INTRODUCTION TO RIVER ENGINEERING

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by

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CONTENT

CHAPTER 1

INTRODUCTION

Fluvial hydraulics studies problems related to natural rivers and artificial channels where walls and beds are made of materials capable of being carried away by water flow, so that rivers and creeks allow water and sediment transport.

Sediments are formed by particles of all sizes which, coming from the soil and rocks of a basin, are carried away and transported by water flow. Therefore, organic materials or solved salts are not included in sediments.

Even if fluvial hydraulics may usually apply different methods to quantify a phenomenon, it is important to remember that they can lead to very different results. And that the best method for a particular case can only be determined by experience or a good knowledge of the river stretch under study.

1.1 System of units

The laws that govern physical phenomena are expressed through relations or equations among physical magnitudes which have to be measured by means of a system of units. To perform this operation it is necessary to choose some fundamental magnitudes and to allot each of them to a unit, being both, the magnitudes derived and their units, a function of the fundamental ones.

In order to study the problems involved in hydraulics, it is generally necessary to deal with three fundamental units. In accordance with the problem studied, two systems of units are considered in this book: the first and most usual is the gravitatory, also known as the technical system. Its fundamental units for hydraulics are

Magnitude	Dimension	Unit	
force	F	kgf	(force kilogram)
length	L	m	(meters)
time	T	s	(seconds)
temperature	θ	°C	(Celsius degrees)

The most important derived magnitude is mass

Mass	M	$kgf \cdot s^2/m$ (technical unit of mass)

One technical unit of mass is the mass which, under the action of a force of 1 kgf, produces an acceleration of 1 m/s^2.

The second system is the International (SI); it was proposed in 1960, during the XI General Conference on Weights and Measures, attended by 40 countries; it is based on the MKS absolute system, being its fundamental units:

Magnitude	Dimension	Unit	
mass	M	kg	(kilogram)
length	L	m	(meters)
time	T	s	(seconds)
temperature	θ	K	(Kelvin)

The most important derived magnitude corresponds to force

force	F	N	(newton)

One newton is the force which has to be applied to a 1 kg mass to produce an acceleration of 1 m/s^2. Its units are $kg \cdot m/s^2$.

Table 1.1 shows the principal magnitudes with their respective units and dimensions for both systems.

To be able to do the conversions of these magnitudes from one system to the other, the Newton's second Law is applied by means of gravity acceleration g, that is

$$F = m g \tag{1.1}$$

The units of gravity acceleration are m/s^2, and its value is obtained by

$$g = 9.7803\,(1 + 5.24 \times 10^{-3}\ \text{sen}^2\ \theta)(1 - 3.15 \times 10^{-7}\ h) \tag{1.2}$$

where

- θ latitude of the place, in degrees
- h altitude of the place over sea level, in m

Even if standard gravity is equal to 9.80665 m/s^2, for engineering purposes 9.81 m/s^2 is normally used. Therefore, and according with eq 1.1

$$1\ \text{kgf} = 9.81\ \text{kg}\cdot\text{m/s}^2 \tag{1.3}$$

or

$$1\ \text{kgf} = 9.81\ \text{N} \tag{1.4}$$

this means that, on the earth surface, one mass kilogram will weight 9.81 N according with the simplification accepted in eq 1.4.

When dealing with temperature, conversion between kelvin units, K, and celcius degrees, °C, is given by

$$K = 273.16 + {}^\circ C \tag{1.5}$$

Formerly, kelvin unit was called kelvin degrees, °K.

1.1.2 Prefixes

To avoid the inconvenience of writing figures too big or small, the use of prefixes before the names of units is employed, especially when their factor is 10^{3n}, where n is a whole number, either positive or negative; however, the exception for exponents 1, 2 and -1, -2 is normal.

The accepted prefixes and their symbols are:

Prefix	Symbol	Factor by which the unit is multiplied
Exa	E	10^{18}
Peta	P	10^{15}
Tera	T	10^{12}
Giga	G	10^{9}
Mega	M	10^{6}
Kilo	k	10^{3}
Hecto	h	10^{2}
Deca	da	10
Deci	d	$10^{\ 1}$
Centi	c	10^{-2}
Mili	m	10^{-3}
Micro	μ	10^{-6}
Nano	n	10^{-9}
Pico	p	10^{-12}
Fenito	f	10^{-15}
Atto	a	10^{-18}

1.2 Water properties

Fluvial hydraulics is mainly concerned with water and sediments. In spite of the fact that fluvial models also include oil and air, this paper only considers water as a fluid.

Some of the properties of water are inherent to matter like, for instance, inertia, specific weight, density and specific gravity, while others are common to fluids, like viscosity and compressibility.

1.2.1 Inertia

According to Newton's First Law, inertia is an intrinsic property of matter. It establishes that a body by itself can not modify its state of equilibrium or its uniform movement. Therefore, it needs an external, unbalanced force to change its state. The external manifestation of this property is the force of inertia and its magnitude is obtained from Newton's Second Law. This says that an unbalanced force acting on a body causes a change in the acceleration in the same direction of the acting force, which is directly proportional to it, and inversely proportional to the body mass. It is expressed as

$$\vec{F} = m \frac{d\vec{V}}{dt} = m\,\vec{a} \tag{1.6}$$

Then, with the application of eq 1.6, if the acting force and the acceleration are known, the inertial mass of a body can be known

$$m = \frac{F}{a} \tag{1.7}$$

Therefore, it can be said that the dimensional expressions of eqs 1.6 and 1.7 are, respectively

$$\{F\} = \{M\,L\,T^{-2}\}$$

$$\{M\} = \{F\,L^{-1}\,T^{2}\}$$

where the dimensions between parenthesis are the already explained: M (mass), F (force), L (length) and T (time).

Within the limits of experimental exactitude, the inertial mass m_i of all bodies is proportional to the gravitatory mass, m_g. According to Eotvos determinations, $m_g/m_i = 1.003$. Consequently, it can be considered without error that $m_g = m_i$; this means that the word mass can be applied regardless which of them is implied.

1.2.2 Density

Density is defined as the mass of a substance contained in a unit of volume

$$\rho = \frac{m}{\forall} \tag{1.8}$$

In homogeneous bodies, density is a property which refers to all parts of the body. If they are heterogeneous, density varies from one point to the other. To obtain the density of a specific point, it is necessary to consider the mass Δm, contained in a volume $\Delta\forall$; then, density is determined as

$$\rho = \lim_{\Delta\forall \to 0} \frac{\Delta m}{\Delta \forall} \tag{1.9}$$

$$\rho = \frac{dm}{d\forall} \tag{1.10}$$

The dimensions of density are $\{\rho\} = \{ML^{-3}\} = \{FL^{-4}T^{2}\}$

According to the SI, its units are

$$kg/m^3$$

and in the technical system

$$kgf \cdot s^2/m^4$$

In the case of fluvial hydraulics, the density of water, of the particles that constitute sediments, and the mixture water-sediment are very important.

Density of water varies with temperature. Tables 1.2 and 1.3 show some values for each of the already mentioned system of units.

1.2.2.1 Specific gravity

The specific gravity of two substances is obtained by comparing them. In practice, the specific gravity of solid and liquid substances are compared with water at 4° C, which is equal to 1000 kg/m³. Therefore, specific gravity is a number or non dimensional parameter

$$S_s = \frac{\rho_s}{\rho} \tag{1.11}$$

where

ρ_s specific density of a substance s

ρ specific density of water at 4 °C

S_s specific gravity of a substance s

1.2.3 Specific weight, γ

It is the weight of a substance contained in the unit of volume, that is why it depends on the gravitational field where the body is. This definition implies that there are no voids in the volume occuppied by the material, and it is expressed as

$$\gamma = \frac{W}{\forall} \tag{1.12}$$

where W is the substance weight in volume ∀, and ∀ is the volume of reference.

Its dimensions are $\{\gamma\} = \{ML^{-2}\} = \{FL^{-3}\}$.

In SI, its units are

$$kg/m^2 \cdot s^2$$
$$N/m^3$$

and in the technical system

$$kgf/m^3$$

According to Newton's Second Law (eq 1.1) specific gravity relates to density, and it can be written as

$$\gamma = \rho\, g \tag{1.13}$$

where g is the acceleration produced by the gravitational field.

Taking into account eq 1.13 in eq 1.11, the specific gravity of a material or substance may also be obtained through the application of the relation

$$S_s = \frac{\gamma_s}{\gamma} \tag{1.14}$$

where γ_s and γ are, respectively, the specific gravity of the substance s and of water at 4°C.

1.2.4 Viscosity

1.2.4.1 Dynamic viscosity, μ

It is the resistance of fluids to internal flow or to angular deformation. This resistance offered by a moving fluid is inversely proportional to the velocity according to which deformation or displacement of the particles is produced.

Newton established that in a moving fluid, the tangential force or the internal friction force F_s, produced by a unit of area or tangential stress τ, is proportional to the transversal gradient of velocities du/dy, that is to say

$$\frac{F_s}{A} = \tau \simeq \frac{du}{dy} \qquad (1.15)$$

or

$$\tau = \mu \frac{du}{dy} \qquad (1.16)$$

the constant of proporcionality μ is called dynamic or absolute viscosity and it is a characteristic magnitude of fluid viscosity.

For eq 1.16 to be dimensionaly correct, μ dimensions must be

$$\{\mu\} = \{M\ L^{-1}\ T^{-1}\} = \{F\ L^{-2}\ T\}$$

being its units, according to SI

$$kg/m \cdot s$$

or

$$N \cdot s/m^2$$

and, in the technical system

$$kgf \cdot s/m^2$$

Another unit largely employed is poise

$$1 \text{ poise} = 1 \text{ gr/cm}\cdot\text{s} = 0.1 \text{ kg/m}\cdot\text{s}$$

or

$$1 \text{ poise} = 0.010194 \text{ kgf}\cdot\text{s/m}^2$$

When in a fluid, μ is constant for being independent of the deformations it suffers, it is called Newtonian, because in 1687, Newton was the first to postulate the law of resistance expressed by eq 1.16.

Therefore, Newtonian fluids have a dynamic viscosity, being its value independent of the movement they are subjected to. An exact definition of a Newtonian fluid can be expressed by saying that it is the liquid that presents a linear relationship between velocity of deformation and the stresses that produce it.

Dynamic viscosity varies especially with fluid temperature; this variation is direct for gases and inversal for liquids. Tables 1.2 and 1.3 show some values of the dynamic viscosity of water in relation to temperature. In table 1.2, those values are expressed in decapoises.

1.2.4.2 Kinematic viscosity, ν

In the solution of problems which include viscosity, it is a common practice to apply the relation between the dynamic viscosity of the fluid and its density, relation known as kinematic viscosity, ν. It has the advantage of not presenting force or mass units in its units, but only of length and time (kinematic units).

$$\nu = \frac{\mu}{\rho} \tag{1.17}$$

Its dimensions are $(\nu) = (L^2\ T^{-1})$ and, for both systems, its units are m^2/s.

Another very usual unit is the stoke; its equivalence is

$$1 \text{ stoke} = 0.0001 \text{ m}^2/\text{s} = 1 \text{ cm}^2/\text{s}$$

The kinematic viscosity of fluids only varies with temperature; but in the case of gases it changes with temperature and pressure. Tables 1.2 and 1.3 give some values of the kinematic viscosity of water. In real fluids, viscosity has a value different from zero. Air and water are Newtonian fluids with low viscosity.

1.3 References

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Maza Alvarez, J.A., and García Flores, M., "Hidrodinámica (bases para hidráulica fluvial)" Publicación D-20 del Instituto de Ingeniería, Universidad Nacional Autónoma de México, Mexico, D.F., March 1984

TABLE 1.1. MAGNITUDES AND UNITS USED IN FLUVIAL HYDRAULICS

Magnitude	Symbol	International System			Technical System	
		Dimension	Units		Dimension	Units
FUNDAMENTAL MAGNITUDES						
Mass	M	$\{M\}$	kg mass	kg	———	———
Force	F	———	———	———	$\{F\}$	kg force kgf
Length	L	$\{L\}$	meter	m	$\{L\}$	meter m
Time	T	$\{T\}$	second	s	$\{T\}$	second s
GEOMETRICAL MAGNITUDES						
Area	S	$\{L^2\}$		m^2	$\{L^2\}$	m^2
Volume	$\forall$	$\{L^3\}$		m^3	$\{L^3\}$	m^3
Angle		Non dimensional	Radians	rad	Non dimensional	rad
KINEMATIC MAGNITUDES						
Velocity	V	$\{LT^{-1}\}$		m/s	$\{LT^{-1}\}$	m/s
Acceleration	a	$\{LT^{-2}\}$		m/s^2	$\{LT^{-2}\}$	m/s^2
Discharge	Q	$\{L^3 T^{-1}\}$		m^3/s	$\{L^3 T^{-1}\}$	m^3/s
Unit discharge	q	$\{L^2 T^{-1}\}$	$m^3/s \cdot m =$	m^2/s	$\{L^2 T^{-1}\}$	$m^3/s \cdot m = m^2/s$
Kinematic viscosity	ν	$\{L^2 T^{-1}\}$	Stoke (*)	m^2/s	$\{L^2 T^{-1}\}$	m^2/s
DYNAMIC MAGNITUDES						
Force	F	$\{MLT^{-2}\}$	Newton (*)	$kg \cdot m/s^2$	$\{F\}$	kgf
Mass	M	$\{M\}$	kg mass(*)	kg	$\{FT^2 L^{-1}\}$	UTM(*)$kgf \cdot s^2/m$
Specific weight	γ	$\{ML^{-2}T^{-2}\}$		$kg/m^2 \cdot s^2$	$\{FL^{-3}$	kgf/m^3
Density	ρ	$\{ML^{-3}\}$		kg/m^3	$\{FT^2 L^{-4}\}$	$kgf \cdot s^2/m^4$
Relative density	S_s	Non dimensional			Non dimensional	
Pressure	p	$\{ML^{-1} T^{-2}\}$	Pascal (*)	$kg/m \cdot s^2$	$\{FL^{-2}\}$	kgf/m^2
Shear stress	τ	$\{ML^{-1} T^{-2}\}$		$kg/m \cdot s^2$	$\{FL^{-2}\}$	kgf/m^2
Elastic module	E	$\{ML^{-1} T^{-2}\}$		$kg/m \cdot s^2$	$\{FL^{-2}\}$	kgf/m^2
Compressibility	$\bar{C}$	$\{M^{-1} LT^2\}$		$m \cdot s^2/kg$	$\{F^{-1} L^2\}$	m^2/kgf
Surface tension	σ	$\{MT^{-2}\}$		kg/s^2	$\{FL^{-1}\}$	kgf/m
Dynamic viscosity	μ	$\{ML^{-1} T^{-1}\}$	Poise (*)	$kg/m \cdot s$	$\{FL^{-2} T\}$	$kgf \cdot s/m^2$
Impulse-momentum	C	$\{MLT^{-1}\}$		$kg \cdot m/s$	$\{FT\}$	$kgf \cdot s$
Work, energy	W, E	$\{ML^2 T^{-2}\}$	Joule(J)(*)	$kg \cdot m^2/s^2$	$\{FL\}$	$kgf \cdot m$
Power	P	$\{ML^2 T^{-3}\}$	Watt (W)(*)	$kg \cdot m^2/s^3$	$\{FLT^{-1}\}$	$kgf \cdot m/s$

(*) 1 stoke = 1 cm^2/s
1 newton = 1N = 1 $kg \cdot m/s^2$
1 kg = 1 000 gr
1 pascal = $1N/m^2$ = 1 $kg/m \cdot s^2$
1 poise = 1 $gr/cm \cdot s$ = 0.1 $kg/m \cdot s$
1 joule = 1J = $1N \cdot m$ = 1 $kg \cdot m^2/s^2$
1 watt = 1W = 1J/s = 1 $kg \cdot m^2/s^3$
1 UTM = 1 $kgf \cdot s^2/m$ (technical unit of mass)

TABLE 1.2 VALUES OF SOME WATER PROPERTIES, ACCORDING TO SI

Temperature		Density	Specific weight (1)	Kinematic viscosity	Dynamic viscosity	Surface tension (water-air)
θ		ρ	γ	$\nu \times 10^6$	$\mu \times 10^3$	$\sigma \times 10^2$
°C	K	kg/m^3 $N{\cdot}s^2/m^4$	$kg/m^2.s^2$ N/m^3	m^2/s	kg/m.s $N{\cdot}s/m^2$	kg/s^2 N/m
0	273.16	999.8	9 808.04	1.7925	1.7921	7.564
1	274.16	999.9	9 809.02	1.7312	1.7311	7.550
2	275.16	999.9	9 809.02	1.6738	1.6736	7.536
3	276.16	999.9	9 809.02	1.6194	1.6192	7.521
4	277.16	1 000.0	9 810.00	1.5677	1.5677	7.507
5	278.16	1 000.0	9 810.00	1.5188	1.5188	7.493
6	279.16	999.9	9 802.02	1.4725	1.4723	7.479
7	280.16	999.9	9 809.02	1.4283	1.4281	7.465
8	281.16	999.8	9.808.04	1.3863	1.3860	7.450
9	282.16	999.8	9.808.04	1.3462	1.3459	7.436
10	283.16	999.73	9 807.35	1.3077	1.3077	7.422
11	284.16	999.64	9 806.47	1.2716	1.2712	7.407
12	285.16	999.53	9 805.39	1.2368	1.2362	7.393
13	286.16	999.40	9 804.11	1.2036	1.2029	7.377
14	287.16	999.27	9 802.84	1.1718	1.1709	7.363
15	288.16	999.13	9.801.47	1.1413	1.1303	7.349
16	289.16	999.07	9 800.88	1.1119	1.1109	7.334
17	290.16	998.80	9 798.23	1.0841	1.0828	7.320
18	291.16	998.62	9 796.46	1.0572	1.0558	7.304
19	292.16	998.44	9 794.70	1.0314	1.0298	7.290
20	293.16	998.23	9 792.64	1.0066	1.0049	7.275
21	294.16	998.04	9.790.77	0.9828	0.9809	7.260
22	295.16	997.80	9 788.42	0.9599	0.9578	7.245
23	296.16	997.57	9 786.16	0.9378	0.9353	7.228
24	297.16	997.32	9 783.71	0.9166	0.9142	7.212
25	298.16	997.08	9.781.35	0.8961	0.8935	7.197
26	229.16	996.81	9 778.71	0.8764	0.8736	7.182
27	300.16	996.54	9 776.06	0.8574	0.8544	7.167
28	301.16	996.26	9 77331	0.8390	0.8359	7.150
29	302.16	995.94	9 770.17	0.8213	0.8180	7.135
30	303.16	995.62	9 767.03	0.8043	0.8007	7.118
35	308.16	994.04	9 751.53	0.7269	0.7225	7.037
40	313.16	992.16	9 733.090	0.6611	0.6560	6.956
45	318.16	990.20	9 713.862	0.6047	0.5982	6.873
50	323.16	988.14	9 693.653	0.5560	0.5494	6.791
55	328.16	985.22	9 665.008	0.5140	0.5064	6.704
60	333.16	893.28	9 645.977	0.4766	0.4687	6.618
65	338.16	980.39	9 617.626	0.4442	0.4355	6.529
70	343.16	977.52	9 589.471	0.4154	0.4060	6.440
75	348.16	974.66	9 561.415	0.3897	0.3799	6.350
80	353.16	971.82	9 533.554	0.3668	0.3564	6.260
85	358.16	968.99	9 505.792	0.3462	0.3354	6.167
90	363.16	965.25	9 469.103	0.3279	0.3165	6.075
95	368.16	961.54	9 432.707	0.3114	0.2994	5.982
100	373.16	957.85	9 396.509	0.2963	0.2838	5.890

1 Considering $g = 9.81\ m/s^2$

TABLE 1.3. VALUES OF SOME WATER PROPERTIES, ACCORDING TO TECHNICAL SYSTEM

Temperature	Density	Specific weight	Kinematic viscosity	Dynamic viscosity	Surface tension
T	ρ	γ	$\nu \times 10^6$	$\mu \times 10^4$	$\sigma \times 10^3$
°C	$kgf \cdot s^2/m^4$	kgf/m^3	m^2/s	$kgf \cdot s/m^2$	kgf/m
0	101.95	999.8	1.7935	1.8269	7.71
1	101.96	999.9	1.7312	1.7651	7.70
2	101.96	999.9	1.6738	1.7066	7.68
3	101.96	999.9	1.6194	1.6511	7.67
4	101.97	1 000.0	1.5677	1.5986	7.65
5	101.97	1 000.0	1.5188	1.5487	7.64
6	101.96	999.9	1.4725	1.5014	7.64
7	101.96	999.9	1.4283	1.4563	7.61
8	101.95	999.8	1.3863	1.4133	7.59
9	101.95	999.8	1.3462	1.3725	7.58
10	101.94	999.73	1.3077	1.3331	7.57
11	101.93	999.64	1.2716	1.2961	7.55
12	101.92	999.53	1.2368	1.2605	7.54
13	101.91	999.40	1.2036	1.2266	7.52
14	101.90	999.27	1.1718	1.1941	7.51
15	101.88	999.13	1.1413	1.1628	7.49
16	101.88	999.07	1.1119	1.1328	7.48
17	101.85	998.80	1.0841	1.1042	7.46
18	101.83	998.62	1.0572	1.0765	7.44
19	101.81	998.44	1.0314	1.0501	7.43
20	101.79	998.23	1.0066	1.0246	7.42
21	101.77	998.04	0.9828	1.0002	7.40
22	101.75	997.80	0.9599	0.9767	7.38
23	101.72	997.57	0.9378	0.9539	7.37
24	101.70	997.32	0.9166	0.9322	7.35
25	101.67	997.08	0.8961	0.9111	7.34
26	101.65	996.81	0.8764	0.8909	7.32
27	101.62	996.54	0.8574	0.8713	7.31
28	101.59	996.26	0.8390	0.8523	7.29
29	101.56	995.94	0.8213	0.8341	7.27
30	101.52	995.62	0.8043	0.8165	7.26
35	101.36	994.04	0.7269	0.7368	7.17
40	101.17	992.16	0.6611	0.6688	7.09
45	100.97	990.20	0.6047	0.6106	7.01
50	100.76	988.14	0.5560	0.5602	6.92
55	100.46	985.22	0.5140	0.5164	6.83
60	100.27	983.28	0.4766	0.4779	6.75
65	99.97	980.39	0.4442	0.4441	6.66
70	99.68	977.52	0.4154	0.4141	6.56
75	99.39	974.66	0.3867	0.3873	6.47
80	99.10	971.82	0.3668	0.3635	6.38
85	98.81	968.99	0.3462	0.3421	6.29
90	98.43	965.25	0.3279	0.3228	6.19
95	98.05	961.54	0.3114	0.3053	6.10
100	97.67	957.85	0.2963	0.2894	6.00

CHAPTER 2

PROPERTIES OF SEDIMENTS

If we only consider the resistance that materials offer to being carried away and their behaviour when transported in a channel or river, three kinds of materials can be distinguished

- non cohesive or granular
- cohesive and
- rocky

Granular material is formed by loose particles. The force a liquid must exert to move them is a function of the weight of each particle. Another factor to take into account is friction, which is also a function of weight.

Cohesive material is formed by very small particles kept together by cohesive forces which oppose particle transport. The tiding or cohesive force that hinders the transport of particles by current flows is considerably higher than particle weight, that is why, once cohesion is overcome, the particle lifted up may behave as if it were granular, although it will be transported in suspension due to its reduced weight and size.

On the other side, rocky material is not moved or eroded by a current flow during the useful life of hydraulic works.

If the material is fractured and flow energy is very high, it behaves as if it were granular.

Because of its frequency, the interaction between the flow and the granular soils has been studied more thoroughly and widely than that between the current and the cohesive soils. This frequency of occurrence also results in a greater number of studies about the properties of granular materials. However, some of their characteristics can also be applied to cohesive ones.

The properties of granular soil particles which must be known to solve problems of fluvial hydraulics are

1. density and specific weight
2. shape
3. size
4. fall velocity

Natural sediments are formed by an enourmous variety of particles with various sizes and shapes and, therefore, with different fall velocities.

But the behaviour of an isolated particle subject to the action of a flow differs from its behaviour when it is part of a sediment. Thus, in order to understand sediment dynamics, the properties of a bulk of particles must be known, among which the most important are

5. granulometric distribution
6. volumetric weight

When suspended, fine particles can remain some time in that state. In this case, it is necessary to know the

7. concentration of suspended particles in a unit weight or volume of water
8. specific weight of the fluid which contains suspended material
9. viscosity of the fluid which contains suspended material

In connection to fluvial hydraulics and when dealing with cohesive soils, the most important points to be taken into account are dry volumetric weight and resistance to shear stress.

In the following paragraphs and in the given order these nine properties will be dealt with.

2.1 Density and specific weight of solids

A particle density is the relation of its mass divided by its volume. It is designed by ρ_s and expressed in kg/m^3, according to SI, and kgf·s^2/m^4 in the technical or gravitational system (see 1.2.2).

A particle specific weight is defined as the ratio of its weight divided by its volume. It is designed by γ_s and expressed in kg/m^2 · s^2 according to SI, or kgf/m^3 of the technical or gravitational system (see 1.2.3).

The relation between the specific weight and the density of a particle is given by Newton's Second Law (eq I.13).

$$\gamma_s = g\,\rho_s \tag{2.1}$$

where g, is the acceleration due to gravity, in m/s^2

Specific gravity S_s is defined as the ratio between the density of a given material and the density of water at 4°C (see 1.2.2.1).

$$S_s = \frac{\rho_s}{\rho} = \frac{\gamma_s}{\gamma} \tag{2.2}$$

Both the density and the specific weight of granular soil particles have a very small range of variation because they only depend on the original rock composition and the minerals it contains. This case differs from size and shape which can vary in a wide range.

Generally, the specific weight value for boulders and cobbles varies from 1800 to 2800 kgf/m^3, for gravels from 2100 to 2400 kgf/m^3 and, for sands, from 2600 to 2700 kgf/m^3. However, particles with quite different specific weights may exist.

Since a considerable proportion of sand is formed by quartz particles, the density specific weight and specific gravity of these particles are normally considered with the following values

Parameter	S I	G S
ρ_s	2650 kg/m³	270.13 kgf·s²/m⁴
γ_s	25996.5 N/m³	2650 kgf/m³
S_s	2.65	2.65

2.1.1 Density and specific weight of a submerged solid particle

The density of a submerged solid particle is given by the expression

$$\rho'_s = \rho_s - \rho \tag{2.3}$$

The specific weight of a submerged solid particle corresponds to the weight of the substance the particle is made of and contained in the volume unit inside water. It is given by the relation

$$\gamma'_s = \gamma_s - \gamma \tag{2.4}$$

Being the units of ρ'_s and γ'_s equal to those already given for density and specific weight, respectively.

2.1.2 Relative density of a submerged solid particle

It corresponds to the ratio between specific density of a submerged solid particle and the specific density of water. It is expressed as Δ and it is equal to

$$\Delta = \frac{\rho_s - \rho}{\rho} = \frac{\gamma_s - \gamma}{\gamma} \tag{2.5}$$

or

$$\Delta = S_s - 1 \tag{2.6}$$

ρ and γ are normally applied for water at 4°C. For quartz particles, the common values of ρ'_s, γ'_s and Δ are

Parameter	S I	G S
ρ'_s	1650 kg/m^3	168.20 kgf$\cdot$ s^2/m^4
γ'_s	16186.50 N/m^3	1650 kgf/m^3
Δ	1.65	1.65

2.2 Shape of particles

The shape of particles can vary greatly and it has an important influence in their behaviour at the start of transport or when carried by water.

Particles may have the approximate shape of spheres, disks, ellipsis, cilinders, etc, but they can also be completely irregular. In general the shape of particles is not considered in the equations, however, the best known attempt to take it into account is the called shape factor (S.F.), proposed by Corey

$$S.F. = \frac{c}{\sqrt{a \cdot b}} \tag{2.7}$$

where a, b and c are three lengths of the particle measured in three perpendicular directions. a represents the maximum length of the particle, c is the minimum length perpendicular to a, while b is perpendicular to a and b.

When calculating particle fall velocity it is necessary to consider its shape. Fig 2.3 shows the value of fall velocity for different shapes, as a function of the diameter and the shape factor of a particle (see 2.4.2). Even if when studying the start of particle movement its shape ought to be taken into account, up to now this has not been done.

2.3 Particle size

Particles found in natural beds may vary their size from colloidal to rocks several meters in diameter.

With the purpose of giving a common designation to particles of the same size, the *American Geophysical Union* classification of table 2.1 is adopted.

Following a geometrical progression of two, each group in this classification is subdivided in several intermediate classes. Table 2.1 shows the complete list.

Size of particles is known by direct measurement if they are boulders or cobbles; by size distribution analysis if they are gravels and sands, and silts and clays through sedimentary studies. However, all of them must consider one of the following dimensions:

a) *Sieve diameter*, which is the minimum sieve opening through which the particle passes. It is used in the determination of the size of gravels or sands and its value is similar to the length b of the particle.

b) *Sedimentation diameter*. It is the diameter of a sphere which has the same density of a particle, and which, being in water, reaches the same fall velocity at the same temperature. It is useful in the size determination of very fine particles such as silts and clays.

c) *Fall diameter or standard fall diameter*, which is the diameter of a sphere with a relative density of 2.65, and which, being in water at 4°C, reaches the same fall velocity of the particle.

d) *Nominal diameter*, which corresponds to the diameter of a sphere with the same density and volume of the particle.

e) *Triaxial diameter*, it is the arithmetic mean of three lengths measured along three mutually perpendicular axis

$$D = \frac{a + b + c}{3} \tag{2.8}$$

being a the maximum length of the particle, c the minimum length perpendicular to the maximum one, and b the remaining length which is perpendicular to the other two.

The triaxial diameter is used to determine the size of boulders and cobbles and, occasionally, of gravels too.

Due to the fact that for natural particles the seiving and the triaxial diameter are approximately the same, whenever possible the first is used because it is easier to obtain.

2.4 Fall velocity of a particle

Fall velocity is the maximum velocity a particle can reach when falling into water. It takes place when the submerged weight is in equilibrium with the drag force the water exerts against the particle. Besides particle weight, fall velocity takes into account the size and shape of the particle. All these element make of it a useful tool in fluvial hydraulics. It has to be noted that submerged weight refers to the weight a body has within water.

The fall velocity of a particle depends on its diameter, shape, relative position of its shape with respect to fall direction, specific weight both of particle and water, surface texture of the particle and liquid viscosity. Besides, fall velocity is influenced by conditions like the proximity of the particle to the container walls where it falls, the presence of other particles and water flow (see 2.4.3).

When working with river problems, the engineers, and also quite often the investigators, do not know, the real fall velocity of particles. To obtain it they generally apply other authors' curves or formulae or considere it as a function of the diameter or, occasionally, of shape factor and water temperature. This is not in general carefully measured but simply supposed and almost always considered as constant for long periods of time, even a whole year. Such a common attitude is in part due to the fact that there is not a simple and standardized operational system with which to obtain the fall velocity. This is why this parameter loses effectiveness and the true theoric utility it has.

2.4.1 Fall velocity of a sphere

The general expression to obtain the fall velocity of a sphere is

$$\omega = \left(\frac{4\, g\, \Delta\, D}{3\, C_D} \right)^{1/2} \qquad (2.9)$$

where

- ω fall velocity, in m/s
- D particle diameter, in m
- C_D drag coefficient which depends on the Reynolds number, $\mathbb{R}_D = \omega D/\nu$, obtained using fig 2.1
- Δ $(\gamma_s - \gamma)/\gamma$, (eq 2.5)

To obtain the fall velocity of a spheric particle where diameter D is known, the following steps can be followed

a. A drag coefficient C_D is assumed

b. The fall velocity ω is calculated with eq 2.9

c. The Reynolds number for the particle is obtained, $\mathbb{R} = \omega D/\nu$

d. Once the Reynolds number is known, the application of fig 2.1 provides a new drag coefficient, C_D

e. Steps b and d are repeated until the drag coefficient used in step b is equal to the drag coefficient of step d.

In some regions of fig 2.1, coefficient C_D can be written in analytical form as a function of the Reynolds number. For the sake of simplicity, those expressions can be substitute in eq 2.9.

a) When the Reynolds number is under 0.1

For a Reynolds number $\mathbb{R}_D = \omega D/\nu < 0.1$ (or $\mathbb{R}_D < 1$ for engineering purposes) Stokes obtained

$$C_D = \frac{24}{\mathbb{R}_D} = \frac{24\nu}{\omega D} \qquad (2.10)$$

where ν is the kinematic viscosity, in m^2/s

Substituting eq 2.10 in eq 2.9, it results

$$\omega = \frac{g(\gamma_s - \gamma)D^2}{18\,\gamma\,\nu} = \frac{g\,\Delta\,D^2}{18\,\nu} \qquad (2.11)$$

For the established condition, the fall velocity of a particle depends on the square of its diameter. If $\Delta = 1.65$ and $T = 20°C$, the previous condition is approximately fulfilled when $D < 0.1$ mm (see also fig 2.2)

b) When the Reynolds number is under 800

In 1933, Schiller and Newmann obtained an acceptable expression for the coefficient C_D when the Reynolds number is under 800

If $\mathrm{IR}_D < 800$

$$C_D = \frac{24}{\mathrm{IR}_D}(1 + 0.15\ \mathrm{IR}_D^{0.687}) \tag{2.12}$$

c) When the Reynolds number is between 1 000 and 100 000
If it falls in this range, C_D, for a spherical particle, can be considered a constant equal to 0.4. The substitution of this value in eq 2.9 results in

$$\omega = \left(\frac{10\, g\, \Delta\, D}{3}\right)^{0.5} \tag{2.13}$$

Within this range, fall velocity depends on the diameter square root. If $\Delta = 1.65$, the previous equation holds when $D > 2.5$ mm (see also fig 2.2).

