# Dredging Processes

The Cutting of Sand, Clay & Rock Soil Mechanics

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# Preface

Lecture notes for the course OE4626 Dredging Processes, for the MSc program Offshore & Dredging Engineering, at the Delft University of Technology.

#### By Dr.ir. Sape A. Miedema, Sunday, January 13, 2013

In dredging, trenching, (deep sea) mining, drilling, tunnel boring and many other applications, sand, clay or rock has to be excavated. The productions (and thus the dimensions) of the excavating equipment range from mm<sup>3</sup>/sec - cm<sup>3</sup>/sec to m<sup>3</sup>/sec. In oil drilling layers with a thickness of a magnitude of 0.2 mm are cut, while in dredging this can be of a magnitude of 0.1 m with cutter suction dredges and meters for clamshells and backhoe's. Some equipment is designed for dry soil, while others operate under water saturated conditions. Installed cutting powers may range up to 10 MW. For both the design, the operation and production estimation of the excavated it is usually transported hydraulically as a slurry over a short (TSHD's) or a long distance (CSD's). Estimating the pressure losses and determining whether or not a bed will occur in the pipeline is of great importance. Fundamental processes of sedimentation, initiation of motion and ersosion of the soil particles determine the transport process and the flow regimes. In TSHD's the soil has to settle during the loading process, where also sedimentation and ersosion will be in equilibrium. In all cases we have to deal with soil and high density soil water mixtures and its fundamental behavior.

In order to understand the cutting processes in sand, clay and rock, it is required to have knowledge of basic soil and rock mechanics. This lecture note covers this knowledge and has been composed entirely from information from the public domain, especially internet. Most information comes from Wikipedia and Answers.com and from the references below.

This book will enable engineers to understand some basic soil mechanics in order to determine the interaction between the soil and the excavating equipment and is the  $2^{nd}$  of 7 books.

Part 1: The Cutting of Sand, Clay & Rock - Excavating Equipment

Part 2: The Cutting of Sand, Clay & Rock – Soil Mechanics

Part 3: The Cutting of Sand, Clay & Rock - Theory

Part 4: The Loading Process of a Trailing Suction Hopper Dredge.

Part 5: The Initiation of Motion of Particles.

Part 6: Hydraulic Transport – Theory.

Part 7: Hydraulic Transport – Experiments.

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#### **Chapter 1: Introduction**

#### **1.1. Soil Mechanics**

McGraw-Hill Science & Technology Encyclopedia gives the following description of Soil Mechanics:

The study of the response of masses composed of soil, water, and air to imposed loads. Because both water and air are able to move through the soil pores, the discipline also involves the prediction of these transport processes. Soil mechanics provides the analytical tools required for foundation engineering, retaining wall design, highway and railway sub base design, tunneling, earth dam design, mine excavation design, and so on. Because the discipline relates to rock as well as soils, it is also known as geotechnical engineering.

Soil consists of a multiphase aggregation of solid particles, water, and air. This fundamental composition gives rise to unique engineering properties, and the description of the mechanical behavior of soils requires some of the most sophisticated principles of engineering mechanics. The terms multiphase and aggregation both imply unique properties. As a multiphase material, soil exhibits mechanical properties that show the combined attributes of solids, liquids, and gases. Individual soil particles behave as solids, and show relatively little deformation when subjected to either normal or shearing stresses. Water behaves as a liquid, exhibiting little deformation under normal stresses, but deforming greatly when subjected to shear. Being a viscous liquid, however, water exhibits a shear strain rate that is proportional to the shearing stresses. When the three phases are combined to form a soil mass, characteristics that are an outgrowth of the interaction of the phases are manifest. Moreover, the particulate nature of the solid particles contributes other unique attributes.

When dry soil is subjected to a compressive normal stress, the volume decreases nonlinearly; that is, the more the soil is compressed, the less compressible the mass becomes. Thus, the more tightly packed the particulate mass becomes, the more it resists compression. The process, however, is only partially reversible, and when the compressive stress is removed the soil does not expand back to its initial state.

When this dry particulate mass is subjected to shear stress, an especially interesting behavior owing to the particulate nature of the soil solids results. If the soil is initially dense (tightly packed), the mass will expand because the particles must roll up and over each other in order for shear deformation to occur. Conversely, if the mass is initially loose, it will compress when subjected to a shear stress. Clearly, there must also exist a specific initial density (the critical density) at which the material will display zero volume change when subjected to shear stress. The term dilatancy has been applied to the relationship between shear stress and volume change in particulate materials. Soil is capable of resisting shear stress up to a certain maximum value. Beyond this value, however, the material undergoes large, uncontrolled shear deformation.

The other limiting case is saturated soil, that is, a soil whose voids are entirely filled with water. When such a mass is initially loose and is subjected to compressive normal stress, it tends to decrease in volume; however, in order for this volume decrease to occur, water must be squeezed from the soil pores. Because water exhibits a viscous resistance to flow in the microscopic pores of fine-grained soils, this process can require considerable time, during which the pore water is under increased pressure. This excess pore pressure is at a minimum near the drainage face of the soil mass and at a maximum near the center of the soil sample. It is this gradient (or change in pore water pressure with change in position within the soil mass) that causes the outflow of water and the corresponding decrease in volume of the soil mass.

Conversely, if an initially dense soil mass is subjected to shear stress, it tends to expand. The expansion, however, may be time-dependent because of the viscous resistance to water being drawn into the soil pores. During this time the pore water will be under decreased pressure. Thus, in saturated soil masses, changes in pore water pressure and time-dependent volume change can be induced by either changes in normal stress or by changes in shear stress.

#### 1.2. Soil Creation

The primary mechanism of soil creation is the weathering of rock. All rock types (igneous rock, metamorphic rock and sedimentary rock) may be broken down into small particles to create soil. Weathering mechanisms are physical weathering, chemical weathering, and biological weathering. Human activities such as excavation, blasting, and waste disposal, may also create soil. Over geologic time, deeply buried soils may be altered by pressure and temperature to become metamorphic or sedimentary rock, and if melted and solidified again, they would complete the geologic cycle by becoming igneous rock.

Physical weathering includes temperature effects, freeze and thaw of water in cracks, rain, wind, impact and other mechanisms. Chemical weathering includes dissolution of matter composing a rock and composition of soils. Physical weathering includes temperature effects, freeze and thaw of water in cracks, rain, wind, impact and other mechanisms. Chemical weathering includes dissolution of matter composing a rock and precipitation

in the form of another mineral. Clay minerals, for example can be formed by weathering of feldspar, which is the most common mineral present in igneous rock. The most common mineral constituent of silt and sand is quartz, also called silica, which has the chemical name silicon dioxide. The reason that feldspar is most common in rocks but silicon is more prevalent in soils is that feldspar is much more soluble than silica. Silt, Sand, and Gravel are basically little pieces of broken rocks. According to the Unified Soil Classification System, silt particle sizes are in the range of 0.002 mm to 0.075 mm and sand particles have sizes in the range of 0.075 mm to 4.75 mm. Gravel particles are broken pieces of rock in the size range 4.75 mm to 100 mm. Particles larger than gravel are called cobbles and boulders.

Soil deposits are affected by the mechanism of transport and deposition to their location. Soils that are not transported are called residual soils -- they exist at the same location as the rock from which they were generated. Decomposed granite is a common example of a residual soil. The common mechanisms of transport are the actions of gravity, ice, water, and wind. Wind-blown soils include dune sands and loess. Water carries particles of different size depending on the speed of the water, thus soils transported by water are graded according to their size. Silt and clay may settle out in a lake, and gravel and sand collect at the bottom of a river bed. Wind-blown soil deposits (aeolian soils) also tend to be sorted according to their grain size. Erosion at the base of glaciers is powerful enough to pick up large rocks and boulders as well as soil; soils dropped by melting ice can be a well graded mixture of widely varying particle sizes. Gravity on its own may also carry particles down from the top of a mountain to make a pile of soil and boulders at the base; soil deposits transported by gravity are called colluvium.

The mechanism of transport also has a major effect on the particle shape. For example, low velocity grinding in a river bed will produce rounded particles. Freshly fractured colluvium particles often have a very angular shape.



Figure 1-1: Fox Glacier, New Zealand: Soil creation and transportation.

#### **1.3.** Soil Classification

**Soil classification** deals with the systematic categorization of soils based on distinguishing characteristics as well as criteria that dictate choices in use.

**Soil texture** is a qualitative classification tool used in both the field and laboratory to determine classes for agricultural soils based on their physical texture. The classes are distinguished in the field by the 'textural feel' which can be further clarified by separating the relative proportions of sand, silt and clay using grading sieves: The Particle Size Distribution (PSD). The class is then used to determine crop suitability and to approximate the soils responses to environmental and management conditions such as drought or calcium (lime) requirements. A qualitative rather than a quantitative tool it is a fast, simple and effective means to assess the soils physical characteristics. Although the U.S.D.A. system uses 12 classes whilst the U.K.-ADAS uses just 11 the systems are mutually compatible as shown in the combined soil textural triangle below.



Figure 1-2: USDA and UK-ADAS textural triangle

Hand analysis, whilst an arbitrary technique, is an extremely simple and effective means to rapidly assess and classify a soils physical condition. Correctly executed the procedure allows for rapid and frequent assessment of soil characteristics with little or no equipment. It is thus an extremely useful tool for identifying spatial variation both within and between plots (fields) as well as identifying progressive changes and boundaries between soil classes and orders.

The method involves taking a small sample of soil, sufficient to roll into a ball of approximately 2.5 cm diameter, from just below the surface. Using a small drop of water or 'spit' the sample is then moisten to the sticky point (the point at which it begins to adhere to the finger). The ball is then molded to determine its workability and its class according to the steps in the chart opposite.

Soil separates are specific ranges of particle sizes. In the United States, the smallest particles are clay particles and are classified by the USDA as having diameters of less than 0.002 mm. The next smallest particles are silt particles and have diameters between 0.002 mm and 0.05 mm. The largest particles are sand particles and are

larger than 0.05 mm in diameter. Furthermore, large sand particles can be described as coarse, intermediate as medium, and the smaller as fine. Other countries have their own particle size classifications.

Table 1-1. Son Classification.						
Nome of soil concrete	Diameter limits (mm)					
Name of son separate	(USDA classification)					
Clay	less than 0.002					
Silt	0.002–0.05					
Very fine sand	0.05–0.10					
Fine sand	0.10-0.25					
Medium sand	0.25–0.50					
Coarse sand	0.50–1.00					
Very coarse sand	1.00-2.00					

Table 1-1: Soil Classification.

Australia	2		20			200			200
Belgium —	2	10	20	50	100	200	500	1000	200
Denmark	2		20	63	125	200	500		200
France —	•			50		~~~~~	11112291		200
Germany —	2	(6.3)	20	63	(125	5) 200	500		200
Greece —	2	6	20	60		200	600	2	200
Italy —	2		58.63	50					200
The Netherlands —	2		16	50	105 1	50 210 30	00		200
Portugal	2		20	693	175.0	200	550		200
Slovac Republic —	69 1 1	10	1993	50		250			200
Spain —	2		20						200
Sweden	2	6	20	60		200	600	0	200
England & Wales	2		20	60	100	200	600	)	200
Nothern Ireland	2		1975-	60	10.57.57/10 	199202		55	200
Scotland —	2			60					200
USA							•		-

Figure 1-3: Particle size classifications used by different countries, diameters in µm.

#### **Chapter 2: Soils**

#### **2.1. Sand**

Sand is any material composed of loose, stony grains between 1/16 mm and 2 mm in diameter. Larger particles are categorized as gravel; smaller particles are categorized as silt or clay. Sands are usually created by the breakdown of rocks, and are transported by wind and water, before depositing to form soils, beaches, dunes, and underwater fans or deltas. Deposits of sand are often cemented together over time to form sandstones.

The most common sand-forming process is weathering, especially of granite. Granite consists of distinct crystals of quartz, feldspar, and other minerals. When exposed to water, some of these minerals (e.g., feldspar) decay chemically faster than others (especially quartz), allowing the granite to crumble into fragments. Sand formed by weathering is termed epiclastic.

Where fragmentation is rapid, granite crumbles before its feldspar has fully decayed and the resulting sand contains more feldspar. If fragmentation is slow, the resulting sand contains less feldspar. Fragmentation of rock is enhanced by exposure to fast-running water, so steep mountains are often source areas for feldspar-rich sands and gentler terrains are often source areas for feldspar-poor sands. Epiclastic sands and the sandstones formed from them thus record information about the environments that produce them. A sedimentologist can deduce the existence of whole mountain ranges long ago eroded, and of mountain-building episodes that occurred millions of years ago from sandstones rich in relatively unstable minerals like feldspar.

The behavior of sand carried by flowing water can inscribe even more detailed information about the environment in sand deposits. When water is flowing rapidly over a horizontal surface, any sudden vertical drop in that surface splits the current into two layers, (1) an upper layer that continues to flow downstream and (2) a slower backflow that curls under in the lee of the drop-off. Suspended sand tends to settle out in the backflow zone, building a slope called a "slip face" that tilts downhill from the drop-off. The backflow zone adds continually to the slip face, growing it downstream, and as the slip face grows downstream its top edge continues to create a backflow zone. The result is the deposition of a lengthening bed of sand. Typically, periodic avalanches of large grains down the slip face (or other processes) coat it with thin layers of distinctive material. These closely-spaced laminations are called "cross bedding" because they angle across the main bed. Cross-bedding in sandstone records the direction of the current that deposited the bed, enabling geologists to map currents that flowed millions of years ago (paleocurrents).

Evidence of grain size, bed thickness, and cross-bedding angle, allows geologists to determine how deep and fast a paleocurrent was, and thus how steep the land was over which it flowed.

Ripples and dunes—probably the most familiar forms created by wind- or waterborne sand—involve similar processes. However, ripples and dunes are more typical of flow systems to which little or no sand is being added. The downstream slip faces of ripples and dunes are built from grains plucked from their upstream sides, so these structures can migrate without growing. When water or wind entering the system (e.g., water descending rapidly from a mountainous region) imports large quantities of sand, the result is net deposition rather than the mere migration of sand forms.

