. For saturated soils, because ϕ_u vanishes in undrained strength tests, the shear strength τ_f is represented as

$$\tau_f = c_u \qquad ......(3.2)$$

The undrained strength c_u of clay is directly dependent on the initial structure (void ratio) after sedimentation, which in turn is affected by the consolidation pressure p in the ground. The increasing rate of c_u to the effective pressure p is almost proportional, and the value of the ratio c_u/p becomes to be around 1/3 in normally consolidated clays.

Shear strength tests on compacted fill materials reveal that the strength envelop consists of a combination of two straight lines, as shown in Fig.3.2. This is due to the fact that compacted soils more or less can have high skeleton strength composed by suction effects and the strength becomes greater than that of normally consolidated state under a confining pressure lower than the pre-compression stress p_c. For practical purposes and for a safe-side design, however, the straight line ABC is usually used in the design to determine c_u and ϕ_u.



Compacted Fill Materials:
- **Compacted soils show higher strength A'B than that in normally consolidated sate AB under lower confining pressure than the pre-compression stress pc.**
- **Pre-compression effects vanish due to wetting of soil, so that c and φ to be used in the design are determined by AB for practical safe-side design.**

Fig.3.2 Pre-compression Effect of Compacted Fill Materials

3.3 Effective Stress Analysis

In the effective stress analysis, the failure criterion is represented in terms of effective stresses by the expression:

$$\tau_f = c' + \sigma'\tan\phi' = c' + (\sigma - u)\tan\phi' \qquad ......(3.3)$$

in which parameters c' and ϕ' are referred to, respectively, as the cohesion intercept and the angle of shearing resistance in terms of effective stresses, and they are determined from CU tests or drained tests (D tests) for saturated samples.

3.4 Shear Strength of Non-cohesive Rock Materials

Failure criterion of granular and non-cohesive materials, such as sand, gravel and rock, can usually be represented in any situations in terms of effective stresses without a cohesion intercept as follows;

$$\tau_f = (\sigma - u)\tan\phi' \qquad \text{......(3.4)}$$

It has recently been known through extensive researches on the shear strength properties of non-cohesive rock materials that they show a steep increase in strength in the range of very low confining pressure. In order to consider such a curvature of the failure envelop in a practical design, two types of expression of shear strength have been proposed, as shown in Fig.3.3. One is the same expression as Eq.(3.4) using, in place of constant ϕ', a variable of the tangential angle ϕ_0 from the origin. The other is the expression of a power of the effective confining pressure σ_n as $\tau_f = A(\sigma_n)b$. Variation of ϕ_0 and the coefficients A and b in the latter are given as in Fig.3.3.