2.4.2 Fall velocity of a natural particle

In 1933, and to obtain the fall velocity of natural particles with sizes between silts and gravels, Rubey proposed the equation

$$\omega = F_1 (g\, \Delta\, D)^{0.5} \tag{2.14}$$

where

$$F_1 = \left(\frac{2}{3} + \frac{36\, \nu^2}{g\, \Delta\, D^3}\right)^{0.5} - \left(\frac{36\, \nu^2}{g\, \Delta\, D^3}\right)^{0.5} \tag{2.15}$$

Figure 2.2 shows the curve which corresponds to eq 2.14 for a temperature of 20°C.

The Inter Agency Committee in Water Resources suggested a second procedure which is shown in fig 2.3; according to it three families of curves were obtained for three shape factors: 0.5, 0.7 and 0.9. A shape factor of approximately 0.7 corresponds to quartz natural particles.

2.4.3 Fall velocity alterations

As already said in 2.4, a particle fall velocity is affected by the proximity of a wall, the presence of other particles in suspension and water flow.

2.4.3.1 Effect of boundary proximity

In theory, when a particle falls in a fluid, part of the fluid moves in the opposite direction. When a particle with diameter D falls in a pipe with diameter d, and the ratio D/d is small, the effect of the relative flow is appreciable and the actual fall velocity ω_a of the particle is smaller than its theoretical fall velocity ω, obtained as mentioned before. This can happen during laboratory measurements of particles fall velocity using small diameter translucid tubes.

ω_a can be obtained with the following expressions

a. If $D/d \leqslant 0.3$ and if the particle Reynolds member is less than 1.0, the use of the McNown, Lee, Mcpherson and Engez expression is recommended

$$\omega_a = K_1^{-1}\,\omega \qquad \text{or} \qquad \omega/\omega_a = K_1 \tag{2.16}$$

being

$$K_1 = 1 + \sum_i \left(\frac{9D}{4d}\right)^i \tag{2.17}$$

where i = 1, 2, 3, 4..., even if only with two terms enough approximation is obtained.

b. If $D/d > 0.4$, it is better to apply the expression proposed by the same authors

$$\omega_a = K_2^{-1}\,\omega \qquad \text{or} \qquad \omega/\omega_a = K_2 \tag{2.18}$$

being

$$K_2 = 0.53\,\pi \left(1 - \frac{D}{d}\right)^{-2.5} \tag{2.19}$$

2.4.3.2 Effect of concentration

The presence of other particles either in group or suspension, reduces the fall velocity of an isolated one. This is because, when falling, it drags a certain amount of fluid in the same direction. However, and following the mass conservation principle, there are places where the fluid goes upwards.

Another slowing down effect upon the falling particles is provoked by the pounding or displacement of the fine particles in suspension when their mass is bigger than that of the liquid they displace.

To evaluate the fall velocity of a particle submerged in water which contains other particles uniformly distributed and with a volume concentration C_s, McNown-Lee-Mcpherson and Engez' formula can be applied

$$\omega_a = K_3^{-1}\,\omega \qquad \text{or} \qquad \omega/\omega_a = K_3 \tag{2.20}$$

and

$$K_3 = 1 + 1.56\,C_s^{1/3} \tag{2.21}$$

where

- C_s concentration by volume, in fraction
- ω fall velocity in clear water
- ω_a fall velocity affected by concentration C_s

2.4.3.3 Effect of flowing water

The turbulence existing in a moving fluid also reduces particle fall velocity. Levine studied this effect and proposed the following expression

$$\omega_a = \omega - \omega_r \tag{2.22}$$

then

$$\omega_r = 0.132\,\frac{U}{d^{0.5}} \tag{2.23}$$

where ω_a is the actual fall velocity, m/s; ω, the theoretical fall velocity in clear water, in m/s; ω_r, the delayed effect in fall velocity, in m/s; U, the mean flow velocity in m/s, and d, the depth of the flow in m. This expression must be used with care.

2.5 Granulometric distribution of sediments

Sieving with mesh and hydrometer analysis are the most commonly used methods for the mechanical analysis of sediments. While the first determines the corresponding fractions for coarse material such as gravels and sands, the second, obtains those fractions for finest material like silts and clays.

The material characteristics of a given river stretch are determined by the average of various samples taken in different points of the longitudinal and transversal profiles. Therefore, to deduce from the information supplied by several samples the representative size frequency distribution of a site, it is necessary to apply descriptive statistics.

The mechanical analysis of a natural sample permits its division in fractions according to their size. With the statistical treatment of basic data like the openings of the mesh and the weight of the material reatained in each of them, a table of distribution of frequencies can be obtained (see table 2.2). However, graphic representations are better than numerical ones for the objective observation of the distribution of the different sizes of particles.

There are several forms of plotting frequency distributions to provide an objective representation of mechanical analysis results like histograms, relative frequency polygons and accumulated relative frequency polygons. In turn, these may be of the major or minor type, depending on whether the accumulated relative frequency is over or under a given diameter. All these diagrams are usually plotted with frequencies or percentages as ordinates and particle size as abscissas. To do this, table 2.2 should be used.

The most common graphic representation is the curve of distribution of frequencies of the *smallest* accumulated dimension. It is usually drawn in a semilogarithmic scale, known as granulometric curve or cumulative size-frequency curve. In this diagram, the ordinates refer to the percentage of the particle weight, which is smaller than the size represented by the mesh, and the abscissas refer to the logarithmic of the size of the mesh openings.

Then, to obtain the points of the granulometric curve, the values of the second column of table 2.2 have to be drawn against those of column 5 in the same table. Finally, the joining of these points with segments of a straight line or with a series of gentle curves produces a continuous one that passes over each point, like the shown in the figure included in table 2.2.

Besides, it has to be noted that the cumulative size-frequency curves can be plotted in different coordinate systems or kinds of paper, that is to say, using different scale laws in the coordinate axis:

- Semilogarithmic paper. Axis of the abscissas in logarithmic scale and axis of the ordinate in arithmetic scale, as shown in table 2.2. This is the most frequently used paper.

- Probability paper or arithmetic-probability paper as shown in fig 2.4. One axis in arithmetic scale and the other axis in normal probability or Gaussian scale (cumulative relative frequency in percentage).

- Log-probability paper. Abscissa axis in logarithmic scale and ordinate axis in normal probability scale, as shown in fig 2.5 (cumulative relative frequency in percentage).

- Circular function paper. The ordinate axis follows a circular law (see fig 2.6).

These different kinds of paper are very useful to examine several samples from a site because they provide an easy and quick determination of the average composition closer to reality, or better still, the straight line, which is more approximate to the data gathered in the field (see section 2.5.2).

2.5.1 Representative diameters and parameters

2.5.1.1 Diameters

Once the granulometric curve is drawn, it is easy to determine any of the D_i diameters of the sample, where the subindex n represents the percentage in weight of the sample with particles smaller than D_i. For instance, if D_{75} = 0.524 mm, it means that 75 per cent of the weight of the sediment has particles with sizes smaller than 0.524 mm.

The granulometric curve or the cumulative size-frequency curve of the sample can be divided in different size classes; to do this, two new parameters have to be known: p_i and D_i

p_i fraction of the weight of each size class divided by the total weight of the sample. It can be expressed in fraction or as a percentage

D_i mean diameter of each size class. It is obtained graphically or using the expression

$$D_i = (D_{i_{max}} \cdot D_{i_{min}})^{1/2} \qquad (2.24)$$

where $D_{i_{max}}$ and $D_{i_{min}}$ are the extreme values for each class.

The granulometric curve of natural sediments transported by rivers usually presents a log-normal distribution.

Keeping all this in mind and with the purpose of determining the distribution of different sizes of sediments it is generally necessary to know the following diameters and parameters

a. Arithmetic mean diameter D_m
It is defined as

$$D_m = 0.01 \; \Sigma \; D_i \; p_i \qquad (2.25a)$$

p_i is expressed as a percentage. If it is expressed as a fraction, the eq 2.25a is written as

$$D_m = \Sigma \; D_i \; p_i \qquad (2.25b)$$

b. Geometric mean diameter D_g
It is defined as

$$\log D_g = 0.01 \; \Sigma \; p_i \log D_i \qquad (2.26a)$$

p_i is expressed as a percentage. When p_i is expressed as a fraction eq 2.26a takes the form

$$\log D_i = \Sigma\ p_i\ \log D_i \qquad (2.26b)$$

Otto (1939) showed that the geometric mean diameter can also be obtained by the intersection of the 50 per cent line with the line passing through the granulometric curve at the points 15.87 and 84.13 per cent, that is

$$D_g = (D_{84.13}\, D_{15.87})^{1/2} \qquad (2.27a)$$

Commonly, it is expressed as

$$D_g = (D_{84}\ D_{16})^{1/2} \qquad (2.27b)$$

c. D_{50}. This diameter represents the median of the distribution. Only when the distribution is symmetric, the mean and median are equal, but they are generally different

$$D_{50} \neq D_m$$

d. D_{16} and D_{84}. When the granulometric curve has a log-normal or normal distribution, $D_{15.87}$ and $D_{83.13}$ are used in different formulae to determine the characteristics of those distributions. For practical purposes they can be rounded off to D_{16} and D_{84}, respectively.

e. D_{10} and D_{60} are applied when trying to find out the uniformity coefficient C_u, which is expressed as

$$C_u = \frac{D_{60}}{D_{10}} \qquad (2.28)$$

Sediment is considered uniform when $C_u < 3$. If $C_u = 1$, the material is uniform, that is, the cumulative size-frequency curve is a vertical line. When $C_u > 3$, the material is not uniform and it is said that it presents an extended size-frequency distribution or that the sample is well graded.

When studying the problems solved by fluvial hydraulics, authors found out or proposed different diameters as representative of sediment samples. Some are these

Diameter	Author	Phenomena
D_{35}	Einstein	Flow resistance
D_{40}	Schoklitsch	Bed load transport
D_{50}	Straub, Shields	Bed load transport
D_{65}	Keulegan, Einstein	Flow resistance
	Strickler	Friction factor
D_{75}	Lane	Critical shear stress
D_{80}	Stelczer	Critical condition
D_{84}	Maza-García	Flow resistance, critical condition
D_{90}	Meyer-Peter & Müller	Friction factor
	Many authors	Scour
D_{max}	Levi	Critical mean velocity
D_m	Many authors	Sediment transportation, scour

2.5.1.2 Parameters

1) Standard deviation, σ

It is a dispersion measure indicative of the data dispersion in relation to the mean line. In general, and for our purposes, the standard deviation is defined as

$$\sigma = \left(\sum_{i=1}^{n} (D_i - D_m)^2 \, f_i \right)^{1/2} \tag{2.29}$$

where f_i is the probability or relative frequency of occurrence of size D_i; n is the total number of classes or *differents* D_i considered, and D_m is the sample mean diameter, considering $f_i' \simeq p_i$ (see eqs 2.25 and 2.26).

In eq 2.28 the following is fulfilled

$$\sum_{i=1}^{n} f_i = 1$$

2) Geometric standard deviation, σ_g

Its meaning is similar to σ, but in the expression 2.29 the logarithms of the diameters are applied. It is used when the distribution is log-normal or logarithmic and it is expressed as

$$\log \sigma_g = (\sum_{i=1}^{n} (\log D_i - \log D_m)^2 f_i)^{1/2} \qquad (2.30)$$

The geometric standard deviation is commonly used by fluvial hydraulics engineers.

The geometric standard deviation of bed materials increases as the mean diameter increases. We have found that in well graded samples in several streams, D_{16} varies approximately between 0.00015 m and 0.0004 m; in other words, the granulometric curves of many bed sediment samples have a fixed point around D_{16} and the greater is D_m, the greater is σ_g.

3) Standard random variable Z_i
Z_i is a random variable with normal distribution mean = 0 and standard deviation $\sigma = 1$. This variable can assume any value in the interval $-\infty \leq Z_i \leq \infty$; according to the satisfaction of a given probability. The value of Z_i for a given fraction or percentage is obtained directly from table 2.3.

It can be verified in table 2.3 that the probabilities 15.87 and 84.13 per cent are satisfied for $Z_{15.87} = -1$ and $Z_{84.13} = 1$, respectively, and that these values of the random variable Z_i correspond to the inflection points in the standard normal distribution curve.

2.5.2 *Theoretical distribution*

Scientists devoted to the study of natural sediments concluded that the distribution of the sizes of particles which constitute sediments do not follow a unique law. However, it has been proved that depending on the conditions of sediments in river beds, many cases present a clearly defined tendency towards a certain type of distribution; this means that there are sediments which adjust better to a distribution than to another.

The agreement between a real and a theoretical distribution is rarely perfect. Disagreements generally appear at the ends or tails of the distributions because both, fine material fractions and very coarse ones tend to deviate from the theoretical model, and most of the time these tails only represent a small percentage of the material. In such cases, the validity of the theoretical model can be accepted, or else, the interval in which the model is satisfied should be expressed.

As will be shown, the different types of paper already mentioned are very useful when determining whether a size frequency adjusts to a theoretical distribution.

In fluvial hydraulics the two most important distributions are the log-normal and the circular, but there are cases when the normal and the log-log normal are also relevant.

2.5.2.1 Log-normal distribution

In most fluvial phenomena sands and gravels are present. It has been shown that in natural beds, particles of these materials have a size-distribution which frequently follows a probability law of the log-normal type.

As easy method to determine if the granulometry of certain sediments follows a log-normal distribution consists in plotting the granulometric curve in a log-probability paper (see fig 2.5). If the points are aligned exactly on a straight line, then the distribution of diameter logarithms, log D_i, follows a normal or gaussian probability law. When this happens it is said that the granulometric distribution is of the log-normal type and it can be described as

$$D_i = D_{50} (\sigma_g)^{z_i} \tag{2.31}$$

when σ_g is the standard geometric deviation, D_i is the diameter of the particle, and n stands for the percentage in weight of the sample which contains particles with a diameter smaller than D_i. Z_i is the standard random variable; its value is obtained directly from table 2.3, according to the required probability percentage or the area under the normal curve. It is important to consider (see figures in table 2.3) that if $i < 50$, the standard random variable is negative, and when $i > 50$, Z_i is positive. Thus, for instance, to calculate or generate D_{35}, the value of Z_i corresponding to 35 per cent is found in that table. So $Z_{35} = -0.38532$. And to obtain D_{80} the Z_i corresponding to 80 percent is derived from the same table and it is $Z_{80} = 0.84162$.

Diameters D_{50}, $D_{84.13}$ or $D_{15.87}$ needed to calculate D_i are obtained from the straight line adjusted in the probability log-paper. But, in practice, only diameters D_{16} and D_{84} are necessary to generate any other diameter; in this case the following is also fulfilled

$$D_{50} = (D_{16}\, D_{84})^{1/2} \tag{2.32}$$

and

$$\sigma_g = \left(\frac{D_{84}}{D_{16}} \right)^{1/2} = \frac{D_{84}}{D_{50}} = \frac{D_{50}}{D_{16}} \tag{2.33}$$

The mean diameter of the log-normal distribution is calculated with eqs 2.24 and also with the following formula

$$D_m = D_{50} \exp \left[\frac{1}{2} (\mathrm{Ln}\, \sigma_g)^2 \right] \tag{2.34}$$

and, because the log-normal distribution is asymmetrical, $D_{50} \neq D_m$.

When points are not exactly aligned in the log-probability paper due to the fact that real distribution deviates from the log-normal one, and there is a tendency of the points to aligne along a straight line, we are in front of a regression problem and the straight line better adjusted to real data must be defined according to a standard criterium or regression method. However, and for practical purposes, the Otto's criterium can be followed.

As already mentioned, Otto's criterium consists in accepting as an adjusted straight line the one which results from joining in the log-probability paper the points corresponding to the values of 16 and 84 per cent. That is to say, once the granulometric curve is plotted in the log-probability paper and the values of D_{16} and D_{84} are determined, all other diameters can be obtained from these two.

2.5.2.2 Circular law

When sediment transport reduces because the river level descends, the bottom tends to get armored. This is much more definite and clear when the diameter of the sediment is bigger or, what is the same, when the standard geometric deviation σ_g of the sample is greater.

When a river bed is armored, the portion of the thick (i > 50 percent) sizes of the particles which form the upper layer follow a circular law.

If the distribution of diameters follows the shape of a circumference, it is called a circular law and can be described by

$$D_i = D_{max}\left[1 - \sqrt{1 - \left(\frac{i}{100}\right)^2}\,\right] \quad (2.35)$$

where D_{max} is the maximum diameter of the sample.

That is, if the scales adopted make equal the representative distances of the maximum diameter and that of the 100 per cent, the resulting diagram in arithmetic paper is one fourth of a circumference with a radius equal to the maximum diameter in the respective scale. On the contrary, if it is plotted on a circular function paper (see fig 2.6), where the ordinates follow the law given by eq 2.35, the result is a straight line.

2.5.2.3 Normal distribution

When in a size frequency distribution curve plotted in probability paper, points are exactly on a straight line, it means that the diameter of the particles which constitute sediments follow a normal or gaussian probability law. When this occurs, size-frequency distribution is considered normal and can be described as

$$D_i = D_{50} + Z_i\,\sigma \quad (2.36)$$

where D_i was previously described, Z_i is the standard random variable, and σ is the standard deviation. σ can be determined if D_{84} or D_{16} are known, since they are at the same distance from D_{50}, taking into account that normal distribution is symmetric. It is defined as

$$\sigma = D_{84} - D_{50} = D_{50} - D_{16} = \frac{1}{2}(D_{84} - D_{16}) \quad (2.37)$$

Diameters D_{50}, D_{84} and D_{16} are obtained from the adjusted straight line in the probability paper but, in practice, all other diameters can be determined from D_{16} and D_{84}, for it is also true that

$$D_{50} = \frac{1}{2}(D_{16} + D_{84}) \quad (2.38)$$

Once all the variables of the eq 2.34 are known, different D_i can be obtained.

For a normal distribution the percentage of variation falls within the following ranges

Percentage variation	Range
68.2	$\pm \sigma$
95.4	$\pm 2\sigma$
99.73	$\pm 3\sigma$

which means, per example, that if a dispersion of 2σ from both sides of the mean is allowed, 95.4 per cent of the data points are within that range.

2.5.2.4 Logarithmic distribution

When diameter logarithms, Log D_i, are distributed according to a linear law, the distribution is said to be logarithmic, and can be written as

$$D_i = D_{50} \exp \left[\frac{i - 50}{34} \operatorname{Ln} \sigma_g \right] \tag{2.39}$$

or

$$D_i = D_{50}\, 10^{\left[\frac{i-50}{34} \log \sigma_g\right]} \tag{2.40}$$

where

σ_g standard geometrical deviation, defined as

$$\sigma_g = \left(\frac{D_{84}}{D_{16}} \right)^{0.5} = \frac{D_{84}}{D_{50}} \tag{2.41}$$

2.5.2.5 Log-log distribution

Diameter logarithms can be distributed according to a logarithmic law, and in this case it is said that the distribution is log-log and is described by

$$D_i = D_{50} \exp \left[\frac{\operatorname{Ln} \left(\frac{i}{50}\right) \operatorname{Ln} \sigma_g}{\operatorname{Ln} \left(\frac{84}{50}\right)} \right] \tag{2.42}$$

or

$$D_i = D_{50}\, 10^{\left(4.4383 \log \left(\frac{i}{50}\right) \log \sigma_g\right)} \tag{2.43}$$

where

$$\sigma_g = \frac{D_{84}}{D_{50}} = \left(\frac{D_{50}}{D_{16}}\right)^{0.45531} = \left(\frac{D_{84}}{D_{16}}\right)^{0.31286} \tag{2.44}$$

from which

$$D_{50} = D_{16}^{0.31286}\ D_{84}^{0.68714} \tag{2.45}$$

2.6 Volumetric weight or bulk specific weight, $\gamma_{\forall}$

The volumetric weight $\gamma_{\forall}$ of a group of particles is the material weight divided by its total volume including empty spaces. Its units and dimensions are those indicated for specific weight (section 2.1).

When considering the volumetric weight of a sediment sample, volumetric dry weight must be defined by determining all empty spaces which hold air. The partially saturated weight and the submerged weight hold water in part or in all non-sediment spaces.

In evaluating the real volume held by depositionary sediments, the most interesting relation used in fluvial hydraulics is (see fig 2.7).

$$\gamma_{\forall} = \frac{\text{solid material weight (dry)}}{\text{total volume}} \tag{2.46}$$

The following relations are also relevant.

a. Porosity, n, is defined as the ratio between total volume and the volume of empty spaces

$$n = \frac{\text{voids volume}}{\text{total volume}} = \frac{\forall_V}{\forall_T} \tag{2.47}$$

thus, the following is fulfilled

$$\gamma_{\forall} = \gamma_s\,(1 - n) \tag{2.48}$$

b. Void ratio, e, is defined as the volume of voids divided by the solid material volume; therefore, the following equations can be written

$$e = \frac{\text{voids volume}}{\text{solid volume}} = \frac{V_V}{V_S} \tag{2.49}$$

it holds that

$$n = \frac{e}{1+e} \tag{2.50a}$$

and

$$e = \frac{n}{1-n} \tag{2.50b}$$

thus,

$$\gamma_V = \frac{\gamma_S}{1+e} \tag{2.51}$$

c. Degree of saturation, S, is equal to the volume of water divided by the volume of voids

$$S = \frac{\text{water volume}}{\text{voids volume}} = \frac{V_W}{V_V} \tag{2.52}$$

d. Water content, W, is equal to water weight divided by the weight of solids

$$W = \frac{\text{weight of water}}{\text{weight of solid}} = \frac{W_W}{W_S} \tag{2.53}$$

2.7 Suspended sediment concentration or suspended particle concentration

The amount of particles in suspension in a liquid mass is expressed by its concentration, C_s, taking into account that in fluvial hydraulics it is considered that sediment concentration does not include neither vegetal material nor dissolved solids.

The concentration C_s of suspended sediment is expressed in one of the following ways.

a. Sediment concentration by volume, is the ratio between the volume of sediments $\forall_s$ in the sample and the total volume $\forall_m$ of the mixture. In other words, it gives the volume of sediments per unit volume of water sediment mixture. So C_s can be obtained as

$$C_{S1} = \frac{\forall_s}{\forall_m} \tag{2.54}$$

When C_s is defined as mentioned, it is expressed in m^3/m^3, as a percentage, in fraction, or in parts per million by volume (ppm by volume).

b. Sediment concentration by weight, which is the dry weight of sediments divided by the total volume of the sample. This value also indicates the dry weight of sediments per unit volume of mixture. This is

$$C_{S2} = \frac{\gamma_s \forall_s}{\forall_m} \tag{2.55}$$

When C_s is defined as already mentioned it is expressed in mgf/l, kgf/m^3, or parts per million by weight (ppm by weight).

When the quantity expressed by eq. 2.53 is divided by γ_s, eq 2.52 is obtained, that is to say

$$\gamma_s C_{S1} = C_{S2} \tag{2.56}$$

c. Sediment concentration by weight as a percentage, which is the ratio between the dry weight of sediments in the sample and the total weight of the sample. It gives the dry weight of sediments per unit weight of mixture. So

$$C_{S3} = \frac{\gamma_s \forall_s}{\gamma_m \forall_m} \tag{2.57}$$

where γ_m is the specific weight of the mixture sediment and water. In this conditions, C_s is expressed in kgf/kgf, either in percentage or as a fraction.

d. The last way of expressing C_s is similar to the previous but dividing the dry weight of sediments by the volume of the sample as if all were pure water. That is

$$C_{S4} = \frac{\gamma_s \forall_s}{\gamma \forall_m} \tag{2.58}$$

In this case C_s is expressed with the same units mentioned in paragraph c.

To weight solid material, the mixture or sample has to be filtered or drained, but not evaporated because soluble solids would deposit and they must not be taken into account.

In fluvial hydraulics concentration is generally expressed in parts per million, by weight, by volume or in fraction form. However, when included in formulae or non-dimensional parameters it is necessary to respect their dimensions by expressing them in congruent units. To make conversions the following relations are used.

1. Parts per million, ppm, in volume (see paragraph a)

$$(x)\text{ ppm} = \frac{(x)\text{ mililiter}}{m^3} = (x)\cdot 10^{-6}\ \frac{m^3}{m^3} \tag{2.59}$$

If it is to be expressed in percentage

$$(x)\text{ ppm} = [(x)\cdot 10^{-4}]\ \% \tag{2.60}$$

2. Parts per million, ppm, by weight (see paragraph b)

$$(x)\text{ ppm} = \frac{(x)\text{ mgf}}{\text{liter}} = (x)\cdot 10^{-3}\ \frac{\text{kgf}}{m^3} \tag{2.61}$$

If eqs 2.59 and 2.61 are used, it holds

$$(x)\text{ ppm by weight} = \frac{\gamma_s}{\gamma}\ (x)\text{ ppm by volume} \tag{2.62}$$

If it is expressed in percentage by weight, once the parts per million by weight are known, the following expression can be used

$$(x)\text{ ppm} = \left(\frac{(x)\cdot 10^{-1}}{\gamma_s}\right)\ \% \tag{2.63}$$

Example. Suppose a sample $\forall_m$ = 1 liter containing 6.5 gf of sediments. So

$$\forall_m = 1\text{ liter}$$

$$W_s = 6.5\text{ gf}$$

The volume of sediments is given by

$$\forall_s = \frac{W_s}{\gamma_s} \tag{2.64}$$

$$\forall_s = \frac{0.0065}{2650} = 2.45283 \times 10^{-6}\ m^3$$

The volume of water in the sample is obtained by

$$\forall_W = \forall_m - \forall_s \tag{2.65}$$

$$\forall_W = 0.997547 \text{ liter}$$

$$\forall_W = 9.97547 \times 10^{-4}\ m^3$$

If it is accepted that γ = 1000 kgf/m³, the total weight of the sample is

$$W_m = \gamma_s \forall_s + \gamma \forall_W \tag{2.66}$$

$$W_m = 6.5 + 997.547$$

$$W_m = 1.004047 \text{ kgf} \tag{2.66a}$$

The concentration of sediments expressed in different units is

1. In parts per million by volume. From eq 2.54

$$C_{S1} = \frac{2.45283 \times 10^{-6}}{0.001} = 0.00245283 \frac{m^3}{m^3}$$

from eq 2.59

$$C_{S1} = 2452.83 \text{ ppm by volume}$$

from eq 2.60

$$C_{S1} = 0.24528\ \%\ \text{in volume}$$

2. In parts per million by weight

From eq 2.55

$$C_{S2} = \frac{0.0065}{0.001} = 6.5\ kgf/m^3$$

From eq 2.61

$$C_{S2} = 6500 \text{ ppm by weight}$$

If eq 2.62 is applied and as $C_{S1} = 2453$ ppm by volume, it also holds that

$$C_{S2} = 2452.83 \times 2.65 = 6500 \text{ ppm by weight}$$

3. In percentage by weight eq 2.57 is used multiplied by 100

$$C_{S3} = \frac{0.0065}{1.0040471} \times 100 = 0.64738\ \%$$

If eq 2.58 is considered, multiplying by 100

$$C_{S4} = \frac{0.0065}{1} \times 100 = 0.65\ \%$$

2.8 Specific weight of a liquid containing suspended material

The specific weight γ_m of a mixture of water and suspended material can be obtained using the following relation (eq. 2.66)

Weight of the mixture = weight of liquid + weight of solid material

$$\gamma_m \forall_m = \gamma \forall + \gamma_s \forall_s \tag{2.67}$$

where $\forall$ is the volume, γ is the specific weight, subindex m refers to the mixture and s to solid material; when there is no subindex, it refers to water.

From the previous relation the following is deduced

$$\gamma_m = \gamma + \frac{\forall_s (\gamma_s - \gamma)}{\forall_m} \tag{2.68}$$

since

$$\forall = \forall_m - \forall_s \tag{2.69}$$

Generally, in eq 2.68 the value of $\forall_s$ is not known, but the value of concentration is known, and it has to be expressed in fractions, according to eq 2.54, 2.57 and 2.58 of section 2.7.

1. If C_s is expressed in volume according to eq 2.54, then the following relation is reached

$$\gamma_{m1} = \gamma + C_{S1}(\gamma_s - \gamma) \tag{2.70}$$

2. If C_s is expressed by weight according to eq 2.57, it reaches

$$\gamma_{m3} = \frac{\gamma\,\gamma_s}{\gamma_s - (\gamma_s - \gamma)\,C_{S3}} \tag{2.71}$$

3. If C_s is expressed by weight according to eq 2.58, it is obtained

$$\gamma_{m4} = \gamma\,[1 + \frac{(\gamma_s - \gamma)\,C_{S4}}{\gamma_s}] \tag{2.72}$$

Example. Considering the data of the problem in section 2.7 and γ = 1000 kgf/m³, concentration of C_s expressed in decimal form would be

$$C_{S1} = 0.00245283 \quad \text{(by volume)}$$

$$C_{S3} = 0.064738 \quad \text{(by weight)}$$

$$C_{S4} = 0.0065 \quad \text{(by weight)}$$

Thus, from eq 2.70

$$\gamma_m = 1004.04717 \text{ kgf/m}^3$$

From eq 2.71

$$\gamma_m = 1004.04717 \text{ kgf/m}^3$$

From eq 2.72 it is obtained

$$\gamma_m = 1004.04717 \text{ kgf/m}^3$$

Considering the weight of the sample given by eq 2.66a and due to the fact that its volume is exactly 1 liter, it can be immediately deduced that the equations used to obtain volumetric weight are correct, in spite of the differences in the definition of the various concentrations.

Because of these differences and with the purpose of avoiding errors, it is important to know exactly the mechanism employed in the laboratory, for the mere knowledge of (x) ppm is not enough.

2.9 Viscosity of a liquid containing suspended material

The presence of fine suspended material changes the viscosity of liquids.

For those cases where the concentration of particles expressed in volume is C_s and under 0.03 m^3/m^3, Einstein proposed the following relation to obtain the dynamic viscosity of the mixture

$$\mu_{mixture} = \mu\,(1 + 2.5\,C_s) \qquad (2.73)$$

where μ is the water dynamic viscosity at the same temperature of the mixture (tables 1.2 and 1.3), and C_s is the concentration of solid material expressed in fraction by volume.

For concentrations over 0.03 m^3/m^3, Ward proposed in 1955 a relation similar to the above method

$$\mu_{mixture} = \mu\,(1 + 4.5\,C_s) \qquad (2.74)$$

The eq 2.73, suggested by Einstein, assumed that particles were spheres of smooth surface and small inertia force. When this is very small, it can be negligible in the Navier-Stokes equations.