Grain shape, too, records history. All epiclastic grains of sand start out angular and become more rounded as they are polished by abrasion during transport by wind or water. Quartz grains, however, resist wear. One trip down a river is not enough to thoroughly round an angular grain of quartz; even a long sojourn on a beach, where grains are repeatedly tumbled by waves, does not suffice. The well-rounded state of many quartz sands can be accounted for only by crustal recycling. Quartz grains can survive many cycles of erosion, burial and cementation into sandstone, uplift, and re-erosion. Recycling time is on the order of 200 million years, so a quartz grain first weathered from granite 2.4 billion years ago may have gone through 10 or 12 cycles of burial and re-erosion to reach its present day state. An individual quartz grain's degree of roundness is thus an index of its antiquity. Feldspar grains can also survive recycling, but not as well, so sand that has been recycled a few times consists mostly of quartz.

Sand can be formed not only by weathering but by explosive volcanism, the breaking up of shells by waves, the cementing into pellets of finer-grained materials (pelletization), and the precipitation of dissolved chemicals (e.g., calcium carbonate) from solution.

Pure quartz sands are mined to make glass and the extremely pure silicon employed in microchips and other electronic components.

#### 2.2. Clay

Clay is a fine-grained (small particle size) sedimentary rock. Clay is so fine-grained it is rarely possible to see the individual mineral particles with the naked eye. The definition of clays describes rocks with particle sizes of

less than 4 µm in diameter. Most sedimentary rocks are described using both mineral content and particle size. While this is also true for clays, the particle size description is most reliable and most often used.

The majority of common types of minerals found in clays are kaolinite (a soapy-feeling and lightweight mineral), talc, pyrophyllite, all types of micas, minerals from the chlorite group, feldspars, and a lesser amount of tectosilicates (including quartz).

The mineral content of clays is less variable than other types of sedimentary rock. This is a direct result of the way clays are formed. Water carries the bulk of sediments to their resting place where they are cemented together. The transport of sediments is directly related to the force or velocity of water carrying them. The stronger the velocity of water, the larger and heavier the particle it can move. Conversely, the weaker the flow, the smaller the particle that is carried by the water. As a result, water acts as a winnowing filter for certain types of minerals. The heavier minerals are not carried as far by water currents as are the lighter ones. When water finally comes to rest, it deposits its load of minerals. The last to be released are the lighter and smaller particles, the clay minerals.

Where rivers meet oceans, the clay minerals are so light they are usually carried far out to sea where they fall gently to the bottom forming a fine-grained sediment. These deposits cover organic materials and trap them at the edges of deltas and continental slopes. Over millions of years, the organic materials convert to petroleum and remain trapped by the clays. This relationship makes the study of clays extremely important for petroleum geologists. In addition to this important economic consideration, clays provide important economic resources for a wide variety of other industries.

Depending on the academic source, there are three or four main groups of clays: kaolinite, montmorillonite, smectite, illite, and chlorite. Chlorites are not always considered a clay, sometimes being classified as a separate group within the phyllosilicates. There are approximately 30 different types of "pure" clays in these categories, but most "natural" clays are mixtures of these different types, along with other weathered minerals.

Varve (or varved clay) is clay with visible annual layers, formed by seasonal differences in erosion and organic content. This type of deposit is common in former glacial lakes. When glacial lakes are formed there is very little movement of the water that makes the lake, and these eroded soils settle on the lake bed. This allows such an even distribution on the different layers of clay.

Quick clay is a unique type of marine clay indigenous to the glaciated terrains of Norway, Canada, Northern Ireland, and Sweden. It is highly sensitive clay, prone to liquefaction, which has been involved in several deadly landslides.

Clays exhibit plasticity when mixed with water in certain proportions. When dry, clay becomes firm and when fired in a kiln, permanent physical and chemical changes occur. These reactions, among other changes, cause the clay to be converted into a ceramic material. Because of these properties, clay is used for making pottery items, both utilitarian and decorative. Different types of clay, when used with different minerals and firing conditions, are used to produce earthenware, stoneware, and porcelain. Prehistoric humans discovered the useful properties of clay, and one of the earliest artifacts ever uncovered is a drinking vessel made of sun-dried clay. Depending on the content of the soil, clay can appear in various colors, from a dull gray to a deep orange-red.

Clay tablets were used as the first known writing medium, inscribed with cuneiform script through the use of a blunt reed called a stylus.

Clays sintered in fire were the first form of ceramic. Bricks, cooking pots, art objects, dishware, and even musical instruments such as the ocarina can all be shaped from clay before being fired. Clay is also used in many industrial processes, such as paper making, cement production, and chemical filtering. Clay is also often used in the manufacture of pipes for smoking tobacco. Until the late 20th century bentonite clay was widely used as a mold binder in the manufacture of sand castings.

Clay, being relatively impermeable to water, is also used where natural seals are needed, such as in the cores of dams, or as a barrier in landfills against toxic seepage (lining the landfill, preferably in combination with geotextiles).

Recent studies have investigated clay's absorption capacities in various applications, such as the removal of heavy metals from waste water and air purification.

#### 2.3. Rock

To the geologist, the term rock means a naturally occurring aggregate of minerals that may include some organic solids (e.g., fossils) and/or glass. Rocks are generally subdivided into three large classes: igneous, sedimentary, and metamorphic. These classes relate to common origin, or genesis. Igneous rocks form from cooling liquid rock or related volcanic eruptive processes. Sedimentary rocks form from compaction and cementation of sediments. Metamorphic rocks develop due to solid-state, chemical and physical changes in pre-existing rock because of elevated temperature, pressure, or chemically active fluids.

With igneous rocks, the aggregate of minerals comprising these rocks forms upon cooling and crystallization of liquid rock. As crystals form in the liquid rock, they become interconnected to one another like jigsaw puzzle pieces. After total crystallization of the liquid, a hard, dense igneous rock is the result. Also, some volcanic

lavas, when extruded on the surface and cooled instantaneously, will form a natural glass. Glass is a mass of disordered atoms, which are frozen in place due to sudden cooling, and is not a crystalline material like a mineral. Glass composes part of many extrusive igneous rocks (e.g., lava flows) and pyroclastic igneous rocks. Alternatively, some igneous rocks are formed from volcanic processes, such as violent volcanic eruption. Violent eruptions eject molten, partially molten, and non-molten igneous rock, which then falls in the vicinity of the eruption. The fallen material may solidify into a hard mass, called pyroclastic igneous rock. The texture of igneous rocks (defined as the size of crystals in the rock) is strongly related to cooling rate of the original liquid. Rapid cooling of liquid rock promotes formation of small crystals, usually too small to see with the unaided eye. Rocks with this cooling history are called fine-textured igneous rocks. Slow cooling (which usually occurs deep underground) promotes formation of large crystals. Rocks with this cooling history are referred to as coarse-textured igneous rocks.

The mineral composition of igneous rocks falls roughly into four groups: silicic, intermediate, mafic, and ultramafic. These groups are distinguished by the amount of silica  $(SiO_4)$ , iron (Fe), and magnesium (Mg) in the constituent minerals. Mineral composition of liquid rock is related to place of origin within the body of the earth. Generally speaking, liquids from greater depths within the earth contain more Fe and Mg and less  $SiO_4$  than those from shallow depths.

In sedimentary rocks, the type of sediment that is compacted and cemented together determines the rock's main characteristics. Sedimentary rocks composed of sediment that has been broken into pieces (i.e., clastic sediment), such as gravel, sand, silt, and clay, are clastic sedimentary rocks (e.g., conglomerate, sandstone, siltstone, and shale, respectively). Sedimentary rocks composed of sediment that is chemically derived (i.e., chemical sediment), such as dissolved elements like calcium (Ca), sodium (Na), iron (Fe), and silicon (Si), are chemical sedimentary rocks. Examples of chemical sedimentary rocks are limestone (composed of calcium carbonate), rock salt (composed of sodium chloride), rock gypsum (composed of calcium sulfate), ironstones (composed of iron oxides), and chert (composed of hydrated silica). Biochemical sedimentary rocks are a special kind of chemical sedimentary rock wherein the constituent particles were formed by organisms (typically as organic hard parts, such as shells), which then became sedimentary particles. Examples of this special kind of sediment in two stages: compaction and cementation. Compaction occurs when sediments pile up to sufficient thickness that overlying mass squeezes out water and closes much open space. Cementation occurs when water flowing through the compacted sediment deposits mineral crystals upon particles thus binding them together. The main cement minerals are calcite (CaCO<sub>3</sub>), hematite (Fe<sub>2</sub>O<sub>3</sub>), and quartz (SiO<sub>2</sub>).

With metamorphic rocks, the nature of the pre-existing rock (protolith) determines in large part the characteristics of the ultimate metamorphic rock. Regardless of protolith, however, almost all metamorphic rocks are harder and more dense than their protoliths. A protolith with flat or elongate mineral crystals (e.g., micas or amphiboles) will yield a metamorphic rock with preferentially aligned minerals (due to directed pressure). Such metamorphic rocks are called foliated metamorphic rocks (e.g., slate and schist). Non-foliated metamorphic rocks (e.g., marble and quartzite) come from protoliths that have mainly equidimensional mineral crystals (e.g., calcite and quartz, respectively). For example, a protolith shale will yield a foliated metamorphic rock, and a protolith limestone will yield marble, a non-foliated metamorphic rock. Metamorphic rocks possess distinctive grades or levels of metamorphic change from minimal to a maximum near total melting. Low-grade metamorphic rocks generally have fine-textured crystals and low-temperature indicator minerals like the mica chlorite. High-grade metamorphic rocks generally have coarse-textured crystals and very distinctive foliation, plus high-temperature indicator minerals like the silicate mineral staurolite.

Rock is a brittle natural solid found mainly in the outer reaches of Earth's crust and upper mantle. Material that would be brittle rock at such shallow depths becomes to one degree or another rather plastic within the body of the earth. The term "rock" is not generally applied to such non-brittle internal Earth materials. Therefore, rock is a concept related to the outer shell of the earth. The term rock may also be properly applied to brittle natural solids found on the surfaces of other planets and satellites in our solar system. Meteorites are rock. Naturally occurring ice (e.g., brittle water ice in a glacier,  $H_2O$ ) is also a rock, although we do not normally think of ice this way.

Rock has been an important natural resource for people from early in human evolution. Rocks' properties are the key to their specific usefulness, now as in the past. Hard, dense rocks that could be chipped into implements and weapons were among the first useful possessions of people. Fine-textured and glassy rocks were particularly handy for these applications. Later on, rock as building stone and pavement material became very important, and this continues today in our modern world. All of Earth's natural mineral wealth, fossil energy resources, and most groundwater are contained within rocks of the earth's crust.

Rock is a natural occurrence mass of cohesive organic or inorganic material, which forms a part earth crest of which most rocks are composed of one or more minerals. Rocks can be classified in different ways. The most used classification is based on their origin, in which the following classes can be distinguished.

Igneous rock; a rock that has solidified from molten rock material (magma), which was generated within the Earth. Well known are granite and basalt

Sedimentary rock; a rock formed by the consolidation of sediment settle out in water, ice of air and accumulated on the Earth's surface, either on dry land or under water. Examples are sandstone, lime stone and clay stone

Metamorphic rock; any class of rocks that are the result of partial or complete recrystallization in the solid state of pre-existing rocks under conditions of temperature and pressure that are significantly different from those obtaining at the surface of the Earth.

When deterring the dredge-ability of rock, distinction has to be made between the properties of intact rock and that of a rock mass. Depending on the fracture density of the rock the cutter will cut intact rock or break out rock blocks.

In the first case the strength (tensile- and compressive strength), deformation properties (E-value) and the petrography (mineralogical proposition) of the intact rock determines the production completely. The second case the fracture frequency and the weathering of the rock is more important than the strength of the intact rock. It is known that the absence of water in rock is important for the rock strength. When saturated with water the rock strength can be 30 to 90 % of the strength of dry rock. Therefore rock samples have to be sealed immediately after drilling in such a way that evaporation of or intake of water is avoided. It has to be mentioned that this does not mean that cutting forces in saturated rock are always lower than in dry rock.

The petrography is important for the weir of rock cutting tools.

GRAIN SIZE	Bedded rocks					Foliated rocks		Massive and crystalline rocks				GRAIN SIZE			
(mm)	SEDIM	DIMENTARY METAMORPHIC				IGNEOUS			(mm)						
Coarse	CONGLOMERATE (rounded particles in a finer matrix) BRECCIA (angular particles in a finer matrix)		calcirudite	E conglomerate limestone	¢ Dolomite	gypsum, anhydrite		GNEISS Weil developed but often widely spaced foliation sometimes with schistose bands MIGMATITE mixture of gneiss and velns of igneous nock	Som mass	etimes ive	GRANITE (1,2)	DIORITE (1,2)	GABBRO (1,3)	PERIDOTITE	Coarse 2
0.6 - Medium 0.2 - 0.06	SANDSTONES (Quartz) ARENITE Quartz grains and silicocos cement Quartz 1 ARKOSE Many foldspar grains usually with some mica Quartz (grey) WACKE Many rock chips comentification cost)	on (Clark and Walker 1977	calcarenite	STIC LIMESTON	E LIMESTONE &	rES Halite,	OAL & LIGNITE	SCHIST Well developed foliation generally much mica	PHOSED LIMESTONE (A		MICRO- GRANITE (1,2) (Porphyry)*	MICRO- DIORITE (1,2) (Porphyry)*	DOLERITE (3) (Porphyry)*		Medium 0.06
Fine 0.002	CLAYSTONE Mostly silt UCLAYSTONE Mostly clay (massive texture) V (massive texture)	rocks with mixed compositi	calcilutite calcisiltite	CLAS fine grained limestone	CRYSTALLIN	EVAPORI	0	PHYLLITE Closely spaced foliation, mica luster, but crystals not visible with handlens SLATE narow spaced well developed plane of foliation, (mica is absent	METAMOR	HORNFELLS	RHYOLITE (3,4)	ANDESITE (3,4)	BASALT (3,4)		Fine 0.002
Amorphous	Flint, Chert	۷	weak strong Mylonite Obsidian					Mylonite					Amorphous		
	CLASTIC CRYSTALLINE Organic			CRYSTALLINE											
	SILICEOUS CALCAREOUS -accous			Carbon -aceous	Foliated rocks may be layered or banded. This banding may be undulating or controted, created in the case of shates where it may be exactly parallel. Metamorphic rocks often spit testily water foliation planes.			Quartz p	c oor						
	GENERAL NOTES: Bedding in sedimentary rocks may not, because of its spacing, be seen in hand specimen but only in outcrop. Fossils may be found in sedimentary rocks. The mineral calcite, in calcarcous rocks, may be scratched with a knife, and will react with dilute hydrochloric acid Quartz scratches steel. Broken crystals in crystalline rocks reflect light. **alloceus and calcarcous rocks may not generate (e.g. siliceous fine grained limestone)					MODE OF 0 1. Batholiths 2. * Porphyrics are than the surrou	OCCURRENCE OF IG Stocks 3. Sills and c rocks in which some mi nding matrix. All igneo	NEOUS ROCKS: dykes 4. Lava flow ineral grains are very m us rocks can be "porph	vs iuch l yritic	arger					

Figure 2-1: Aid to identification of rock for engineering purposes (After BS 5930:1981).