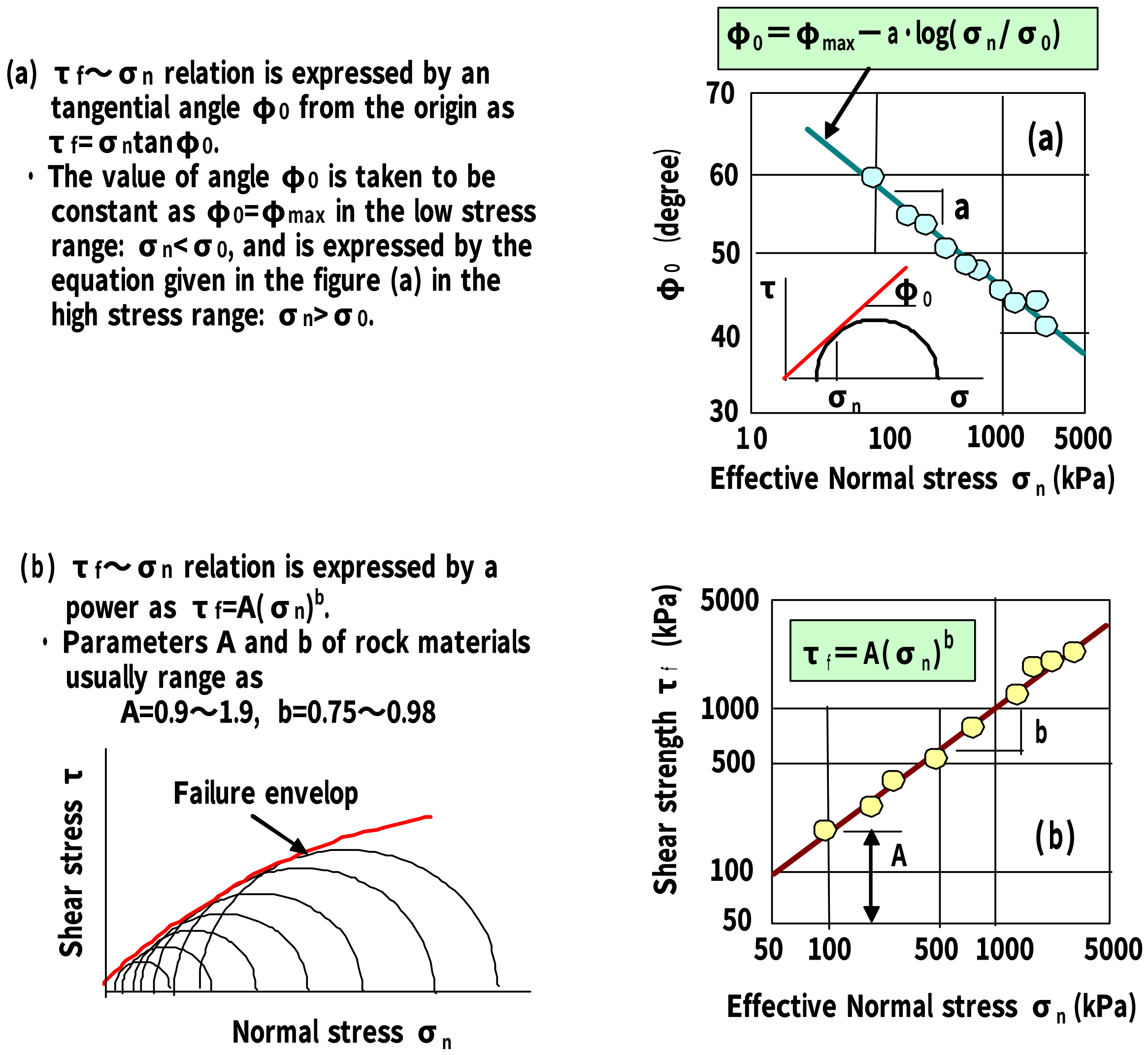


Fig.3.3 Shear Strength of Non-cohesive Rock Materials

4. COMPACTION OF FILL MATERIALS

4.1 Preface

Compaction of soils is one of methods of soil stabilization. Principal purposes of compaction of fill materials are

1) increasing stiffness, to minimize settlements during and after construction,
2) increasing strength, to prevent sliding shear failure of embankment, and
3) making water tight, to obtain required imperviousness of the core zone

Variations of grain size distribution curves of fill materials employed in twelve representative earth and rockfill dams in Japan are shown in Fig.4.1. Soil particles of fairly large grain size are used even as impervious core material because of recent development of heavy equipment and techniques for construction control.