2.10 References

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TABLE 2.1 SEDIMENT CLASSIFICATION OF THE AMERICAN GEOPHYSICAL UNION

Group	Class	Size, in mm	
Boulders	Very large	2 000	- 4 000
	Large	1 000	- 2 000
	Medium	500	- 1 000
	Small	150	- 500
Cobbles	Large	130	- 250
	Small	64	- 130
Gravel	Very coarse	32	- 64
	Coarse	16	- 32
	Medium	8	- 16
	Fine	4	- 8
	Very fine	2	- 4
Sand	Very coarse	1	- 2
	Coarse	0.5	- 1
	Medium	0.25	- 0.5
	Fine	0.125	- 0.25
	Very fine	0.062	- 0.125
Silt	Coarse	0.031	- 0.062
	Medium	0.016	- 0.031
	Fine	0.008	- 0.016
	Very fine	0.004	- 0.008
Clay	Coarse	0.002	- 0.004
	Medium	0.001	- 0.002
	Fine	0.0005	- 0.001
	Very fine	0.00024	- 0.0005

TABLE 2.2 GRANULOMETRIC ANALYSIS

Project______________________ Location______________________
Sounding______________ Sample______________ Depth______________m
Test______________ Date______________ Operator______________

Sample dry weight + tare = 533
− tare = 20
Original sample dry weight, Wi = 513 g

Mesh	Opening mm	Retained weight g	% retained	% that passes by weight
3"	76.2			
2"	50.8			
1½"	38.1			
1"	25.4			
¾"	19.1			
½"	12.7			
⅜"	9.52			
Nº 4	4.76			100.0
Nº 10	2.00	52	10.1	89.9
Nº 20	0.84	143	27.9	62.0
Nº 40	0.42	138	26.9	35.1
Nº 60	0.25	65	12.7	22.4
Nº 100	0.149	62	12.1	10.3
Nº 200	0.074	48	9.3	1.0
Recipient		5	1.0	

(Rows Nº 4 to Nº 200: Example)

Final dry weight Wf = __________ g

$$\% \text{ retained} = \frac{\text{Retained weight in a mesh, gr}}{\text{Original sample dry weight, gr}} \times 100$$

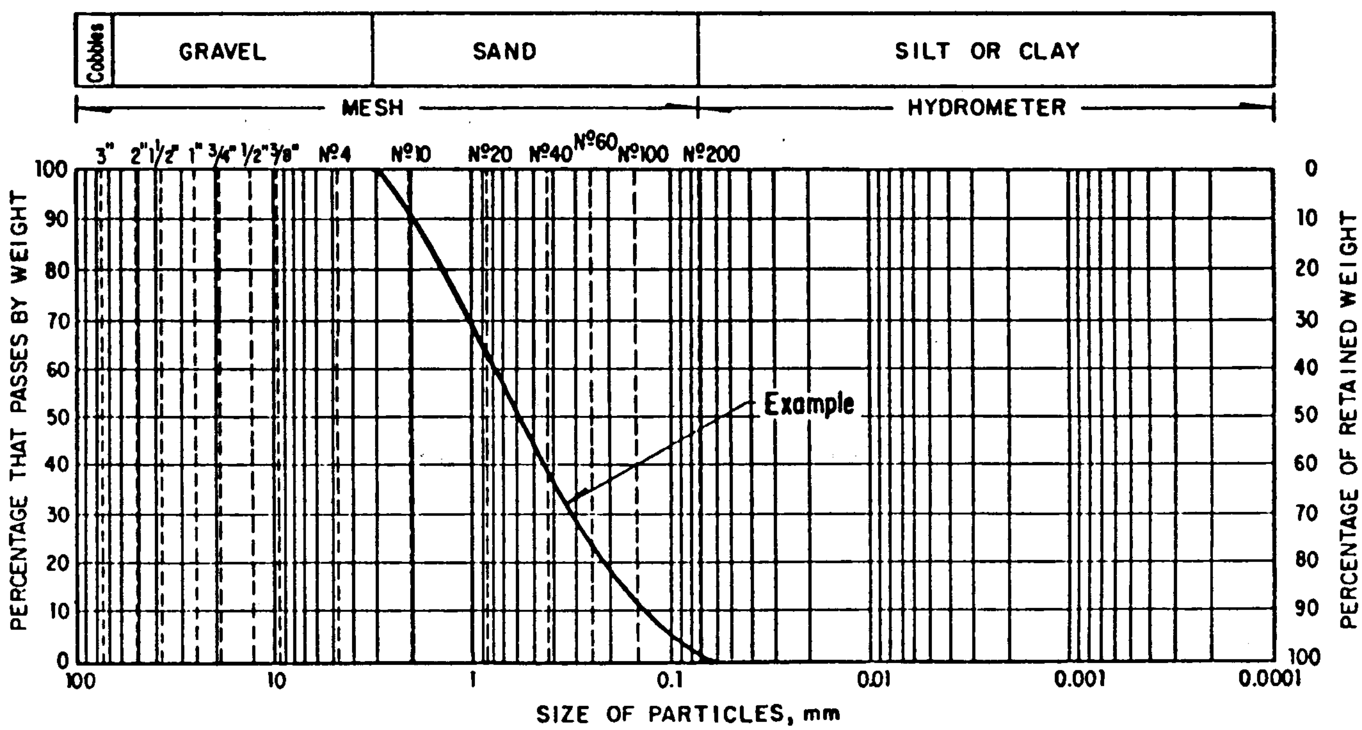
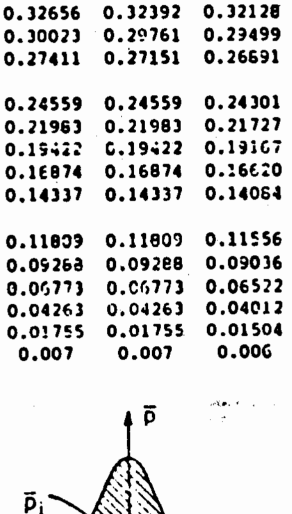
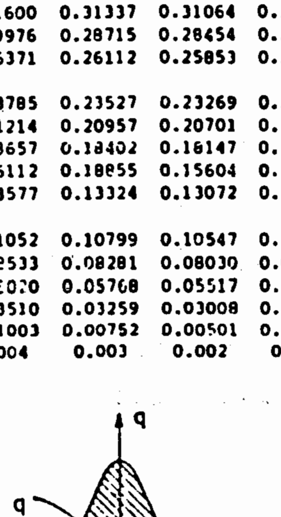


TABLE 2.3 NORMAL PROBABILITY FUNCTION. VALUE OF Z_i

$q, \bar{p}_i$	0.000	0.001	0.002	0.003	0.004	0.005	0.006	0.007	0.008	0.009	1.010	$q, \bar{p}_i$
0.00	∞	3.09023	2.87816	2.74778	2.65207	2.57583	2.51214	2.45726	2.40892	2.36562	2.32635	0.99
0.01	2.32635	2.29037	2.25713	2.22621	2.19729	2.17009	2.14441	2.12007	2.09693	2.07485	2.05375	0.98
0.02	2.05375	2.03352	2.01409	1.99539	1.97737	1.95996	1.94313	1.92684	1.91104	1.89570	1.88079	0.97
0.03	1.88079	1.86630	1.85218	1.83842	1.82501	1.81191	1.79912	1.78661	1.77438	1.76241	1.75069	0.96
0.04	1.75069	1.73920	1.72793	1.71689	1.70604	1.69540	1.68494	1.67466	1.66456	1.65463	1.64485	0.95
0.05	1.64485	1.63523	1.62576	1.61644	1.60725	1.59819	1.58927	1.58047	1.57179	1.56322	1.55477	0.94
0.06	1.55477	1.54643	1.53820	1.53007	1.52204	1.51410	1.50626	1.49851	1.49085	1.48326	1.47579	0.93
0.07	1.47579	1.46838	1.46106	1.45381	1.44603	1.43953	1.43250	1.42554	1.41865	1.41183	1.40507	0.92
0.08	1.40507	1.39838	1.39174	1.38517	1.37866	1.37220	1.35681	1.35946	1.35317	1.34694	1.34076	0.91
0.09	1.34076	1.33462	1.32854	1.32251	1.31652	1.31058	1.30469	1.29884	1.29303	1.28727	1.28155	0.90
0.10	1.28155	1.27597	1.27024	1.26464	1.25908	1.25357	1.24808	1.24264	1.23723	1.23186	1.22653	0.89
0.11	1.22653	1.22123	1.21596	1.21072	1.20553	1.20036	1.19522	1.19012	1.18504	1.18000	1.17499	0.88
0.12	1.17499	1.17000	1.16505	1.16012	1.15522	1.15036	1.14551	1.14069	1.13590	1.13113	1.12639	0.87
0.13	1.12639	1.12168	1.11699	1.11232	1.10768	1.10306	1.09847	1.09390	1.08935	1.08482	1.08032	0.86
0.14	1.08032	1.07584	1.07138	1.06694	1.06252	1.05812	1.05374	1.04939	1.04505	1.04073	1.03643	0.85
0.15	1.03643	1.03215	1.02789	1.02365	1.01943	1.01522	1.01103	1.00686	1.00271	0.99358	0.99446	0.84
0.16	0.99446	0.99036	0.98627	0.98220	0.97815	0.97411	0.97009	0.96609	0.96210	0.95812	0.95416	0.83
0.17	0.95416	0.95022	0.94629	0.94238	0.93848	0.93458	0.93072	0.92686	0.92301	0.91918	0.91537	0.82
0.18	0.91537	0.91156	0.90777	0.90399	0.90023	0.89647	0.89273	0.88901	0.88529	0.86152	0.87790	0.81
0.19	0.87790	0.87422	0.87055	0.86689	0.86325	0.85962	0.85600	0.85239	0.64879	0.84520	0.84162	0.80
0.20	0.84162	0.83805	0.83450	0.83095	0.82742	0.82390	0.82038	0.81687	0.81338	0.80990	0.80642	0.79
0.21	0.60642	0.80296	0.79950	0.79606	0.78919	0.78577	0.78237	0.77897	0.77897	0.77557	0.77219	0.78
0.22	0.77219	0.76882	0.76546	0.76210	0.75875	0.75542	0.75208	0.74876	0.74545	0.74214	0.73885	0.77
0.23	0.73885	0.73556	0.73228	0.72900	0.72474	0.72248	0.71923	0.71599	0.71275	0.70952	0.70630	0.76
0.24	0.70630	0.70309	0.69988	0.69668	0.69349	0.69031	0.68713	0.68396	0.68080	0.67764	0.67449	0.75
0.25	0.67449	0.67135	0.66821	0.66508	0.66196	0.65884	0.65573	0.65262	0.64952	0.64643	0.64335	0.74
0.26	0.64335	0.64027	0.63719	0.63412	0.63106	0.62801	0.62496	0.62191	0.61887	0.61584	0.61281	0.73
0.27	0.61261	0.60979	0.60678	0.60376	0.60076	0.59776	0.59477	0.59178	0.58879	0.58581	0.58284	0.72
0.28	0.58284	0.57987	0.57691	0.57395	0.57100	0.56805	0.56511	0.56217	0.55924	0.55631	0.55338	0.71
0.29	0.55338	0.55047	0.54755	0.54464	0.54174	0.53884	0.53594	0.53305	0.53016	0.52728	0.52440	0.70
0.30	0.52440	0.52153	0.51866	0.51579	0.51293	0.51007	0.50772	0.50437	0.50153	0.49869	0.49585	0.69
0.31	0.49585	0.49302	0.49019	0.48736	0.48453	0.48173	0.47891	0.47610	0.47330	0.47050	0.46770	0.68
0.32	0.46770	0.46490	0.46211	0.45933	0.45654	0.45376	0.45099	0.44821	0.44544	0.44268	0.43991	0.67
0.33	0.43991	0.43715	0.43440	0.43164	0.42689	0.42615	0.42340	0.42066	0.41793	0.41519	0.41246	0.66
0.34	0.41246	0.40974	0.40701	0.40429	0.40157	0.39886	0.39614	0.39343	0.39073	0.38802	0.38532	0.65
0.35	0.38532	0.38262	0.37993	0.37723	0.37454	0.37186	0.36917	0.36649	0.36381	0.36113	0.35846	0.64
0.36	0.35846	0.35579	0.35312	0.35045	0.34769	0.34513	0.34247	0.33981	0.33716	0.33450	0.33185	0.63
0.37	0.33185	0.32921	0.32656	0.32392	0.32128	0.31864	0.31600	0.31337	0.31064	0.30811	0.30548	0.62
0.38	0.30548	0.30286	0.30023	0.29761	0.29499	0.29237	0.29976	0.28715	0.28454	0.28193	0.37932	0.61
0.39	0.27932	0.27671	0.27411	0.27151	0.26691	0.26631	0.26371	0.26112	0.25853	0.25594	0.25335	0.60
0.40	0.25335	0.25076	0.24559	0.24559	0.24301	0.24043	0.23785	0.23527	0.23269	0.23012	0.22754	0.59
0.41	0.22754	0.22497	0.21963	0.21983	0.21727	0.21470	0.21214	0.20957	0.20701	0.20445	0.20189	0.58
0.42	0.20180	0.19934	0.19422	0.19422	0.19167	0.16912	0.18657	0.18402	0.16147	0.17892	0.17637	0.57
0.43	0.17637	0.17383	0.16874	0.16874	0.16620	0.16376	0.16112	0.18855	0.15604	0.15351	0.15097	0.56
0.44	0.15097	0.14843	0.14337	0.14337	0.14084	0.13830	0.13577	0.13324	0.13072	0.12619	0.12566	0.55
0.45	0.12566	0.12314	0.11809	0.11809	0.11556	0.11304	0.11052	0.10799	0.10547	0.10295	0.10043	0.54
0.46	0.10043	0.09731	0.09268	0.09288	0.09036	0.08784	0.08533	0.08281	0.08030	0.07778	0.07527	0.53
0.47	0.07527	0.07276	0.06773	0.06773	0.06522	0.06271	0.06020	0.05768	0.05517	0.05266	0.05015	0.52
0.48	0.05015	0.04764	0.04263	0.04263	0.04012	0.03761	0.03510	0.03259	0.03008	0.02756	0.02507	0.51
0.49	0.02507	0.02256	0.01755	0.01755	0.01504	0.01253	0.01003	0.00752	0.00501	0.00251	0.00000	0.50
	0.010	0.009	0.007	0.007	0.006	0.005	0.004	0.003	0.002	0.001	0.000	





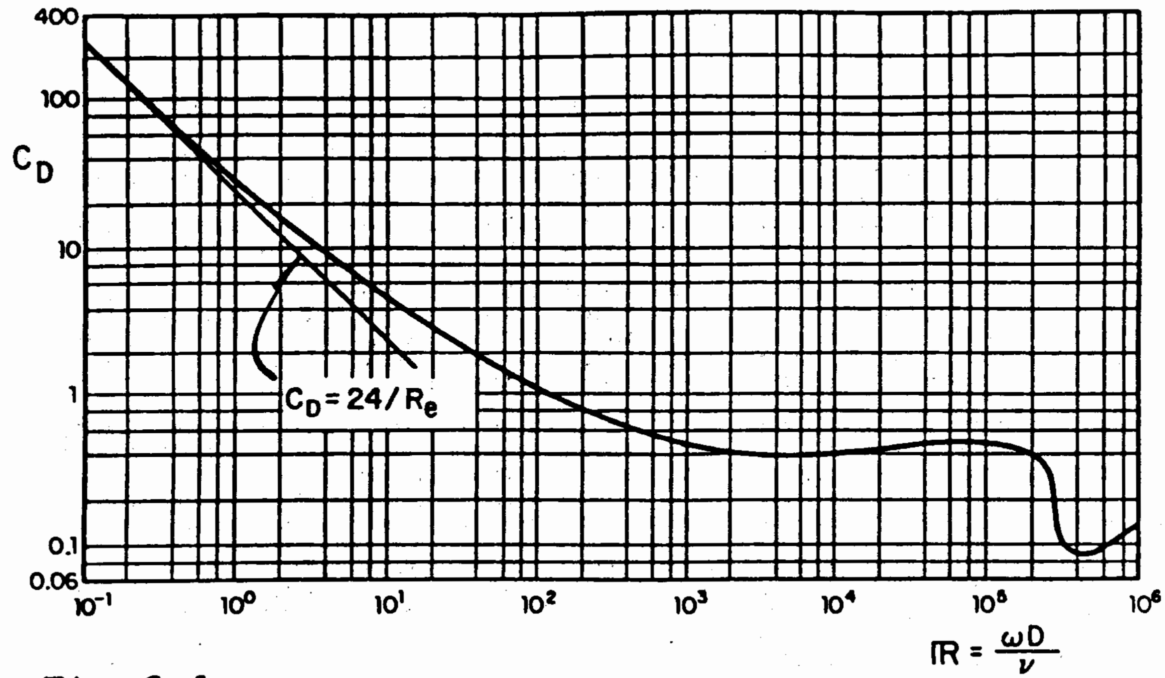


Fig 2.1
Drag coefficient, C_D, versus Reynolds number, for spheres

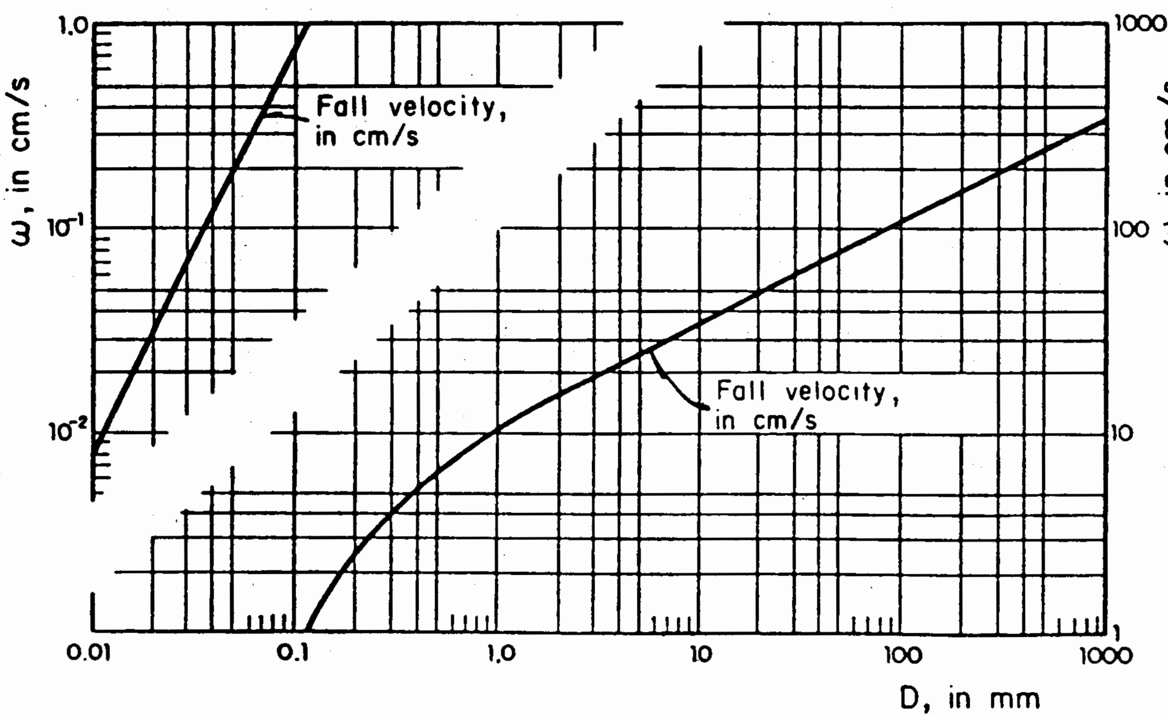


Fig 2.2
Fall velocity for natural particles, for 20°C, after Rubey

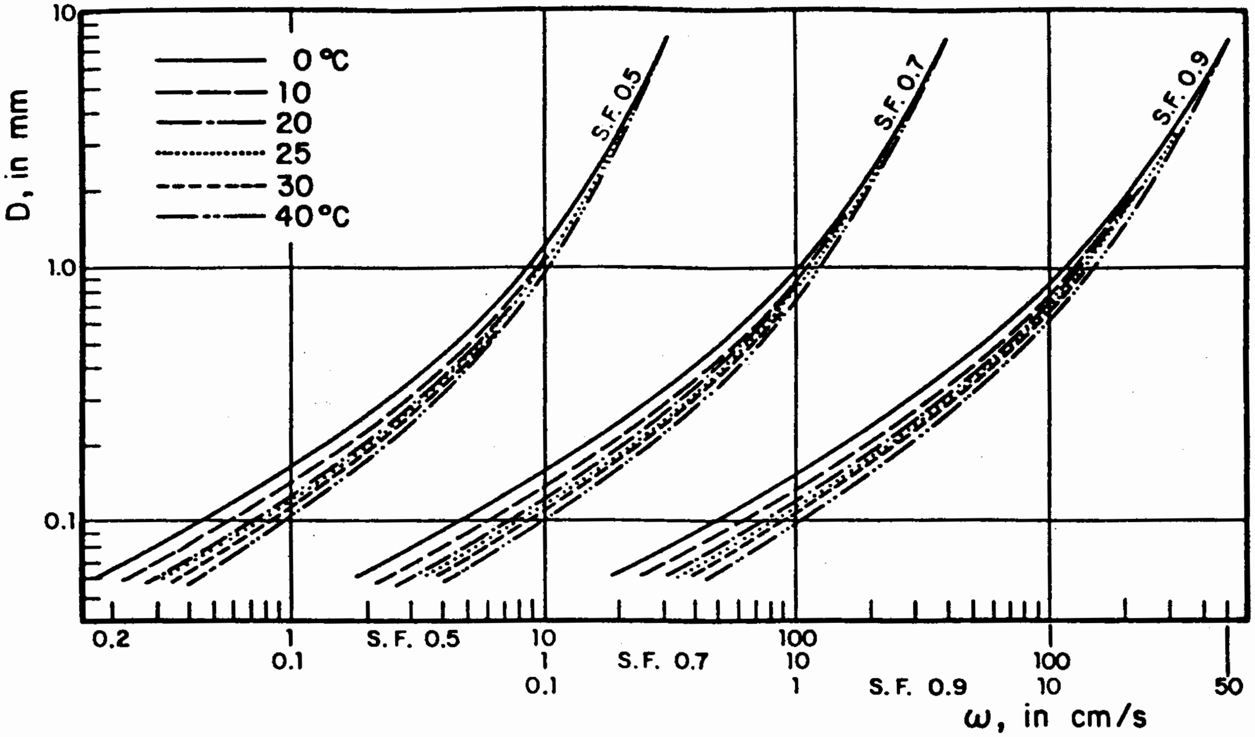


Fig 2.3
Fall velocity as a function of D, shape factor and temperature

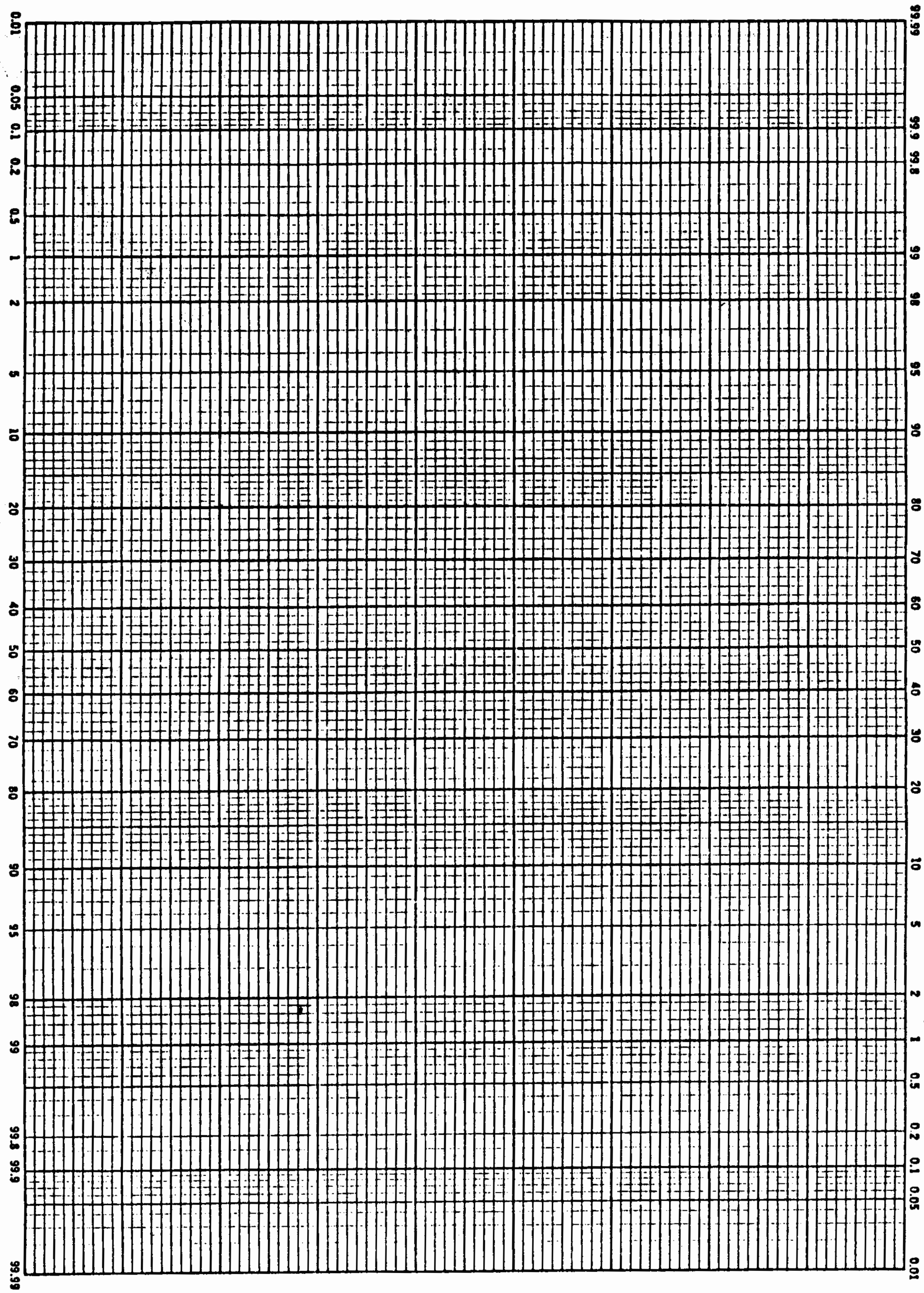


Fig 2.4

Arithmetic probability paper

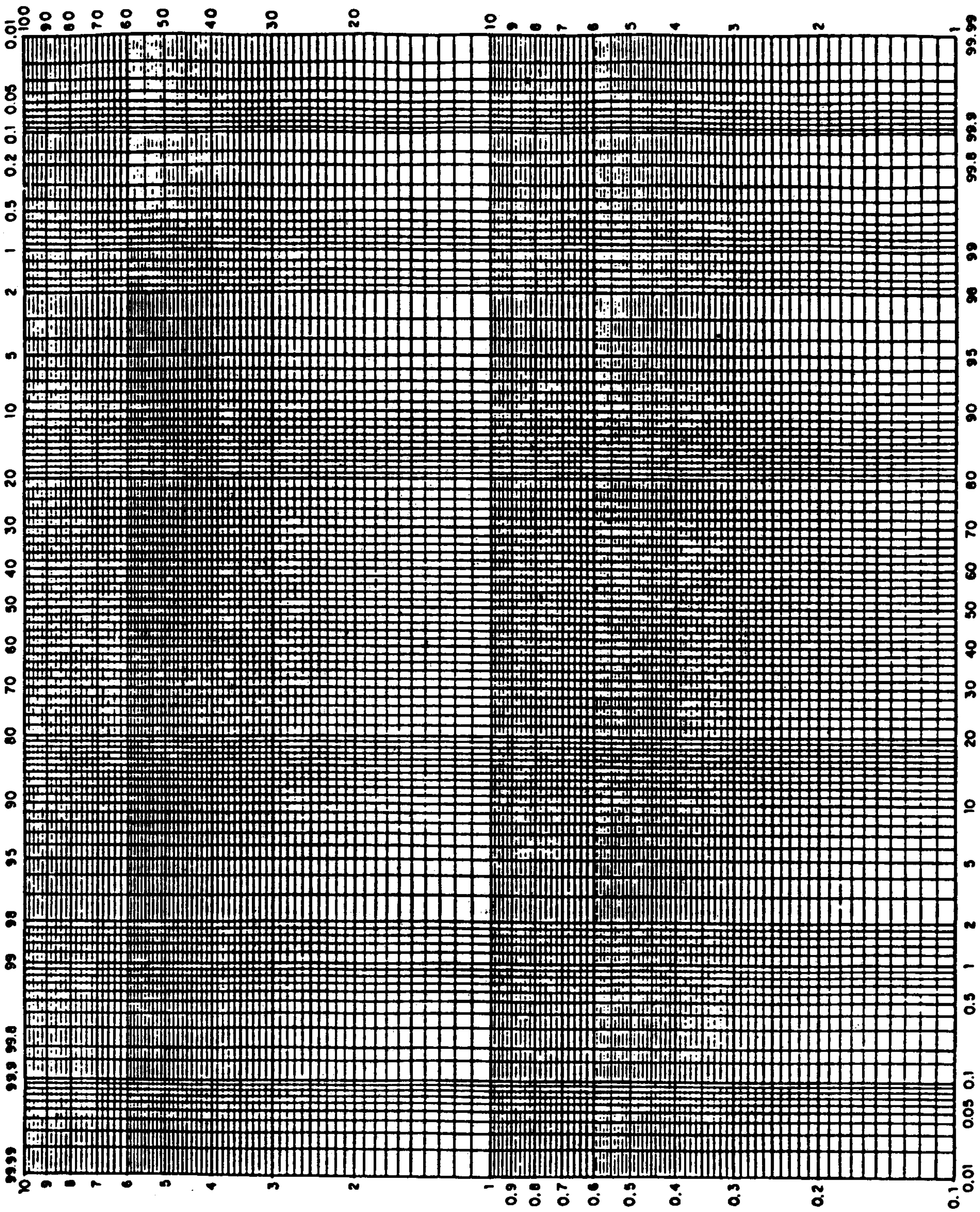


Fig 2.5
Log-normal probability distribution paper

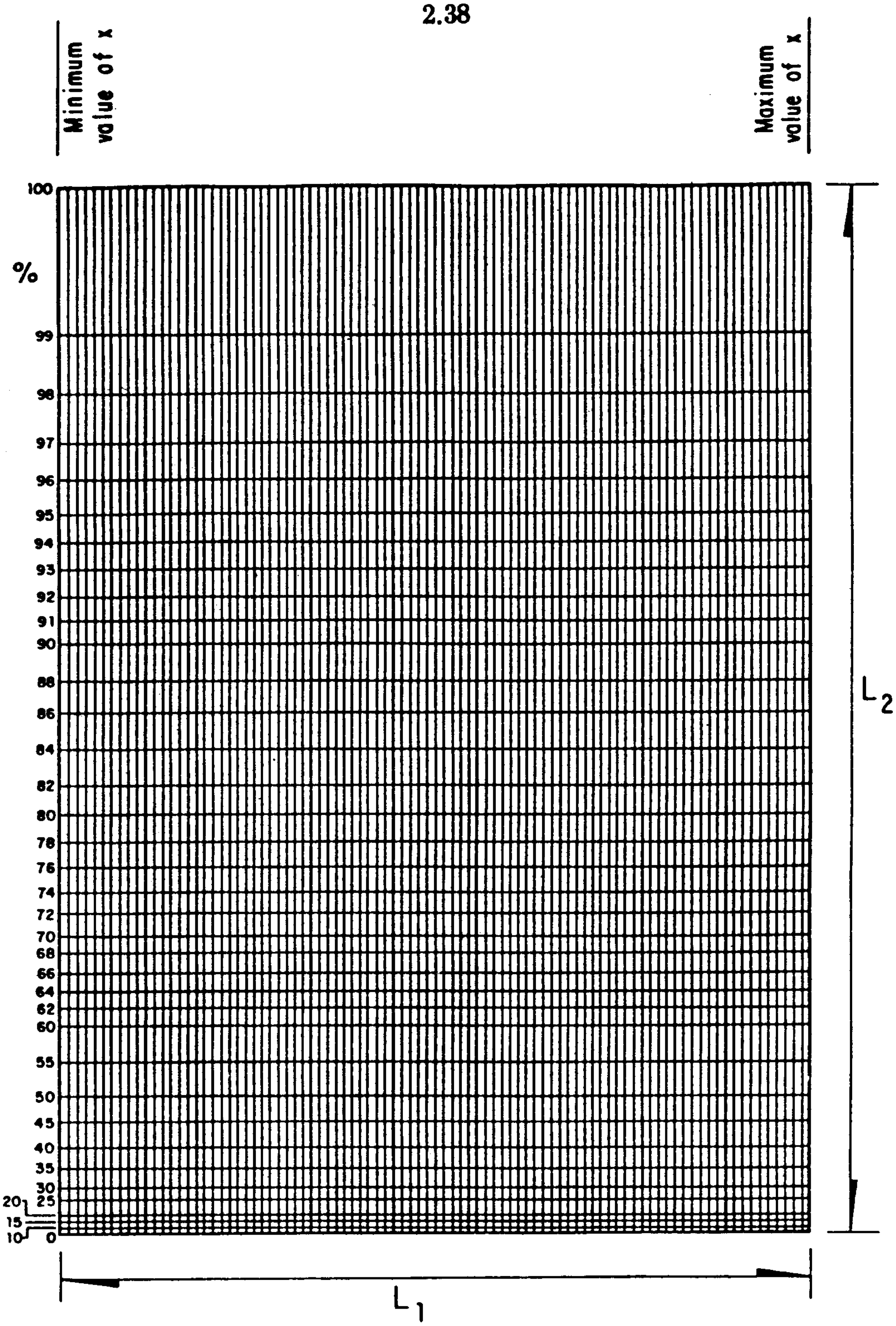


Fig 2.6
Circular distribution paper

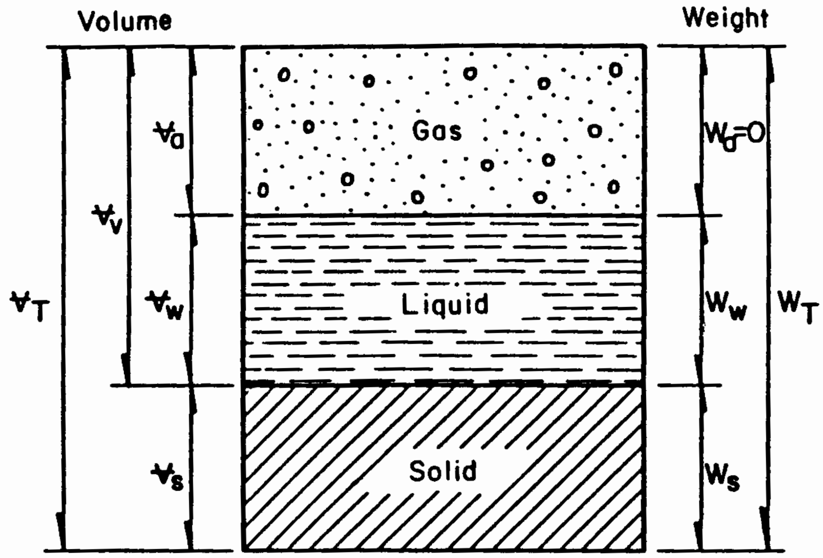


Fig 2.7
Scheme of the soil

CHAPTER 3

RESISTANCE TO FLOW

3.1 Bed forms

Friction is the main force which opposes to the movement of a liquid in pipes or in free surface flows. This is the case of artificial and natural channels, where beds are fixed when constituted by rock, concrete, asphalt, etc., or altered by flow and their components transported by water. The latter normally happens when beds are made of sand, gravel, cobbles, etc., but they can behave like the first group if flow velocities are too small to move the particles of the bottom and walls of the channel.