#### **Chapter 3: Soil Mechanical Parameters**

#### 3.1. Grain Size Distribution

Soils consist of a mixture of particles of different size, shape and mineralogy. Because the size of the particles obviously has a significant effect on the soil behavior, the grain size and grain size distribution are used to classify soils. The grain size distribution describes the relative proportions of particles of various sizes. The grain size is often visualized in a cumulative distribution graph which, for example, plots the percentage of particles finer than a given size as a function of size. The median grain size,  $d_{50}$ , is the size for which 50% of the particle mass consists of finer particles. Soil behavior, especially the hydraulic conductivity, tends to be dominated by the smaller particles; hence, the term "effective size", denoted by  $d_{10}$ , is defined as the size for which 10% of the particle mass consists of finer particles.

Sands and gravels that possess a wide range of particle sizes with a smooth distribution of particle sizes are called well graded soils. If the soil particles in a sample are predominantly in a relatively narrow range of sizes, the soil is called uniformly graded soils. If there are distinct gaps in the gradation curve, e.g., a mixture of gravel and fine sand, with no coarse sand, the soils may be called gap graded. Uniformly graded and gap graded soils are both considered to be poorly graded. There are many methods for measuring particle size distribution. The two traditional methods used in geotechnical engineering are sieve analysis and hydrometer analysis.

#### 3.1.1. Sieve Analysis

The size distribution of gravel and sand particles are typically measured using sieve analysis. The formal procedure is described in ASTM D6913-04(2009). A stack of sieves with accurately dimensioned holes between a mesh of wires is used to separate the particles into size bins. A known volume of dried soil, with clods broken down to individual particles, is put into the top of a stack of sieves arranged from coarse to fine. The stack of sieves is shaken for a standard period of time so that the particles are sorted into size bins. This method works reasonably well for particles in the sand and gravel size range. Fine particles tend to stick to each other, and hence the sieving process is not an effective method. If there are a lot of fines (silt and clay) present in the soil it may be necessary to run water through the sieves to wash the coarse particles and clods through.

A variety of sieve sizes are available. The boundary between sand and silt is arbitrary. According to the Unified Soil Classification System, a #4 sieve (4 openings per inch) having 4.75mm opening size separates sand from gravel and a #200 sieve with an 0.075 mm opening separates sand from silt and clay. According to the British standard, 0.063 mm is the boundary between sand and silt, and 2 mm is the boundary between sand and gravel.



Figure 3-1: Sieves.



The Cutting of Sand, Clay & Rock - Soil Mechanics





Figure 3-3: A medium sand.

#### **3.1.2. Hydrometer analysis**

The classification of fine-grained soils, i.e., soils that are finer than sand, is determined primarily by their Atterberg limits, not by their grain size. If it is important to determine the grain size distribution of fine-grained soils, the hydrometer test may be performed. In the hydrometer tests, the soil particles are mixed with water and shaken to produce a dilute suspension in a glass cylinder, and then the cylinder is left to sit. A hydrometer is used to measure the density of the suspension as a function of time. Clay particles may take several hours to settle past the depth of measurement of the hydrometer. Sand particles may take less than a second. Stoke's law provides the theoretical basis to calculate the relationship between sedimentation velocity and particle size. ASTM provides the detailed procedures for performing the Hydrometer test.

Clay particles can be sufficiently small that they never settle because they are kept in suspension by Brownian motion, in which case they may be classified as colloids.

#### **3.2.** Atterberg Limits

The **Atterberg limits** are a basic measure of the nature of a fine-grained soil. Depending on the water content of the soil, it may appear in four states: solid, semi-solid, plastic and liquid. In each state the consistency and behavior of a soil is different and thus so are its engineering properties. Thus, the boundary between each state can be defined based on a change in the soil's behavior. The Atterberg limits can be used to distinguish between silt and clay, and it can distinguish between different types of silts and clays. These limits were created by Albert Atterberg, a Swedish chemist. They were later refined by Arthur Casagrande. These distinctions in soil are used in picking the soils to build structures on top of. These tests are mainly used on clayey or silty soils since these are the soils that expand and shrink due to moisture content. Clays and silts react with the water and thus change sizes and have varying shear strengths. Thus these tests are used widely in the preliminary stages of building any structure to insure that the soil will have the correct amount of shear strength and not too much change in volume as it expands and shrinks with different moisture contents.

#### 3.2.1. Shrinkage limit

The shrinkage limit (SL) is the water content where further loss of moisture will not result in any more volume reduction. The test to determine the shrinkage limit is ASTM International D4943. The shrinkage limit is much less commonly used than the liquid and plastic limits.

#### 3.2.2. Plastic limit

The plastic limit (PL) is the water content where soil transitions between brittle and plastic behavior. A thread of soil is at its plastic limit when it begins to crumble when rolled to a diameter of 3 mm. To improve test result consistency, a 3 mm diameter rod is often used to gauge the thickness of the thread when conducting the test. The Plastic Limit test is defined by ASTM standard test method D 4318.

#### 3.2.3. Liquid limit

The liquid limit (LL) is the water content at which a soil changes from plastic to liquid behavior. The original liquid limit test of Atterberg's involved mixing a pat of clay in a round-bottomed porcelain bowl of 10-12cm diameter. A groove was cut through the pat of clay with a spatula, and the bowl was then struck many times against the palm of one hand. Casagrande subsequently standardized the apparatus and the procedures to make the measurement more repeatable. Soil is placed into the metal cup portion of the device and a groove is made down its center with a standardized tool of 13.5 millimeters (0.53 in) width. The cup is repeatedly dropped 10mm onto a hard rubber base during which the groove closes up gradually as a result of the impact. The number of blows for the groove to close is recorded. The moisture content at which it takes 25 drops of the cup to cause the groove to close over a distance of 13.5 millimeters (0.53 in) is defined as the liquid limit. The test is normally run at several moisture contents, and the moisture content which requires 25 blows to close the groove is interpolated from the test results. The Liquid Limit test is defined by ASTM standard test method D 4318. The test method also allows running the test at one moisture content where 20 to 30 blows are required to close the groove; then a correction factor is applied to obtain the liquid limit from the moisture content.

The following is when you should record the N in number of blows needed to close this 1/2-inch gap:

The materials needed to do a Liquid limit test are as follows

- Casagrande cup ( liquid limit device)
- Grooving tool

- Soil pat before test
- Soil pat after test

Another method for measuring the liquid limit is the fall cone test. It is based on the measurement of penetration into the soil of a standardized cone of specific mass. Despite the universal prevalence of the Casagrande method, the fall cone test is often considered to be a more consistent alternative because it minimizes the possibility of human variations when carrying out the test.





Figure 3-5: Liquid limit device.

Figure 3-4: Liquid limit device.

#### 3.2.4. Importance of Liquid Limit test

The importance of the liquid limit test is to classify soils. Different soils have varying liquid limits. Also to find the plasticity index of a soil you need to know the liquid limit and the plastic limit.

#### 3.2.5. Derived limits

The values of these limits are used in a number of ways. There is also a close relationship between the limits and properties of a soil such as compressibility, permeability, and strength. This is thought to be very useful because as limit determination is relatively simple, it is more difficult to determine these other properties. Thus the Atterberg limits are not only used to identify the soil's classification, but it allows for the use of empirical correlations for some other engineering properties.

#### 3.2.6. Plasticity index

The plasticity index (PI) is a measure of the plasticity of a soil. The plasticity index is the size of the range of water contents where the soil exhibits plastic properties. The PI is the difference between the liquid limit and the plastic limit (PI = LL-PL). Soils with a high PI tend to be clay, those with a lower PI tend to be silt, and those with a PI of 0 (non-plastic) tend to have little or no silt or clay. PI and their meanings

- 0 Non-plastic
- (1-5)- Slightly Plastic
- (5-10) Low plasticity
- (10-20)- Medium plasticity
- (20-40)- High plasticity
- >40 Very high plasticity

#### **3.2.7. Liquidity index**

The liquidity index (LI) is used for scaling the natural water content of a soil sample to the limits. It can be calculated as a ratio of difference between natural water content, plastic limit, and plasticity index:

LI=(W-PL)/(LL-PL) where W is the natural water content.

#### 3.2.8. Activity

The activity (A) of a soil is the PI divided by the percent of clay-sized particles (less than 2  $\mu$ m) present. Different types of clays have different specific surface areas which controls how much wetting is required to move a soil from one phase to another such as across the liquid limit or the plastic limit. From the activity one can predict the dominant clay type present in a soil sample. High activity signifies large volume change when wetted and large shrinkage when dried. Soils with high activity are very reactive chemically. Normally the activity of clay is between 0.75 and 1.25, and in this range clay is called normal. It is assumed that the plasticity index is approximately equal to the clay fraction (A = 1). When A is less than 0.75, it is considered inactive. When it is greater than 1.25, it is considered active.

#### 3.3. Mass Volume Relations

There are a variety of parameters used to describe the relative proportions of air, water and solid in a soil. This section defines these parameters and some of their interrelationships. The basic notation is as follows:

 $V_a,\,V_w,\,\text{and}\,\,V_s$  represent the volumes of air, water and solids in a soil mixture;

 $\mathbf{W}_{a}, \mathbf{W}_{w},$  and  $\mathbf{W}_{s}$  represent the weights of air, water and solids in a soil mixture;

 $M_a$ ,  $M_w$ , and  $M_s$  represent the masses of air, water and solids in a soil mixture;

 $\rho_a$ ,  $\rho_w$ , and  $\rho_s$  represent the densities of the constituents (air, water and solids) in a soil mixture;

Note that the weights, **W**, can be obtained by multiplying the mass, **M**, by the acceleration due to gravity, **g**; e.g.,  $\mathbf{W}_s = \mathbf{M}_s \mathbf{g}$ 



Figure 3-6: A phase diagram of soil indicating the masses and volumes of air, solid, water and voids.

#### 3.4. Specific Gravity

Specific Gravity is the ratio of the density of one material compared to the density of pure water ( $\rho_w = 1000 \text{ kg/m}^3$ ).

$$G_{s} = \frac{\rho_{s}}{\rho_{w}}$$
(3-1)

With:

,, 1011.		
Gs	Specific gravity	-
$\rho_{s}$	Density of the soil	kg/m <sup>3</sup>
ρ <sub>w</sub>	Density of water	kg/m <sup>3</sup>

**g** Gravitational constant  $(9.81 \text{ m/s}^2)$ 

#### 3.5. Density

The terms density and unit weight are used interchangeably in soil mechanics. Though not critical, it is important that we know it. Density, Bulk Density, or Wet Density,  $\rho_t$ , are different names for the density of the mixture, i.e., the total mass of air, water, solids divided by the total volume of air, water and solids (the mass of air is assumed to be zero for practical purposes. To find the formula for density, divide the mass of the soil by the volume of the soil, the basic formula for density is:

$$\rho_t = \frac{\mathbf{M}_t}{\mathbf{V}_t} = \frac{\mathbf{M}_s + \mathbf{M}_w + \mathbf{M}_a}{\mathbf{V}_s + \mathbf{V}_w + \mathbf{V}_a}$$
(3-2)

With:

$\mathbf{M}_{\mathbf{t}}$	Mass of the soil, total mass	kg
$M_s$	Mass of the solids	kg
$\mathbf{M}_{\mathbf{w}}$	Mass of the water	kg
$\mathbf{M}_{\mathbf{a}}$	Mass of the air	kg
Vt	Volume of the soil, total volume	m <sup>3</sup>
Vs	Volume of the solids	m <sup>3</sup>
$\mathbf{V}_{\mathbf{w}}$	Volume of the water	m <sup>3</sup>
$\mathbf{V}_{\mathbf{a}}$	Volume of the air	m <sup>3</sup>
ρ <sub>t</sub>	Density of the soil	kg/m <sup>3</sup>
γ <sub>t</sub>	Unit weight of the soil	$N/m^3$
g	Gravitational constant $(9.81 \text{ m/s}^2)$	m/s <sup>2</sup>

Unit weight of a soil mass is the ratio of the total weight of soil to the total volume of soil. Unit Weight,  $\gamma_t$ , is usually determined in the laboratory by measuring the weight and volume of a relatively undisturbed soil sample obtained from a brass ring. Measuring unit weight of soil in the field may consist of a sand cone test, rubber balloon or nuclear densitometer, the basic formula for unit weight is:

$$\gamma_t = \frac{\mathbf{M}_t \cdot \mathbf{g}}{\mathbf{V}_t} \tag{3-3}$$

Dry Density,  $\rho_d$ , is the mass of solids divided by the total volume of air water and solids:

$$\rho_d = \frac{M_s}{V_t} = \frac{M_s}{V_s + V_w + V_a}$$
(3-4)

Buoyant Density,  $\rho'$ , defined as the density of the mixture minus the density of water is useful if the soil is submerged under water:

$$\rho' = \rho - \rho_{\rm w} \tag{3-5}$$

Table 3-1: Empirical values for $\rho_t$ , of granular soils based on the standard penetration number,
(from Bowels, <i>Foundation Analysis</i> ).