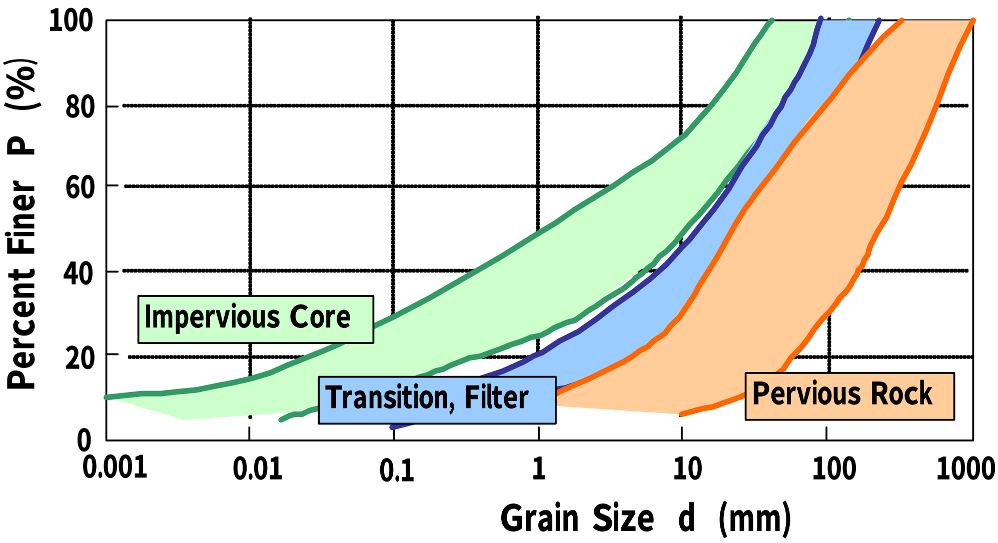


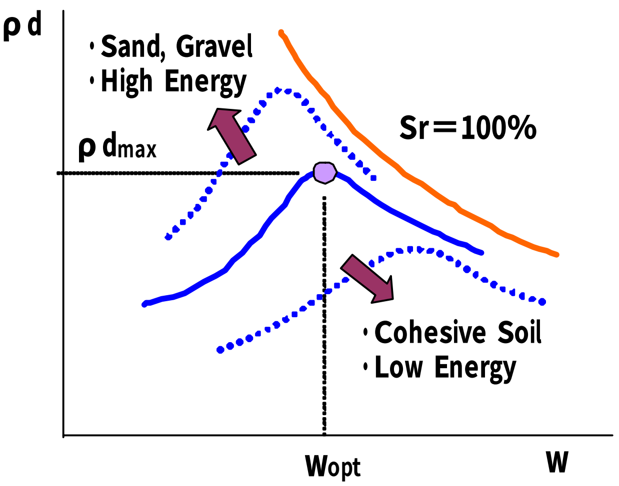
Fig.4.1 Representative Fill Materials

When soil is compacted, with a constant energy, by increasing water content step by step, dry density of soil increases first monotonically because lubrication effect of water allows particle movement. Density of soil decreases, however, as water content increases beyond a certain value of wopt because more part of void of soil is filled with water which interferes filling of soil particles. Compaction curve is then formed like a mountain where peak point is defined by the maximum dry density ρ_{dmax} and the optimum water content wopt. General characteristic of compaction curves are described in Fig.4.2.

4.2 Compressibility

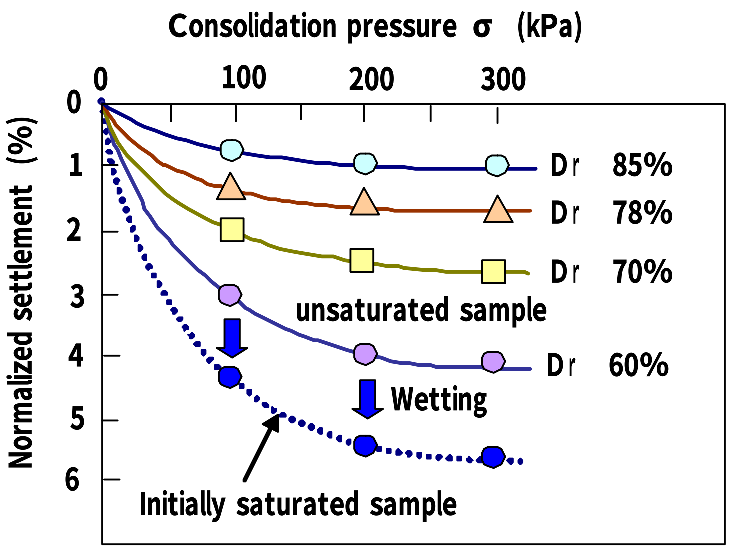
In Fig.4.3, the results from compression tests performed on compacted granular materials are summarized for MAKIO Dam materials, showing relations between settlements and applied stresses for different values of relative density. It is seen that compressibility of fill materials is gradually improved as compaction dry density increases.