Even if fluvial hydraulics deals with natural channels, whatever is said in this chapter can also be applied to artificial ones.

While the surface of natural river beds can be plane or ondulated, ondulations may be of different types. The most important consideration when studying flow resistance in natural channels lies in the fact that the bottom configuration or bed form is not fixed or constant but varies and modifies as a function of the characteristics of the flow and of the particles that conform the bottom.

There is a plane bed when there is no bed transport or when particles are bigger than 5 mm. In river beds, ondulations form when they consist mainly of sand and there is sediment transportation. As already said, the shape and size of ondulations depend on current depth, flow velocity and particle diameter. There is not yet a criterium for the correct prediction of the geometry and dimensions of the ondulations, but only its type and approximate shape.

These are the different configurations sandy beds adopt (see fig 3.1):

1. Plane bed with no transport
2. Ripples, when size sediment is under 0.5 mm
3. Dunes
4. Plane bed with transport
5. Stationary waves
6. Antidunes

Besides these configurations, intermediate ones may exist. For instance, dunes may either tend to disappear forming a plane bed or they may present overimposed ripples. If the diameter of the material is more than 0.5 mm, ripples can not form. But if it is bigger than 5 mm, there are only plane beds with or without transport.

When studying a sandy bed with a given granulometry, by increasing little by little the Froude's number of the flow, bed configuration will change from the plane condition to antidunes. The Froude's number is equal to

$$\mathbb{F} = \frac{U}{\sqrt{gd}} \tag{3.1}$$

where: U is the mean velocity, in m/s, and d, is the depth, in m.

Ripples are triangular ondulations with a gentle slope in the upstream face and the natural slope of the material towards downstream. Their distribution in the bottom is irregular and the Manning's roughness coefficient varies from 0.018 to 0.035.

In this condition, particles are transported mainly on the bottom (see chapter 5). When the material consists of very fine sand and the flow velocity decreases, the ripple condition does not transform into a plane bed.

Dunes are ondulations bigger than ripples with smoother upstream and repose downstream slopes. Their distribution is even more irregular because their height is also very irregular. The Manning's roughness coefficient increases for a particular stretch and varies from 0.18 to 0.40.

For sands, when the flow Froude's number decreases, plane bed or ripples can reappear.

Intermitent vortexes with vertical axis are formed in the downstream face of dunes and lift big quantities of sand bed particles, which are afterwards temporarily transported in suspension.

When the Froude's number increases, dunes tend to disappear and *plane bed* conditions may present, provoking at the same time a considerable decrement in bottom roughness. Thus, for instance, Manning's roughness coefficient may reach values between 0.012 and 0.014. A bed with dunes can change directly into one with stationary waves, without necessarily implying the formation of a plane bed with transport condition.

Stationary waves and *antidunes* adopt an approximately sinusoidal form. Although particles move downstream, ondulations either remain in place or slowly move upstream.

While stationary waves keep their shape for a time, antidunes follow a cyclic shaping process which takes from 0.5 to 2 minutes. They grow as they move upstream, then they are destroyed and afterwards are swept by the flow, to begin once again the cycle. The Manning's roughness coefficient varies from 0.11 to 0.016 for stationary waves and between 0.011 and 0.022 for antidunes.

When configurations like plane bed without transport, ripples and dunes are present, it is said that a *lower regime* condition exists in the flow. And when there are stationary waves and antidunes, it is said that an *upper regime* exists. Plane beds with transport can be associated to *transition conditions* or to any of the two already mentioned regimes. The transition regime occurs when the Froude's number is between 0.60 and 1.20.

In lower regimes roughness normally increases as bed ondulations change from plane bed to dunes, and in water surface there is not a type of wave that reveals the configuration of the bottom. When there is a plane bed with transport, roughness decreases.

In case of upper regimes, the maximum possible roughness is always smaller than the maximum of lower regimes. In water free surfaces, waves are indicative of bottom configuration: if waves advance upstream and break, there are antidunes in the bottom. Although eventually swept, when waves remain in place, they indicate the existence of stationary waves in the bed.

Among all these possibilities, for the calculation of losses due to friction two bed configurations can be distinguished:

1. Flat or plane beds, where particles do not move, and
2. Ondulated bottoms with transport

3.2 Resistance to flow in channels with no transport or fixed boundaries

3.2.1 Generalities

In the flow direction, the liquid weight component is the force which makes the liquid move and which tends to balance the friction developed against the bed and sides. In permanent regimes both forces are in equilibrium according to the following equation

$$\tau_0 = \gamma R S \tag{3.2}$$

where

τ_0 shear stress produced by the liquid on the bottom, in kgf/m²
R hydraulic radius of the cross section, in m
S hydraulic slope, non dimensional

Some definitions and concepts to keep in mind when studying water flow are the following

a) Velocity associated to shear stress, or shear velocity U_*, which is defined as

$$U_* = \left(\frac{\tau_0}{\rho}\right)^{1/2} \tag{3.3}$$

Substituting in eq 3.3 what has been indicated for eq 3.2, it is obtained

$$U_* = \sqrt{g R S} \tag{3.4}$$

b) The relative roughness ϵ is equal to the height of the boundary roughness k divided by the hydraulic radius R

$$\epsilon = \frac{k}{R} \tag{3.5}$$

c) Boundary layer, correctly named as laminar sublayer or viscous sublayer, is a liquid layer of width δ contiguous to the wall, where flow movement depends on its viscosity; out of the layer, there is properly turbulent flow. δ is expressed as

$$\delta = \frac{5\nu}{U_*} \tag{3.6}$$

In fluvial hydraulics and with the object of reducing the zones (buffer zone) in the flow velocity distribution to a vertical, it is considered that

$$\delta_o = \frac{11.64\,\nu}{U_*} \tag{3.7}$$

The meaning of δ_0 given by eq 3.7 represents the mean distance from the bottom at which the linear velocity distribution curve of the viscous or boundary layer intersects the logarithmic velocity distribution curve of the turbulent zone, when the flow is turbulent with smooth walls.

d. In a moving liquid two flow conditions may present: laminar or turbulent. While in the first viscosity directly intervenes in the movement of the liquid, in turbulent flows the influence of viscosity is very low and the flow is especially affected by wall roughness. To know whether a flow is laminar or turbulent Reynold's criterion is applied. He found out that it depends on the non dimensional number which takes his name

$$\mathbb{R} = \frac{U\,R}{\nu} \quad \text{Reynold s number} \tag{3.8a}$$

Thus, for free surface flows, it holds that

1. Laminar flow, when

$$\mathbb{R} < 500$$

2. Turbulent flow, when

$$\mathbb{R} > 1000$$

The following expression is often applied for the Reynolds number, which is useful in pipes and free surface flows

$$\mathbb{R} = \frac{4\,U\,R}{\nu} = \frac{U\,D}{\nu} \tag{3.8b}$$

When eq 3.8b is used, flow is laminar if $\mathbb{R} < 2000$ and it is turbulent if $\mathbb{R} > 4000$.

It is important to remember that in the eqs 3.8, for pipes R = D/4

R hydraulic radius
D pipe diameter

In nature, practically all rivers, brooks and artificial channels have turbulent flows.

e. When the flow is turbulent and depending on the height of the wall protuberances (or boundary roughness) and on Reynold's number, three behaviour conditions can appear on the wall.

1. If the heights of the wall roughness are very small or the Reynold's number is quite low, these heights remain within the viscous sublayer and therefore wall resistance is only a function of viscosity. In this case and from the hydraulic point of view, the wall is considered to be smooth, thus fulfilling that

$$\frac{k\,U_*}{\nu} \leqslant 5 \quad \text{hydraulically smooth boundary or wall}$$

2. If protuberances project outside the viscous layer, the resistance of the wall is a function of the boundary roughness heights. When this occurs, from the hydraulic point of view, the wall is considered to be rough, thus fulfilling that

$$\frac{k\,U_*}{\nu} \geqslant 70 \quad \text{hydraulically rough boundary}$$

3. The transition state takes place between the two already described conditions

$$5 < \frac{k\,U_*}{\nu} < 70 \quad \text{transition boundary}$$

Generally, natural river beds present turbulent flow conditions with hydraulically rough boundaries.

3.2.2 Empirical formulae

In the determination of free surface flows in channels or natural streams, empirical or semiempirical formulae can be applied. To the first group belong those of Chezy, Manning and Darcy.

3.2.2.1 Chezy's equation

In 1775 Chezy proposed an equation based on the forces discussed in 3.2.1. This equation, which is the most widely used, establishes that

$$U = C \sqrt{RS} \tag{3.9}$$

where U, mean velocity in m/s; C, friction coefficient according to Chezy, in $m^{1/2}/s$ and, finally, S, hydraulic slope.

The eq 3.9 may also be expressed using a non dimensional Chezy's coefficient C_a that holds that

$$C_a = \frac{C}{\sqrt{g}} \tag{3.10}$$

therefore, eq 3.9 becomes

$$U = C_a \sqrt{g R S} = C_a U_* \tag{3.11}$$

Once the coefficient C is known, the flow velocity or the friction losses can be obtained, since

$$S = \frac{\Delta h}{L} \tag{3.12}$$

where Δh, head loss between two sections, in m, and L, distance between the two sections, in m.

To evaluate coefficient C in very broad sections, the following equation is recommended (see paragraph 3.2.3)

$$C = 7.83 \, Ln \frac{11.1 \, R}{k_s} \tag{3.13}$$

where k_s is the roughness of the bed, equivalent to the roughness used by Nikuradse in his tests. Some values of k_s are given here

k_s	condition
2ϵ	lined channels
2D	laboratory channels; plane bed without transport and uniform particles with diameter D
$2D_{84}$	natural sediment; plane bed without transport
h	bed with ripples or dunes without transport. h = height of the bed forms
$3.5D_{84}$	channels with macroroughness $0.03\ m < D_{84} < 0.47\ m$ and $(R/D_{84}) < 20$

For trapezoidal sections, the equation proposed by Keulegan may be used

$$C = 7.83\ Ln\ \frac{12.27\ R}{k_s} \qquad (3.14)$$

Eqs 3.13 and 3.14 were obtained from Prandtl and Von Karman's theories and from Nikuradse's experiences. Both can be applied only in case of turbulent flows with hydraulically rough walls. For other wall conditions see 3.2.3.

The advantage of obtaining coefficient C with eqs 3.13 and 3.14 or similar (see 3.2.3), lies in the fact that if a mistake is made when determining k_s, it will be very small in U and C. For instance, if there is a 100 percent error in the determination of k_s, it will result under 12 percent in Chezy's roughness coefficient.

3.2.2.2 Manning's equation

In 1889 Manning proposed the following equation to evaluate mean flow velocity

$$U = \frac{1}{n}\ R^{2/3}\ S^{1/2} \qquad (3.15)$$

where n is Manning's total roughness coefficient which depends on the boundary characteristic, in $s/m^{1/3}$. When using the English Unit System, n is non dimensional and keeps its value, and eq 3.15 is multiplied by 1.49.

The Manning's equation is the most widely known and used by engineers. To obtain the value of n for plane beds with granular graded material and channels with uniform section, the following equations can be applied:

According to Strickler (1923)

$$n' = \frac{(D_{65})^{1/6}}{7.66\sqrt{g}} = \frac{(D_{65})^{1/6}}{24} \tag{3.16}$$

and to Meyer-Peter and Müller (1948)

$$n' = \frac{(D_{90})^{1/6}}{8.3\sqrt{g}} = \frac{(D_{90})^{1/6}}{26} \tag{3.17}$$

Both equations use particle diameter in m.

If eqs 3.16 and 3.17 are substituted in eq 3.15, and as $U_* = \sqrt{g R S}$, the following equations are obtained

$$\frac{U}{U_*} = 7.66 \left(\frac{R}{D_{65}}\right)^{1/6} \tag{3.18}$$

$$\frac{U}{U_*} = 8.3 \left(\frac{R}{D_{90}}\right)^{1/6} \tag{3.19}$$

Eqs 3.16 and 3.17 and also 3.18 and 3.19 can be applied to uniform sections with plane bed and walls of loose granular material.

The Manning's coefficient n for other materials different from the already mentioned may be found in many books on hydraulics.

3.2.2.3 Darcy's equation

Using non dimensional parameters, Darcy proposed

$$U = \left(\frac{8 g R S}{f}\right)^{1/2} \tag{3.20}$$

where

f non dimensional Darcy-Weisbach roughness coefficient

Darcy obtained his equation to calculate the losses produced by friction in pipes.

In order to apply Darcy's eq 3.20 to channels and streams without transportation, the coefficient can be obtained from the Moody's diagram, which is the most widely applied and which has been elaborated for commercial pipes. In the figure, Reynold's number is a function of diameter D. Therefore, when dealing with channels, Reynold's number expression is to be modified as indicated here, considering that in a circular pipe R = D/4

$$\mathrm{IR} = \underset{\text{pipes}}{\frac{UD}{\nu}} = \underset{\text{channels}}{\frac{4\,UR}{\nu}} \tag{3.21}$$

Coefficient f also depends on the relative roughness and it appears in the diagram as ϵ/D. Therefore, in the case of channels, this transformation must also be done; that is to say, k/4R is to be applied because

$$\frac{\epsilon}{D} = \frac{k}{4\,R}$$

The Moody's diagram is shown in fig 3.2, where both the Reynold's number and the relative roughness are in function of 4R for channels and rivers and D for pipes.

It is interesting to note that Darcy-Weisbach's criteria is not much applied for channels and streams. The usual way of obtaining f is by means of semiempiric formulae, as shown in 3.2.3. The best known for trapezoidal sections is

$$\frac{1}{\sqrt{f}} = 2.03 \log \frac{12.2\,R}{k_s} = 0.884 \,\mathrm{Ln}\, \frac{12.2\,R}{k_s} \tag{3.22}$$

and for very wide sections, the following is recommended

$$\frac{1}{\sqrt{f}} = 2.03 \log 11.1 \,\frac{R}{k_s} = 0.884 \,\mathrm{Ln}\, \frac{11.1\,R}{k_s} \tag{3.23}$$

Both equations were deduced in similar form to the indicated for eqs 3.13 and 3.14, and are equivalent to them.

Opposite to coefficients C and n, Darcy's roughness coefficient is non dimensional, that is why eqs 3.20, 3.22 and 3.23 can be used with any congruent system of units.

3.2.2.4 Relations among roughness coefficients

The coefficients obtained with eqs 3.9, 3.15 and 3.20 can be interrelated. Between the Chezy and the Manning's coefficients, the following relation exists

$$nC = R^{1/6} \tag{3.24}$$

and between the Chezy and Darcy's coefficients, the relation is

$$C\sqrt{f} = \sqrt{8g} \tag{3.25}$$

Taking into account eqs 3.4, 3.11 and 3.25, it is obtained

$$\frac{C}{\sqrt{g}} = \frac{\sqrt{8}}{\sqrt{f}} = \frac{U}{U_*} = C_a \tag{3.26}$$

Finally, it holds that between Manning and Darcy-Weisbach's coefficients

$$\frac{R^{1/6}}{n} = \left(\frac{8g}{f}\right)^{1/2} \tag{3.27a}$$

or

$$\frac{R^{1/3}}{n^2} = \frac{8g}{f} \tag{3.27b}$$

Engineers generally deal with only one of these coefficients, mainly the Manning's roughness coefficient. On the other hand, when one of the other two is used, it should be verified if its value has been properly calculated, starting from the one they know best. To make these comparisons easily, eqs 3.24 and 3.25 should be used. What is expressed by eq 3.25 fulfills between eqs 3.13 and 3.22, and also between eqs 3.14 and 3.22.

3.2.3 Semiempirical formulae

From Prandtl's theories about the development of the boundary layer, the length of the mixture and Von-Karman's works, the velocity distribution in a pipe or channel can be evaluated for turbulent flow and, from this knowledge, mean flow velocity can be deduced.

Thus Prandtl-Von Karman's Universal Law for velocity distribution is represented by the expression

$$u = \frac{U_*}{\kappa} \operatorname{Ln} \frac{y}{y_0} \tag{3.28}$$

where

- u mean velocity of a point situated at a distance y, measured from the bottom of the channel
- κ Von Karman's constant, with a value equal to 0.4 for rigid walls and water free of sediments. This value reduces when channels transport suspended sediments.
- y_0 ordinate of the velocity distribution curve at the point of intersection with the y axis (y value for u = 0)

y_0 is the only unknown variable and was experimentally obtained by Nikuradse.

The general equation for mean flow velocity is obtained from eq 3.28 and is given by

$$U = \frac{U_*}{\kappa} \operatorname{Ln} \frac{d}{e\, y_0} \tag{3.29}$$

where

- e base of neperian logarithms, e = 2.718281828
- d depth

Eqs 3.28 and 3.29 are dimensionally correct and can be used with any congruent system of units.

Thanks to Nikuradse's experiences y_0 was obtained. Some authors followed the same procedure with different roughness conditions and transversal sections. The equations to obtain flow distribution or mean flow velocity when flow is turbulent and boundaries are hydraulically smooth, rough or in transition, in trapezoidal and very wide rectangular channels are shown in the following paragraphs.

3.2.3.1 Turbulent flow with hydraulically smooth boundaries

As already said, a flow is turbulent with hydraulically smooth boundaries when two conditions are satisfied:

$$\mathbb{R} = \frac{4UR}{\nu} > 4000 \qquad \text{guarantees turbulent flow}$$

$$\mathbb{R}_* = \frac{U_* k}{\nu} \leqslant 5 \qquad \text{guarantees hydraulically smooth walls}$$

In this kind of flow, three zones can be distinguished, needing each of them a different equation to determine its velocity distribution. They are the viscous sublayer with apparently laminar flow, which is closest to the bed; the turbulent zone and a transition one between the other two. The viscous sublayer and the transition constitute what is called the boundary region.

To facilitate the calculus without error, fluvial hydraulics only considers two zones: the viscous sublayer or boundary layer and the turbulent one.

The boundary between both zones is determined by the place of intersection of the linear velocity distribution of the viscous sublayer with the properly said logarithmic distribution of the turbulent flow; this happens when

$$y = \frac{11.64\,\nu}{U_*} = \delta_o \qquad (3.30)$$

Therefore, the thickness of both zones is

Zone	Boundaries	Flow velocity distribution
Boundary layer (laminar sublayer)	$0 < y < \frac{11.64\,\nu}{U_*}$	linear
Turbulent layer	$\frac{11.64\,\nu}{U_*} \leq y \leq d$(depth)	logarithmic

a) **Theoretical velocity distribution for very wide channels**

a.1 **Within the boundary layer**

$$u = \frac{U_*^2\,y}{\nu} \tag{3.31}$$

a.2 **In the turbulent zone**

$$u = 2.5\,U_*\,\text{Ln}\,9.025\,\frac{U_*\,y}{\nu} \tag{3.32}$$

b) **Mean flow velocity**

Based on the works of Prandtl, Von Karman and Nikuradse, some equations were elaborated for the obtaintion of the mean flow velocity for different sections. The present work only takes into account very wide channels, according to Nikuradse (1933), and trapezoidal ones, according to Keulegan (1938). From the practical point of view, a channel is very wide when

$$B > 40\,d \tag{3.33}$$

because it can be considered that the hydraulic radius is approximately equal to

$$R \simeq d \tag{3.34}$$

b.1 **For very wide channels**

$$U = 2.5\,U_*\,\text{Ln}\,\frac{3.32\,U_*\,R}{\nu} \tag{3.35}$$

b.2 For trapezoidal channels (after Keulegan)

$$U = 2.5\, U_* \, Ln \, \frac{3.67\, U_* \, R}{\nu} \qquad (3.36)$$

3.2.3.2 Turbulent flow with hydraulically rough boundaries

The viscous sublayer does not exist in this kind of flow, and this happens when

$$\frac{U_* k}{\nu} > 70$$

Therefore, there is only one equation for velocity distribution because only one zone corresponds to turbulent flows.

c) Velocity distribution for very wide channels

$$u = 2.5\, U_* \; Ln \, \frac{30.2\, y}{k_s} \qquad (3.37)$$

where k_s is, the equivalent roughness according to Nikuradse (see comments to eq 3.13).

d) Mean flow velocity

d.1 For very wide channels

$$U = 2.5\, U_* \, Ln \, \frac{11.1\, R}{k_s} \qquad (3.38)$$

d.2 For trapezoidal channels (after Keulegan)

$$U = 2.5\, U_* \, Ln \, \frac{12.27\, R}{k_s} \qquad (3.39)$$

3.2.3.3 Turbulent flow with transition boundary

This condition holds when

$$5 < \frac{U_* \, k_s}{\nu} < 70$$

The formulae for evaluating the distribution of velocities and the mean flow velocity with transition boundary also work for the other two conditions already explained. It is better to apply one of the following two criteria: Einstein's or Colebrook-White's formulae, both based on Keulegan. The first is very well known in fluvial hydraulics books but needs a figure for its application. On the other hand, Colebrook-White's formulae are equal to those obtained by other authors but they are presented in a more general way. We shall only see the equation for trapezoidal channels.

e) Velocity distribution

e.1 According to Einstein

$$u = 2.5\, U_* \;\mathrm{Ln}\, \frac{30.2\, y\, x}{k_s} \tag{3.40}$$

where

x constant, function of k_s/y_0, obtained by applying fig 3.3

e.2 According to Colebrook-White with Keulegan's coefficient

$$u = -2.5\, U_* \;\mathrm{Ln}\left(\frac{\nu}{9.025\, U_*\, \nu} + \frac{k_s}{30.2\, y}\right) \quad \text{wide channel} \tag{3.41a}$$

$$u = -2.50\, U_* \;\mathrm{Ln}\left(\frac{\nu}{9.98\, U_* y} + \frac{k_s}{33.3\, y}\right) \quad \text{trapezoidal channel} \tag{3.41b}$$

f) Mean flow velocity

f.1 According to Einstein

$$U = 2.5\, U_* \;\mathrm{Ln}\, \frac{12.27\; R\, x}{k_s} \tag{3.42}$$

f.2 According to Colebrook-White with Keulegan's coefficients (compare with eqs 3.41)

$$U = -2.5\, U_* \mathrm{Ln}\left(\frac{\nu}{3.32\, U_* R} + \frac{k_s}{11.1\, R}\right) \quad \text{wide channels} \tag{3.43a}$$

$$U = -2.5\, U_* \mathrm{Ln}\left(\frac{\nu}{3.67\, U_* R} + \frac{k_s}{12.27 R}\right) \quad \text{trapezoidal channels} \tag{3.43b}$$

3.2.4 Comparison between empirical and semiempirical formulae

The semiempirical equations presented in 3.2.3 to calculate mean flow velocity can be written as

$$U/U_* = 2.5 \, Ln(A) \qquad (3.44)$$

where A is an expression which depends on the hydraulic condition of the boundary and on the form of the section. Thus, for instance, in case of rough boundaries and trapezoidal sections $A = 12.27 \, (R/k_s)$ (see eq 3.39).

The comparison between semiempirical equations and those of Chezy and Darcy-Weisbach can be easily done by means of what has been said in eq 3.26.

3.2.4.1 Comparing Chezy's equation

Chezy's equation can be obtained from eqs 3.26 and 3.44, and takes the form of

$$C = 2.5 \sqrt{g} \, Ln\,(A) = 7.83 \, Ln\,(A) \qquad (3.45)$$

From eq 3.45 the value of C is obtained directly as a function of the characteristics of the flow without the need of using tables or of assuming its value.

From eqs 3.26 and 3.44 Darcy-Weisbach coefficient can be obtained, and it is expressed as

$$1/\sqrt{f} = 2.5/\sqrt{8} \, Ln(A) \qquad (3.46a)$$

or

$$1/\sqrt{f} = 0.884 \, Ln(A) \qquad (3.46b)$$

3.3 Flow resistance in beds with sediment transport

It is not so easy to predict resistance to flow in natural river beds with transport because

1. Bed configuration changes as flow intensity varies.

2. Sometimes bed particles are transported in suspension and the increase in concentration modifies the liquid and flow characteristics.

In natural beds, total resistance can be divided in two resistances: one due to particles and the second to ondulations or bed forms; therefore, it is possible to refer to hydraulic radius, roughness coefficients, slope, etc., associated to total roughness, particles and bed forms.

That is why most authors assume that shear stress in beds can be divided in two parts

$$\tau_0 = \tau_0' + \tau_0'' \tag{3.47}$$

where

τ_0 total shear stress

τ_0' shear stress associated to particles

τ_0'' shear stress associated to bed forms

From eq 3.46 and given $\tau = \rho U_*^2$ (see eq 3.3) the following is obtained

$$U_*^2 = (U_*')^2 + (U_*'')^2 \tag{3.48}$$

where

U_* total shear stress velocity

U_*' shear stress velocity associated to particles

U_*'' shear stress velocity associated to bed forms

As already seen, $\tau_0 = \gamma RS$. When substituting this value in eq 3.47, most authors accept that slope S is the same for τ_0' and τ_0'' and that both, roughness variations due to particles and bed ondulations, affect the hydraulic radius. That is why there is an hydraulic radius R' associated to particles and to plane beds and another one, R'', which corresponds to bed forms. Therefore, substituting eq 3.1 in eq 3.47 it is obtained

$$\gamma RS = \gamma R'S + \gamma R''S \tag{3.49a}$$

and then

$$R = R' + R'' \tag{3.49b}$$

Other authors prefer to consider that there are changes in slope while accepting R to be constant. Thus, it holds that

$$S = S' + S'' \tag{3.50}$$

For the purposes of this work eq 3.49b is normally considered as valid.

In the previous equation and in the following ones, variables with prime index are associated to particles, those with double prime, to bed forms, and variables without index are related to total roughness; this means that they give the total value of each variable.

When working with roughness coefficients, it is possible to obtain the relations among them. For example, from Manning's equation, the following is fulfilled (see eq 3.15)

$$n^{3/2} = (n')^{3/2} + (n'')^{3/2} \qquad (3.51)$$

from Darcy's equation (eq 3.20)

$$f = f' + f'' \qquad (3.52)$$

and from Chezy's (eq 3.9)

$$\frac{1}{C^2} = \frac{1}{C'^2} + \frac{1}{C''^2} \qquad (3.53)$$

While mean velocity (like slope) has a single value.

To find out the mean flow velocity or slope, equations and criteria can be divided in

a) Those which only consider total resistance

b) Those obtained by dividing total resistance into resistance associated to particles and to bed forms

Two methods for each group are discussed in the next paragraphs.

It has to be remembered that the equations and methods proposed for evaluating resistance to flow do not take into account neither the real shape of sections, contractions and expansions nor the obstacles in sections and that, therefore, these methods only work for straight reaches with constant sections.

Provided that some of the methods exposed in 3.3.1 and 3.3.2 are used and, therefore, U, R and S are known, it is convenient for the engineer to apply any of the eqs 3.9, 3.15 or 3.20 to obtain the roughness coefficient most familiar to him, like, for instance, Manning's. At least he will be able to discard absurd results.

Due to the empirical character of the methods used to evaluate the resistance to flow, there will be as many results as methods applied, and only the real knowledge of the river under study will make the discernment of the most precise value possible.

3.3.1 Methods which consider total resistance

Among the methods which permit to calculate total flow resistance without separating the one due to particles and ondulations, there are those proposed by Cruickshank-Maza (1973), Garde-Ranga (1966), Paris (1980) and Chimeka (1967). The first three are shown here.

3.3.1.1 Cruickshank-Maza's method

To obtain the basic equations to evaluate mean flow velocity, these authors took into account the relative roughness of grains and, implicitly, the bed form due to flow variation. They proposed two equations: one for lower regime with ripples and dunes, and the other for upper regime with stationary waves or antidunes.

For lower regime

$$U = 7.58\, \omega_{50} \left(\frac{d}{D_{84}}\right)^{0.634} \left(\frac{S}{\Delta}\right)^{0.456} \tag{3.54}$$

which is fulfilled if

$$\frac{1}{S} \geqslant 83.5 \left(\frac{d}{\Delta\, D_{84}}\right)^{0.35} \tag{3.55}$$

For upper regime

$$U = 6.25\, \omega_{50} \left(\frac{d}{D_{84}}\right)^{0.644} \left(\frac{S}{\Delta}\right)^{0.352} \tag{3.56}$$

which is fulfilled if

$$\frac{1}{S} \leqslant 66.5 \left(\frac{d}{\Delta D_{84}} \right)^{0.382} \tag{3.57}$$

where ω_{50}, fall velocity of particles with diameter D_{50}, in m, solved with eq 2.14; U, mean velocity; d, mean depth; S, hydraulic slope and $\Delta = (\gamma_s - \gamma)/\gamma$.

Due to the fact that these equations are dimensionally correct, they can be applied whatever system of units is used.

In fig 3.4, eqs 3.54 and 3.56 and their limits of application are shown. In practice, it is functional to consider that in transition regimes, mean velocity increases without increasing depth.

Even if the application of this method does not need a roughness coefficient, it implies the careful analysis of bed sediments, basically because the possible error in the determination of the fall velocity ω_{50} has a direct effect in the value of U.

This method has the advantage of been the easiest to apply because it is direct and does not need the use of additional figures for the obtaintion of its variables. But when bed material and slope are given, the disadvantage lies in the fact that velocity depends only on depth, even if it takes into account changes in regime.

When the flow regime belongs to the transition zone it is better to use the equation for lower regime, because it is a more stable condition.

3.3.1.2 Garde-Ranga's method

Another method which also considers total resistance is the one suggested by Garde-Ranga. Starting from the dimensional analysis of some meaningful variables, they proposed the following

$$\frac{U}{\sqrt{g \Delta D_{50}}} = K \left(\frac{R}{D_{50}} \right)^{2/3} \left(\frac{S}{\Delta} \right)^{0.5} \tag{3.58}$$

Where the coefficient K depends on the bed configuraton; it is equal to 7.66 for plane bed without transport, to 3.20 for ripples and dunes, and to 6 for transition and dunes. The equation is dimensionally correct.

This method presents the disadvantage of requiring the knowledge of the bed configuration, even if it may be supposed with some experience, especially when dealing with sandy beds and the Froude's number is less than 0.5, because often K is equal to 3.2. It has the same advantages and disadvantages of Cruickshank-Maza's method.

3.3.1.3 Paris' method

It divides the condition of the bed without transport from that with ondulations and bed sediment transport. The boundary between both conditions is defined by the Shield's parameter, eq 4.5 (see point 4.1.2), but associated with the D_{35} of the particle mixture; that is to say

$$\tau_{*c} = f(D_{35}) \tag{3.59}$$

$$\tau_{*c} = \frac{d_c S}{\Delta D_{35}} \tag{3.60}$$

where the variables not yet defined are

- τ_{*c} Shield's parameter for the transport critical condition. It can be obtained with the Shield's criterion, using fig 4.2 according to what is explained in 4.1.2, but with $D = D_{35}$

- d_c critical depth associated with incipient sediment transport, in m

To calculate τ_{*c} as a function of the sediment diameter, Paris reccomends to use Ackers and White's criterion —which is also shown in fig 4.2— instead of Shields'.

The flow mean velocity is given by Chezy's equation

$$U = C_a \sqrt{gRS} \qquad \text{(eq 3.11)}$$

where C_a is Chezy's non dimensional coefficient which is valued as a function of the relation

$$\frac{\tau_*}{\tau_{*c}} = \frac{d}{d_c}$$

being

$$\tau_* = \frac{d\,S}{\Delta\,D_{35}} \tag{3.61}$$

where

d the flow real depth

S the flow hydraulic slope

Two conditions may be distinguished

a) If $\frac{\tau_c}{\tau_{*c}} \leqslant 1$, there is no sediment transport and the bed becomes flat. Then, the value of C_a is

$$C_a = 5.66 \log \frac{10\,d}{D_{35}} \tag{3.62a}$$

or

$$C_a = 2.46 \,\mathrm{Ln} \frac{10\,d}{D_{35}} \tag{3.62b}$$

b) If $\frac{\tau_*}{\tau_{*c}} > 1$, there is sediment transport and C_a takes the value of

$$C_a = [1 - 0.47 \log \frac{\tau_*}{\tau_{*c}} + 0.12 (\log \frac{\tau_*}{\tau_{*c}})^2]\, C_{ac} \tag{3.63}$$

where C_{ac} is valued according to

$$C_{ac} = 5.66 \log \frac{10\,d_e}{D_{35}} \tag{3.64}$$

and d_c is obtained from eq 3.60

$$d_c = \frac{\tau_{*c}\,\Delta\,D_{35}}{S} \tag{3.65}$$

Paris has applied his method to more than 3000 field and laboratory data and has found a good agreement. The ranges of the most important variables are: velocity, 0.1 to 2.81 m/s; depth, 0.015 to 17.2 m; slope, 0.00056 to 2.5 percent and sediment diameter, 0.04 to 68 mm.