(If our Dowels, I our autor in a pais)						
SPT Penetration, N-Value	$\mathbf{\rho_t} \ (\text{kg/m}^3)$					
(blows/ foot)						
0 - 4	1120 - 1520					
4 - 10	1520 - 1800					
10 - 30	1800 - 2080					
30 - 50	2080 - 2240					
>50	2240 - 2400					

Table 3-2: Empirical values for  $\rho_s$ , of cohesive soils based on the standard penetration number,

 $m/s^2$ 

(II olii Dowels, Foundation Analysis).						
SPT Penetration, N-Value (blows/ foot)	$\rho_{s, sat} (kg/m^3)$					
0 - 4	1600 - 1840					
4 - 8	1840 - 2000					
8 - 32	2000 - 2240					

#### (from Bowels, Foundation Analysis).

# Table 3-3: Typical Soil Characteristics (from Lindeburg, Civil Engineering Reference Manual for the PE Exam, 8th ed.)

Soil Type	$\rho_{s}$ (kg/m <sup>3</sup> )	$\rho_{s, sat}$ (kg/m <sup>3</sup> )
Sand, loose and uniform	1440	1888
Sand, dense and uniform	1744	2080
Sand, loose and well graded	1584	1984
Sand, dense and well graded	1856	2160
Glacial clay, soft	1216	1760
Glacial clay, stiff	1696	2000

#### Table 3-4: Typical Values of Soil Index Properties (from NAVFAC 7.01)

Soil Type	$\rho_{s} (kg/m^{3})$
Sand; clean, uniform, fine or medium	1344 - 2176
Silt; uniform, inorganic	1296 - 2176
Silty Sand	1408 - 2272
Sand; Well-graded	1376 - 2368
Silty Sand and Gravel	1440 - 2480
Sandy or Silty Clay	1600 - 2352
Silty Clay with Gravel; uniform	1840 - 2416
Well-graded Gravel, Sand, Silt and Clay	2000 - 2496
Clay	1504 - 2128
Colloidal Clay	1136 - 2048
Organic Silt	1392 - 2096
Organic Clay	1296 - 2000

#### 3.6. Relative Density

Relative density is an index that quantifies the state of compactness between the loosest and densest possible state of coarse-grained soils. The relative density is written in the following formulas:

$$\mathbf{D}_{\mathbf{r}} = \frac{\mathbf{e}_{\max} - \mathbf{e}}{\mathbf{e}_{\max} - \mathbf{e}_{\min}} = \frac{\mathbf{n}_{\max} - \mathbf{n}}{\mathbf{n}_{\max} - \mathbf{n}_{\min}}$$
(3-6)

With:

Dr	Relative density	-
e	Current void ratio of the soil in-situ	-
e <sub>max</sub>	Void ratio of the soil at its loosest condition	-
e <sub>min</sub>	Void ratio of the soil at its densest condition	-
n	Porosity of the soil in-situ	-
n <sub>max</sub>	Porosity of the soil at its loosest condition	_
n <sub>min</sub>	Porosity of the soil in its densest condition	-

Tuble 5 5. Designation of Grandult Son Dased on Relative Density.			
D <sub>r</sub> (%)	Description		
0 - 20	Very loose		
20 - 40	Loose		
40 - 70	Medium dense		
70 - 85	Dense		
85 - 100	Very dense		

Table 3-5: Designation of Granul	ar Soil Based on Relative Density.
----------------------------------	------------------------------------

Lambe & Whitman (1979), page 78 (Figure 3-7) give the relation between the SPT value, the relative density and the hydrostatic pressure in two graphs. With some curve-fitting these graphs can be summarized with the following equation (Miedema (1995)):





Figure 3-7: SPT values versus relative density (Miedema (1995).

#### 3.7. Porosity

Porosity is the ratio of the volume of openings (voids) to the total volume of material. Porosity represents the storage capacity of the geologic material. The primary porosity of a sediment or rock consists of the spaces between the grains that make up that material. The more tightly packed the grains are, the lower the porosity. Using a box of marbles as an example, the internal dimensions of the box would represent the volume of the sample. The space surrounding each of the spherical marbles represents the void space. The porosity of the box

(3-7)

of marbles would be determined by dividing the total void space by the total volume of the sample and expressed as a percentage.

The primary porosity of unconsolidated sediments is determined by the shape of the grains and the range of grain sizes present. In poorly sorted sediments, those with a larger range of grain sizes, the finer grains tend to fill the spaces between the larger grains, resulting in lower porosity. Primary porosity can range from less than one percent in crystalline rocks like granite to over 55% in some soils. The porosity of some rock is increased through fractures or solution of the material itself. This is known as secondary porosity.

$$\mathbf{n} = \frac{\mathbf{V}_{\mathbf{v}}}{\mathbf{V}_{\mathbf{t}}} = \frac{\mathbf{V}_{\mathbf{v}}}{\mathbf{V}_{\mathbf{s}} + \mathbf{V}_{\mathbf{v}}} = \frac{\mathbf{e}}{1 + \mathbf{e}}$$
(3-8)

With:

#### 3.8. Void ratio

The ratio of the volume of void space to the volume of solid substance in any material consisting of void space and solid material, such as a soil sample, a sediment, or a powder.

$$e = \frac{V_v}{V_s} = \frac{V_v}{V_t - V_v} = \frac{n}{1 - n}$$
 (3-9)

With:

The relations between void ratio and porosity are:

$$\mathbf{e} = \frac{\mathbf{n}}{1-\mathbf{n}} \text{ and } \mathbf{n} = \frac{\mathbf{e}}{1+\mathbf{e}}$$
(3-10)

#### 3.9. Permeability

Permeability is a measure of the ease with which fluids will flow though a porous rock, sediment, or soil. Just as with porosity, the packing, shape, and sorting of granular materials control their permeability. Although a rock may be highly porous, if the voids are not interconnected, then fluids within the closed, isolated pores cannot move. The degree to which pores within the material are interconnected is known as effective porosity. Rocks such as pumice and shale can have high porosity, yet can be nearly impermeable due to the poorly interconnected voids. In contrast, well-sorted sandstone closely replicates the example of a box of marbles cited above. The rounded sand grains provide ample, unrestricted void spaces that are free from smaller grains and are very well linked. Consequently, sandstones of this type have both high porosity and high permeability.

The range of values for permeability in geologic materials is extremely large. The most conductive materials have permeability values that are millions of times greater than the least permeable. Permeability is often directional in nature. The characteristics of the interstices of certain materials may cause the permeability to be significantly greater in one direction. Secondary porosity features, like fractures, frequently have significant impact on the permeability of the material. In addition to the characteristics of the host material, the viscosity and pressure of the fluid also affect the rate at which the fluid will flow.

Hydraulic conductivity or permeability  $\mathbf{k}$  can be estimated by particle size analysis of the sediment of interest, using empirical equations relating either  $\mathbf{k}$  to some size property of the sediment. Vukovic and Soro (1992) summarized several empirical methods from former studies and presented a general formula:

$$\mathbf{k} = \mathbf{C} \cdot \frac{\mathbf{g}}{\mathbf{v}} \cdot \mathbf{f}(\mathbf{n}) \cdot \mathbf{d}_{\mathbf{e}}^2$$
(3-11)

The kinematic viscosity v is related to dynamic viscosity  $\mu$  and the fluid (water) density  $\rho_w$  as follows:

$$v = \frac{\mu}{\rho_{\rm w}} \tag{3-12}$$

The values of C, f(n) and  $d_e$  are dependent on the different methods used in the grain-size analysis. According to Vukovic and Soro (1992), porosity n may be derived from the empirical relationship with the coefficient of grain uniformity U as follows:

$$\mathbf{n} = \mathbf{0.255} \cdot \left(1 + \mathbf{0.83^{U}}\right) \tag{3-13}$$

Where U is the coefficient of grain uniformity and is given by:

$$\mathbf{U} = \left(\frac{\mathbf{d}_{60}}{\mathbf{d}_{10}}\right) \tag{3-14}$$

Here,  $\mathbf{d}_{60}$  and  $\mathbf{d}_{10}$  in the formula represent the grain diameter in (mm) for which, 60% and 10% of the sample respectively, are finer than. Former studies have presented the following formulae which take the general form presented in equation (3-11) above but with varying **C**,  $\mathbf{f}(\mathbf{n})$  and  $\mathbf{d}_{\mathbf{e}}$  values and their domains of applicability.

Hazen's formula (1982) was originally developed for determination of hydraulic conductivity of uniformly graded sand but is also useful for fine sand to gravel range, provided the sediment has a uniformity coefficient less than 5 and effective grain size between 0.1 and 3mm.

$$\mathbf{k} = 6 \cdot 10^{-4} \cdot \frac{g}{v} \cdot \left(1 + 10 \cdot (n - 0.26)\right) \cdot d_{10}^2$$
(3-15)

The Kozeny-Carman equation is one of the most widely accepted and used derivations of permeability as a function of the characteristics of the soil medium. The Kozeny-Carman equation (or Carman-Kozeny equation) is a relation used in the field of fluid dynamics to calculate the pressure drop of a fluid flowing through a packed bed of solids. It is named after Josef Kozeny and Philip C. Carman. This equation was originally proposed by Kozeny (1927) and was then modified by Carman (1937) and (1956) to become the Kozeny-Carman equation. It is not appropriate for either soil with effective size above 3 mm or for clayey soils. The equation is only valid for laminar flow. The equation is given as:

$$\mathbf{k} = \mathbf{d}_{e}^{2} \cdot \frac{\gamma_{w}}{\mu} \cdot \frac{\mathbf{e}^{3}}{1+\mathbf{e}} \cdot \mathbf{C} \text{ or } \mathbf{k} = 8.3 \cdot 10^{-3} \cdot \frac{\mathbf{g}}{\mathbf{v}} \cdot \frac{\mathbf{n}^{3}}{(1-\mathbf{n})^{2}} \cdot \mathbf{d}_{10}^{2} \text{ with } \mathbf{v} = \frac{\mu}{\rho_{w}} \text{ and } \gamma_{w} = \rho_{w} \cdot \mathbf{g}$$
(3-16)

This equation holds for flow through packed beds with particle Reynolds numbers up to approximately 1.0, after which point frequent shifting of flow channels in the bed causes considerable kinetic energy losses. This equation can be expressed as "*flow is proportional to the pressure drop and inversely proportional to the fluid viscosity*", which is known as Darcy's law.

The Breyer method does not consider porosity and therefore, porosity function takes on value 1. Breyer formula is often considered most useful for materials with heterogeneous distributions and poorly sorted grains with uniformity coefficient between 1 and 20, and effective grain size between 0.06mm and 0.6mm.

$$\mathbf{k} = 6 \cdot 10^{-4} \cdot \frac{\mathbf{g}}{\mathbf{v}} \cdot \log\left(\frac{500}{\mathbf{U}}\right) \cdot \mathbf{d}_{10}^2 \tag{3-17}$$

The Slitcher formula is most applicable for grain-sizes between 0.01 mm and 5 mm.

$$\mathbf{k} = 1 \cdot 10^{-2} \cdot \frac{\mathbf{g}}{\mathbf{v}} \cdot \mathbf{n}^{3.287} \cdot \mathbf{d}_{10}^2$$
(3-18)

Where the  $C_t$  = sorting coefficient and  $6.1 \times 10^{-3} < C_t < 10.7 \times 10^{-3}$ . The Terzaghi (1964) formula is most applicable for coarse sand. The Terzaghi equation:

$$\mathbf{k} = \mathbf{C}_{t} \cdot \frac{\mathbf{g}}{\mathbf{v}} \cdot \left(\frac{\mathbf{n} - \mathbf{0.13}}{\sqrt[3]{1 - \mathbf{n}}}\right)^{2} \cdot \mathbf{d}_{10}^{2}$$
(3-19)

With:

Ct	Sorting coefficient	-
С	Sorting coefficient	-
K	Hydraulic conductivity	$m^2$
k	Permeability	m/s
f(n)	porosity function	-
С	sorting coefficient	
de	effective grain diameter	mm
<b>d</b> <sub>10</sub>	Grain diameter where 10% is smaller	mm
<b>d</b> <sub>60</sub>	Grain diameter where 60% is smaller	mm
U	Grain uniformity coefficient	-
v	kinematic viscosity	
μ	Dynamic viscosity	Pa.s
$\rho_w$	Water density	kg/m <sup>3</sup>
$\gamma_{w}$	Unit weight of water	N/m <sup>3</sup>

#### **3.10.** The Angle of Internal Friction

Angle of internal friction for a given soil is the angle on the graph (Mohr's Circle) of the shear stress and normal effective stresses at which shear failure occurs. Angle of Internal Friction,  $\varphi$ , can be determined in the laboratory by the Direct Shear Test or the Triaxial Stress Test. Typical relationships for estimating the angle of internal friction,  $\varphi$ , are as follows:

Table 3-6: Empirical values for  $\varphi$ , of granular soils based on the standard penetration number, (from Bowels, *Foundation Analysis*).

SPT Penetration, N-Value (blows/ foot)	φ (degrees)
0	25 - 30
4	27 - 32
10	30 - 35
30	35 - 40
50	38 - 43

Table 3-7: Relationship between $\phi$ , and standard penetration number for sands,
(from Peck 1974, Foundation Engineering Handbook).

SPT Penetration, N-Value (blows/ foot)	Density of Sand	φ (degrees)	
<4	Very loose	<29	
4 - 10	Loose	29 - 30	
10 - 30	Medium	30 - 36	
30 - 50	Dense	36 - 41	
>50	Very dense	>41	

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SPT Penetration, N-Value (blows/ foot)	Density of Sand	φ (degrees)
<4	Very loose	<30
4 - 10	Loose	30 - 35
10 - 30	Medium	35 - 40
30 - 50	Dense	40 - 45
>50	Very dense	>45

 Table 3-8: Relationship between φ, and standard penetration number for sands,

 (from Meyerhof 1956, Foundation Engineering Handbook).



Figure 3-8: SPT values reduced to 10m water depth (Miedema (1995).



Figure 3-9: Friction angle versus SPT value (Miedema (1995).

Lambe & Whitman (1979), page 148 (Figure 3-9) give the relation between the SPT value and the angle of internal friction, also in a graph. This graph is valid up to 12 m in dry soil. With respect to the internal friction, the relation given in the graph has an accuracy of 3 degrees. A load of 12 m dry soil with a density of 1.67 ton/m<sup>3</sup> equals a hydrostatic pressure of 20 m.w.c. An absolute hydrostatic pressure of 20 m.w.c. equals 10 m of water depth if cavitation is considered. Measured SPT values at any depth will have to be reduced to the value that would occur at 10 m water depth. This can be accomplished with the following equation (see Figure 3-8):

$$SPT_{10} = \frac{1}{(0.646 + 0.0354 \cdot z)} \cdot SPT_{z}$$
(3-20)

With the aim of curve-fitting, the relation between the SPT value reduced to 10 m water depth and the angle of internal friction can be summarized to:

$$\phi = 54.5 - 25.9 \cdot e^{-0.01753 \cdot SPT_{10}}$$
 (+ 3 degrees value)

(3-21)



Figure 3-10: Coefficients of internal friction for a variety of rock types (Wijermars (1997-2011)).