It is generally known that soils compacted in unsaturated states, especially in the dry side of the optimum water content, have a certain skeleton strength composed by suction effect between soil particles. This skeleton strength readily disappears by wetting (saturation) during the first filling of the reservoir, which results in large settlement and drugs in the upstream shell of a rockfill dam and also in differential settlement and opening cracks in the core zone. Compressibility due to wetting can be easily evaluated by conducting compression tests on saturated samples compacted in the same dry density.



- Dry density of soil increases first as water content increases, because lubrication effect of water allows particle movement
- Density decrease beyond wopt, because void is filled by water which can interfere movement of soil particles
- Compaction curve becomes sharp and moves left (to dry side) for granular (non-cohesive) soils like sand and gravel, and when compaction energy is high
- Sharping of comaction curve reveals that variation of dry density becomes sensitive to the change in water content
- Granular soils can have a certain suction forces with less water content because contact points of particles are fewer than cohesive soils

Fig.4.2 General Characteristics of Compaction Curves



* Relation of ρd and e

$\rho d = \rho s/(1+e)$

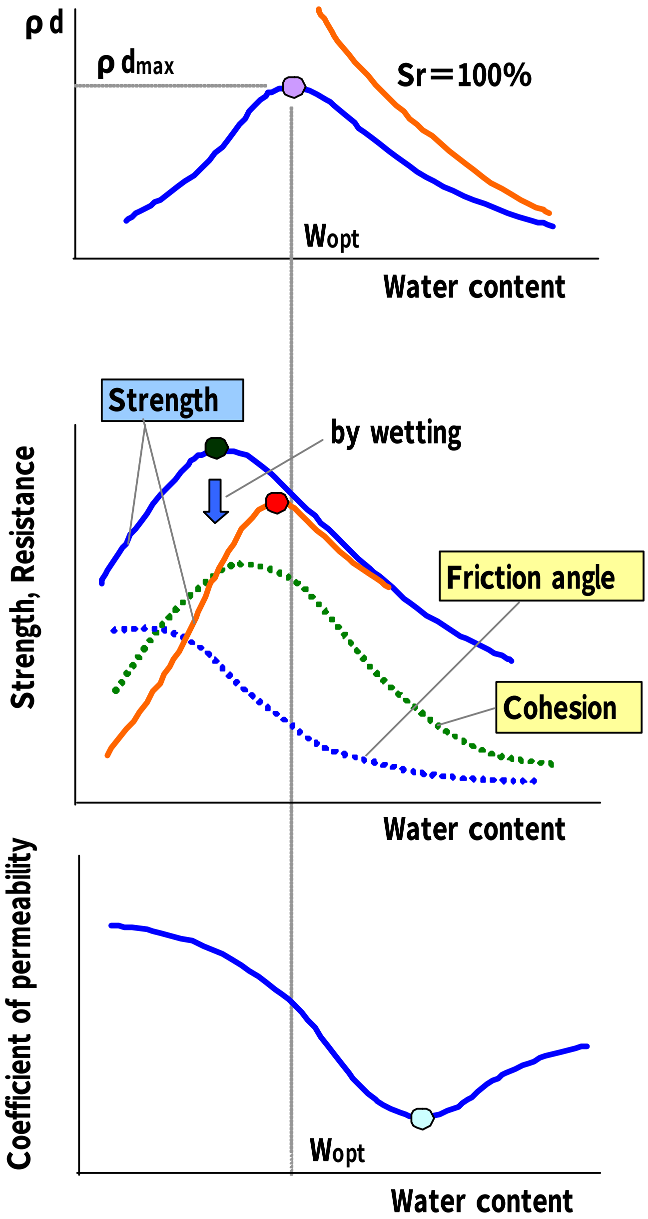
* Relative Density

$Dr = (e_{max} - e) / (e_{max} - e_{min})$

Fig.4.3 Compressibility of Compacted Soils

4.3 Strength and Permeability

Characteristics of shear strength of compacted soil are almost the same as those of compressibility mentioned above. Correlations between shear strength and permeability of compacted soils and the compaction dry density and water content are summarized in Fig.4.4. Strength of compacted soils generally shows a peak on the dry side of the optimum water content. Severe strength reduction, however, is anticipated on the dry side when the fill is submerged, so that dry side compaction should be avoided especially in the upstream fill. The permeability of compacted soils, on the other hand, has a minimum peak on the wet side of the optimum.