3.3.2 Methods which subdivide total resistance

Among the methods to find out the mean flow velocity that separately take into account the resistance due to particles and bed forms are those of Engelund, Alam-Lovera-Kennedy, Einstein-Barbarossa and Raudkivi, the last of which usually gives a higher mean velocity than the real one. The first two are described below.

3.3.2.1 Engelund's method

He proposed his method in 1966 and 1967 with the following neccesary equations

$$\tau'_* = f(\tau_*) \tag{3.66}$$

$$\tau_* = \frac{R\,S}{\Delta\, D_{50}} = \frac{d\,S}{\Delta\, D_{50}} \tag{3.67}$$

$$\tau_* = \frac{R'S}{\Delta\, D_{50}} = \frac{d'\,S}{\Delta\, D_{50}} \tag{3.68}$$

where

- τ_* Shields' non dimensional parameter
- τ'_* Shields' non dimensional parameter associated to particles
- R' hydraulic radius related to grain roughness
- R total hydraulic radius of the cross section

Although the relation between τ_* and τ'_* can be seen in fig 3.5 for lower regimes, the following expression can also be used

$$\tau'_* = 0.06 + 0.4\,\tau_*^2 \tag{3.69}$$

Then, to obtain flow mean velocity, Engelund proposed the equation

$$U = 2.5\,U'_*\,\mathrm{Ln}\,\frac{11.1\,R'}{2\,D_{65}} \tag{3.70}$$

where

R' = d' depth related to grain roughness, in m

U'_* shear velocity related to grain roughness, in m/s, which is equal to

$$U'_* = \sqrt{g R' S} \tag{3.71}$$

Once R, S and bed particle size are known, mean velocity is obtained as follows.

1. τ_* is calculated from eq 3.67, assuming R equal to d

2. τ'_* is obtained from fig 3.5 or eq 3.69

3. The value of d' from eq 3.68 is obtained assuming R' equal to d'

4. Shear velocity U'_* is calculated from eq 3.71

5. Finally, mean velocity is calculated from eq 3.70

According to fig 3.5, two values of τ'_* are obtained for the interval $0.4 < \tau_* \leqslant 1.6$: the first for upper regime and the other for the lower one, therefore, bed configuration should be known. For this, fig 3.6 is applied, which includes non dimensional parameters $U/\sqrt{gd}$ and U/U'_*.

6. It is necessary to verify whether the selected branch in fig 3.5 is of the same type of regime.

3.3.2.2 Alam-Lovera-Kennedy's method

In 1969 they proposed their method which started with the assumption that S = S' + S'' and a constant hydraulic radius R.

The equations suggested are those of Darcy-Weisbach which are only applicable to lower regimes.

$$U^2 = \frac{8 g R S}{f} \tag{3.72}$$

where coefficient f is expressed as

$$f = f' + f'' \tag{3.73}$$

Coefficient f' is associated to particle diameter and to plane beds. It is evaluated by means of Lovera-Kennedy's fig 3.7, as a function of (UR/ν; R/D_{50}). On the other hand, coefficient f'' is related to bed forms in lower regime (ripples and dunes) and it is obtained as a function of ($U/\sqrt{gD_{50}}$, $U/\sqrt{gR}$ and R/D_{50}), using fig 3.8, obtained by Alam-Kennedy.

The method is applied by trial and error as shown below:

1. Velocity U is assumed; hydraulic radius R is known.

2. Non dimensional parameters $\frac{UR}{\nu}$, $\frac{R}{D_{50}}$, $\frac{U}{\sqrt{gD_{50}}}$ and $\frac{U}{\sqrt{gR}}$ are obtained.

3. f' is obtained by means of fig 3.7. If the point defined in the figure falls under the line which gives the value of f' for smooth wall conditions, the value given by UR/ν at that line is adopted.

4. f'' is calculated with fig 3.8 with values $\frac{R}{D_{50}}$ and $\frac{U}{\sqrt{gD_{50}}}$ or $\frac{U}{\sqrt{gR}}$.

5. f is derived from eq 3.73.

6. U is obtained from eq 3.72 as a function of the f found in step 5 and the known S.

7. The value of U is compared to the assumed value. If they are different, new iterations will be done with the U which results from step 6 until both values are equal.

The same process can be applied to obtain the hydraulic ratio when velocity is known. But if the mean velocity and hydraulic radius are known and the slope is to be found, the process is direct without the need of the trial and error process.

The method works for sandy materials in lower regime, and even if it has the disadvantage of requiring the use of figures for its application, it yields reliable results.

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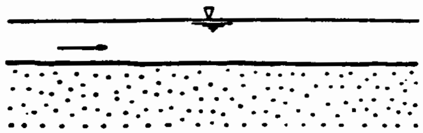
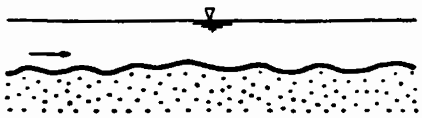
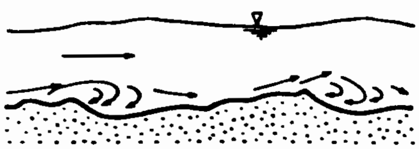


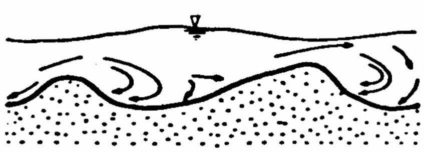
1. Plane bed without transport $F_r << 1$



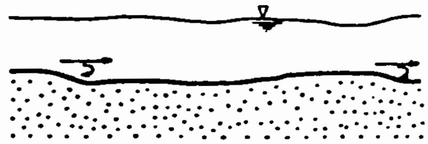
2. Ripples, $F << 1$ and $D_m < 0.5$ mm



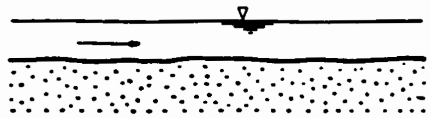
3a. Dunes with ripples, $F_r << 1$ and $D_m < 0.5$ mm



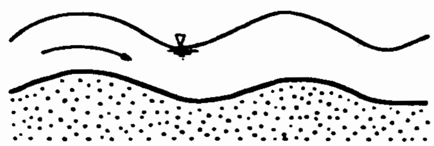
3b. Dunes, $F_r < 1$



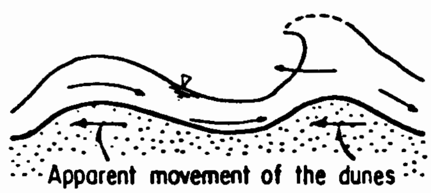
3c. Transition from dunes to plane bed, $F_r < 1$



4. Plane bed with transport, $F_r < 1$



5. Standing waves, $F_r \geq 1$



6. Antidunes breaking waves, $F_r > 1$

1,2,3 lower Regime

4,5,6 upper Regime

Fig 3.1
Forms of bed roughness in sand channels

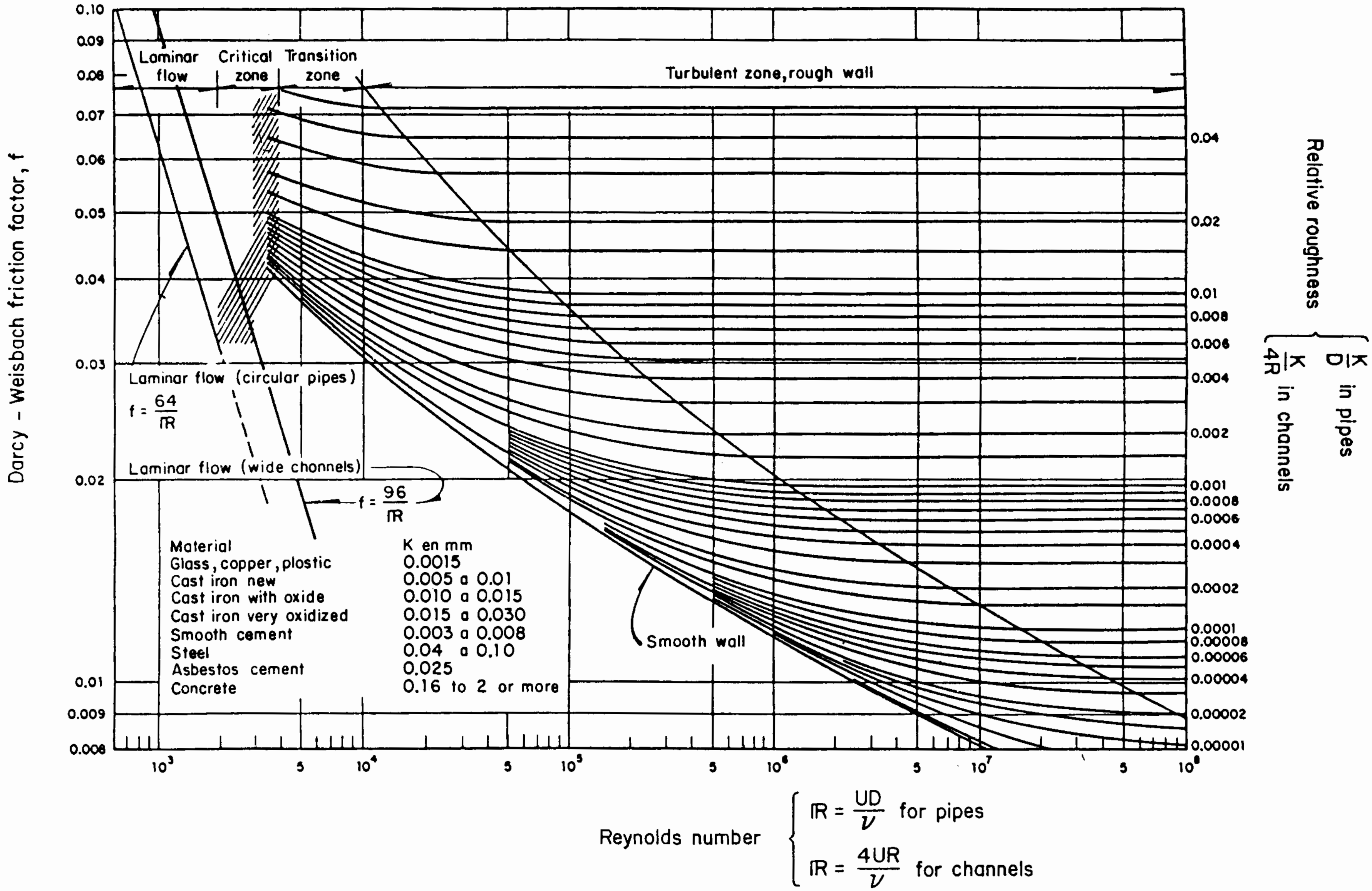


Fig 3.2
Moody diagram

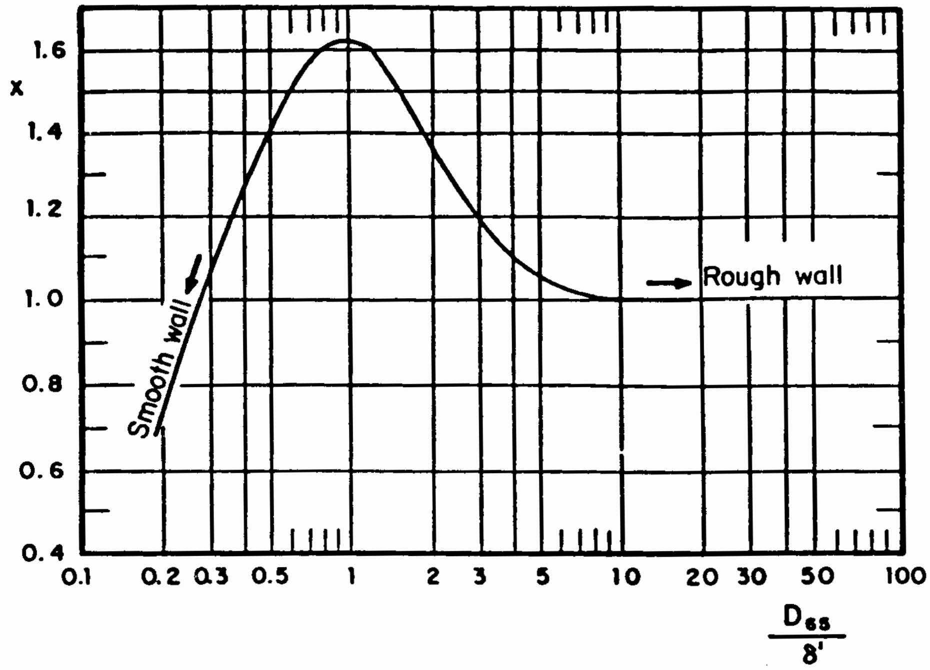


Fig 3.3

Correction factor x. Einstein method

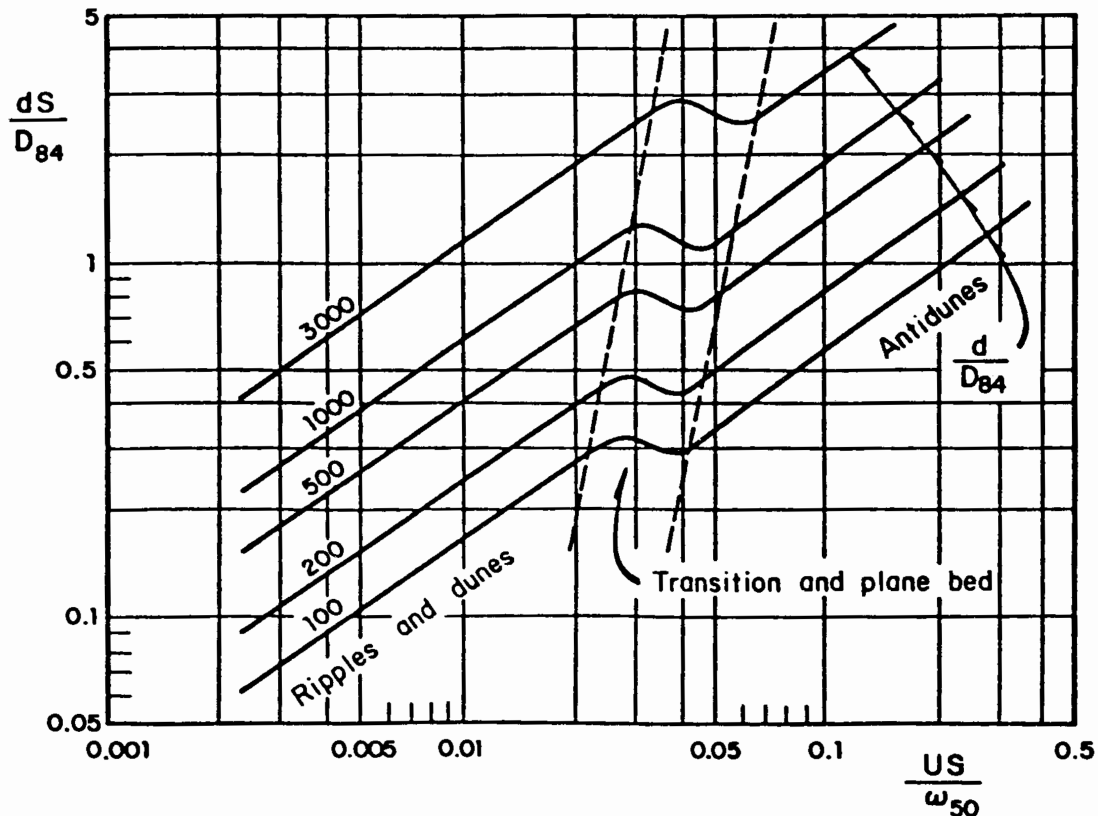


Fig 3.4

Diagram to obtain flow resistence in sandy river beds, after Cruickshank-Maza

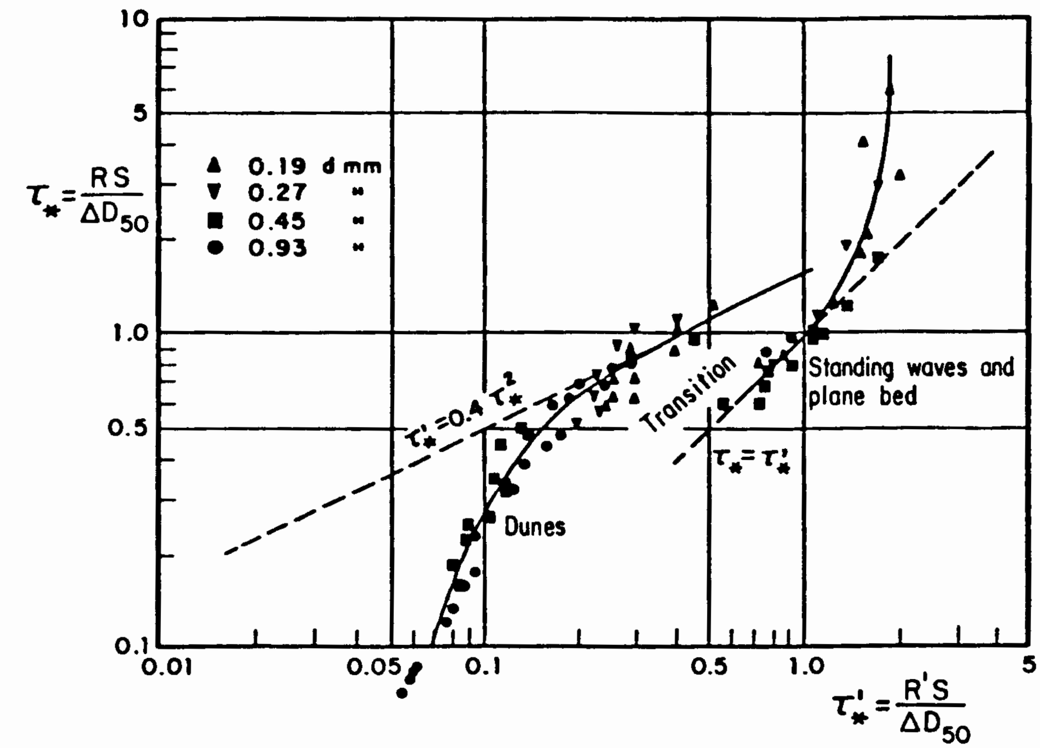


Fig 3.5

Relation between τ_* and τ'_*, after Engelund

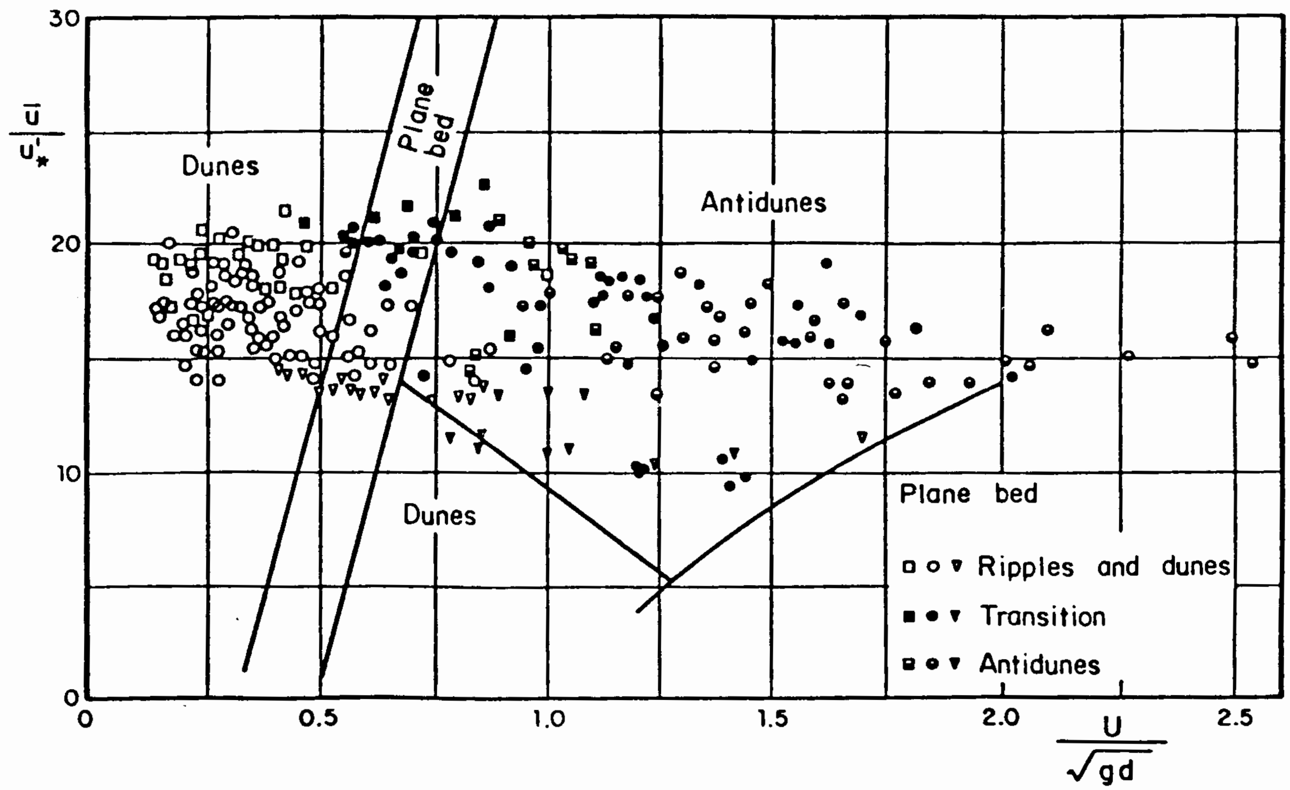


Fig 3.6

Engelund – Hansen criterion to obtain the bed form

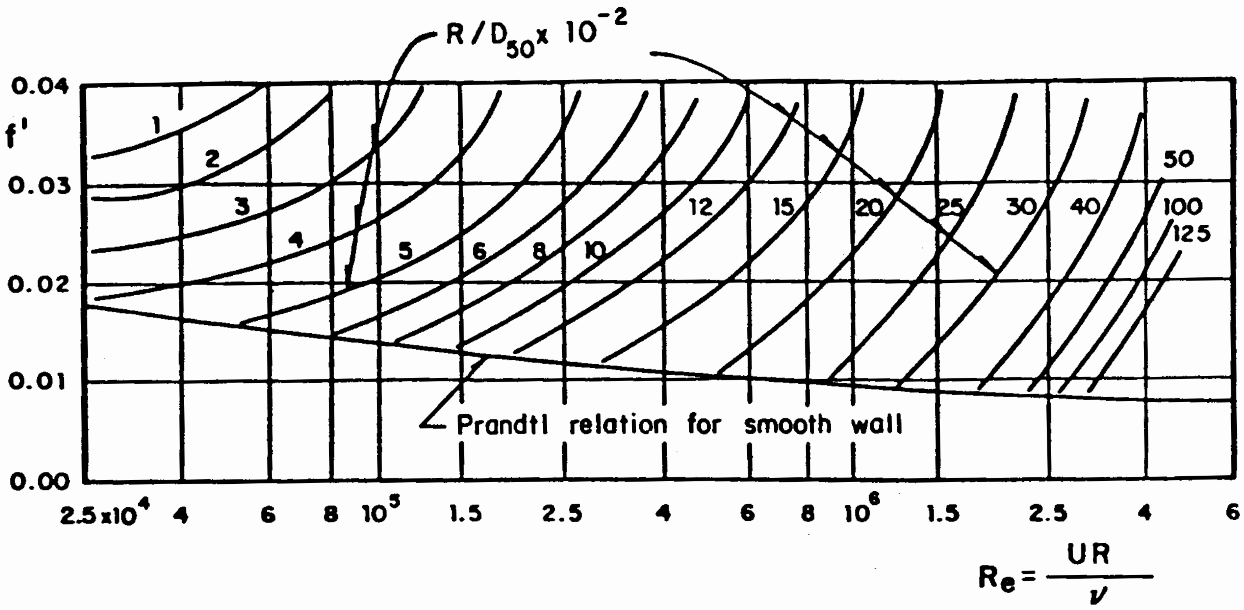


Fig 3.7
Friction coefficient f' for alluvial channels with plane bed, after Lovera – Kennedy

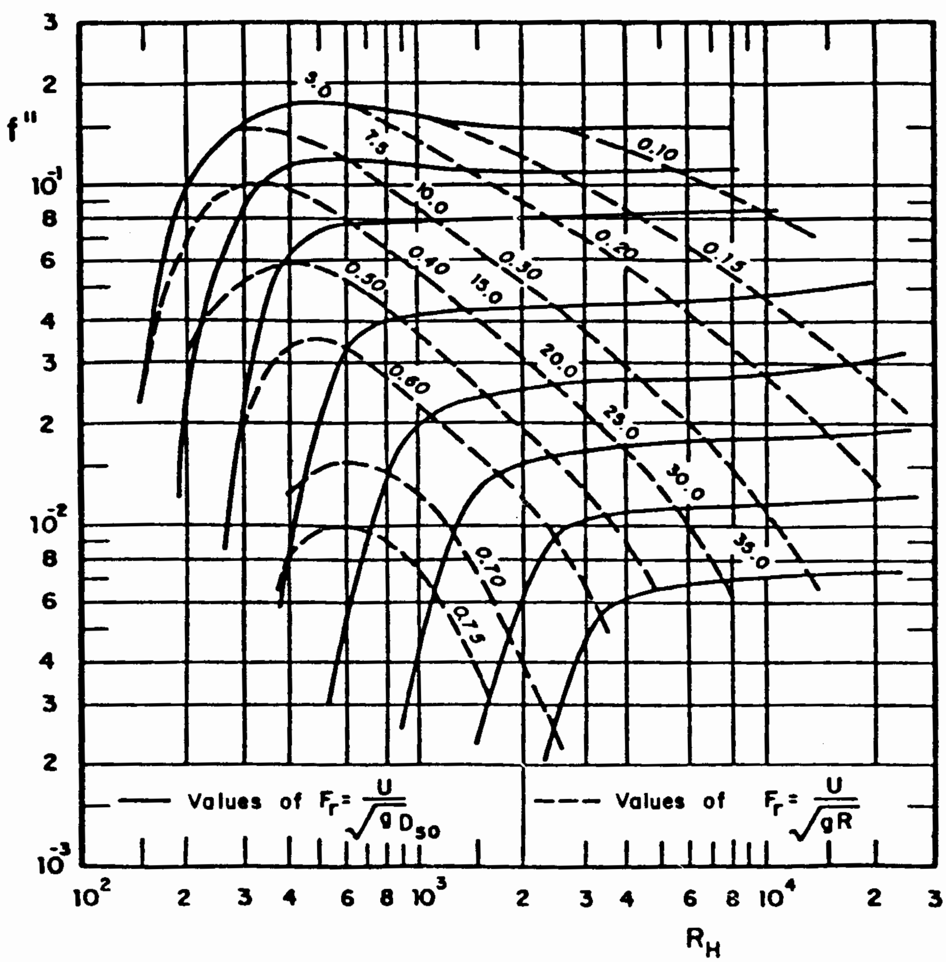


Fig 3.8
Friction coefficient f'' as a function of the Froude number and R/D50, after Alam – Kennedy

CHAPTER 4

INCIPIENT MOTION OF SEDIMENT PARTICLES

A flow capable of starting particle movement or transport in a river or channel reach has certain hydraulic characteristics which is necessary to determine. It is very important to know the critical transport conditions the current has when designing channels with no erosional process, diversion or intake channels with no sediment transport, and the conditions under which particles move, either to be transported or not deposited.

The beginning of motion is in relation to the shear stress a current produces on the bed, the mean flow velocity in the cross section, the mean velocity in a vertical or the bottom velocity. All these situations imply what is known as *critical* conditions.

The determination in a flow of the hydraulic conditions which start the movement of bed particles seems to be an easy matter. This is not so: in practice, there is not a single criterion capable by itself of solving the problem. Several situations can be distinguished in the observation of the flow characteristics when:

a. One particle moves within the observation field. This represents the minimum critical condition

b. Several particles move, of which a percentage can be determined beforehand

c. General particle movement exists with very small transport. This represents the maximum critical condition

d. Setting a limit in the sediment transport equation for which theoretical sediment transportation is null.

Besides, the problem complicates still further because many authors do not indicate the different conditions their methods or formulae are based on.

However, and unless explicitly expressed, most authors give for the critical condition a mean value between the minimum and the maximum ones, and this is the criterium also followed in this work.

When dealing with the critical condition, it is relevant to distinguish between the scientist and the engineer's point of view. For the former, it is important since the critical condition is defined up to the determination of all the intervining parameters and their circumstances. On the other hand, the engineer only needs a simple formula to design, for instance, channels without erosion and to build the smallest possible section with the maximum economy.

That is why, when this phenomenon occurs, it is better to let the highest possible flow run through the smallest section. The size of the cross section can vary according to the condition which is assumed to start movement.

Trying to be a little more conservative, we normally use mean values in this work; however, other methods are presented to help the engineer choose the most suitable for his particular problem.

When referring to critical transport conditions or to incipient motion, it is necessary to distinguish between: a) the hydraulic characteristics of the current and, b) the shear stress of the particle or any of the three critical particle velocities already mentioned; that is to say, the resistance made by the particle to start moving and, on the other hand, one of the following flow hydraulic characteristics:

a) Bed shear stress
b.1) Mean flow velocity
b.2) Mean velocity in the vertical
b.3) Velocity near the bottom

a) The bed shear stress is given by eq 3.1

$$\tau_0 = \gamma R S \qquad (4.1)$$

b.1) Mean flow velocity in the cross section is obtained by eqs 3.9, 3.13, 3.18, 3.33, 3.34, 3.36, 3.37, 3.40 and 3.41a and b, being the last two the more general ones. Eq 3.41b can also be written as

$$U = 2.5\, U_* \ \mathrm{Ln}\left[\frac{12.27\, R}{3.34 \dfrac{\nu}{U_*} + k_s}\right] \qquad (4.2)$$

this equation is used in trapezoidal channels and turbulent flow with hydraulically smooth, rough or transition boundaries.

b.2) Mean velocity in a vertical can be obtained with eqs 3.33, 3.36 and 3.41a. Considering these equations, a general one, similar to eq 4.2, can be presented, but using d instead of R

$$U = 2.5\, U_* \ \mathrm{Ln}\left[\frac{11.1\, d}{3.34 \dfrac{\nu}{U_*} + k_s}\right] \qquad (4.3)$$

Eq 4.3 is useful to calculate the mean velocity in a vertical when flow is turbulent with hydraulically smooth, rough or transition boundaries and the cross section is very wide.

b.3) Close to the bottom, the velocity which acts directly on the particles can be determined with eqs 3.29, 3.35, 3.38 and 3.39a and b. They give current velocity u at a given distance y, measured from the bottom. A general expression for trapezoidal channels is

$$u = 2.5\, U_* \ \mathrm{Ln}\left[\frac{33.3\, y}{3.34 \dfrac{\nu}{U_*} + k_s}\right] \qquad (4.4)$$

In eqs 4.2 to 4.4, and for the design of channels with plane beds and critical transport condition, the following value of k_s must be used

$k_s = 2D_{90}$ when sediment size distribution is log normal

$k_s = 2D_{84}$ for any other distribution

Next, the critical condition to start particle movement will be explained.

4.1 Critical shear stress for granular soils

Many criteria have been proposed to evaluate the critical shear stress which starts particle motion. Among the most important are those by Lane (1937), Shields (1936), Meyer-Peter and Müller (1948), Laursen (1958), Straub (1935), White (1940), Iwagaki (1956), Gessler (1965), etc. Only the first four are described in the following paragraphs. Even if Gessler's method has the best formulation, it is the most difficult to apply.

4.1.1 Lane's method

Curves proposed by this author are shown in fig 4.1. The maximum shear stress a particle can resist before starting to move can be directly derived from them as a function of particle diameter. Fig 4.1, which Lane suggested, also includes other criteria. The main advantage of this method lies in the fact that, whatever the case, τ_c can be obtained if the fluid carries little, much or no material at all. It must be noticed that the increase in concentration mainly affects particles smaller than 2 mm, but that the effect decreases with the increase of particle size.

Lane allows the current to produce a bigger shear stress in the bottom than other authors. This is because his method was conceived for the design of earth channels with some transport at the beginning of their useful life (before armoring the bottom) which would not damage or affect the transversal section.