#### 3.11. The Angle of External Friction

The external friction angle,  $\delta$ , or friction between a soil medium and a material such as the composition from a retaining wall or pile may be expressed in degrees as the following:

<b>20°</b>	steel piles (NAVFAC)	
$0.67 \cdot \varphi - 0.83 \cdot \varphi$	USACE	
<b>20°</b>	steel (Broms)	
$\frac{3}{4} \cdot \varphi$	concrete (Broms)	
$\frac{2}{3} \cdot \varphi$	timber (Broms)	
0.67 · φ	Lindeburg	
$\frac{2}{3} \cdot \varphi$	for concrete walls (Coulomb)	

Fable 3-9:	External	friction	angle	φ	values
				T	

The external friction angle can be estimated as  $1/3\varphi$  for smooth retaining walls like sheet piles or concrete surfaces against timber formwork, or as  $1/2\phi$  to  $2/3\phi$  for rough surfaces. In the absence of detailed information the assumption of  $2/3\varphi$  is commonly made.

#### 3.12. Cohesion (Internal Shear Strength)

Cohesion (n. lat. cohaerere "stick or stay together") or cohesive attraction or cohesive force is the action or property of like molecules sticking together, being mutually attractive. This is an intrinsic property of a substance that is caused by the shape and structure of its molecules which makes the distribution of orbiting electrons irregular when molecules get close to one another, creating electrical attraction that can maintain a macroscopic structure such as a water drop. Cohesive soils are clay type soils. Cohesion is the force that holds together molecules or like particles within a soil. Cohesion, c, is usually determined in the laboratory from the Direct Shear Test. Unconfined Compressive Strength, U.C.S., can be determined in the laboratory using the Triaxial Test or the Unconfined Compressive Strength Test. There are also correlations for U.C.S. with shear strength as estimated from the field using Vane Shear Tests. With a conversion of 1 kips/ft<sup>2</sup>=47.88 kN/m<sup>2</sup>, tables Table 3-10 and Table 3-11 are basically the same.

$$c = \frac{U.C.S.}{2}$$
(3-22)

With:

с	Cohesion
U.C.S.	Unconfined Compressive Strength

-11. 2 10.	C	<b>C</b>	C.			0 7 00
adle 3-10:	Guiae lor	Consistency	of rine-Gra	inea Son	, INA V Ľ A	AC 7.02

Table 3-10: Guide for Consistency of Fine-Grained Soil, NAVFAC 7.02							
<b>SPT Penetration</b> (blows/ foot)	Estimated Consistency	<b>U.C.S.</b> (kPa)					
<2	Very Soft	<24					
2 - 4	Soft	24 - 48					
4 - 8	Medium	48 - 96					
8 - 15	Stiff	96 - 192					
15 - 30	Very Stiff	192 - 388					
>30	Hard	>388					

Table 3-11: Empirical Values	for Consistency of Cohesiv	e Soil, (from Foundation	Analysis, Bowels)
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2)

kPa or kN/m<sup>2</sup> kPa or kN/m<sup>2</sup>

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SPT Penetration (blows/ foot)	Estimated Consistency	<b>U.C.S.</b> (kips/ft <sup>2</sup> )		
0 - 2	Very Soft	0 - 0.5		
2 - 4	Soft	0.5 - 1.0		
4 - 8	Medium	1.0 - 2.0		
8 - 16	Stiff	2.0 - 4.0		
16 - 32	Very Stiff	4.0 - 8.0		
>32	Hard	>8		

#### 3.13. Adhesion (External Shear Strength)

**Adhesion** is any attraction process between dissimilar molecular species that can potentially bring them in close contact. By contrast, cohesion takes place between similar molecules.

Adhesion is the tendency of dissimilar particles and/or surfaces to cling to one another (cohesion refers to the tendency of similar or identical particles/surfaces to cling to one another). The forces that cause adhesion and cohesion can be divided into several types. The intermolecular forces responsible for the function of various kinds of stickers and sticky tape fall into the categories of chemical adhesion, dispersive adhesion, and diffusive adhesion.

#### **3.14. Dilatation**

Dilation (or dilatation) refers to an enlargement or expansion in bulk or extent, the opposite of contraction. It derives from the Latin dilatare, "to spread wide". It is the increase in volume of a granular substance when its shape is changed, because of greater distance between its component particles. Suppose we have a volume V before the enlargement and a volume V+dV after the enlargement. Before the enlargement we name the porosity  $n_i$  (i from initial) and after the enlargement  $n_{cv}$  (the constant volume situation after large deformations). For the volume before the deformation we can write:

$$\mathbf{V} = (1 - \mathbf{n}_i) \cdot \mathbf{V} + \mathbf{n}_i \cdot \mathbf{V}$$
(3-23)

With:

VThe total volume of soil $m^3$  $n_i$ Initial porosity-pDeregity at constant volume

 $\mathbf{n}_{\mathbf{cv}}$  Porosity at constant volume

The first term on the right hand side is the sand volume, the second term the pore volume. After the enlargement we get:

$$\mathbf{V} + \mathbf{dV} = (1 - \mathbf{n}_{cv}) \cdot (\mathbf{V} + \mathbf{dV}) + \mathbf{n}_{cv} \cdot (\mathbf{V} + \mathbf{dV})$$
(3-24)

Again the first term on the right hand side is the sand volume. Since the sand volume did not change during the enlargement (we assume the quarts grains are incompressible), the volume of sand in both equations should be the same, thus:

$$(1-n_i) \cdot V = (1-n_{cv}) \cdot (V+dV)$$
 (3-25)

From this we can deduce that the dilatation  $\varepsilon$  is:

$$\varepsilon = \frac{dV}{V} = \frac{n_{cv} - n_i}{1 - n_{cv}} = \frac{dn}{1 - n_{cv}}$$
(3-26)

#### 3.15. UCS or Unconfined Compressive Strength

UCS is one of the most basic parameters of rock strength, and the most common determination performed for bore ability predictions. It is measured in accordance with the procedures given in ASTM D2938, with the length to diameter ratio of 2 by using NX-size core samples. 3 to 5 UCS determinations are recommended to achieve statistical significance of the results. If the sample length to diameter ratio was greater or less than 2, ASTM recommends a correction factor that is applied to the UCS value determined from testing. UCS measurements are made using an electronic-servo controlled MTS stiff testing machine with a capacity of 220 kips. Loading data and other test parameters are recorded with a computer based data acquisition system, and the data is subsequently reduced and analyzed with a customized spreadsheet program.

The most important test for rock in the field of dredging is the uniaxial unconfined compressive strength (UCS). In the test a cylindrical rock sample is axial loaded till failure. Except the force needed, the deformation is measured too. So the complete stress-strain curve is measured from which the deformation modulus and the specific work of failure can be calculated. The unconfined compressive strength of the specimen is calculated by dividing the maximum load at failure by the sample cross-sectional area:

$$\sigma_{\rm c} = \frac{\rm F}{\rm A} \tag{3-27}$$

With:

σ	Unconfined Compressive Strength	kPa
F	Maximum Failure Load	kN
Α	Cross-sectional area of the core sample	m <sup>2</sup>
Е	Deformation modulus	N/m <sup>2</sup>
W	Specific work of failure	J/m <sup>3</sup>



Figure 3-11: A UCS test facility.

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Figure 3-12: The stress strain relation during a UCS test (Vlasblom (2003-2007)).

#### 3.16. Unconfined Tensile Strength

The uniaxial unconfined tensile strength is defined in the same way as the compressive strength Figure 3-13. Sample preparation and testing procedure require much effort and not commonly done. Another method to determine the tensile strength , also commonly not used , is by bending a sample see Figure 3-14.



Figure 3-13: Unconfined

Figure 3-14: Bending (Vlasblom (2003-2007)).

#### 3.17. BTS or Brazilian Tensile Strength

Indirect, or **Brazilian**, tensile strength is measured using **NX**-size core samples cut to an approximate 0.5 length-to-diameter ratio, and following the procedures of **ASTM D3967**. **BTS** measurements are made using an electronic-servo controlled **MTS** stiff testing machine with a capacity of 220 kips. Loading data and other test parameters are recorded with a computer based data acquisition system, and the data is subsequently reduced and analyzed with a customized spreadsheet program. **BTS** provides a measure of rock toughness, as well as strength. The indirect tensile strength is calculated as follows (Fairhurst (1964)):

$$\sigma_{\rm T} = \frac{2 \cdot F}{\pi \cdot L \cdot D}$$
(3-28)
With:

σTBrazilian Tensile StrengthkPaDDiameter of the core samplemFMaximum Failure LoadkNLLength of the core samplem



Figure 3-15: Directions of foliation.



Figure 3-16: The Brazilian split test (Vlasblom (2003-2007)).

In bedded/foliated rocks, particular attention needs to be given to loading direction with respect to bedding/foliation. The rock should be loaded so that breakage occurs in approximately the same direction as fracture propagation between adjacent cuts on the tunnel face. This is very important assessment in mechanical excavation by tunnel boring machines.

The most common used test to estimate, in an indirect way, the tensile strength is the Brazilian split test. Here the cylindrical sample is tested radial.

The validity of BTS to determine de UTS is discussed by many researchers. In general it can be stated that the BTS over estimates the UTS. According to Pells (1993) this discussion is in most applications in practice largely academic.

#### 3.18. Hardness

Hardness is a loosely defined term, referring the resistance to rock or minerals against an attacking tool. Hardness is determined using rebound tests (f.i. Schmidt hammer), indentation tests, (Brinell, Rockwell) or scratch tests (Mohs). The last test is based on the fact that a mineral higher in the scale can scratch a mineral lower in the scale. Although this scale was established in the early of the 19<sup>th</sup> century it appeared that the increment of Mohs scale corresponded with a 60% increase in indentation hardness.

	Mohs Table					
No.	Mineral					
1	Talc					
2	Gypsum					
3	Calcite					
4	Fluorite					
5	Apatite					
6	Orthoglass					
7	Quartz					
8	Topaz					
9	Corundum					
10	Diamond					

#### **3.19. Pore Water Pressure**

#### 3.19.1. Hydrostatic conditions

If there is no pore water flow occurring in the soil, the pore water pressures will be hydrostatic. The water table is located at the depth where the water pressure is equal to the atmospheric pressure. For hydrostatic conditions, the water pressure increases linearly with depth below the water table:

$\mathbf{u} = \rho_v$	$\mathbf{w} \cdot \mathbf{g} \cdot \mathbf{z}_{\mathbf{w}}$	(3-29)
With:		
$\rho_w$	Density of water	kg/m <sup>3</sup>
$z_w$	Depth below the water table	m
u	Hydrostatic pressure	kPa
g	Gravitational constant	m/s <sup>2</sup>

#### 3.19.2. Capillary action

Due to surface tension water will rise up in a small capillary tube above a free surface of water. Likewise, water will rise up above the water table into the small pore spaces around the soil particles. In fact the soil may be completely saturated for some distance above the water table. Above the height of capillary saturation, the soil may be wet but the water content will decrease with elevation. If the water in the capillary zone is not moving, the water pressure obeys the equation of hydrostatic equilibrium,  $\mathbf{u} = \rho_w \cdot \mathbf{g} \cdot \mathbf{z}_w$ , but note that  $z_w$ , is negative above the water table. Hence, hydrostatic water pressures are negative above the water table. The thickness of the zone of capillary saturation depends on the pore size, but typically, the heights vary between a centimeter or so for coarse sand to tens of meters for a silt or clay.

The surface tension of water explains why the water does not drain out of a wet sand castle or a moist ball of clay. Negative water pressures make the water stick to the particles and pull the particles to each other, friction at the particle contacts make a sand castle stable. But as soon as a wet sand castle is submerged below a free water surface, the negative pressures are lost and the castle collapses. Considering the effective stress equation,  $\sigma' = \sigma - u_{,,i}$  if the water pressure is negative, the effective stress may be positive, even on a free surface (a surface where the total normal stress is zero). The negative pore pressure pulls the particles together and causes compressive particle to particle contact forces.

Negative pore pressures in clayey soil can be much more powerful than those in sand. Negative pore pressures explain why clay soils shrink when they dry and swell as they are wetted. The swelling and shrinkage can cause major distress, especially to light structures and roads.

#### 3.20. Shear Strength

Shear strength is a term used in soil mechanics to describe the magnitude of the shear stress that a soil can sustain. The shear resistance of soil is a result of friction and interlocking of particles, and possibly cementation or bonding at particle contacts. Due to interlocking, particulate material may expand or contract in volume as it is subject to shear strains. If soil expands its volume, the density of particles will decrease and the strength will decrease; in this case, the peak strength would be followed by a reduction of shear stress. The stress-strain relationship levels off when the material stops expanding or contracting, and when inter-particle bonds are broken. The theoretical state at which the shear stress and density remain constant while the shear strain increases may be called the critical state, steady state, or residual strength.

The volume change behavior and inter-particle friction depend on the density of the particles, the inter-granular contact forces, and to a somewhat lesser extent, other factors such as the rate of shearing and the direction of the shear stress. The average normal inter-granular contact force per unit area is called the effective stress.

If water is not allowed to flow in or out of the soil, the stress path is called an undrained stress path. During undrained shear, if the particles are surrounded by a nearly incompressible fluid such as water, then the density of the particles cannot change without drainage, but the water pressure and effective stress will change. On the other hand, if the fluids are allowed to freely drain out of the pores, then the pore pressures will remain constant and the test path is called a drained stress path. The soil is free to dilate or contract during shear if the soil is drained. In reality, soil is partially drained, somewhere between the perfectly undrained and drained idealized conditions. The shear strength of soil depends on the effective stress, the drainage conditions, the density of the particles, the rate of strain, and the direction of the strain.

For undrained, constant volume shearing, the Tresca theory may be used to predict the shear strength, but for drained conditions, the Mohr–Coulomb theory may be used.

Two important theories of soil shear are the critical state theory and the steady state theory. There are key differences between the steady state condition and the steady state condition and the resulting theory corresponding to each of these conditions.