Strength

- Strength of compacted soil in general shows a peak value on the dry side.
- Friction and cohesion components also have large values on the dry side.
- Strength decreases drastically due to wetting on the dry side, and the peak of strength moves to the optimum.
- Dry side compaction therefore should be avoided especially in the upstream fill, which causes large settlement and drug against the core.
- High energy compaction with heavy roller is recommended for very dry fill materials to have the optimum state at low water content.

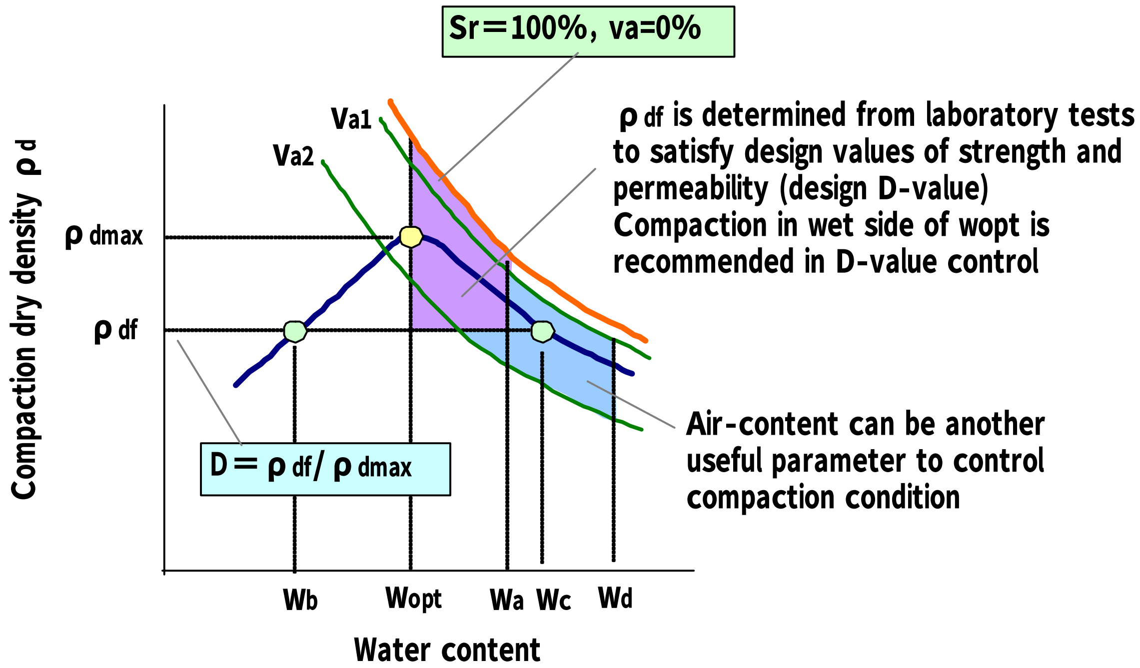
Permeability

- Permeability of compacted soil decreases as water content (degree of saturation) increases, having the minimum value on the wet side of the optimum.

Fig.4.4 Strength and Permeability of Compacted Soils

4.4 Compaction Control

In order to maintain required strength and permeability in the field, compaction conditions should be specified in the design stage and placement process is severely controlled in the construction stage. In general, strength and permeability of compacted soils are related in advance with dry density by use of laboratory test results. Required field compaction dry density ρ_{df} is then determined to satisfy design values of strength and permeability, usually expressed by *D*-value as the ratio of ρ_{df} to ρ_{dmax}. Considering wet side compaction and allowable field water content, compaction is controlled to maintain dry density greater than the specified *D*-value, as presented in Fig.4.5. For fairly compressible materials, in which *D*-value control is difficult to be applied, air-content can be another useful index parameter to control compaction condition.