4.1.2 Shields' method

Shields was the first to propose a method to calculate the value of the shear stress at which a particle starts to move. Fig 4.2 shows the curve he obtained as a function of two non dimensional parameters, τ_* and $\mathbb{R}_*$, given by the following expressions

$$\tau_* = \frac{\tau_0}{(\gamma_s - \gamma)D} = \frac{\gamma R S}{(\gamma_s - \gamma)D} = \frac{R S}{\Delta D} \tag{4.5}$$

$$\mathbb{R}_* = \frac{U_* D}{\nu} = 11.64 \frac{D}{\delta_0} \tag{4.6}$$

where

τ_* Shields' non dimensional parameter

τ_0 shear stress exerted on the bottom by flow, in kgf/m²; for the points $\tau_c = \tau_0$ in Shields' curve

$\mathbb{R}_*$ Reynold's number associated to shear velocity, non dimensional

δ_0 thickness of the viscous sublayer, in m, given by eq 3.7

D particle diameter, in m. If different sizes are present $D = D_{50}$

As D is found in eqs 4.5 and 4.6, and R and S too, even if in an implicit way a trial and error process must be used to obtain τ_c. It is as follows

1. Since S and D_{50} are known, a hydraulic radius R is assumed

2. $U_* = \sqrt{gRS}$ is calculated

3. $\mathbb{R}_* = \dfrac{U_* D}{\nu}$ is calculated considering $D = D_{50}$

4. τ_{*c} is obtained by means of fig 4.2

5. The value of $\tau_0 = \tau_c$ is solved from the resulting relationship of step 4 and eq 4.2:

$$\tau_c = \tau_{*c} (\gamma_s - \gamma) D_{50} \tag{4.7}$$

6. The value of $R = \tau_c / \gamma_S$ is obtained

7. If the value of R obtained from 6 and that of the assumed R are not equal, the procedure is to be repeated until they are

8. Finally, R_c and τ_c are known.

According to Shields, if $\mathbb{R}_*$ is higher than 1.0, the parameter τ_{*c} takes a constant value of 0.60, and τ_c is directly obtained from eq 4.7

$$\tau_c = 0.06 (\gamma_s - \gamma) D_{50} \tag{4.8}$$

4.1.3 Meyer-Peter and Müller's formulae

From the Meyer-Peter and Müller's bed load equation presented in 5.1.1, the following results are obtained for transport equal to zero; that is to say, when particle transport stops

$$\frac{\tau_c}{(\gamma_s - \gamma)\, D_m} = 0.047 \qquad (4.9)$$

$$\tau_c = 0.047\, (\gamma_s - \gamma)\, D_m \qquad (4.10)$$

where D_m is the mean sample diameter.

The main difference between eqs 4.8 and 4.10 lies not only in the different value of parameter τ_*, but in the fact that for Meyer-Peter and Müller its value is constant and equal to 0.047 for all particle sizes.

4.1.4 Laursen's formula

When expressing his formula for sediment transport (see 5.1.1) Laursen introduces τ_c. To value it, he proposes an equation similar to those already shown, but changing the value of τ_{*c} to 0.039

$$\tau_c = 0.039\, (\gamma_s - \gamma)\, D_m \qquad (4.11)$$

4.2 Critical shear stress for cohesive soils

The critical shear stress resisted by cohesive soils is a function of the voids ratio and clay content; it can be obtained with the curves of The Bureau of Reclamation shown in fig 4.3.

Although there are natural river beds with cohesive walls, beds are generally covered by granular material.

Also frequent is the fact that channels in cohesive materials are not lined, that is why the geometric and hydraulic characteristics which do not produce erosion must be calculated.

Cohesive soils with high volumetric weight are more resistant to shear stress than those formed by granular or loose soils. Therefore, when a dike built with compact cohesive material breaks, the resulting failure does not advance so much to the sides as if it were made of sand.

4.3 Critical mean flow velocity of granular material

Another way of determining the hydraulic characteristics of a current when the bed particles start to move requires the knowledge of the critical mean velocity a particle can resist before beginning to move.

All vertical lines of very wide channels present a logarithmic distribution of velocities, and mean velocity is measured at about 0.368 d (d = depth) above the bed. Then, mean velocity is a function of the current depth and for its evaluation particle diameter and depth should be known. The logarithmic velocity distribution implies that the smaller the depth, the smaller is the vertical or cross section mean velocity needs to start particle movement.

Therefore, to calculate mean critical velocity, bed particle diameter and flow depth must be known. This is why critical shear stress is so frequently used, since it is determined only as a function of diameter. However, for engineers, mean velocity has a much more clear meaning.

An aditional advantage in favour of the critical shear stress is that most of the methods that propose critical mean velocity only deal with materials with a specific weight of 2650 kgf/m^3, or do not indicate the density for which they are applicable. Besides, others only evaluate mean critical velocity for one meter depth.

Empirical or semiempirical formulae can be used to determine how much mean flow velocity is necessary to start bed material movement. The first are obtained either in the field or laboratories and the second start from formulae similar to eq 4.2; thus, when assuming that $U_* = (\tau/\rho)^{1/2}$, critical velocity becomes a function of critical shear stress.

4.3.1 Empirical methods

Among other empirical methods to evaluate the critical mean velocity there are those of Maza-García (1978), Lischtvan-Levediev (1960), Bogardi (1971), Hjülstrom (1935), Levi (1945) and Helley (1969). Many other investigators proposed formulae where only diameter particle intervenes, without considering the depth of the current, a parameter which has to be taken into account when dealing with engineering problems. In this work, the first two are presented.

4.3.1.1 Maza-García's method

In 1978, and based on the results obtained by many investigators, Maza-García proposed the following equation

$$U_c = 4.71\, \Delta^{1/2}\, D^{0.35}\, R^{0.5} \tag{4.12}$$

or, if the expression is a function of the critical Froude number, $\mathbf{F}_c$

$$\mathbb{F}_c = 1.504\, \Delta^{1/2}\, (D/R)^{0.35} \tag{4.13}$$

In eqs 4.12 and 4.13, D and R are expressed in m and U in m/s.

Taking into account that for very wide channels, it holds that $R \simeq d$.

The two previous equations are equivalent and can be applied in the interval $0.0001 \text{ m} < D < 0.2 \text{ m}$. For river beds where material is almost uniform it is better $D = D_m$; and for well graded size distribution $D = D_{90}$ is recommended if the sediment size distribution is log-normal, and $D = D_{84}$ for any other distribution. When a conservative design is desired it is better to choose $D = D_m$.

Comparing the Maza-García's method with Shields', the results are similar with $0.0001 \text{ m} < D < 0.0015 \text{ m}$. In case of coarser particles, Shields' mean velocity rises up to a 30 percent.

If particles are considered with $\gamma_s = 2650 \text{ kgf/m}^3$, eq 4.12 becomes

$$U_c = 6.05\, D^{0.35}\, R^{0.15} \tag{4.14}$$

4.3.1.2 Lischtvan-Levediev's method

In his book, Lischtvan (1960) presents two tables based on Levediev's results; they show the maximum mean velocities a current may have without undergoing bottom erosion even if there is some particle movement. Table 4.1 gives the values of such velocity for granular material as a function of particle mean diameter and flow depth.

4.3.2 Semiempirical formulae

In cross section critical mean flow velocity can be obtained from eq 4.2 as a function of τ_c, whatever the hydraulic condition of the wall is. The equation can be written as

$$U = 2.50 \left(\frac{\tau}{\rho}\right)^{0.5} \text{Ln}\left(\frac{12.27\ R}{k_s + \frac{3.34\nu}{(\tau/\rho)^{0.5}}}\right) \tag{4.15}$$

If $U_* = (\tau/\rho)^{1/2}$, and assuming that $\rho = \gamma/g = 101.97\ \text{kgf}\cdot\text{s}^2/\text{m}^4$, eq 4.15 becomes

$$U = 0.2476\ \tau^{1/2}\ \text{Ln}\left(\frac{12.27\ R}{k_s + \frac{33.7\ \nu}{\tau^{1/2}}}\right) \tag{4.16}$$

If $\tau = \tau_c$, then, mean flow velocity is equal to the critical one, and it holds that

$$U_c = 0.2476\ \tau_c^{1/2}\ \text{Ln}\left(\frac{12.27\ R}{k_s + \frac{33.7\ \nu}{\tau_c^{1/2}}}\right) \tag{4.17}$$

With eq 4.17, mean critical velocity in a cross section can be calculated for whatever hydraulic condition of the wall: smooth, rough or in transition, as a function of critical shear stress. For well graded sediments, eq 4.17 permits the transport of finer particles until a certain degree of armoring is achieved.

Some interesting simplifications of eq 4.17 are:

a) If Laursen's critical shear stress value is accepted

$$\tau_c = 0.039\ (\gamma_s - \gamma)\ D \tag{4.18}$$

Substituting eq 4.18 in eq 4.17, and if it holds that $(\gamma_s - \gamma) = 1650\ \text{kgf/m}^3$ and $k_s = 2D_{84}$, then eq 4.17 takes the form

$$U_c = 2 D^{1/2} \; \mathrm{Ln} \left(\frac{6 R D_{50}^{1/2}}{D_{84} D_{50}^{1/2} + 2\nu} \right) \qquad (4.19)$$

Taking into account what was said in the preceeding paragraph, eq 4.19 works whatever the hydraulic condition of the wall is. D can be equal to D_{84} as already explained.

b) Other two important simplifications refer to hydraulically smooth or rough bottoms. In the first case, eq 4.19 is reduced to

$$U_c = 2 D_{50}^{1/2} \; \mathrm{Ln} \left(\frac{3 R D_{50}^{1/2}}{\nu} \right) \qquad (4.20)$$

and for rough bottoms, where viscosity is not important, eq 4.19 can be written as

$$U_c = 2 D_{50}^{1/2} \; \mathrm{Ln} \left(\frac{6 R}{D_{84}} \right) \qquad (4.21)$$

4.4 Critical bottom velocity for granular soils

To obtain the bottom flow velocity a particle can resist when starting to move the following relations can be used

$$U_b = 1.92 \, \Delta \, D^{0.45} \quad \text{when } D > 0.002 \text{ m} \qquad (4.22)$$

$$U_b = 0.41 \, \Delta \, D^{0.2} \quad \text{when } 0.0002 < D < 0.002 \text{ m} \qquad (4.23)$$

Eqs 4.22 and 4.23 were obtained through the analysis of other author's results —mainly Bogardi—, and afterwards adjusted to the results of eqs 4.2, 4.3 and 4.12.

4.5 Mean critical velocity for cohesive soils

To calculate the mean critical velocity at which erosion starts in cohesive soils, the application of Lischtvan-Levediev's method is recommended. They suggested the results of table 4.2 as a function of depth and dry bulk volumetric weight.

When the bed consists in cohesive soils, the expression that "movement starts" is no longer appropriate, it is better to refer to it as the condition of the flow capable of producing erosion or of wearing the soil, or to the ability to transport soil fragments.

4.6 Design of channels with no sediment transport

The value of the critical condition is not only useful in the design of channels without transport but also in scour calculation, especially from the theoretical point of view. Besides, some authors take it into account in their methods to evaluate sediment transportation. The reach of a river downstream and close to a big dam undergoes an erosive process because sediments are retained in the reservoir. The knowledge of the equilibrium condition and of its evolution in the process requires the application of the concepts discussed in this chaper and afterwards in 9.7. They are also useful in the design of river closures, where diversions start.

From what has been said in this chapter, two criteria or methods are applied in the design of channels and river beds without transport: while the first considers critical shear stress, the second implies the critical mean velocity of a cross section.

Both methods have different applications: since side or bank slope stability is considered in the shear stress criterion, this should be used in the design of soil channels. On the other hand, the simplicity of the critical mean velocity method makes it more suitable in case of wide sections, where some small side erosion is acceptable.

4.6.1 The critical shear stress method

To calculate the hydraulic section of a channel without sediment transportation, the shear stress caused by the current in the bed and sides τ_0 and τ_t, respectively, must be equal to the critical shear stress resisted by the material they are made of, τ_{c0} and τ_{ct}.

The following paragraphs describe the procedure recommended by the Bureau of Reclamation.

The starting data are: the materials the channel is made of, its side slope and the design dicharge.

1. A slope is selected for its banks or sides, and it has to be equal or smaller than the slope recommended in table 4.3. The angle formed by the side slope is designated as α, which is the minimum recommended for easy construction equal to 2:1.

2. The repose angle of the material is found. When it is granular, it is obtained from fig 4.4 and it is designated as ϕ. Necessarily, α must be smaller than ϕ. If more accuracy is desired, the angle of repose can be obtained experimentally.

3. A value of K is obtained. K is the ratio between the shear stress a particle resists on the side slope and the corresponding shear stress resisted at the bottom. K is written as

$$K = \left(1 - \frac{\operatorname{sen}^2 \alpha}{\operatorname{sen}^2 \phi}\right)^{1/2} \qquad (4.24)$$

For cohesive soils K = 1, because particle weight is reduced compared to cohesive force.

4. For channel material, critical shear stress is calculated applying the criteria shown in 4.2; it represents the shear stress a bed particle can resist before starting to move in an almost horizontal plane. It is expressed as τ_{co}.

5. The critical shear stress of the slope side material τ_{ct} is a function of τ_{co} and

$$\tau_{ct} = K \tau_{co} \qquad (4.25)$$

A side slope particle is less resistant to shear stress because its own weight tends to make it roll over the slope. That is why, in this case, shear stress has to be calculated separately.

6. The ratio b/d between the width b of the bottom and the depth d is assumed. Here the trial and error process starts.

7. The maximum shear stress produced by the current in the bed and slopes is obtained as a function of depth

$$\tau_o = \epsilon_o \gamma dS \qquad (4.26)$$

$$\tau_t = \epsilon_t \gamma dS \qquad (4.27)$$

where

ϵ_o and ϵ_t coefficients that take into account shear stress distribution, according to the bank slope of the cross section and when depth appears in the formulae instead of wetted perimeter. They are respectively obtained from figs 4.5 and 4.6, as a function of the cross section shape and of ratio b/d.

τ_o Maximum shear stress produced by the flow at the bottom, in kgf/m²

τ_t Maximum shear stress produced by the flow at the slope side, in kgf/m².

When in eqs 4.26 and 4.27 the known values are substituted, two expressions remain in function of d.

In wide channels, the shear stress in the bottom is γdS, considering coefficient $\epsilon_o = 1$ and the hydraulic radius equal to the depth.

8. Shear stress τ_{ct} and τ_t from steps 5 and 7 are made equal, and also τ_{co} and τ_o. Thus, two values of d result, the smallest of which is chosen.

   Normally, by using slope shear stress, the smallest depth can be obtained from the relation $\tau_{ct} = \tau_t$. When the channel is not well designed, there is erosion in the bank slope and the material moved accumulates at its toe in the form of a parabolic cross section.

9. When depth d is known, the width of the bottom is solved from b/d, according to step 6. Finally, d and b are known.

10. The geometry of the cross section is checked by means of a friction equation like, for instance, 3.13, to verify whether the design discharge passes through.

11. If the calculated and the designed discharges are not equal, a new value of b/d is chosen and steps 6 to 10 repeated. If both values are the same, we continue to step 12. Generally, it takes two trial and error processes to solve the problem.

12. A free board of 0.5 m is given and the dimensions of the cross section adjusted to make of them a practical value. If no doubt exists about the design flow, the recommended free board of 0.5 m is accepted. But when there are doubts about the maximum possible discharge or when it has great variations, a free board of at least 1.00 m is recommended.

When in the design of a channel side erosion must be prevented, it is better to apply the critical shear stress method, because that of the mean critical velocity —which will be later explained— does not contemplate the equilibrium condition of particles on slope banks.

4.6.2 The mean critical velocity method

This method tries to make equal the mean flow velocity and the maximum velocity particles can stand before beginning to move,

$$U_c \text{ (flow)} = U_c \text{ (particle)} \tag{4.28a}$$

$$U_{cf} = U_{cs} \tag{4.28b}$$

To calculate mean flow velocity, the following equation is recommended

$$U_{cf} = 2.5\, U_* \, \text{Ln} \left(\frac{12.27\, R}{2\, D_{84}} \right) \tag{4.29}$$

But eq 3.13 can also be used.

While the critical flow velocity of non cohesive soils is calculated by means of eq 4.12

$$U_{cs} = 6.05\, R^{0.15}\, D_m^{0.35} \tag{4.30}$$

When the transportation of smaller particles is allowed, eq 4.30 must be a function of D_{84} or D_{90} (see explanation of eq 4.12)

To value U_c for cohesive soils, table 4.2 —proposed by Lischtvan-Levediev— is more convenient.

To obtain the geometry of a channel without transport, these steps are followed

1. The shape of the section is chosen: trapezoidal, rectangular, etc. For trapezoidal ones, the slopes are selected according to 4.5.1. The longitudinal slope S of the channel, the design discharge and the characteristics of the bed and bank materials are known.

2. Eqs 4.29 and 4.30 are made equal while leaving an expression as a function of R, which is afterwards solved by trial and error. The resulting value is R.

3. R is substituted either in eqs 4.29 or 4.30 and the mean flow velocity U is obtained.

4. The cross section area A is obtained as $A = Q/U_c$.
Besides, A is a function of depth and width. If it were trapezoidal

$$A = (b + kd)\,d \tag{4.31}$$

where $k = \cot\alpha$, and α is the angle formed by the side slope and the horizontal line.

5. The value of the wetted perimeter is obtained: $P = A/R$, which can be written as a function of the section geometric characteristics. If it were trapezoidal

$$P = b + 2d\,(k^2 + 1)^{1/2} \tag{4.32}$$

6. Eqs 4.31 and 4.32 have two variables: b and d, which are found by solving them.

7. As in 4.5.1, a free board of 0.5 m is selected and the results adjusted to practical values. A cross section designed with this method provides equilibrium for bed particles, but not for those in the slope which result in small erosion.

4.7 Armoring of bed material

The material of the bed and banks of natural channels only very seldom presents the uniformity of those used in laboratory tests. These materials are generally well-graded or with extended granulometry, $\sigma_g > 3$. That is why it is necessary to determine the representative size of a material with extended granulometry which is not transported by a given current.

When a flow transports materials of extended granulometry it carries more fine particles than coarse ones; these form an armor coat on the bed upper layer which covers and protects the original material that rests below.

a) Lower point of incipient motion. It takes place when the flow shear stress is ready to start the motion of the finer particles. For this condition there is no erosion or lowering of the bed.

b) Upper point of incipient motion. It occurs when the flow shear stress is ready to move the particles of maximum diameter.

c) Maximum armoring. It takes place when the armor mean diameter is maximum. This condition is under the upper point; between this and the maximum armoring some big particles move and uncover part of the finer particles which are also transported. Hence, the maximum shear stress a material with extended granulometry can endure under stable conditions occurs when it reaches its maximum armoring. This is generically called critical shear stress.

Between the lower point of incipient motion and the maximum armoring there may exist many degrees of partial armoring. Starting at the lower point, when the shear stress grows, bigger particles are transported, increasing in this way the armour mean diameter until the maximum armoring is attained.

When studying the transport critical condition and once the bed material of a channel or river is known, two practical problems generally present. The first refers to obtaining the maximum discharge that can pass through a stable channel without transporting particles. To do this, the condition of maximum armoring, the armor coat granulometry and the maximum shear stress the flow can exert must be determined.

The second problem is to obtain the characteristics of the partial armorings, that is to say, to calculate the granulometry and the mean diameter of the armor formed in the bottom, given the discharge that flows through the section (less than the critical maximum).

The armoring characteristics are a function of the particle specific weight, their granulometric distribution, the fluid physical properties and the hydraulic characteristics of the flow. Besides, given the aleatory character of the turbulence, there will always exist a probability associated to every particle of forming part of the armor, which is higher for the coarser grains and lower for the finer.

Here a summary of Gessler's method is presented. It permits to find out the critical shear stress a material with extended granulometry can resist, the granulometry of the maximum armor coat, that of the transported material, and the characteristics of the partial armorings.

Later the contributions by Cruickshank-García and Maza-García will be presented; they facilitate the obtaintion of the maximum armor coat and of the critical shear stress.

4.7.1 Gessler's method

When considering the aleatory character of the turbulence in a current and when analyzing experimentally the transported material and that which forms the armor, Gessler obtained the distribution of probabilities. According to them, and given the mean shear stress of the flow, a specific size of grain is not transported by the flow becoming part of the armor. He also found out that that probability depended most of all on Shields' parameter τ_* and, in a much lesser degree, on the Reynolds number of particle $\mathbb{R}_p$. Thus, the well known Shields' curve means that the probability of the particle of being transported is of 0.5.

4.7.1.1 Armor material

The probability a particle has of not being transported is described as

$$q = p\left[\frac{\tau'_o}{\tau_o} < \frac{\tau_c}{\tau_o}\right] = f\left(\frac{\tau_c}{\tau_o}\right) \tag{4.33}$$

where

- τ'_o instantaneous shear stress on the bed over the particle
- τ_o mean shear stress on the bed
- τ_c critical shear stress necessary to move a particle of diameter D. It is obtained with Shields' criterion and accepting the mean value of the coefficient given by Meyer-Peter and Müller

$$\tau_c = 0.047 \ (\gamma_s - \gamma) \ D \tag{4.34}$$

The eq 4.33 may be also explained as the probability that the shear stress on the bed (which varies in aleatory form) is not greater than the critical value for the particle, that is

$$q = p\left[\frac{\tau_c}{\tau_o} \geqslant 1\right] \tag{4.35}$$

this means that if the relation τ_c/τ_o is greater than one, the particle does not move.

With the results of his experiences, Gessler drew fig 4.7, where the straight line equation has the relation τ_c/τ_o as the aleatory variable, which is expressed as

$$q = \frac{1}{\sigma\sqrt{2\pi}} \int_{-\infty}^{\tau_c/\tau_o} \exp\{-\frac{1}{2}(\frac{x-\mu}{\sigma})^2\}\, dx \tag{4.36}$$

where q is the probability a particle has, with a critical shear stress τ_c (according to eq 4.34), of not being transported by a flow with a shear stress τ_o. Besides, μ and σ are, respectively, the mean and the standard deviation of the probability distribution. Their values are $\mu = 1$ and $x = 0.57$. x is the aleatory variable.

Therefore, if $\tau_o = \tau_c$, the probability for a grain of size D to form part of the armor is 0.5 (fig 4.7). Besides, if $\tau_o = 0.5\,\tau_c$, the probability it has of staying in the bed is of 0.96; but if $\tau_o = 2\,\tau_c$, that probability diminishes to 0.21.

The aleatory variable x may be standardized by means of the transformation

$$Z = \frac{x-\mu}{\sigma} \tag{4.37}$$

or, according to Gessler's results

$$Z = \frac{x-1}{0.57} \tag{4.38}$$

The change indicated in eq 4.37 will entirely transform any aleatory variable x, to one with $\mu = 0$ and $\sigma = 1$.

When eq 4.36 is normalized, it takes the form

$$q = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^{Z_c} e^{-Z^2/2}\, dZ \tag{4.39}$$

where

$$Z_c = \frac{\frac{\tau_c}{\tau_o} - 1}{0.57} \tag{4.40}$$

If the symbol $N(\mu, \sigma)$ is used to denote a probability distribution of normal type, then N(0, 1) will be the notation which corresponds to the standard, unit normal, or the normally centered and reduced probability distribution. According to this and taking into account eq 4.37, a point in $N(\mu, \sigma)$ corresponds to a point $Z = (x-\mu)/\sigma$ in N(0, 1); then, because of analogy, a point τ_c/τ_o in N(1, 0.57), corresponds to a point $Z_c = ((\tau_c/\tau_o)/-1)/0.57$ in N(0, 1); therefore, if $\tau_c/\tau_o = 1$, then $Z_c = 0$, and the value of the probability is

$$q = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^{0} e^{-\frac{Z^2}{2}} dZ \qquad (4.41)$$

it corresponds to the value of q = 0.5, which is half the area under the normal standard curve. When proceeding in similar form it is obtained that if $\tau_c/\tau_o = 2$, the centered and reduced aleatory variable takes the value of Z = 1/0.57 = 1.7544; therefore

$$q = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^{1.7544} e^{-\frac{Z^2}{2}} dZ \simeq 0.96 \qquad (4.42)$$

this means that if $\tau_c = 2\,\tau_o$, a particle of size D has a high probability q = 0.96 of not being moved, while the probability of being transported is of (1−q) = 0.04.

Once the probability q is known, the granulometric curves of the armor coat material and of the transported material can be obtained as a function of the original granulometry; the size distribution of the original sample is given by

$$P_o(D) = \int_{D_{min}}^{D} p_o(y)\, dy \qquad (4.43)$$

where $P_o(D)$ is the probability distribution function (cumulative frequency distribution function) of the sizes of the original material; $p_o(y)$ is the probability density function (relative frequency function or histogram) of the particle size of the sediment, before being under the action of the flow. Variable y is the integration variable.

Besides, the fraction of the material smaller than D which forms the armor, is given by

$$\int_{D_{min}}^{D} q\, p_o(y)\, dy \qquad (4.44)$$

and the fraction of the material which forms the armor coat, smaller than D_{max}, is expressed as

$$\int_{D_{min}}^{D_{max}} q\, p_o(y)\, dy \tag{4.45}$$

therefore, the probability distribution function of the sizes of the armor coat material up to the size D

$$P_a(D) = \frac{\int_{D_{min}}^{D} q p_o(y)\, dy}{\int_{D_{min}}^{D_{max}} q\, p_o(y)\, dy} \tag{4.46}$$

Due to the fact that the original granulometric curve is divided in fractions ΔP_o (generally deciles or percentiles), the distribution function $P_a(D)$ may also be valued with those increments by means of the expression

$$P_a(D) = \frac{\sum_{D_{min}}^{D} q\, \Delta\, p_o}{\sum_{D_{min}}^{D_{max}} q\, \Delta\, p_o} \tag{4.47}$$

where $P_a(D)$ is the probability distribution function or cumulative frequency distribution function of the sizes of the armor material up to size D.

Therefore, the value of the probability or frequency of each fraction of the armor is

$$\Delta\, p_a = \frac{q\, \Delta\, p_o}{\sum_{D_{min}}^{D_{max}} q \Delta\, p_o} \tag{4.48}$$

In the former paragraphs subindex a refers to the armor coat material while subindex o to the original material.

4.7.1.2 Transported or moving material

Since q is the probability a particle of size D has a forming part of the armor coat, it holds that (1—q) is the probability that same grain has of being transported without forming part of that armor coat. Consequently, the probability distribution function of the sizes of the transported material is obtained by means of the expression

$$P_e(D) = \frac{\int_{D_{min}}^{D} (1-q)\, p_o(y)\, dy}{\int_{D_{min}}^{D_{max}} (1-q)\, p_o(y)\, dy} \tag{4.49}$$

Subindex e refers to the transported material which therefore is not part of the armor coat.

If the original granulometric curve is divided into fractions, $P_e(D)$ is expressed as

$$P_e(D) = \frac{\sum_{D_{min}}^{D} (1-q)\,\Delta p_o}{\sum_{D_{min}}^{D_{max}} (1-q)\,\Delta p_o} \tag{4.50}$$

and the probability for the mean grain of every fraction to be transported is

$$\Delta P_e = \frac{(1-q)\,\Delta p_o}{\sum_{D_{min}}^{D_{max}} (1-q)\,\Delta p_o} \tag{4.51}$$

4.7.2 Calculation sequence of the Gessler's method

As already said, when studying the incipient transport of bed material with extended granulometry, two very important problems present:

a) To obtain the critical shear stress of the original bed material, that is to say, the one that makes the armor coat mean diameter maximum, and the armor granulometry which fulfills that condition.

b) Given a current where the bed mean shear stress is smaller than the critical, to obtain the armor coat granulometry, its mean diameter and its critical representative diameter.

In the following paragraphs the steps to solve point b) are given for a known τ_o. Point a) is solved through successive iterations and following the same steps, increasing τ_o in each of them until obtaining the armor coat maximum mean diamter. To do this table 4.4 is filled, where its columns show what is explained afterwards.

The calculation starts by knowing $\tau_o = \gamma RS$ of the flow (problem b). If problem a) is to be solved, the computation can begin accepting Lane's criterion (see fig 4.1).

$$\tau_{o\ initial} = \tau_c = f(D_{75}) \tag{4.52}$$

The granulometric curve representative of the bed material is divided into increments or intervals Δp_o which, to facilitate the process, is better to make equal. If working with a computer, it is advisable to use percentiles, $\Delta p_o = 0.01$.

Column 1: i — relative cumulative frequency distribution (also cumulative percentage) of the original granulometric curve, which corresponds to every limit of the class intervals the granulometric curve has been divided into.

Column 2: Δp — relative frequency of every class interval the granulometric curve has been divided into. It is better if this value and those of the following columns, except 12 and 17, are written between the values of column 1 (table 4.4).

Column 3: p_i — relative cumulative frequency distribution of the original granulometric curve, which corresponds to every class mark or midpoint of every class interval.

Column 4: Z_n — standard random variable. The values of Z_n are obtained from table 2.3 according to the relative cumulative frequency (probability) of the class mark of each class interval; for instance, if the relative cumulative frequency of the first interval defines a probability of 0.05 (percentage of 5 %), by looking up in the table what value of Z_n satisfies a 5 percent probability, then it is obtained $Z_5 = -1.64485$. In other words, the lower figures of table 2.3 represent the density function of the unit normal distribution; if $Z_n = -1.64485$, the probability or the area under the density curve from $-\infty$ to Z_n, is 0.05. So if $p < 0.5$, Z_n is negative.

Column 5: D_i — diameter or representative size of the interval, in mm. It is obtained from the theoretical distribution the granulometric curve has been adjusted to using eqs 2.31 and 2.36. If that distribution does not exist, it is directly obtained from the granulometric curve and it is not necessary to fill column 4.

Column 6: τ_c — critical shear stress resisted by every particle, in kgf/m^2 or N/m^2. For conditions of turbulent flow it is calculated by means of eq 4.34

$\tau_c = 0.047\,(\gamma_s - \gamma)\,D_i$ $\quad \tau_c$ in kgf/m² (eq 4.34a)

$\tau_c = 4.523\,(\rho_s - \rho)\,D_i$ $\quad \tau_c$ in N/m² (eq 4.34b)

The calculation is done for every representative size, by substituting in eqs 4.34 the diameters of column 5, expressed in m.

Column 7: τ_c/τ_o — the indicated relation of shear stresses is calculated. The values in this column are obtained by dividing every value of τ_c in column 6 by the known or selected τ_o.

Column 8: Z — value of the abscissa of the unit normal distribution with mean $\mu = 0$ and standard deviation $\sigma = 1$. It is obtained from eq 4.40

$$Z = \frac{\dfrac{\tau_c}{\tau_o} - 1}{0.57} \qquad \text{(eq 4.40)}$$

This computation is done to determine the probability every representative diameter has of not being transported and of forming part of the armor coat (column 9).

Column 9: q — probability the particles of size D_i have of remaining in the armor coat. Every probability q is determined with table 2.3, according to the area under the normal standard curve which defines every corresponding value of Z. When the value of Z does not agree with that in the table, the probability q is determined through linear interpolation. q may also be obtained using fig 4.7, but it implies a loss of accuracy.

Column 10: $q\Delta p$ — frequency of the original material that becomes part of the armor. It is obtained by multiplying the class width Δp_o by the probability q which corresponds to the representative diameter of the class interval considered. The products are added to know

$$\sum_{D_{min}}^{D_{max}} q\Delta p_o \qquad \text{(from eq. 4.47)}$$

Column 11: Δp_a — relative frequency of the armor coat material or percentage of particles of size D_i which stays to form the armor. It is obtained through dividing every value in column 10 by its total sum (eq 4.47).

Column 12: P_a — distribution of relative cumulative frequencies (cumulative percentage that passes) of the armor coat material. It is obtained in the following way: in the first line of column 11 (i = 0) zero is written down and added to the first value in column 11; the result is written down in the fifth line of column 12, which corresponds to the second i. The amount obtained is added to the second amount of column 11, and the result is written down in the fifth line of column 12, which corresponds to the third i; and so on.

Column 13: $D_i \Delta p_a$ — the quantities that form this column are obtained by multiplying the diameters of column 4 by the corresponding values p_a in column 11. The addition of all these products gives the mean diameter of the granulometric distribution of the armor material, because, by definition

$$D_{a_m} = \sum_{i=0}^{n} D_{a_i} \Delta P_a \qquad \text{(eq 2.25)}$$

Thus the calculation of the characteristics of the armor coat formed with a given τ_o comes to an end.

If what is desired to find out is the maximum armoning, here finishes the iteration to determine whether D_{a_m} corresponds to the critical condition, that is, if the armor coat diameter does no longer increase.

The following iterations begin in column 6, supposing other values of τ_o and calculating the corresponding D_{a_m}. It is advisable to draw the supposed values of τ_o versus the calculated D_m, because this can show for what intervals of τ_o values D_{a_m} increases or diminishes. The process comes to an end when $(D_{a_m})_{max}$ is obtained and, with this, $\tau_c = \tau_{c_{max}}$ is also obtained.

Once the critical condition of maximum (problem a) or partial (problem b) armoring is determined, if the granulometric curve of the eroded material is to be known, it is necessary to go on with columns

Column 14: $1-q$ — probability the particles of size D_i have of being transported or of not forming part of the armor.

Column 15: $(1-q)\Delta p$ frequency of the original material which is eroded or transported

Column 16: Δp_e relative frequency of the transported material or percentage of particles of size D_i which is transported or washed by the flow.

Column 17: P_e relative cumulative frequency distribution (percentage cumulative distribution) of the transported material.

4.8 Simplifications to obtain the maximum armoring

It has already been seen that to know the maximum armoring of a material with extended granulometry or well graded sediment, or, what is the same, the maximum critical shear stress it can resist, the Gessler's method with successive iterations is used.

To make the process easier, Cruickshank and García made what was just explained for two theoretical size distributions of the bed material. Later, Maza and García carried on this idea analyzing with precision the log-normal distribution; they also obtained the critical shear stress resisted by the maximum armoring and its granulometry. Their results are stated below.