#### 3.21. Undrained Shear Strength

This term describes a type of shear strength in soil mechanics as distinct from drained strength. Conceptually, there is no such thing as the undrained strength of a soil. It depends on a number of factors, the main ones being:

- Orientation of stresses
- Stress path
- Rate of shearing
- Volume of material (like for fissured clays or rock mass)

Undrained strength is typically defined by Tresca theory, based on Mohr's circle as:

$$\sigma_1 - \sigma_3 = 2 \cdot S_u = U.C.S. \tag{3-30}$$

With:

kPa
kPa
kPa
kPa

It is commonly adopted in limit equilibrium analyses where the rate of loading is very much greater than the rate at which pore water pressures, that are generated due to the action of shearing the soil, may dissipate. An example of this is rapid loading of sands during an earthquake, or the failure of a clay slope during heavy rain, and applies to most failures that occur during construction. As an implication of undrained condition, no elastic volumetric strains occur, and thus Poisson's ratio is assumed to remain 0.5 throughout shearing. The Tresca soil model also assumes no plastic volumetric strains occur. This is of significance in more advanced analyses such as in finite element analysis. In these advanced analysis methods, soil models other than Tresca may be used to model the undrained condition including Mohr-Coulomb and critical state soil models such as the modified Cam-clay model, provided Poisson's ratio is maintained at 0.5.

#### 3.22. Drained Shear Strength

The drained shear strength is the shear strength of the soil when pore fluid pressures, generated during the course of shearing the soil, are able to dissipate during shearing. It also applies where no pore water exists in the soil (the soil is dry) and hence pore fluid pressures are negligible. It is commonly approximated using the Mohr-Coulomb equation. (It was called "Coulomb's equation" by Karl von Terzaghi in 1942.) combined it with the principle of effective stress. In terms of effective stresses, the shear strength is often approximated by:

$$\tau = \sigma' \cdot \tan(\phi) + c' \tag{3-31}$$

With:

σ'	$(\sigma - u)$ the effective stress	kPa
σ	Total stress applied normal to the shear plane	kPa
u	Pore water pressure acting on the same plane	kPa
φ	Effective stress friction angle or the angle of internal friction after Coulomb friction	deg
c'	Cohesion	kPa
τ	The shear strength $\tau = S_u$ (or sometimes $c_u$ )	kPa

The coefficient of friction  $\mu$  is equal to tan( $\phi$ ). Different values of friction angle can be defined, including the peak friction angle,  $\phi'_{p}$ , the critical state friction angle,  $\phi'_{cv}$ , or residual friction angle,  $\phi'_{r}$ .

c' is called cohesion, however, it usually arises as a consequence of forcing a straight line to fit through measured values of  $(\tau, \sigma')$  even though the data actually falls on a curve. The intercept of the straight line on the shear stress axis is called the cohesion. It is well known that the resulting intercept depends on the range of stresses considered: it is not a fundamental soil property. The curvature (nonlinearity) of the failure envelope occurs because the dilatancy of closely packed soil particles depends on confining pressure.

#### **Chapter 4: Criteria & Concepts**

#### 4.1. Failure Criteria

After a soil reaches the critical state, it is no longer contracting or dilating and the shear stress on the failure plane  $\tau_{crit}$  is determined by the effective normal stress on the failure plane  $\sigma_n'$  and critical state friction angle,  $\phi_{cv}$ :

$$\tau_{\rm crit} = \sigma_{\rm n} \cdot \tan(\phi_{\rm cv}) \tag{4-1}$$

The peak strength of the soil may be greater, however, due to the interlocking (dilatancy) contribution. This may be stated:

$$\tau_{\text{peak}} = \sigma_n' \cdot \tan(\phi_{\text{peak}}) \tag{4-2}$$

Where  $\varphi_{peak} > \varphi_{cv}$ . However, use of a friction angle greater than the critical state value for design requires care. The peak strength will not be mobilized everywhere at the same time in a practical problem such as a foundation, slope or retaining wall. The critical state friction angle is not nearly as variable as the peak friction angle and hence it can be relied upon with confidence. Not recognizing the significance of dilatancy, Coulomb proposed that the shear strength of soil may be expressed as a combination of adhesion and friction components:

$$\tau = \sigma' \cdot \tan(\phi) + c' \tag{4-3}$$

It is now known that the **c'** and  $\varphi$  parameters in the last equation are not fundamental soil properties. In particular, **c'** and  $\varphi$  are different depending on the magnitude of effective stress. According to Schofield (2006), the longstanding use of **c'** in practice has led many engineers to wrongly believe that **c'** is a fundamental parameter. This assumption that **c'** and  $\varphi$  are constant can lead to overestimation of peak strengths.

#### 4.2. The Phi=0 Concept

When a fast triaxial test is carried out, so an unconsolidated undrained test, it is very well possible that the pore pressures will be equal to the increase of the cell pressure. If a test at high cell pressure is carried out, the only difference with a test with a low cell pressure is the value of the pore pressures. The grain pressures will be almost equal in both cases and the result is, that we will find the same critical Mohr circle. So let's consider a series of unconsolidated undrained (UU) tests. Three specimens are selected and all are consolidated to 110 kPa. This brings the specimens to the end of step 1 in the UU test program. Now the confining pressures are changed to say 70, 140 and 700 kPa, without allowing further consolidation and the sheared undrained. The result, within experimental scatter, is that the shear stress or radius of the Mohr circle is about 35 kPa for each specimen. So what happened?

When the confining pressure was changed, the pore pressure in the fully saturated specimens changed just as much as did the confining pressure, and the effective stress remained unchanged and equal in each specimen. Thus the effective stress remained 110 kPa and each specimen behaved during shear just as did the CU specimen. The shear stress and thus the radius of the Mohr circle did not increase and apparently the specimens did not encounter internal friction. This is called the phi=0 concept. In clays with a very low permeability and at a high deformation rate, like during the cutting of clay, the clay behaves like the internal friction angle is zero. So for cutting processes the phi=0 concept will be applied.

#### 4.3. Factors Controlling Shear Strength of Soils

The stress-strain relationship of soils, and therefore the shearing strength, is affected by:

- 1. Soil composition (basic soil material): mineralogy, grain size and grain size distribution, shape of particles, pore fluid type and content, ions on grain and in pore fluid.
- 2. State (initial): Defined by the initial void ratio, effective normal stress and shear stress (stress history). State can be described by terms such as: loose, dense, over consolidated, normally consolidated, stiff, soft, contractive, dilative, etc.

- 3. Structure: Refers to the arrangement of particles within the soil mass; the manner the particles are packed or distributed. Features such as layers, joints, fissures, slickensides, voids, pockets, cementation, etc., are part of the structure. Structure of soils is described by terms such as: undisturbed, disturbed, remolded, compacted, cemented; flocculent, honey-combed, single-grained; flocculated, deflocculated; stratified, layered, laminated; isotropic and anisotropic.
- 4. Loading conditions: Effective stress path, i.e., drained, and undrained; and type of loading, i.e., magnitude, rate (static, dynamic), and time history (monotonic, cyclic).

The shear strength and stiffness of soil determines whether or not soil will be stable or how much it will deform. Knowledge of the strength is necessary to determine if a slope will be stable, if a building or bridge might settle too far into the ground, and the limiting pressures on a retaining wall. It is important to distinguish between failure of a soil element and the failure of a geotechnical structure (e.g., a building foundation, slope or retaining wall); some soil elements may reach their peak strength prior to failure of the structure. Different criteria can be used to define the "shear strength" and the "yield point" for a soil element from a stress-strain curve. One may define the peak shear strength as the peak of a stress strain curve, or the shear strength at critical state as the value after large strains when the shear resistance levels off. If the stress-strain curve does not stabilize before the end of shear strength test, the "strength" is sometimes considered to be the shear resistance at 15% to 20% strain. The shear strength of soil depends on many factors including the effective stress and the void ratio.

The shear stiffness is important, for example, for evaluation of the magnitude of deformations of foundations and slopes prior to failure and because it is related to the shear wave velocity. The slope of the initial, nearly linear, portion of a plot of shear stress as a function of shear strain is called the shear modulus

#### 4.4. Friction, Interlocking & Dilation

Soil is an assemblage of particles that have little to no cementation while rock (such as sandstone) may consist of an assembly of particles that are strongly cemented together by chemical bonds. The shear strength of soil is primarily due to inter-particle friction and therefore, the shear resistance on a plane is approximately proportional to the effective normal stress on that plane.<sup>[3]</sup> But soil also derives significant shear resistance from interlocking of grains. If the grains are densely packed, the grains tend to spread apart from each other as they are subject to shear strain. The expansion of the particle matrix due to shearing was called dilatancy by Osborne Reynolds. If one considers the energy required to shear an assembly of particles there is energy input by the shear force, T, moving a distance, x and there is also energy input by the normal force, N, as the sample expands a distance, y. Due to the extra energy required for the particles to dilate against the confining pressures, dilatant soils have greater peak strength than contractive soils. Furthermore, as dilative soil grains dilate, they become looser (their void ratio increases), and their rate of dilation decreases until they reach a critical void ratio. Contractive soils become denser as they shear, and their rate of contraction decreases until they reach a critical void ratio.

The tendency for a soil to dilate or contract depends primarily on the confining pressure and the void ratio of the soil. The rate of dilation is high if the confining pressure is small and the void ratio is small. The rate of contraction is high if the confining pressure is large and the void ratio is large. As a first approximation, the regions of contraction and dilation are separated by the critical state line.

#### 4.5. Effective Stress

Karl von Terzaghi (1964) first proposed the relationship for effective stress in 1936. For him, the term 'effective' meant the calculated stress that was effective in moving soil, or causing displacements. It represents the average stress carried by the soil skeleton. Effective stress ( $\sigma$ ') acting on a soil is calculated from two parameters, total stress ( $\sigma$ ) and pore water pressure ( $\mathbf{u}$ ) according to:

$$\sigma' = \sigma - u$$

Typically, for simple examples:

 $\sigma = \gamma_{soil} \cdot \mathbf{H}_{soil}$  and  $\mathbf{u} = \gamma_{w} \cdot \mathbf{H}_{w}$ 

slip plane, assuming it is acting as a confined layer.

(4-4)

(4-5)

However, the formula becomes confusing when considering the true behavior of the soil particles under different measurable conditions, since none of the parameters are actually independent actors on the particles.

Consider a grouping of round quartz sand grains, piled loosely, in a classic 'cannonball' arrangement. As can be seen, there is a contact stress where the spheres actually touch. Pile on more spheres and the contact stresses increase, to the point of causing frictional instability (dynamic friction), and perhaps failure. The independent parameter affecting the contacts (both normal and shear) is the force of the spheres above. This can be calculated by using the overall average density of the spheres and the height of spheres above.

If we then have these spheres in a beaker and add some water, they will begin to float a little depending on their density (buoyancy). With natural soil materials, the effect can be significant, as anyone who has lifted a large rock out of a lake can attest. The contact stress on the spheres decreases as the beaker is filled to the top of the spheres, but then nothing changes if more water is added. Although the water pressure between the spheres (pore water pressure) is increasing, the effective stress remains the same, because the concept of 'total stress' includes the weight of all the water above. This is where the equation can become confusing, and the effective stress can be calculated using the buoyant density of the spheres (soil), and the height of the soil above.

The concept of effective stress truly becomes interesting when dealing with non-hydrostatic pore water pressure. Under the conditions of a pore pressure gradient, the ground water flows, according to the permeability equation (Darcy's law). Using our spheres as a model, this is the same as injecting (or withdrawing) water between the spheres. If water is being injected, the seepage force acts to separate the spheres and reduces the effective stress. Thus, the soil mass becomes weaker. If water is being withdrawn, the spheres are forced together and the effective stress increases. Two extremes of this effect are quicksand, where the groundwater gradient and seepage force act against gravity; and the 'sandcastle effect', where the water drainage and capillary action act to strengthen the sand. As well, effective stress plays an important role in slope stability, and other geotechnical engineering and engineering geology problems, such as groundwater-related subsidence.

#### 4.6. Darcy's law

Darcy's law states that the volume of flow of the pore fluid through a porous medium per unit time is proportional to the rate of change of excess fluid pressure with distance. The constant of proportionality includes the viscosity of the fluid and the intrinsic permeability of the soil.

![](_page_41_Figure_7.jpeg)

Figure 4-1: Diagram showing definitions and directions for Darcy's law.

Q =	$\frac{\mathbf{K} \cdot \mathbf{A}}{\mu} \cdot \frac{\left(\mathbf{u_b} - \mathbf{u_a}\right)}{\mathbf{L}}$	(4-6)
With:		
Q	units of volume per time	m³/s
K	intrinsic permeability	m <sup>2</sup>
k	permeability	m/s
Α	cross sectional area	m <sup>2</sup>
L	Length	m
ua	Start excess pore pressure	Pa
u <sub>b</sub>	End excess pore pressure	Pa

#### μ dynamic viscosity of the fluid

The negative sign is needed because fluids flow from high pressure to low pressure. So if the change in pressure is negative (in the *x*-direction) then the flow will be positive (in the *x*-direction). The above equation works well for a horizontal tube, but if the tube was inclined so that point b was a different elevation than point a, the equation would not work. The effect of elevation is accounted for by replacing the pore pressure by *excess pore pressure*,  $u_e$  defined as:

$$\mathbf{u}_{\mathbf{c}} = \mathbf{u} - \boldsymbol{\rho}_{\mathbf{w}} \cdot \mathbf{g} \cdot \mathbf{z} \tag{4-7}$$

Where z is the depth measured from an arbitrary elevation reference (datum). Replacing **u** by  $\mathbf{u}_{e}$  we obtain a more general equation for flow:

$$\mathbf{Q} = \frac{-\mathbf{K} \cdot \mathbf{A}}{\mu} \cdot \frac{\left(\mathbf{u}_{c,b} - \mathbf{u}_{c,a}\right)}{\mathbf{L}}$$
(4-8)

Dividing both sides of the equation by A, and expressing the rate of change of excess pore pressure as a derivative, we obtain a more general equation for the apparent velocity in the x-direction:

$$\mathbf{q}_{\mathbf{x}} = \frac{-\mathbf{K}}{\mu} \cdot \frac{\mathbf{d}\mathbf{u}_{\mathbf{c}}}{\mathbf{d}\mathbf{x}}$$
(4-9)

Where  $q_x$  has units of velocity and is called the Darcy velocity, or discharge velocity. The seepage velocity ( $v_{sx}$  = average velocity of fluid molecules in the pores) is related to the Darcy velocity, and the porosity, n:

$$\mathbf{v}_{\mathbf{s},\mathbf{x}} = \frac{\mathbf{q}_{\mathbf{x}}}{\mathbf{n}} \tag{4-10}$$

Civil engineers predominantly work on problems that involve water and predominantly work on problems on earth (in earth's gravity). For this class of problems, civil engineers will often write Darcy's law in a much simpler form:

$$\mathbf{q}_{\mathbf{x}} = \mathbf{k} \cdot \mathbf{i}_{\mathbf{x}} \tag{4-11}$$

Where k is called permeability, and is defined as:

$$\mathbf{k} = \mathbf{K} \cdot \frac{\boldsymbol{\rho}_{\mathrm{W}} \cdot \mathbf{g}}{\boldsymbol{\mu}_{\mathrm{W}}} \tag{4-12}$$

And i is called the hydraulic gradient. The hydraulic gradient is the rate of change of total head with distance. Values are for typical fresh groundwater conditions — using standard values of viscosity and specific gravity for water at 20°C and 1 atm.