ρ_{dmax}: Maximum dry density
w_{opt}: Optimum water content
(wet side compaction is recommended)
ρ_{df}: Required field compaction dry density to satisfy design values of strength and permeability
wb～wc: Field water content to obtain required D-value
wa: Allowable field water content
wd: Upper limit of field water content

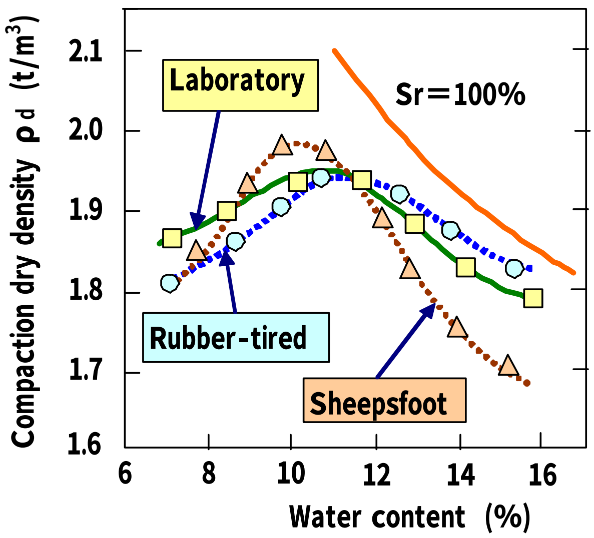
Fig.4.5 Idea of Compaction Control

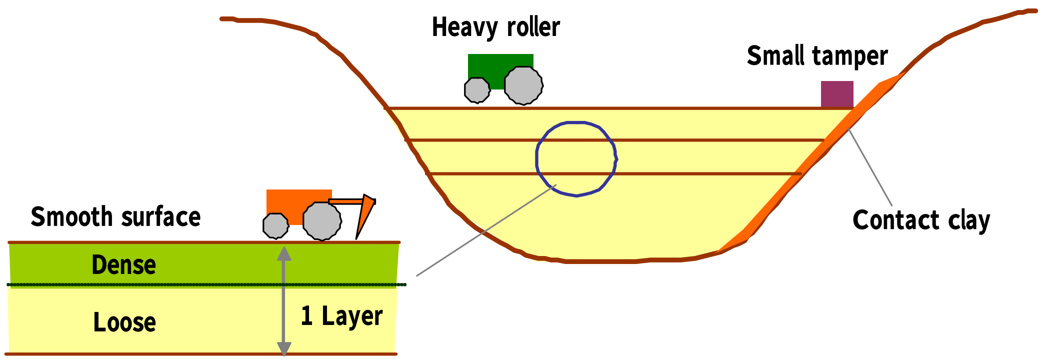
4.5 Laboratory and Field Compaction

Field compaction of fill materials is usually carried out by using compaction rollers such as a tamping (sheepsfoot) roller, a rubber-tired roller and a vibration roller. In general, either a sheepsfoot roller or a rubber-tired roller is used for impervious or semi-impervious materials. For pervious materials such as sand, gravel and rock, rubber-tired or vibration rollers are usually employed. The usefulness of each roller depends on its compaction characteristics and soil types. The results of field compaction tests are compared with those of laboratory standard Procter compaction tests in Fig.4.6.

The surface of the fill under compaction likely becomes smooth when a rubber-tired or a vibration roller is employed. This is remarkable when comparatively soft rock materials are compacted by a vibration roller or when impervious materials are compacted by a rubber-tired roller. It is readily recognized that the formation of smooth surface is undesirable in the stability of embankment slopes, causing reduction in shear resistance along this plane. For narrow areas adjacent to the abutment foundation and concrete structures, small compactors such as hand and air tamper are effectively used in place of heavy equipment.

Fig.4.6
Laboratory and Field Compaction





- Thickness of soil to be compacted in a layer is important to keep good quality of fill, because densely and loosely compacted zones exist even in one layer
- Surface treatment is needed when smoothly compacted or after heavy rain, in order to avoid reduction in shear resistance in the fill
- Small compactors such as hand and air tamper are effective for use in compaction near abutment foundation

4.6 Particle Breakage

Breakage of solid particles is one of important issues to be considered in the design and construction of rockfill dams. Marsal has proposed a practically useful index of grain breakage, *Bg*, to classify rockfill materials. The index *Bg* is defined, as illustrated in Fig.4.7, by the sum of positive values of weight differences of the initial and final fractions of rock particles. Fig.4.8 demonstrates a classification of rockfills, where the particle strength, hardness, of rockfill is related to the *Bg*-value obtained in triaxial and unconfined compression tests.