The maximum or critical shear stress a sediment with extended granulometry can endure is obtained from the expressions

1) $\tau_c = 0.028\,(\gamma_s - \gamma)\,D_{50}\,\sigma_g^{2.042}$ (4.53)

   applicable when $\sigma_g \geqslant 3$

2) $\tau_c = 0.0356\,(\gamma_s - \gamma)\,D_{50}\,\sigma_g^{1.823}$ (4.54)

   applicable when $1.7 \leqslant \sigma_g < 3$

where

- D_{50} diameter fifty of the original sample, in m
- σ_g geometrical standard deviation of the original sample
- τ_c critical shear stress of the maximum armor coat

From the $\tau_{c_{max}}$ the representative diameter of the original material that produces the critical shear stress can be calculated. It is obtained with Shield's expression

$$D_r = \frac{\tau_c}{0.047\,(\gamma_s - \gamma)} \qquad (4.55)$$

For instance, if the distribution is log-normal and $\sigma_g = 4$; D_r has a value almost equal to the D_{95} of the original sample.

Due to the fact that the matters already mentioned are theoretical, when designing canals o when studying the maximum armoring in rivers, it is advisable to consider as $\tau_{c_{max}}$ of design, the theoretical one obtained from eqs 4.53 and 4.54 divided by 1.50, because of the uncertainties related with the determination of the granulometric curve of the original material. Then it holds that

$$\tau_{design} = \frac{\tau_{c_{max}}}{1.5} \qquad (4.56)$$

Besides, the granulometric curve of the maximum theoretic armor can be calculated in a simple, though approximate way, with the help of the following expressions

$$(D_{84})_a = 0.723\, D_{50}\, \sigma_g^{2.32} \qquad (4.57)$$

$$(D_{70})_a = 0.674\, D_{50}\, \sigma_g^{2.03} \qquad (4.58)$$

$$(D_{50})_a = 0.676\, D_{50}\, \sigma_g^{1.5} \qquad (4.59)$$

$$(D_{30})_a = 0.757\, D_{50}\, \sigma_g^{0.695} \qquad (4.60)$$

$$(D_{110})_a = 0.858\, D_{50}\, \sigma_g^{-0.086} \qquad (4.61)$$

The subindex a which affects the parenthesis refers to the armor coat particles. D_{50} and σ_g refer to the original sample.

With eqs 4.53 to 4.61 the engineer can calculate in a simpler way the characteristics of the maximum armor coat when the original granulometry may be adjusted to a log-normal distribution. When this is not possible, he will have to apply Gessler's method.

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TABLE 4.1 NON ERODIBLE MEAN VELOCITY FOR NON COHESIVE MATERIALS, in m/s; AFTER LISCHTVAN-LEVEDIEV

Mean diameter, mm	Mean depth of the flow, in m					
	0.40	1.00	2.00	3.00	5.00	más de 10
0.005	0.15	0.20	0.25	0.30	0.40	0.45
0.05	0.20	0.30	0.40	0.45	0.55	0.65
0.25	0.35	0.45	0.55	0.60	0.70	0.80
1.0	0.50	0.60	0.80	0.75	0.85	0.95
2.5	0.65	0.75	0.80	0.90	1.00	1.20
5	0.80	0.85	1.00	1.10	1.20	1.50
10	0.90	1.05	1.15	1.30	1.45	1.75
15	1.10	1.20	1.35	1.50	1.65	2.00
25	1.25	1.45	1.65	1.85	2.00	2.30
40	1.50	1.85	2.10	2.30	2.45	2.70
75	2.00	2.40	2.75	3.10	3.30	3.60
100	2.45	2.80	3.20	3.50	3.80	4.20
150	3.00	3.35	3.75	4.10	4.40	4.50
200	3.50	3.80	4.30	4.65	5.00	5.40
300	3.85	4.35	4.70	4.90	5.50	5.90
400		4.75	4.95	5.30	5.60	6.00
Above 500			5.35	5.50	6.00	6.20

TABLE 4.2 NON ERODIBLE MEAN VELOCITY FOR COHESIVE MATERIAL, in m/s; AFTER LISCHTVAN-LEVEDIEV

Soil	Percentage of particle contents		$\gamma_{\psi} \leq 1200$ kgf/m^3				$1200 < \gamma_{\psi}$ 1660 kgf/m^3				$1660 < \gamma_{\psi}$ 2040 kgf/m^3				$2040 < \gamma_{\psi} \leq 2140$ kgf/m^3			
			Mean depth, in m															
	<0.005	0.005-0.05	0.4	1.0	2.0	3.0	0.4	1.0	2.0	3.0	0.4	1.0	2.0	3.0	0.4	1.0	2.0	3.0
Clays Heavy clayed soils	30-50 20-30	70-50 80-70	0.35	0.4	0.45	0.5	0.7	0.85	0.95	1.1	1.0	1.2	1.4	1.5	1.4	1.7	1.9	2.1
Lean clayed soils	10-20	90-80	0.35	0.4	0.45	0.5	0.65	0.8	0.9	1.0	0.95	1.2	1.4	1.5	1.4	1.7	1.9	2.1
Alluvial silts							0.6	0.7	0.8	0.85	0.8	1.0	1.2	1.3	1.1	1.3	1.5	1.7
Sandy soils	5-10	20-40	According to Table I.2a.															

TABLE 4.3 SUITABLE SIDE SLOPES FOR CHANNELS BUILT IN DIFFERENT KINDS OF MATERIALS

Material	Side slope
Rock	Nearly vertical
Muck and peat soils	¼:1
Stiff clay or earth with concrete lining	½:1 to 1:1
Earth with stone lining, or earth for large channels	1:1
Firm clay or earth for small ditches	1½:1
Loose sandy earth	1:1
Sandy loam or porous clay	3:1

TABLA 4.4 CALCULATION SEQUENCE FOR THE DETERMINATION OF τ_c . GESSLER´S METHOD

τ_o = kgf/m²

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
i	Δp	$\bar{p}_i$	z_n	D_i en mm	τ_c, en kgf/m²	$\frac{\tau_c}{\tau_o}$	z	q	$q\Delta p$	Δp_a	p_a	$D_i \Delta p_a$, en mm	1−q	$(1-q)\Delta p$	Δp_e	p_e
0											0.00000					0.0000
	0.10	0.05	−1.645	0.20	0.016	0.017	−1.725	0.042	0.0042	0.0239	0.01195	0.0049	0.958	0.0958	0.1163	0.0582
0.10											0.239					0.1163
	0.10	0.15	−1.036	0.48	0.037	0.040	−1.685	0.046	0.0046	0.0260	0.0369	0.0124	0.954	0.0954	0.1159	0.1743
0.20											0.0499					0.2322
	0.10	0.25	−0.674	0.79	0.061	0.065	−1.640	0.051	0.0051	0.0286	0.0642	0.0224	0.949	0.0949	0.1153	0.2899
0.30											0.0785					0.3475
	0.10	0.35	−0.385	1.17	0.091	0.098	−1.583	0.057	0.0057	0.0321	0.0945	0.0376	0.943	0.0943	0.1146	0.4048
0.40											0.1160					0.4621
	0.10	0.45	−0.126	1.68	0.130	0.140	−1.509	0.066	0.0066	0.0372	0.1292	0.0625	0.934	0.0934	0.1135	0.5188
0.50											0.1478					0.5756
	0.10	0.55	0.126	2.38	0.185	0.198	−1.406	0.080	0.0080	0.0452	0.1704	0.1076	0.920	0.0920	0.1118	0.6315
0.60											0.1930					0.6873
	0.10	0.65	0.385	3.41	0.265	0.284	−1.256	0.105	0.0105	0.0592	0.2226	0.2020	0.895	0.0895	0.1088	0.7417
0.70											0.2522					0.7961
	0.10	0.75	0.674	5.09	0.395	0.424	−1.010	0.156	0.0156	0.0884	0.2964	0.4504	0.844	0.0844	0.1025	0.8473
0.80											0.3406					0.8986
	0.10	0.85	1.036	8.41	0.653	0.701	−0.525	0.300	0.0300	0.1697	0.4255	1.4279	0.700	0.0700	0.0850	0.9411
0.90											0.5103					0.9836
	0.10	0.95	1.645	19.56	1.517	1.629	1.104	0.865	0.0865	0.4896	0.7552	9.5757	0.135	0.0135	0.0164	0.9918
1.00											1.0000					1.0000

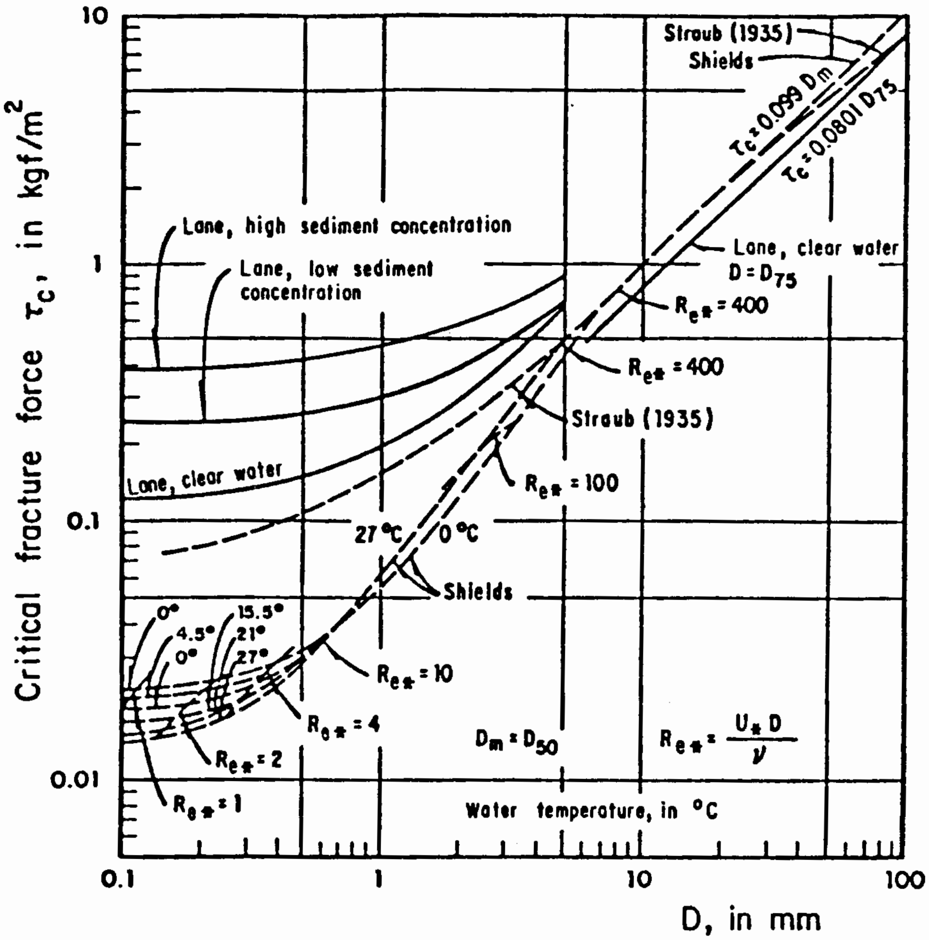

Fig 4.1

Critical shear stress as a funtion of grain diameter

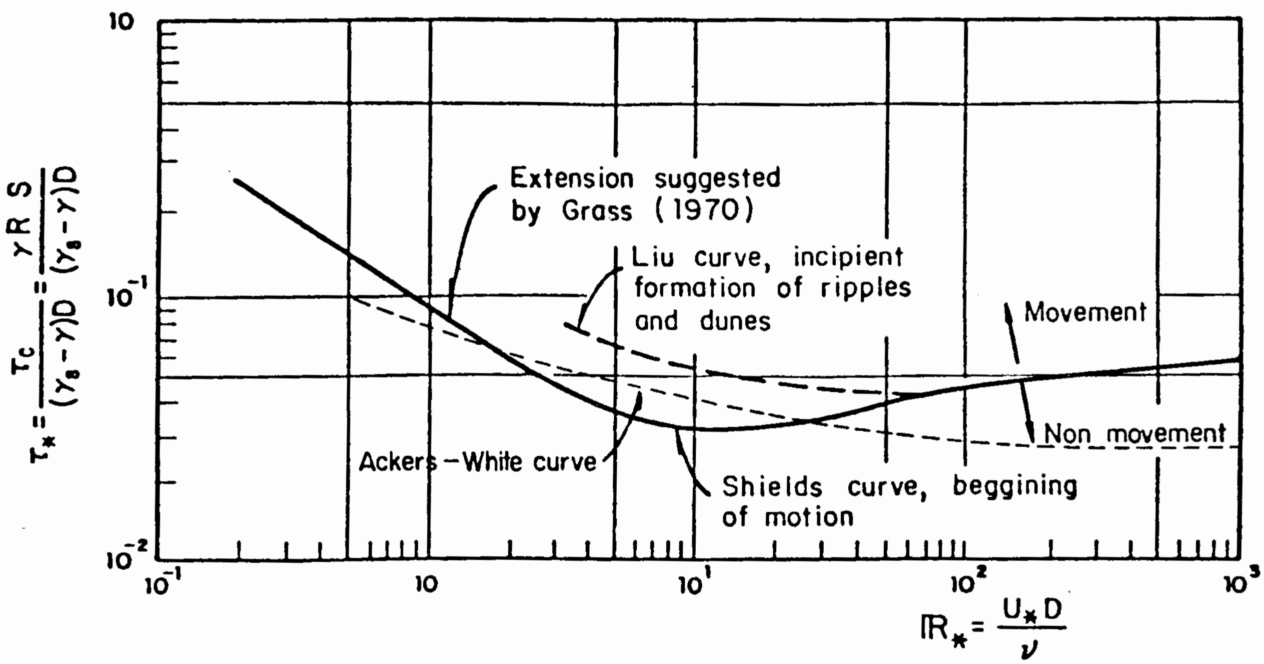

Fig 4.2

Shields diagram to obtain incipient motion of sediment

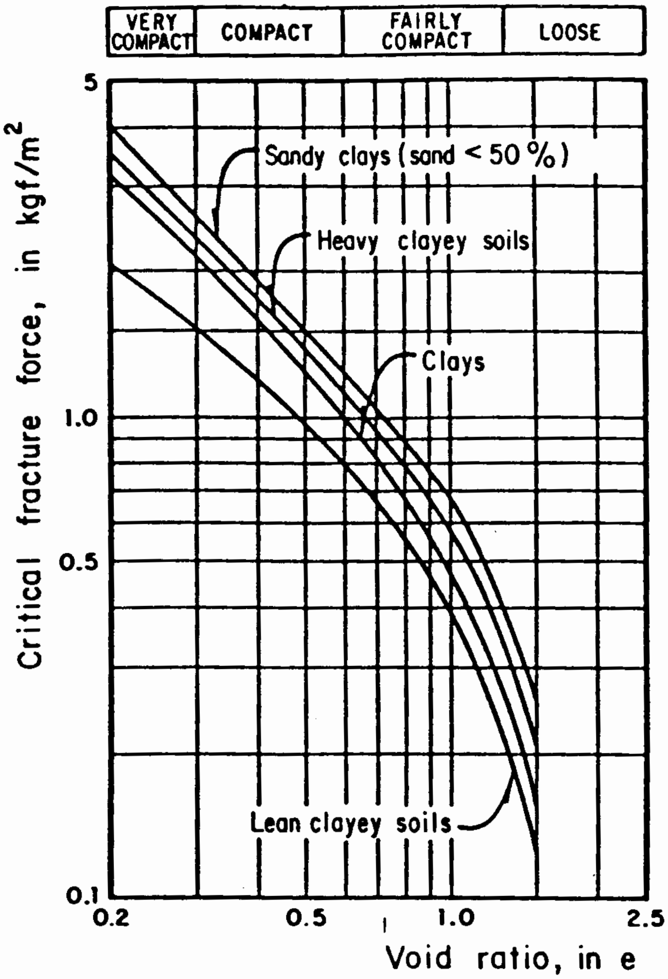

Fig 4.3
Critical shear stress for canals in cohesive soil

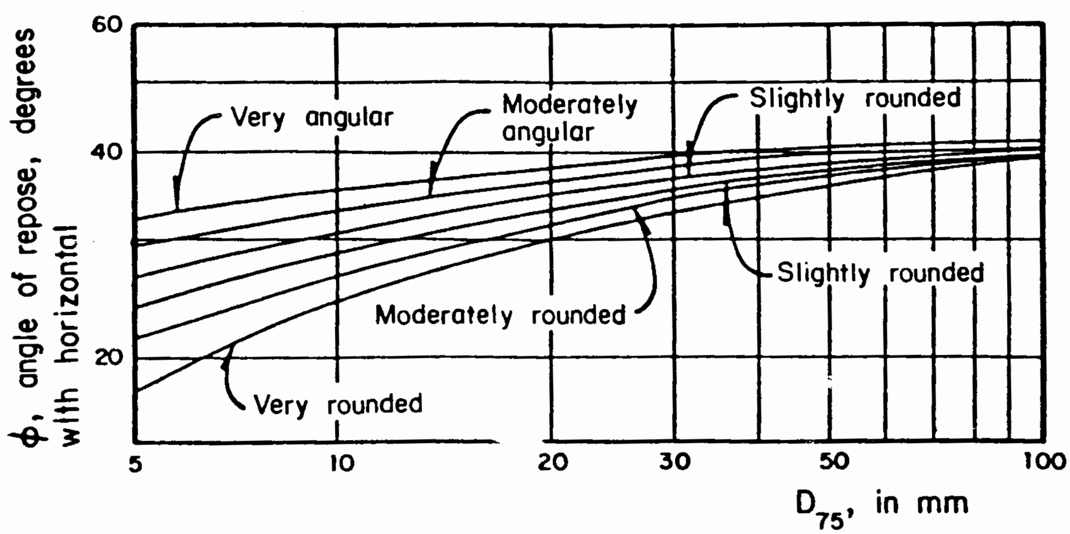

Fig 4.4
Angle of repose of noncohesive materials

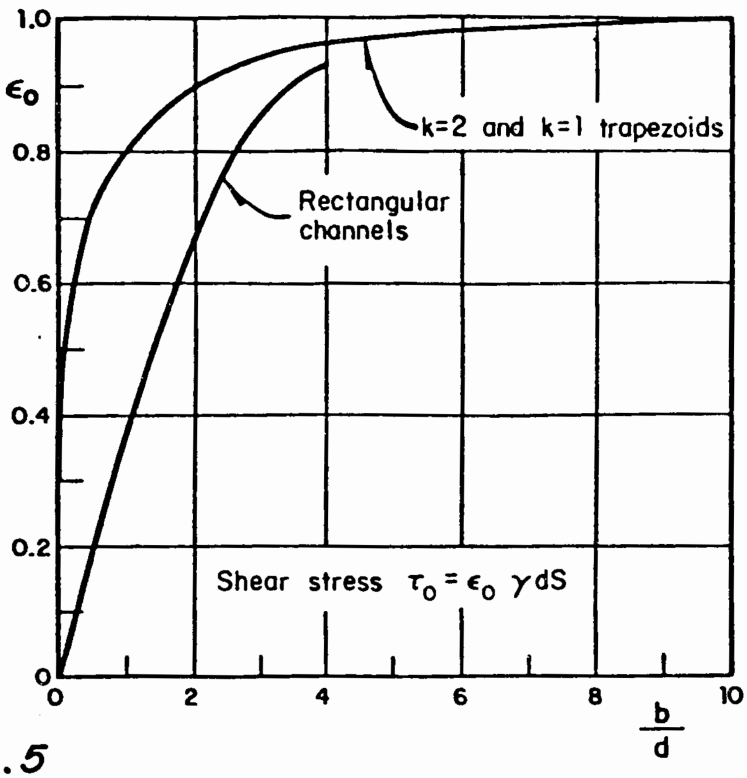

Fig 4.5
Coefficient ϵ_0, on bottom of channels in terms of b/b

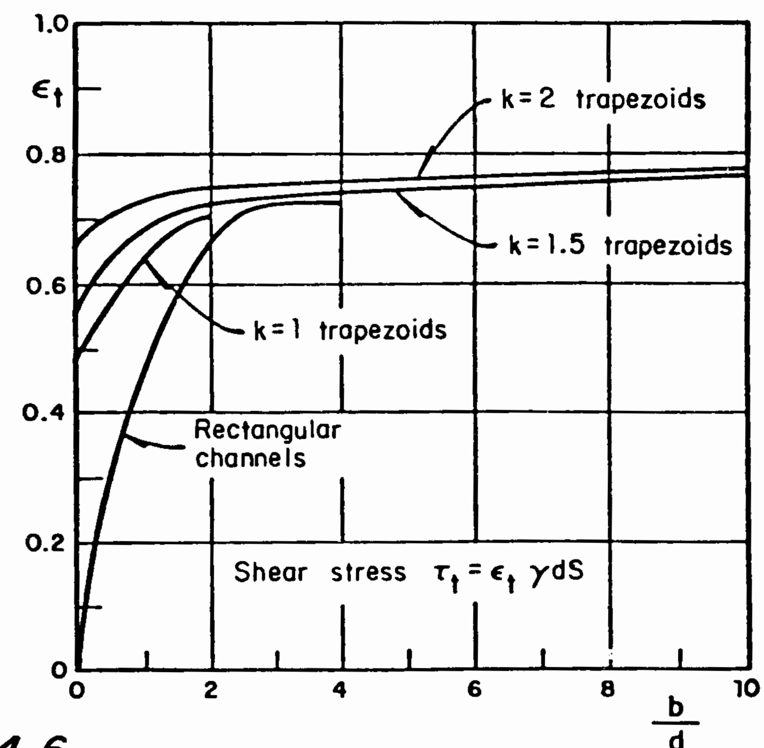

Fig 4.6
Coefficient ϵ_t, on side of channels, in terms of b/b

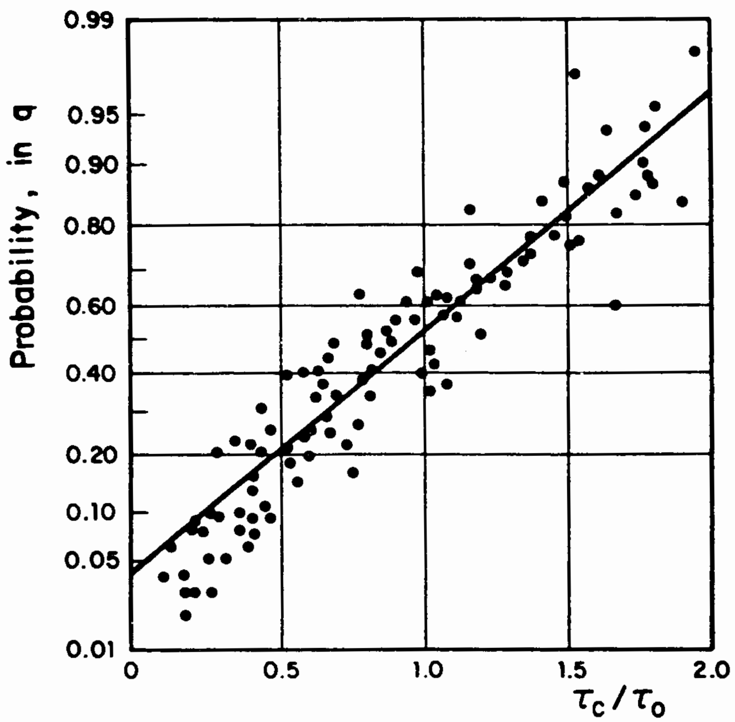

Fig 4.7

Probability for a sediment particle to be part of the armour coat

CHAPTER 5

SEDIMENT TRANSPORTATION

5.1 Definition and concepts

Sediments are all the basin rock and soil particles a fluid carries away sliding, rolling or jumping in suspension or over the bed. Sediments liable to be transported are those from the beds and banks of rivers and the particles from the watershed.

When fluid velocity is low, granular particles generally roll or slide over themselves. However, at high velocities, sands and even gravels can be also transported in suspension.

Very fine particles move in suspension. The finer the particle or the stronger the flow turbulence, the greater the transport in suspension. But the discharges of bed suspended sediments stop when flow velocity decreases.

In a river bed covered by dunes, vortexes form downstream of each of them and spread to the surface carrying in suspension big quantities of bottom particles which, at surface, look like bubbles. When finally the vortexes disappear or loose intensity after 20 to 60 seconds, particles fall back to the bottom.

When in suspension, the highest particle concentration takes place within these vortexes and this is why samples should not be taken from them.

According to their behaviour when transported by a flow, sediments may be divided in two big groups: bed load, which forms in the bottom and, the other, which corresponds to the wash load. This name is given to the sediments formed by very fine particles, especially silts and clays, which are easily kept in suspension.

The principal difference between the behaviour of bed sediment transport and that of wash load lies in the hydraulic characteristics of the flow and the physical characteristics of the material. Therefore, similar stretches with identical bed material of two different streams will transport the same quantities of bed sediment under similar hydraulic conditions. In other words, it is considered that in a river stretch and under the same hydraulic conditions, bed transport will always be the same. However, this is not so with wash load: generally, a river can transport all the wash load that comes to it, independently of the river hydraulic conditions. Thus, if there are two rivers with the same bed sediments, but one with a protected or forested basin, and the other unprotected, with high slopes and loose material on the surface, they will carry quite different amounts of wash load, up to the point that while the first may transport none, the second can present conditions of 300 kgf/m^3 or more.

Wash load is the name given to all the fine materials which are not represented in the bed. When their diameters are not known, the limit between them is established at 0.062 mm, so that wash load transport is made of all those particles with less than 0.062 mm of diameter.

Bed sediment transport may either take place over the bottom as bed load or in suspension, while wash load transport is always in suspension. Due to the fact that wash load consists of very fine particles, it is considered to be uniformly distributed over the river cross section.

Unitary sediment transport may be expressed either by weight or volume. In the first case, it is designated with the letter g, being its units kgf/s·m; if by volume, with letter q and m^3/s·m. q represents the volume occuppied by all solid particles without considering voids among them. The relation between g and q is given by

$$g = \gamma_s q \qquad (5.1)$$

There are times when the submerged weight of the sediment is also used, and it is considered as g'. Its relation with g and q is

$$g' = \left(\frac{\gamma_s - \gamma}{\gamma_s} \right) g \qquad (5.2)$$

and

$$g' = (\gamma_s - \gamma) q \qquad (5.3)$$

where

γ_s specific gravity of particles, in kgf/m³

γ specific gravity of water, in kgf/m³

G and Q represent the transports which take place all over the cross section of the river, expressed either by weight (kgf/s) or volume (m³ /s), respectively.

In order to simplify sediment transport calculation, to understand the data needed for its quantification and to classify the related criteria and formulae, we distinguish six types of transportation (see fig 5.1)

a) Bed load transport or discharge, g_B

b) Bed suspended sediment discharge, g_{BS}

c) Bed sediment discharge or total bed sediment discharge, g_{BT}

d) Wash load transport or discharge, g_L

e) Suspended sediment discharge, g_S

f) Total sediment discharge, g_T

a) The *bed load transport or bed load discharge* is the material of the bottom which is carried by the current in a layer adjacent to the bottom with a width equal to twice the diameter of the particle considered. This transport is represented by subindex B.

b) The *bed suspended sediment discharge* is the bed material which is transported in suspension by the flow in the fluid, over the bed load. Due to its velocity and turbulence, the liquid raises the particles and keeps them in suspension until they fall again to the bottom when both velocity and turbulence decrease. This transport is represented by subindex BS

In most streams bed material is granular because it consists of loose sand particles, gravel and cobbles. The forces that try to move them are those of drag and lift the flow exerts on them. The forces which oppose to movement are particle weight and the friction particles provoke when resting on other particles. When raised and put in suspension, the finer the material and the greater the turbulence, the more uniform the vertical distribution of these materials is. When the material is coarse or the turbulence not so great, there is not much suspended material on the water surface and the concentrations close to the bottom are higher.

c) The *bed sediment discharge or total bed sediment discharge* is formed by the bed material transported by the flow either on the bed layer or in suspension. Therefore, bed sediment transport is equal to the sum of the bed load transport and the bed suspended sediment transport. It is represented by subindex BT

$$g_{BT} = g_b + g_{BS} \tag{5.4}$$

d) The *wash load discharge* is the very fine material the current transports in suspension and which is not represented in the bed. When considering a section, it has to be remembered that all the wash load comes from upstream. It originates in the soils eroded by the effects of the rain or, sometimes, it is the result of the erosion the stream produces on the banks. It is represented by subindex L.

e) The *suspended sediment transport* is constituted by all the particles kept in suspension as results of the turbulence or of their low weight. Therefore, suspended transport is equal to bed suspended sediment transport plus wash load, and it is represented by subindex S. Thus

$$g_S = g_{BS} + g_L \tag{5.5}$$

A water sample taken from a natural stream is always representative of its concentration in relation to suspended transport, because it includes wash load and particles from the bed.

A sample with only wash load material may be obtained in a zone of a river with very low velocities, because sand or bigger particles deposit very quickly. A sample with only bed suspended material, that is to say, without wash load, can be obtained in a laboratory channel where clean water is used. In nature, it is sometimes found in tributary affluentes of mountain areas and in the channels which communicate coastal lagoons with the sea, because when the tide rises and the sea water enters, water is generally free of fine material.

f) The *total transport* includes all the particles transported by the river which come from the river bed or from the watershed. It is represented by subindex T.

According to what was said, the following relations can be expressed

$$g_T = g_L + g_{BT} \tag{5.6}$$

$$g_T = g_L + g_{BS} + g_B \tag{5.7}$$

$$g_T = g_S + g_B \tag{5.8}$$

Most of the first methods developed to quantify sediment transport tried to obtain the material carried over the bottom; however, the tests were made in laboratory channels and sediment transport was known by measuring all the transported particles, even those in suspension, up to the end of the channel. That is why, when flow velocities were high, the transport obtained corresponded to the bottom (total bed sediment transport) and not really and exclusively to the bed layer.

In 1950, Einstein introduced the bed layer concept by dividing the bed load from the bed suspended transport; however, he took into account other of his results which quantified the sum of both loads and not bed load transport independently. The concept of the bed layer is a rather confused one and not very useful when the bottom consists of well graded material, because if the bed layer is twice the particle diameter, this can give a value of up to 10 cm for a big particle; besides, the layer width of fine sand in a same sample can only give half a millimeter, value which corresponds to an other bed layer.

5.1.1 Methods for sediment transport quantification

There is a large number of methods to quantify sediment transport which only work for some of the transport conditions indicated in the previous chapter. However, the most complete give total transport, a value that can also be obtained by applying the methods which calculate the partial transports of eqs 5.6 to 5.8. However, not all problems need the total transport value for its solution.

Here we present the transports which have to be calculated in order to solve some of the practical problems the fluvial hydraulics specialists normally encounter.

Problem	Intervening sediment transport
Time needed to fill with sediments a very low diversion dam	g_B
Time needed to fill with sediments a normal diversion dam	g_{BT}
Determination of a dam dead capacity	g_{BT} or g_T
Design of direct intakes to channels	g_{BS} or g_S
Design of pumping plants	g_{BS} or g_{BT}
River rectifications	g_{BT}
Deviation to lagoons for aquaculture	g_L
Treatment plants for drinking water	g_L
Dredging studies	g_{BT}
Scour studies downstream of big dams, etc.	g_{BT}

These methods have been classified in six groups according to the type of transport they evaluate and to the data needed for their application

I. Methods to quantify bed load transport

II. Methods to quantify bed sediment transport g_{BT}, without distinguishing bed load transport from bed suspended sediment transport

III. Methods to quantify suspended transport g_S, where bed suspended transport g_{BS} and wash load g_L transport may be distinguished.

IV and V. Methods for the quantification of the bed sediment transport g_{BT}, separating what is carried on the bottom layer g_B from what is transported in suspension g_{BS}. Two groups may be distinguished in this classification: IV) that which requires the additional datum of the suspended material concentration at least in one point of the vertical, and V) the one which does not need this datum because bed sediment transport is calculated from bed material characteristics.

VI. Methods for the quantification of the total transport g_T, which separate its three components g_B , g_{BS} and g_L.

While to explain the methods grouped in I, II and V, bed material physical characteristics must be known, groups III and IV only require the suspended material and its concentration at a given depth. On the other hand, some methods of group VI include both the datum of the bed material and that in suspension with its mean concentration between the surface and a given depth. Besides, others only need the bed material characteristics and the hydraulic and geometric ones of the channel.

First, some methods of groups I, II and V are shown, and afterwards those of groups III and IV, all of them with their formulae and coefficient values. Also mentioned are their limits of application and the conditions under which they were obtained.

5.1.2 Data required

For their application, groups I, II and V need the following data

1. Granulometric curve of the bed material. Every researcher establishes the diameter which he considers representative of the distribution. The most usual are mean diameters D_m and D_{50}, even if others prefer D_{35}, D_{65} and D_{90}.

2. Bed particle specific weight.

3. River cross section from which the hydraulic radius, the wetted perimeter and the hydraulic area are obtained in relation to the elevation considered.

4. Liquid discharge, which is not necessary to know explicitly.

5. Water elevation at which sediment transport is to be known. For each elevation,

mean flow velocity and discharge can be calculated. If one of these is known, the remaining calculations are more precise and some coefficients may be calibrated.