Pa.s

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k (cm/s)	10 <sup>2</sup>	101	100=1	10 <sup>-1</sup>	10 <sup>-2</sup>	10-3	10-4	10-5	10-6	10 <sup>-7</sup>	10 <sup>-8</sup>	10-9	10 <sup>-10</sup>
k (ft/day)	105	10,000	1,000	100	10	1	0.1	0.01	0.001	0.0001	10-5	$10^{-6}$	10 <sup>-7</sup>
Relative Permeability	Pervious		Semi-Pervious				Impervious						
Aquifer	Good				Poor			None					
Unconsolidated Sand & Gravel	We	ll Sorted Gravel	Well Sorted Sand or Sand & Gravel			r Very Fine Sand, Silt, Loess, Loam							
Unconsolidated Clay & Organic			Pe			eat Layered Clay			Fat / Unweathered Clay				
Consolidated Rocks	Н	ighly Fra	ctured R	ocks	ks Oil F		voir	Fr San	resh dstone	Fres Limest Dolon	sh one, nite	Fr Gra	esh mite

#### Table 4-1: Saturated hydraulic conductivity/permeability (k) values found in nature.

Table 4-2: Ranges of commo	n intrinsic permeabilities (K).
----------------------------	---------------------------------

Permeability		Pervious			Semi-Pervious				Impervious				
Unconsolidated Sand & Gravel	Well Gr	Sorted avel	Well Sa	Sorted nd & (	l Sand or Gravel	Very I	Fine loess,	Sand, , Loam	Silt, 1				
Unconsolidated Clay & Organic					Pea	ıt	Lay	yered (	Clay	Ur	nweathe	red Cl	ay
Consolidated Rocks	High	ly Fractu	ired R	ocks	Oil Rese	rvoir R	.ocks	Fre Sand	esh stone	Fresh Limestone, Dolomite		esh mite	
K (cm <sup>2</sup> )	0.001	0.0001	10-5	10-6	10 <sup>-7</sup>	$10^{-8}$	10-9	10 <sup>-10</sup>	10 <sup>-11</sup>	10 <sup>-12</sup>	10 <sup>-13</sup>	10 <sup>-14</sup>	10 <sup>-15</sup>
K (millidarcy)	10+8	10+7	10+6	10+5	10,000	1,000	100	10	1	0.1	0.01	0.001	0.0001

#### 4.7. Brittle versus Ductile Failure

In rock failure a distinction is made between brittle, brittle ductile and ductile failure. Factors determining those types of failure are the ductility number (ratio compressive strength over tensile strength), the confining pressure and the temperature. During dredging the temperature will have hardly any influence, however when drilling deep oil wells temperature will play an important role. The corresponding failure diagrams are shown in Figure 4-2. The confining pressure, where the failure transit from brittle to ductile is called  $\sigma_{bp}$ .

Brittle failure occurs at relative low confining pressures  $\sigma_3 < \sigma_{bp}$  en deviator stress  $q=\sigma_1-\sigma_3 > \frac{1}{2}q_u$ . The strength increases with the confining pressure, but decreases after the peak strength to a residual value. The presence of pore water can play an important role. Brittle failure types are shown in Figure 4-3.

- Pure tensile failure with or without a small confining pressure.
- Axial tensile failure
- shear plane failure

Brittle ductile failure is also called semi brittle. In the transition area where  $\sigma_3 \approx \sigma_{bp}$ , the deformations are not restricted to local shear planes or fractures but are divided over the whole area. The residual- strength is more or less equal to the peak strength.

Ductile failure. A rock fails ductile when  $\sigma_3 >> q_u$  en  $\sigma_3 > \sigma_{bp}$  while the force stays constant or increases some what with increasing deformation.

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![](_page_44_Figure_1.jpeg)

Figure 4-2: Failure diagrams ductile-brittle (Vlasblom (2003-2007)).

![](_page_44_Figure_3.jpeg)

Figure 4-3: Brittle failure types (Vlasblom (2003-2007)).

#### **Chapter 5: Soil Mechanical Tests**

#### 5.1. Standard Penetration Test

The standard penetration test (SPT) is an in-situ dynamic penetration test designed to provide information on the geotechnical engineering properties of soil. The test procedure is described in the British Standard BS EN ISO 22476-3, ASTM D1586 and Australian Standards AS 1289.6.3.1.

The test uses a thick-walled sample tube, with an outside diameter of 50 mm and an inside diameter of 35 mm, and a length of around 650 mm. This is driven into the ground at the bottom of a borehole by blows from a slide hammer with a weight of 63.5 kg (140 lb) falling through a distance of 760 mm (30 in). The sample tube is driven 150 mm into the ground and then the number of blows needed for the tube to penetrate each 150 mm (6 in) up to a depth of 450 mm (18 in) is recorded. The sum of the number of blows required for the second and third 6 in. of penetration is termed the "standard penetration resistance" or the "N-value". In cases where 50 blows are insufficient to advance it through a 150 mm (6 in) interval the penetration after 50 blows is recorded. The blow count provides an indication of the density of the ground, and it is used in many empirical geotechnical engineering formulae.

![](_page_45_Figure_5.jpeg)

Figure 5-1: The Standard Penetration Test.

The main purpose of the test is to provide an indication of the relative density of granular deposits, such as sands and gravels from which it is virtually impossible to obtain undisturbed samples. The great merit of the test, and the main reason for its widespread use is that it is simple and inexpensive. The soil strength parameters which can be inferred are approximate, but may give a useful guide in ground conditions where it may not be possible to obtain borehole samples of adequate quality like gravels, sands, silts, clay containing sand or gravel and weak rock. In conditions where the quality of the undisturbed sample is suspect, e.g. very silty or very sandy clays, or hard clays, it is often advantageous to alternate the sampling with standard penetration tests to check the strength. If the samples are found to be unacceptably disturbed, it may be necessary to use a different method for measuring strength like the plate test. When the test is carried out in granular soils below groundwater level, the soil may become loosened. In certain circumstances, it can be useful to continue driving the sampler beyond the distance specified, adding further drilling rods as necessary. Although this is not a standard penetration test, and should not be regarded as such, it may at least give an indication as to whether the deposit is really as loose as the standard test may indicate.

The usefulness of SPT results depends on the soil type, with fine-grained sands giving the most useful results, with coarser sands and silty sands giving reasonably useful results, and clays and gravelly soils yielding results which may be very poorly representative of the true soil conditions. Soils in arid areas, such as the Western United States, may exhibit natural cementation. This condition will often increase the standard penetration value.

The SPT is used to provide results for empirical determination of a sand layer's susceptibility to earthquake liquefaction, based on research performed by Harry Seed, T. Leslie Youd, and others.

Despite its many flaws, it is usual practice to correlate SPT results with soil properties relevant for geotechnical engineering design. The reason being that SPT results are often the only test results available, therefore the use of direct correlations has become common practice in many countries.

Different correlations are proposed for granular and cohesive soils.

#### **5.2.** Cone Penetration Test

The cone penetration test (CPT) is an in situ testing method used to determine the geotechnical engineering properties of soils and delineating soil stratigraphy. It was initially developed in the 1950s at the Dutch Laboratory for Soil Mechanics in Delft to investigate soft soils. Based on this history it has also been called the "Dutch cone test". Today, the CPT is one of the most used and accepted in situ test methods for soil investigation worldwide.

The test method consists of pushing an instrumented cone, with the tip facing down, into the ground at a controlled rate (usually 2 centimeters/second). The resolution of the CPT in delineating stratigraphic layers is related to the size of the cone tip, with typical cone tips having a cross-sectional area of either 10 or 15 cm<sup>2</sup>, corresponding to diameters of 3.6 and 4.4 cm.

The early applications of CPT mainly determined the soil geotechnical property of bearing capacity. The original cone penetrometers involved simple mechanical measurements of the total penetration resistance to pushing a tool with a conical tip into the soil. Different methods were employed to separate the total measured resistance into components generated by the conical tip (the "tip friction") and friction generated by the rod string. A friction sleeve was added to quantify this component of the friction and aid in determining soil cohesive strength in the 1960s (Begemann, 1965). Electronic measurements began in 1948 and improved further in the early 1970s (de Reister, 1971). Most modern electronic CPT cones now also employ a pressure transducer with a filter to gather pore water pressure data. The filter is usually located either on the cone tip (the so-called U1 position), immediately behind the cone tip (the most common U2 position) or behind the friction sleeve (U3 position). Pore water pressure data aids determining stratigraphy and is primarily used to correct tip friction values for those effects. CPT testing which also gathers this piezometer data is called CPTU testing. CPT and CPTU testing equipment generally advances the cone using hydraulic rams mounted on either a heavily ballasted vehicle or using screwed-in anchors as a counter-force. One advantage of CPT over the Standard Penetration Test (SPT) is a more continuous profile of soil parameters, with CPTU data recorded typically at 2cm intervals.

In addition to the mechanical and electronic cones, a variety of other CPT-deployed tools have been developed over the years to provide additional subsurface information. One common tool advanced during CPT testing is a geophone set to gather seismic shear wave and compression wave velocities. This data helps determine the shear modulus and Poisson's ratio at intervals through the soil column for soil liquefaction analysis and low-strain soil strength analysis. Engineers use the shear wave velocity and shear modulus to determine the soil's behavior under low-strain and vibratory loads. Additional tools such as laser-induced fluorescence, X-ray fluorescence<sup>[1]</sup>, soil conductivity/resistivity, membrane interface probe and cameras for capturing video imagery are also increasingly advanced in conjunction with the CPT probe. An additional CPT deployed tool used in Britain, Netherlands, Germany, Belgium and France is a piezocone combined with a tri-axial magnetometer. This is used to attempt to ensure that tests, boreholes, and piles, do not encounter unexploded ordnance (UXB) or duds. The magnetometer in the cone detects ferrous materials of 50 kg or larger within a radius of up to about 2 m distance from the probe depending on the material, orientation and soil conditions.

CPT for geotechnical applications was standardized in 1986 by ASTM Standard D 3441 (ASTM, 2004). ISSMGE provides international standards on CPT and CPTU. Later ASTM Standards have addressed the use of CPT for various environmental site characterization and groundwater monitoring activities. Particularly for geotechnical soil investigations, CPT is gaining popularity compared to standard penetration testing as a method of geotechnical soil investigation by its increased accuracy, speed of deployment, more continuous soil profile and reduced cost over other soil testing methods. The ability to advance additional in situ testing tools using the CPT direct push drilling rig, including the seismic tools described above, are accelerating this process.

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![](_page_47_Picture_1.jpeg)

![](_page_47_Figure_2.jpeg)

Figure 5-3: Several configurations of cones.

## The Cutting of Sand, Clay & Rock - Soil Mechanics

![](_page_48_Picture_1.jpeg)

**Figure 5-4: Several cone configurations.** 

#### 5.3. Triaxial Test

A triaxial shear test is a common method to measure the mechanical properties of many deformable solids, especially soil (e.g. sand, clay) and rock, and other granular materials or powders. There are several variations on the test. Although the name triaxial test suggests that the stresses would be different in three directions, this is not true in the test as is usually done. In this test with oil or water as confining medium, the confining pressures are equal in all directions (i.e. in terms of principal stresses: for a compression test:  $\sigma_1 \neq \sigma_2 = \sigma_3$  and for tensile:  $\sigma_1 = \sigma_2 \neq \sigma_3$ ). Only in a true triaxial test the stresses in all three directions can be different (i.e.  $\sigma_1 \neq \sigma_2 \neq \sigma_3$ ). For loose granular materials like sand or gravel, the material is contained in a cylindrical latex sleeve with a flat, circular metal plate or platen closing off the top and bottom ends. This cylinder is placed into a bath of water (mostly water but may be any other fluid) to provide pressure along the sides of the cylinder. The top platen can then be mechanically driven up or down along the axis of the cylinder to squeeze the material. The distance that the upper platen travels is measured as a function of the force required to move it, as the pressure of the surrounding water is carefully controlled. The net change in volume of the material is also measured by how much water moves in or out of the surrounding bath. The test for cohesive (non-loose) materials (e.g. clay, rock) is similar to the test for loose granular materials. For rock testing the sleeve may be a thin metal sheeting rather than latex. Triaxial testing on rock is fairly seldom done because the high forces and pressures required to break a rock sample imply very costly and cumbersome testing equipment available at few laboratories in the world. During the test the pore pressures of fluids (e.g. water, oil) or gasses in the sample may be measured.

The principle behind a triaxial shear test is that the stress applied in the vertical direction (along the axis of the cylindrical sample) can be different from the stresses applied in the horizontal directions perpendicular to the sides of the cylinder, i.e. the confining pressure). In a homogeneous and isotropic material this produces a non-hydrostatic stress state, with shear stress that may lead to failure of the sample in shear. In non-homogeneous and anisotropic samples (e.g. bedded or jointed samples) failure may occur due to bending moments and, hence, failure may be tensile. Also combinations of bending and shear failure may happen in inhomogeneous and anisotropic material.