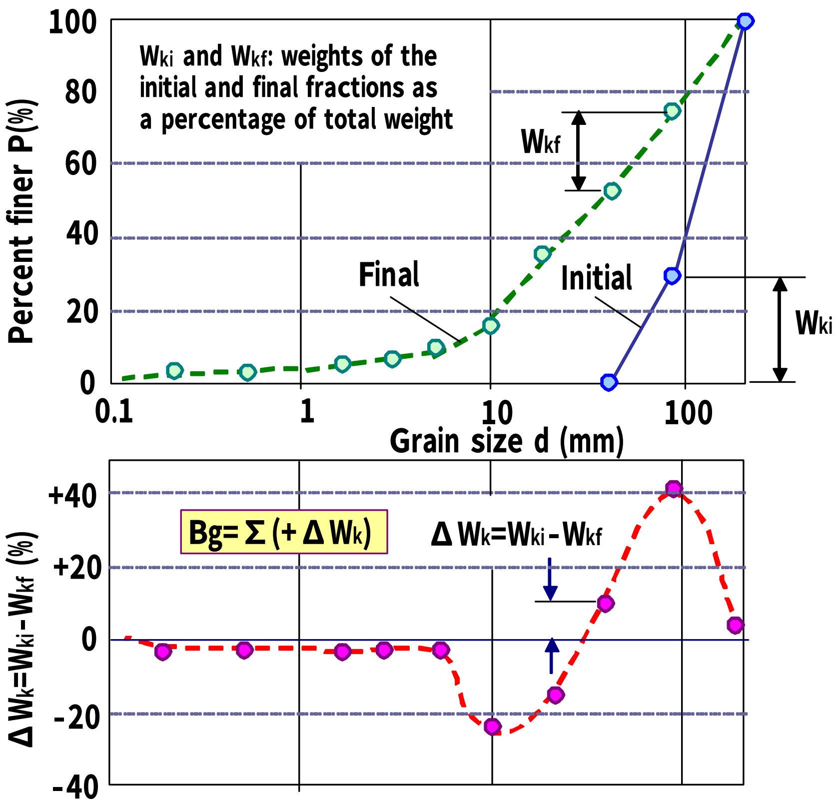


Fig.4.7　Definition of grain breakage Bg

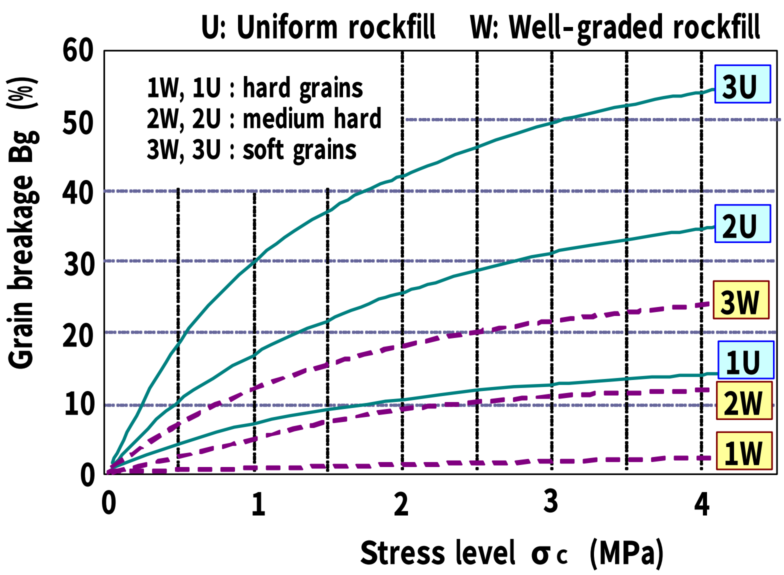


Fig.4.8　relation between Bg-index and hardness of rockfi