A solid is defined as a material that can support shear stress without moving. However, every solid has an upper limit to how much shear stress it can support. The triaxial test is designed to measure that limit. The stress on the platens is increased until the material in the cylinder fails and forms sliding regions within itself, known as shear bands. A motion where a material is deformed under shear stress is known as shearing. The geometry of the shearing in a triaxial test typically causes the sample to become shorter while bulging out along the sides. The stress on the platen is then reduced and the water pressure pushes the sides back in, causing the sample to grow taller again. This cycle is usually repeated several times while collecting stress and strain data about the sample. During the shearing, a granular material will typically have a net gain or loss of volume. If it had originally been in a dense state, then it typically gains volume, a characteristic known as Reynolds' dilatancy. If it had originally been in a very loose state, then contraction may occur before the shearing begins or in conjunction with the shearing.

From the triaxial test data, it is possible to extract fundamental material parameters about the sample, including its angle of shearing resistance, apparent cohesion, and dilatancy angle. These parameters are then used in computer models to predict how the material will behave in a larger-scale engineering application. An example would be to predict the stability of the soil on a slope, whether the slope will collapse or whether the soil will support the shear stresses of the slope and remain in place. Triaxial tests are used along with other tests to make such engineering predictions.

The triaxial test can be used to determine the shear strength of a discontinuity. A homogeneous and isotropic sample (see above) fails due to shear stresses in the sample. If a sample with a discontinuity is orientated such that the discontinuity is about parallel to the plane in which maximum shear stress will be developed during the test, the sample will fail due to shear displacement along the discontinuity, and hence, the shear strength of a discontinuity can be calculated.

![](_page_49_Figure_4.jpeg)

Figure 5-5: The Triaxial apparatus.

![](_page_50_Figure_1.jpeg)

Figure 5-6: The Triaxial apparatus cross-section.

There are several variations of the triaxial test:

#### 5.3.1. Consolidated Drained (CD)

In a consolidated drained test the sample is consolidated and sheared in compression with drainage. The rate of axial deformation is kept constant, i.e. is strain controlled. The idea is that the test allows the sample and the pore pressures to fully consolidate (i.e. adjust) to the surrounding stresses. The test may take a long time to allow the sample to adjust, in particular low permeability samples need a long time to drain and adjust stain to stress levels.

#### 5.3.2. Consolidated Undrained (CU)

In a consolidated undrained test the sample is not allowed to drain. The shear characteristics are measured under undrained conditions and the sample is assumed to be fully consolidated under the stresses applied that should be similar to the field conditions. Test in particular used if a change in stress is to happen without time for further consolidation.

#### 5.3.3. Unconsolidated Undrained (UU)

In an unconsolidated undrained test the sample is not allowed to drain. The sample is compressed at a constant rate (strain-controlled).

#### 5.4. **Shear Test**

A direct shear test also known as shear box test is a laboratory or field test used by geotechnical engineers to measure the shear strength properties of soil or rock material, or of discontinuities in soil or rock masses. For soil the U.S. and U.K. standards defining how the test should be performed are ASTM D 3080 and BS 1377-7:1990 respectively to establish the shear strength properties of soil. It is also possible to estimate typical values of the shear strength parameters based on the type and classification of the soils. For rock the test is generally restricted to rock with (very) low (shear) strength. The test is, however, standard practice to establish the shear strength properties of discontinuities in rock.

![](_page_51_Figure_1.jpeg)

Figure 5-8: The vane shear test.

The test is performed on three or four specimens from a relatively undisturbed soil sample. A specimen is placed in a shear box which has two stacked rings to hold the sample; the contact between the two rings is at approximately the mid-height of the sample. A confining stress is applied vertically to the specimen, and the upper ring is pulled laterally until the sample fails, or through a specified strain. The load applied and the strain induced is recorded at frequent intervals to determine a stress-strain curve for the confining stress.

Direct Shear tests can be performed under several conditions. The sample is normally saturated before the test is run, but can be run at the in-situ moisture content. The rate of strain can be varied to create a test of undrained or

drained conditions, depending whether the strain is applied slowly enough for water in the sample to prevent pore-water pressure buildup.

Several specimens are tested at varying confining stresses to determine the shear strength parameters, the soil cohesion (c) and the angle of internal friction (commonly friction angle) ( $\phi$ ). The results of the tests on each specimen are plotted on a graph with the peak (or residual) stress on the x-axis and the confining stress on the y-axis. The y-intercept of the curve which fits the test results is the cohesion, and the slope of the line or curve is the friction angle.

#### 5.5. Point Load Test

The Point Load Strength test is intended as an index test for the strength classification of rock materials. It may also be used to predict other strength parameters with which it is correlated, for example the unconfined compressive and the tensile strength. It is measured in accordance with the procedures recommended in **ASTM D5731**, usually with **NX**-size core samples. The testing machine consists of a loading frame, which measures the force required to break the sample, and a system for measuring the distance between the two platen contact points. Rock specimens in the form of either core, cut blocks, or irregular lumps are broken by application of concentrated load through a pair of spherically truncated, conical platens. The applied force at failure of the sample and distance between the platen tips are recorded in order to calculate the point load index as follows:

$$\mathbf{I}_{s} = \frac{\mathbf{F}}{\mathbf{D}_{e}^{2}}$$
(5-1)

With:		
Is	Point load index	kPa
F	Failure load	kN
D <sub>e</sub>	Distance between platen tips	m
$\mathbf{D}_{\mathbf{e}}^{2}$	= D <sup>2</sup> for diametrical test	m <sup>2</sup>
$\mathbf{D}_{\mathbf{e}}^{2}$	$= 4A/\pi =$ for axial, block and lump test	$m^2$
A	= W.D = minimum cross-sectional area of a plane through the platen contact points	m <sup>2</sup>

![](_page_52_Picture_7.jpeg)

Figure 5-9: Point load test facility.

Another test that is familiar with the Brazilian splitting test is the point load strength test. This test is executed either axial, diametrical or on irregular pieces.

![](_page_53_Figure_1.jpeg)

Figure 5-10: Diametrical point load test (Vlasblom (2003-2007)).

The point load test is frequently used to determine the strength when a large number of samples have to be tested. The tests give for brittle rocks, when tested under diametric loading, values reasonable close to the BTS. Also it is suggested that PLS=0.8\*BTS it is suggested to establish such a relation based on both tests.

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# Chapter 7: List of Symbols Used

Gs	Specific gravity	-
ρ <sub>s</sub>	Density of the soil	kg/m <sup>3</sup>
ρ <sub>w</sub>	Density of water	kg/m <sup>3</sup>
g	Gravitational constant (9.81 m/s <sup>2</sup> )	m/s <sup>2</sup>
$\mathbf{M}_{\mathbf{t}}$	Mass of the soil, total mass	kg
$M_s$	Mass of the solids	kg
$\mathbf{M}_{\mathbf{w}}$	Mass of the water	kg
$M_a$	Mass of the air	kg
$\mathbf{V}_{\mathbf{t}}$	Volume of the soil, total volume	m <sup>3</sup>
$V_s$	Volume of the solids	m <sup>3</sup>
$\mathbf{V}_{\mathbf{w}}$	Volume of the water	m <sup>3</sup>
$V_a$	Volume of the air	m <sup>3</sup>
$\rho_t$	Density of the soil	kg/m <sup>3</sup>
γt	Unit weight of the soil	N/ m <sup>3</sup>
g	Gravitational constant (9.81 m/s <sup>2</sup> )	m/s <sup>2</sup>
$\mathbf{D}_{\mathbf{r}}$	Relative density	-
e	Current void ratio of the soil in-situ	-
e <sub>max</sub>	Void ratio of the soil at its loosest condition	-
e <sub>min</sub>	Void ratio of the soil at its densest condition	-
n	Porosity of the soil in-situ	-
n <sub>max</sub>	Porosity of the soil at its loosest condition	-
n <sub>min</sub>	Porosity of the soil in its densest condition	-
V <sub>v</sub>	Volume of the voids/pores	m <sup>3</sup>
V <sub>s</sub>	Volume of the solids/grains/particles	m <sup>3</sup>
n	Porosity	-
e	Void ratio	-
Ct	Sorting coefficient	-
C	Sorting coefficient	-
К	Hydraulic conductivity	m <sup>2</sup>
k	Permeability	m/s
<b>f</b> ( <b>n</b> )	porosity function	-
С	sorting coefficient	
d <sub>e</sub>	effective grain diameter	mm
<b>d</b> <sub>10</sub>	Grain diameter where 10% is smaller	mm
d <sub>60</sub>	Grain diameter where 60% is smaller	mm
U	Grain uniformity coefficient	-
v	kinematic viscosity	
μ	Dynamic viscosity	Pa.s
$ ho_w$	Water density	kg/m <sup>3</sup>
$\gamma_{w}$	Unit weight of water	N/m <sup>3</sup>
Q	units of volume per time	m <sup>3</sup> /s
К	intrinsic permeability	m <sup>2</sup>
k	permeability	m/s
Α	cross sectional area	m <sup>2</sup>
L	Length	m
u <sub>a</sub>	Start excess pore pressure	Pa
u <sub>b</sub>	End excess pore pressure	Pa
μ	dynamic viscosity of the fluid	Pa.s
c	Cohesion	kPa or $kN/m^2$
U.C.S.	Unconfined Compressive Strength	kPa or $kN/m^2$
V	The total volume of soil	m <sup>3</sup>
n <sub>i</sub>	Initial porosity	-
n <sub>cv</sub>	Porosity at constant volume	-
3	Dilatation	-
$\sigma_{c}$	Unconfined Compressive Strength	kPa
F	Maximum Failure Load	kN
Α	Cross-sectional area of the core sample	m <sup>2</sup>
Ε	Deformation modulus	N/m <sup>2</sup>

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W	Specific work of failure	J/m <sup>3</sup>
στ	Brazilian Tensile Strength (BTS)	kPa
D	Diameter of the core sample	m
F	Maximum Failure Load	kN
L	Length of the core sample	m
Is	Point load index	kPa
F	Failure load	kN
De	Distance between platen tips	m
$\mathbf{D}_{e}^{2}$	$= D^2$ for diametrical test	$m^2$
$\mathbf{D}_{e}^{2}$	$= 4A/\pi =$ for axial, block and lump test	$m^2$
A	= W.D $=$ minimum cross-sectional area of a plane through the platen contact	m <sup>2</sup>
	points	
$\rho_w$	Density of water	kg/m <sup>3</sup>
Zw	Depth below the water table	m
u	Hydrostatic pressure	kPa
g	Gravitational constant	m/s <sup>2</sup>
σ1	the major principal stress	kPa
σ3	the minor principal stress	kPa
τ	the shear strength $\tau = S_u$ (or sometimes $c_u$ )	kPa
Su	the undrained strength	kPa
σ'	$(\sigma - u)$ the effective stress	kPa
σ	Total stress applied normal to the shear plane	kPa
u	Pore water pressure acting on the same plane	kPa
φ	Effective stress friction angle or the angle of internal friction after Coulomb	deg
-	friction	-
c'	Cohesion	kPa
τ	The shear strength $\tau = S_u$ (or sometimes $c_u$ )	kPa

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![](_page_62_Picture_1.jpeg)

![](_page_62_Picture_2.jpeg)

by Dr.ir. Sape A. Miedema

![](_page_62_Picture_3.jpeg)

![](_page_62_Picture_4.jpeg)

#### **Chapter 10: About the Author.**

![](_page_63_Picture_2.jpeg)

Dr.ir. Sape A. Miedema (November 8<sup>th</sup> 1955) obtained his M.Sc. degree in Mechanical Engineering with honours at the Delft University of Technology (DUT) in 1983. He obtained his Ph.D. degree on research into the basics of soil cutting in relation with ship motions, in 1987. From 1987 to 1992 he was assistant professor at the chair of Dredging Technology. In 1992 and 1993 he was a member of the management board of Mechanical Engineering & Marine Technology of the DUT. In 1992 he became associate professor at the DUT with the chair of Dredging Technology. From 1996 to 2001 he was appointed educational director of Mechanical Engineering and Marine Technology at the DUT, but still remaining associate professor of Dredging Engineering. In 2005 he was appointed educational director of the MSc program of Offshore Engineering and he is also still associate professor of Dredging Engineering. In 2011 he was also appointed as director of the MSc program Offshore Engineering of the Petrovietnam University.

Dr.ir. S.A. Miedema teaches (or has taught) courses on soil mechanics and soil cutting, hopper sedimentation. mechatronics, applied thermodynamics, drive system design principles, mooring systems and mathematics. His research focuses on the mathematical modeling of dredging systems like, cutter suction dredges, hopper dredges, clamshell dredges, backhoe dredges and trenchers. The fundamental part of the research focuses on the cutting processes of sand, clay and rock, sedimentation processes in Trailing Suction Hopper Dredges and the associated erosion processes. Lately the research focuses on hyperbaric rock cutting in relation with deep sea mining and on hydraulic transport of sand/water slurries.

# Dredging Processes The Cutting of Sand, Clay & Rock Soil Mechanics

#### By

# Dr.ir. Sape A. Miedema

In dredging, trenching, (deep sea) mining, drilling, tunnel boring and many other applications, sand, clay or rock has to be excavated. The productions (and thus the dimensions) of the excavating equipment range from mm<sup>3</sup>/sec - cm<sup>3</sup>/sec to m<sup>3</sup>/sec. In oil drilling layers with a thickness of a magnitude of 0.2 mm are cut, while in dredging this can be of a magnitude of 0.1 m with cutter suction dredges and meters for clamshells and backhoe's. Some equipment is designed for dry soil, while others operate under water saturated conditions. Installed cutting powers may range up to 10 MW. For both the design, the operation and production estimation of the excavated it is usually transported hydraulically as a slurry over a short (TSHD's) or a long distance (CSD's). Estimating the pressure losses and determining whether or not a bed will occur in the pipeline is of great importance. Fundamental processes of sedimentation, initiation of motion and ersosion of the soil particles determine the transport process and the flow regimes. In TSHD's the soil has to settle during the loading process, where also sedimentation and ersosion will be in equilibrium. In all cases we have to deal with soil and high density soil water mixtures and its fundamental behavior.

This book will enable engineers to understand some basic soil mechanics in order to determine the interaction between the soil and the excavating equipment and is the  $2^{nd}$  of 7 books.

Part 1: The Cutting of Sand, Clay & Rock – Excavating Equipment

Part 2: The Cutting of Sand, Clay & Rock - Soil Mechanics

- Part 3: The Cutting of Sand, Clay & Rock Theory
- Part 4: The Loading Process of a Trailing Suction Hopper Dredge.
- Part 5: The Initiation of Motion of Particles.

Part 6: Hydraulic Transport – Theory.

Part 7: Hydraulic Transport - Experiments.