6. Mean hydraulic slope along the stream.

7. Water temperature.

Besides the values already mentioned, when applying the methods of groups III and IV the following are also needed

8. Concentration of suspended material at a point where the distance from the bottom is also known.

With the preceeding data, some of the following parameters must be calculated:

9. Mean velocity of the current and the discharge associated to water elevation.

10. Mean critical velocity of the current which starts sediment transport or critical discharge, critical depth and critical shear stress. (The term *critical* is associated with the beginning of transport and not with the condition of minimum energy.).

11. Bed particle fall velocity.

12. Stream roughness, both the total and the one associated with particles and ondulations.

Here, at least one method for each of the first five groups (I to V) indicated in 5.1.1 is shown.

5.2 Methods for bed load transport quantification

There is a considerable number of methods to value bed load transport. This process takes place in a layer adjacent to the bottom, being its width equal to twice the particle diameter. Among others, the following methods can be cited: DuBoys (1879), afterwards complemented by Straub (1935), Schocklitsh (1914, 1950), Shields (1936), Meyer-Peter and Müller (1948), Kalinske (1947), Einstein (1942), Einstein-Brown (1950), Levi (1948), Sato-Kikawa and Ashida (1958), Rottner (1958), Garde and

Albertson (1961), Frijlin (1962), Yalin (1963), Pernecker and Voelmer (1965), Ingles-Lacey (1968), Bogardi (1947), etc. Most of these methods are totally empiric.

The methods for the obtaintion of bed load transport, which at the same time may be applied to obtain suspended transport, have been gathered in group V and are presented in 5.6.

While Shields' parameter is $\tau_* < 1$, with many of these methods it is possible to attein values within the same order of magnitude. But this is not so when $\tau_* > 1.5$: some methods give results too big and similar to those obtained with the formulae of group II because they probably give bed sediment transport and not only bed load transport. This is why the one recommended is that by Meyer-Peter and Müller.

5.2.1 Meyer-Peter and Müller's method

Developed from 1932 to 1948 in the Hydraulic Laboratory at the Ecole Technique Supérieure in Zurich, a summary of all the work was presented in 1948.

The authors did four series of tests, at the end of which they proposed a formula for each of them. But due to the fact that the last includes all the previous results, it has a general character. These tests were made for granular material with the following characteristics

1. Particles with uniform diameter and specific weight of 2860 kgf/m^3.

2. Particles with uniform diameter but different specific weight: baryta, 4220 kgf/m^3; gravel, 2680 kgf/m^3 and lignite, 1250 kgf/m^3.

3. Mixture of different diameter particles, that is to say, with more or less graded granulometries and γ_s = 2680 kgf/m^3.

4. Equal to 3, but with tests for baryta and lignite of different sizes.

The resulting general formula, which can be applied both for materials with different specific weight or with uniform or not uniform diameters, is as follows

$$\left(\frac{n'}{n}\right)^{3/2} \frac{\gamma R S}{(\gamma_s - \gamma) D_m} = 0.047 + \frac{0.25}{D_m (\gamma_s - \gamma)^{1/3}} \left(\frac{\gamma}{g}\right)^{1/3} \left(\frac{g_B}{\gamma_s}\right)^{2/3} \qquad (5.9)$$

where

n total bed roughness. It is obtained from Manning's formula

n' roughness due to particles. It is found using the Meyer-Peter and Müller's expression

$$n' = \frac{D_{90}^{1/3}}{26} \qquad D_{90}, \text{ in m}$$

g_B unit bed load discharge in kgf/s·m

For eq 5.9, g_B is obtained; it is expressed in explicit form as

$$g_B = 8\,\gamma_s (g\,\Delta\,D_m^3)^{1/2}\,[(\frac{n'}{n})^{3/2}\,\tau_* - 0.047]^{3/2} \tag{5.10}$$

where

$$\tau_* \quad \frac{\gamma\,R\,S}{(\gamma_s - \gamma)} = \frac{R\,S}{\Delta\,D_m} = \frac{\tau_0}{(\gamma_s - \gamma) D_m} \tag{5.11}$$

It can be observed that when in eq 5.10 τ_* is much greater than 0.047 (which occurs when flow velocities are high), the sediment discharge does not depend on particle diameter. Thus, the above mentioned equation reduces to

$$g_B = \frac{8\,\gamma\,g^{1/2}}{\Delta}\,(\frac{n'}{n})^{9/4}\,R^{2/3}\,S^{1/2} = \frac{8\,\gamma_s\,U_*\,R\,S}{\Delta}(\frac{n'}{n})^{9/4}$$

Particle size intervenes in an implicit way in the ratio (n'/n) with a value that varies between 0.5 and 1.0; 0.5 corresponds to sandy beds with dunes and 1.0, to plane beds. For natural beds, it generally ranges between 0.6 and 0.8.

Besides, in the same eq 5.10 it can be observed that if transport tends to zero, when becoming null it holds that

$$\tau_* = 0.047 \tag{5.12a}$$

what can be also expressed as

$$\tau_c = 0.047\,(\gamma_s - \gamma)\,D_m \tag{5.12b}$$

where τ_c is the critical shear stress.

Eq 5.12 holds for the critical condition or the beginning of particle motion.

In the tests done by Meyer-Peter and Müller the variables that intervene in their formulae varied among the following values

	From	To
Particle size	0.0004	0.03 m
Specific weight	1250, 2800 and 4220 kgf/m^3	
Hydraulic slope	0.0004	0.02
Depth	0.01	1.20 m
Discharge	0.002	4.0 m^3/s

Eq 5.10 is the most widely used in engineering problems and it is mainly applied in case of coarse material but also of sandy rivers.

5.2.2 Einstein and Bagnold's methods

With the methods by these authors (which are shown in point 5.5), total bed transport is obtained by separating bed load transport from bed suspended transport. Bed suspended transport can be quantified with eq 5.48 and the following, while bed load transport is valued using eq 5.58.

5.3 Methods for the quantification of bed sediment transport g_{BT}

Among the methods proposed to obtain bed sediment transport without separating the part transported in suspension from the one transported in the bed, the following can be mentioned: Laursen (1958), Colby (1964), Engelund (1967), Graf-Acaroglu (1968), Shen-Hung (1971), Bishop, Simons and Richardson (1965), Carstens and Altinbilek (1972), Yang (1973), Ackers and White (1972-1973) and Ranga-Garde and Bhardwaj (1981).

This work presents those by Engelund and Graf-Acaroglu because they yield proper results. Both methods have the same disadvantage which consists in giving sediment transport even for conditions below the critical. Therefore, they should not be applied if

$$\tau_* = \frac{R\,S}{\Delta\,D_m} < 0.047$$

The second disadvantage is that within the range

$$0.047 < \tau_* < 0.1$$

the transport obtained from their application is greater than the real one, because the total considers transport as bed load. This is why it is advisable to use Meyer-Peter and Müller's formula within that interval.

5.3.1 Engelund's method

In 1937, Engelund developed his transport equation based on the data of four sets of experiments done in sands, in a channel 2.44 m wide and 45.72 m long. In these tests the sediments had, respectively, a mean diameter of 0.19 mm, 0.27 mm, 0.45 mm and 0.93 mm, being the standard deviation of 1.3 for the finer sediments and of 1.6 for the rest. These results were reported by Guy in 1966.

The relation proposed by Engelund is

$$f_e\,\phi_E = 0.1\,\tau_*^{5/2} \tag{5.14}$$

where

f_e Engelund's friction coefficient which is equal to

$$f_e = \frac{2\,\tau_0}{\rho\,U^2} = \frac{2\,U_*^2}{U^2} \tag{5.15}$$

ϕ_E Einstein's non dimensional parameter which takes into account sediment transport

$$\phi_E = \frac{g_{B\,T}}{\gamma_s\,(g\,\Delta\,D_{50}^3)^{1/2}} \tag{5.16}$$

Substituting eqs 5.15, 5.16 and τ_* for their value in eq 5.14 and then simplifying, it is obtained

$$g_{BT} = \frac{0.05\,\gamma_s\,U^2\,\tau_0^{3/2}\,\gamma^{1/2}}{g^{1/2}\,(\gamma_s - \gamma)^2\,D_{50}}, \text{ in kgf/s·m} \tag{5.17}$$

If eq 5.17 were expressed in function of D_{35} instead of D_{50}, since $\tau_0 = \gamma$ dS, it could be written as

$$g_{BT} = \frac{0.04\,\gamma_s\,U^2\,(dS)^{3/2}}{\Delta^2\,g^{1/2}\,D_{35}}, \text{ in kgf/s·m} \tag{5.18}$$

5.3.1.1 Comments and limits of application

Eq 5.18 was drawn in 1967 with Hansen's help (see fig 5.2), however, Engelund-Hansen's graphic is only valid when there are dunes in the bottom and when the Reynold's number associated with velocity, shear stress and diameter 50 is more than 12, that is

$$\mathrm{IR}_* = \frac{U_*\,D_{50}}{\nu} = 12 \tag{5.19}$$

Engelund and Hansen recommend its application for sands, but not if D_{50} is smaller than 0.15 mm or if the diameter standard deviation is more than 2 ($\sigma_g > 2$).

The equation may be used with any congruent unit system. As already said, and according to our analysis, it should only be applied when $\tau_* > 0.1$.

5.3.2 Graf-Acaroglu's method

It was presented in 1968 and it permits to obtain bed load transport g_{BT}, both for channels and circular pipes. That is why these authors considered as hydraulic radius the total hydraulic radius of section R, without separating the one associated with grains or particles R' from that related to bed particles R", due to the fact that in a pipe, part of the wetted perimeter corresponds to the pipe itself and part to the sediment.

Graf and Acaroglu took into account the flow over the particle, its submerged weight and the shear stress which acts on the bottom. They concluded that the non dimensional parameters which intervene in sediment transport are the following

a) Shear intensity parameter

$$\psi_{A_*} = \frac{1}{\tau_*} = \frac{(\gamma_s - \gamma)\,D}{\gamma\,R\,S} \tag{5.20}$$

which is similar to that proposed by Einstein but with the total hydraulic radius, and equal to the reciprocal to the parameter or Shields' number

b) Transport parameter

$$\phi_A = \frac{\bar{C}\, U \cdot R}{(g \Delta D^3)^{1/2}} \tag{5.21}$$

where U is the current mean velocity; $\bar{\epsilon}$, the mean concentration by volume, which is equal to the solid volume by the liquid volume that transports it.

To obtain for a wide channel the bed particle volume transported by a unit of width and time, in $m^3/s \cdot m$, this may be done

$$g_{\epsilon T} = \bar{C}_q = C\, U\, R \tag{5.22}$$

because $R \simeq d$.

If $\bar{C}$ is substituted by q_{BT}/UR in eq 4.25, and considering that $q_{BT} = g_{BT}/\gamma_s$, it is obtained

$$\phi_A = \frac{g_{BT}}{\gamma_s (g D^3 \Delta)^{1/2}} \tag{5.23}$$

which is the non dimensional parameter used by Einstein, who called it transport intensity. In this parameter g_{BT} is bed transport expressed in $kgf/s \cdot m$.

In the previous parameter D is the bed material mean diameter if this is practically uniform. When there is a well graded granular distribution, the transport calculation may be done for different fractions of the granulometric curve.

Using the indicated non dimensional number, Graff and Acaroglu plotted on logarithmic paper the data obtained by Einsten (1944), Gilbert (1914), Guy (1966) and Ansley (1963). From the correlation analysis, they obtained the following expression

$$\phi_A = 10.39\, \tau_*^{2.52} \tag{5.24}$$

Substituting ϕ_A, in eq 5.23 and τ_* in eq 5.24, the following is obtained

$$\bar{C} = 10.39 \frac{(g D^3 \Delta)^{1/2}}{U\, R}\, \tau_*^{2.52} \tag{5.25}$$

In a wide channel with rather uniform granular material, the bed transport expressed in dry weight per time and width unit may be calculated by means of the expression

$$g_{BT} = q_{BT}\gamma_s = \bar{C}UR\gamma_s = \bar{C}q\gamma_s \qquad (5.26)$$

thus

$$g_{BT} = 10.39\,\gamma_s(\Delta g D^3)^{1/2}\,\tau_*^{2.52} \qquad (5.27)$$

But, if the granulometry is well graded, the total bed transport may be calculated by means of the expression

$$g_{BT} = 10.39\,\gamma_s(g\Delta)^{1/2}\sum^{i}\frac{p_i}{100}D_i^{3/2}\,\tau_{*i}^{2.52} \qquad (5.28)$$

where p_i is the percentage of each of the fractions in which the granulometric curve is divided. Every fraction mean diameter is D_i.

Since non dimensional parameters without simplification have been used, eqs 5.27 and 5.28 may work with any system of units and, according to our studies, only if $\tau_* > 0.1$.

5.4 Suspended sediments

Two kinds of sediments are transported in suspension

a) Wash load, which consists of very fine particles like silts and clays, and

b) Bed material.

In order to distinguish them, it is necessary to know beforehand the bed material granulometric curve. Once a sample of the material is obtained and analyzed, the wash load consists of the fine particles not included in the bed material. When the size of the bed material is not previously known, particles smaller than 0.062 are considered as wash load. For practical reasons it is advisable to regard as wash load all the material that passes through the mesh 200 with openings of 0.074 mm.

To quantify the sediment transported in suspension, the data indicated in 5.1 must be known plus the concentration C_a of the suspended material, at a point where its distance a above the bottom is known. Besides, that material granulometry (or at least its mean diameter) is also required. Not all these data are easy to obtain. When the material concentration is very low a big volume sample is needed to determine particle size.

The concentration of suspended material is maximum over the bottom layer and minimum on the surface. This distribution is even more remarkable when particle diameter is greater and flow turbulence is smaller, up to the point that at times there is no suspended material near the surface. On the other hand, when turbulence or flow velocity are smaller, concentration distribution is more uniform. Thus, for instance, in most cases it is considered that the distribution of wash load concentrations is practically uniform along all the flow depth. In consequence, it is always necessary to associate a known concentration with its height with respect to the bottom.

When dealing with suspended sediment transport different aspects may be distinguished

a) Wash load determination

b) Distribution of suspended bed material concentrations along a vertical

c) Determination of suspended sediment transport

But before showing the analytic solution of the previous points, some comments are made.

a) Wash load discharge

There are two marked differences between the wash load and the bed suspended sediment. The first one consists in the fact that, without much error, the wash load may be considered as uniformly distributed along the cross section. The second, that wash load does not depend on the current hydraulic characteristics but on the fine material which gets to the basin or is transported by the river when the banks are eroded. In other words, there is no relation among velocity, depth or current slope and the amount of transported wash load. In general, a river carries all the wash load it receives whatever the velocity or discharge is.

Thus, to evaluate in a section the wash load discharge during a certain time period, it is necessary to measure in a continuous form the material concentration in that section.

Due to the fact that continuous gaging is too expensive, a sample of the suspended material is usually taken at least once a day, whenever a gaging or a scale reading is done in the section.

The amount of wash load depends on the condition of the watershed. The smaller the vegetation cover and the greater the negligence or the deterioration of the watershed, the heavier the wash load is. It is clear that it will also depend directly on the rain, because this is the agent which transports unprotected material towards rivers and streams.

In the plain zone close to the river mouth rivers carry large amounts of wash load, and only during floods there is an increase in suspended bed sediment discharge. Because in the reamaining sections along the river higher concentrations of bed material can occur, a larger amount of bed sediment is carried in suspension.

b) Vertical distribution of concentration

When dealing with this point, one generally refers to the distribution of the concentration of the material which comes from the bottom. This distribution may be more or less uniform, but the concentration is always higher near the bottom and lower in the current surface. In order to obtain all the concentration distribution it is necessary to know, at least, that of one point in the vertical. It is worth to remember that in the sample the particles represented in the wash load bottom must be separated. An easy way to do this is to pass the sample through a mesh 200. The material retained by the mesh is the bed suspended sediment load.

It is important to know the concentration distribution because it is basic for the evaluation of the bed suspended transport. All the available methods to value this parameter start from the velocity and concentration distributions, both on the same vertical.

c) Quantification of suspended sediment transport

To value the suspended sediment transport it is necessary to know the concentration in a point of the vertical, right over the bottom layer, in the middle of the depth, or in a stretch of the vertical. Therefore, once the concentration of the point where the sample was obtained is known, the concentration of the point required by the method is to be calculated. This calculation includes both the bed suspended sediment and the wash load, and the bigger the sample and the more carefully it is analized, the much more precise will the results be. If the volume of the sediments obtained is big, the wash load size distribution can also be obtained and it can receive the same treatment given to bed material.

However, in case of engineering problems this is not very practical, and the wash load is separated from the bottom material. The first is considered as uniformly distributed as indicated in a) and shown in 5.4.1. On the other hand, bed material is more carefully analized and it is quantified following what was said in 5.4.3, which are general concepts applicable to any suspended material.

5.4.1 Quantification of wash load transport

In this case the concentration of that material in a point of the section must be known. It is supposed to be uniform all along the section and it should be expressed in kgf/m³ or m³/m³. Besides, it is also necessary to know the flow discharge for the moment when the concentration was obtained. Then, if concentration is expressed in kgf/m³, (C_{LW}) wash load transport is expressed by

$$G_L = C_{LW}\, Q = C_{LW}\, B\, d\, U \tag{5.32}$$

and if C_L is expressed in m³/m³, ($C_{L\forall}$)

$$Q_L = C_{L\forall}\, Q \tag{5.33}$$

where Q is the flow discharge.

The relation between both discharges is

$$G_L = \gamma_s\, Q_L \tag{5.34}$$

When working with unitary discharges, the expressions are

$$g_L = C_{LW}\, q = C_{LW}\, U\, d \tag{5.35}$$

$$q_L = C_{L\forall}\, q \tag{5.36}$$

If the annual wash load transport is to be known, the calculation should be done dayly in a gaging station, considering simultaneously the concentration and the discharge measured every day.

5.4.2 Distribution of sediment concentration

Among the methods proposed to obtain sediment concentration distribution, the following can be listed: Rouse (1937), Lane-Kalinske (1941), Einsten and Chien (1952), Hunt (1954), Velikanov (1954), Chang-Simons and Richardson (1967), Zagustin (1969), Toffaleti (1969), Amtsyferov-Devol'skiy (1969), Ippen (1971), Itakura and Kishi (1980).

Except for the Lane-Kalinske's method, all the others yield very reliable results. Those by Rouse, Einstein-Chien and Velikonov give the same results even if Rouse's is much easier to apply. Those by Hunt, Chang-Simons, Richardson and Hakura-Kishi are slightly more precise than the others, but their application is much more complicated, being this the reason why only Rouse's is presented.

5.4.2.1 Rouse's method

If at a distance a above the bottom a concentration C_a of solid material is known, then the concentration C_y at any point located at a distance y above the bottom can be evaluated by means of the equation suggested by Rouse, which establishes that

$$C_y = C_a \left[\frac{d-y}{y} \, \frac{a}{d-a} \right]^z \tag{5.37}$$

where

- d water depth, in m
- a vertical distance above the bottom for which concentration C_a is known, in m
- y vertical distance, in m, above the bottom at which concentration C_y must be known
- z exponent that takes into account current turbulence; its value is given by

$$z = \frac{2.5\,\omega}{U_*} \tag{5.38}$$

- ω is the fall velocity of particles with a diameter D. D can be the mean diameter of suspended materials, although it is recommended to calculate it for different portions of the granulometric curve.

Eq 5.37 is not valid on its extremes because when y = d or y= 0, it results in a concentration equal to zero in the first case and in an infinite concentration in the second one. Both values are absurd.

It can be observed in eq 5.38 that for a given shear stress or shear velocity, that is, for a constant U_*, the smaller the diameter, the smaller is ω, and, therefore, z. To a smaller z corresponds a more uniform concentration distribution in the vertical line. If particles are big z will be greater and, in consequence, the concentration towards the bed will also be higher.

A similar reasoning can be applied when the diameter remains constant and the shear stress varies thus also varying the flow turbulence: to a bigger shear stress corresponds a greater U_* and also a smaller z; this results in a more uniform distribution of concentrations. If U_* is small, concentration increases towards the bed, and decreases even more towards the surface.

Due to the fact that in suspension every particle is isolated from the others, every size concentration is different, and it is better to give it this treatment. Therefore, for a given sample, the granulometric curve should be divided in fractions. Then, the mean diameter of each portion D_i, its percentage with reference to the total p_i and its concentration C_{a_i} are known. The final concentration in a point of the vertical is obtained by applying eqs 5.37 and 5.38 for every fraction and afterwards adding

$$C_y = \sum_{i=1}^{n} C_{y_i}$$

5.4.3 Suspended sediment quantification

Generally the methods proposed for the quantification of suspended sediment start from a point at a distance a over the bottom, where concentration C_a is known up to the surface. Therefore, they do not determine the amount of material transported in the zone between the bottom and distance a. When the transport along the vertical is to be known, concentration should be calculated over the bottom layer, that is to say, at a distance y = 2D from the bottom.

Among the methods proposed for suspended transport quantification starting from known concentrations there are those of Lane-Kalinske (1941), Einstein (1950), Brooks (1963), Chang *et al.* (1967), Itakura and Kishi (1980).

Einstein's method, which is also presented in 5.5.1, is the most widely known and the most complete because it also permits to obtain g_B and g_{BS} from bed material only. On the other hand, Brooks' method gives results similar to Einsten's and it is the easiest to apply.

5.4.3.1 Einstein's method

It was proposed by Einstein in 1950 and it is one of the most complete.

To calculate suspended material transport, he used the concentration distribution obtained by Rouse and the velocity logarithmic distribution given by Keulegan, which he obtained by following Prandtl-Von Karman and Nikuradse.

The starting equation for this and the other mentioned methods is

$$[g_s]_a^d = \int_a^d u\, C_y\, d_y \qquad (5.39)$$

where u is the velocity in each point of the vertical and C_y is the concentration in those points. In eq 5.39, C_y units are given in kgf/m³.

The first term of the equation corresponds to the suspended material transport expressed in kgf/s·m, which passes through the zone between the distance a measured over the bottom and the surface.

Einstein's expression to obtain g_s is

$$[g_{s_i}]_a^d = 11.6\, C_{a_i}\, U'_*\, a\, (PI_{1_i} + I_{2_i}) \qquad (5.40)$$

therefore

$$[g_s]_a^d = \sum_{i=1}^{n} [g_{s_i}]_a^d \qquad (5.41)$$

where

C_{a_i} and a have already been explained

$U'_* = \sqrt{gR'S}$. Here, it is advisable to obtain R' by means of Engelund's method, explained in

$$P = 2.303 \log \frac{30.2 \times d}{D_{65}} \qquad (5.42)$$

x   coefficient obtained with fig 5.3 as a function of D_{65}/δ', being

$$\delta' = \frac{11.64\, \nu}{U'_*}$$

I_1 and I_2 are two integrals obtained during the algebraic treatment and, for practical ends, they are plotted in figs 5.4 and 5.5. To use these figures the following parameters should be calculated

$$A = \frac{a}{d} \qquad (5.43)$$

$$z = \frac{2.5\, \omega}{U_*} \qquad (5.44)$$

When in all the vertical the suspended transport material is to be known, the concentration C_a should be calculated in point y = 2D, that is, right over the bottom layer. Thus, eq 5.40 remains

$$g_{s_i} = 11.6\, C_{20_i}\, U'_*(2D_i)\, [P I_{1_i} + I_{2_i}] \qquad (5.45)$$

and the total suspended transport in a unit width as

$$g_s = \sum_{i=1}^{n} (g_{s_i}) \qquad (5.46)$$

5.4.3.2 Brooks' method

It was proposed by Brooks in 1963. In order to apply it, particle concentration should be known at half the depth, that is for y = d/2. This can be done by measuring concentration at that point or by calculating concentration $C_y = C_{d/2}$ with eq 5.37, starting from another known concentration C_a.

Accoding to Brooks, the amount of suspended sediment is

$$g_s = q\, C_{d/2}\, [f(z, \frac{\kappa\, U}{U_*}] \qquad (5.47)$$

where

g_s suspended sediment load, in kgf/s·m

q   unitary discharge, in m^3 s·m

z   parameter defined by eq 5.38

$C_{d/2}$ concentration at half the depth and expressed in weight, in kgf/m³

Function $f(z, \frac{\kappa U}{U_*})$ is plotted in fig 5.6. In it, κ is Von Karman's constant which has been considered equal to 0.4.

The previous expressions are valid for uniform material. If the method is applied to natural conditions the granulometric curve is divided in several fractions, each with a representative mean diameter D_i, and a weight percentage p_i referred to the total weight.

Using eq 5.39 g_{s_i} is obtained for each fraction. Their sum will be the total suspended sediment transport, which can be divided in two: a portion with the diameters found at the bed g_{BS}, and that for finer particles g_L.

5.5 Sediment transport quantification by separating bed load transport from bed suspended sediment transport

Among the methods used to obtain g_{BT}, like for instance those that add g_B and g_{BS}, there are those by Einstein (1950), Velékonov (1956), Bagnold (1966), Chang *et al.* (1967) and Toffaleti (1969).

In this chapter we shall finish the exposition of Einstein's method and then Bagnold's will be presented. Even if the others require more graphs and equations they are not more precise.

5.5.1 Einstein's method

Proposed in 1950, it is one of the most widely known.

The final equations Einstein suggests for immediate application are

a) For bed load discharge

$$g_{B_i} = \phi_* P_i \gamma_s (g \cdot \Delta \cdot D_i^3)^{1/2} \tag{5.48}$$

and, therefore,

$$g_B = \sum_{i=1}^{n} g_{B_i} \tag{5.49}$$

where

ϕ_* intensity of bed load transport function. It is obtained as a function of ψ_* with the help of fig 5.7

p_i percentage by weight of particles with diameter D_i or percentage of each one of the fractions into which the granulometric curve is divided

ψ_* flow intensity function, equal to

$$\psi_* = \frac{Y\xi\,(\gamma_s - \gamma)\,D_i}{\gamma\,R'\,S}\left(\frac{\log 10.6}{\log 10.6\,\frac{xX}{D_{65}}}\right)^2 \tag{5.50}$$

Y coefficient which considers the change undergone by the lift force of each particle within the granular mixture. It is obtained as a function of D_{65}/δ' by using fig 5.8

ξ coefficient which takes into account the fact that big particles hide small ones. It is obtained as a function of D_{65}/δ', using fig 5.9

δ'_o thickness of the laminar sublayer with reference to particle roughness

$$\delta'_o = \frac{11.64\,\nu}{U'_*} \tag{5.51}$$

X coefficient which accounts for particle size and current turbulence

$$X = 0.77\,\frac{D_{65}}{x}, \quad \text{when } \frac{D_{65}}{x\delta'} > 1.80 \tag{5.52}$$

$$X = 1.39\,\delta', \quad \text{when } \frac{D_{65}}{x\delta'} > 1.80 \tag{5.53}$$

U_* shear velocity associated with hydraulic radius R'

$$U'_* = \sqrt{g\,R'\,S} \tag{5.54}$$

n number of fractions in which the granulometric curve is divided

D_i mean diameter of each fraction, in m

To calculate R' or U'_* it is better to use another method, Engelund's for instance, and afterwards Einstein's criterion only to evaluate sediment transport.

Once mean velocity is calculated, it is convenient to obtain Manning's roughness coefficient, for it is the quickest way to know if the resulting velocity is absurd. If it were, Engelund's method should be applied to obtain R', U'_* and later δ'.

These controls are justified, because the most important mistakes are generally made when obtaining the hydraulic characteristics. Each line in table 1.4 corresponds to one of the fractions into which the granulometric curve is divided. Bed suspended sediment discharge is obtained for the region which goes from height $2D_i$ up to the surface.

b) For bed suspended sediment discharge
The equation proposed by Einstein is

$$g_{BS_i} = g_{B_i} (PI_{1_i} + I_{2_i}) \tag{5.55}$$

and, therefore,

$$g_{BS} = \sum_{i=1}^{n} g_{BS_i} \tag{5.56}$$

where

$$p = 2.303 \log \frac{30.2 \times d}{D_{65}} \tag{5.57}$$

I_1, I_2 are the values of two integrals found using figs 5.4 and 5.5, respectively, as a function of

$$A = 2D_i/d \quad \text{and of} \quad z = 2.5\,\omega/U_*$$

As can be seen, to evaluate bed suspended transport it is necessary to know beforehand the bed layer load g_{B_i}. To apply Einstein's method the use of table 5.1 is recommended. This procedure is based on laboratory tests, although it was also tested in real cases. To follow it strictly, first the flow hydraulic characteristics should be obtained by the procedure described in the bibliography; also table 5.2 is helpful for its calculation. Hydraulic characteristics provide mean velocity and total hydraulic radius, including those of particles and bedforms. From R', U'_* and δ' bed sediment discharge can be calculated.

5.5.2 *Bagnold's method*

In 1966, Bagnold proposed the following equations

$$g_B = \frac{\gamma \tau_* U D_m e_b}{\tan \alpha} \tag{5.58}$$

$$g_{BS} = \frac{0.01 \gamma \tau_* U^2 D_m}{\omega} \tag{5.59}$$

therefore,

$$g_{BT} = \gamma \tau_* U D_m \left(\frac{e_b}{\tan \alpha} + 0.01 \frac{U}{\omega} \right) \tag{5.60}$$

where

- U mean flow velocity, in m/s
- D_m bed particle mean diameter, in m
- e_b coefficient which depends on mean flow velocity and mean particle diameter. It is calculated using fig 5.10
- $\tan\alpha$ parameter which depends on τ_* and mean particle diameter. It is found using fig 5.11

Based on what Bagnold wrote in 1956, this method was published in 1966 and it should only be applied for particles bigger than 0.015 mm and under 2 mm.

Provided that the equations presented are dimensionally homogeneous, they can be used with any coherent unit system.

If τ_* is substituted by its value, and because $q = Ud$, eq 5.60 becomes

$$g_{BT} = \frac{\gamma q S}{\Delta} \left(\frac{e_b}{\tan \alpha} + 0.01 \frac{U}{\omega} \right) \tag{5.61}$$

It is better to apply this equation to the fractions of the granulometric curve adding then the results.

5.6 References

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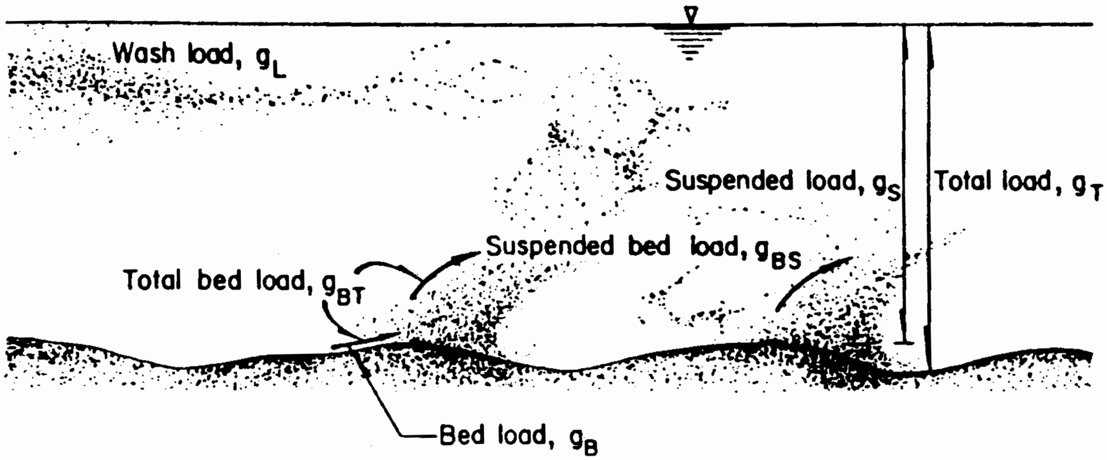
TABLE 5.1 GUIDE TO KEEP AN ORDER IN THE CALCULATION OF BED LOAD AND SUSPENDED BED LOAD. EINSTEIN METHOD

D_i	Write mean diameters of each chosen fraction
P_i	Write percentage, in weight, of each fraction in relation to the sample weight
R'	See Engelund's method in point 3.3.2.1
$\frac{D_i}{X}$	X is calculated with eq 5.52 or 5.53
ψ	$\psi = \frac{\gamma_s - \gamma}{\gamma} \frac{D_i}{R'_H S}$
ξ	Obtained as a function of D_i/X, from fig 5.9
ψ_*	$\psi_* = \xi Y (\frac{\beta}{\beta_*})^2 \psi$
Φ_*	Obtained as a function of ψ_* , from fig 5.7
$D_i^{3/2}$	It is calculated
g_{B_i}	Calculated with eq 5.48
ω_i	Obtained from fig 2.2 for each D_i
Z	$z = 2.5 \frac{\omega_i}{U'_*}$, $U'_* = \sqrt{gR'_H S}$
A	$A = \frac{2D_i}{d}$
I_1	Obtained as a function of A and Z, using fig 5.4
I_2	Obtained as a function of A and Z, using fig 5.5
P	Calculated using eq 5.42
g_{BS_i}	Calculated using eq 5.55
g_{BT_i}	$g_{BT_i} = g_{B_i} + g_{BS_i}$
g_B	$g_B = \sum_{i=1}^{n} g_{B_i}$
g_{BS}	$g_{BS} = \sum_{i=1}^{n} g_{BS_i}$
g_{BT}	$g_{BT} = \sum_{i=1}^{n} g_{BT_i}$



$$g_T = g_S + g_B = g_L + g_{BT}$$

$$g_S = g_L + g_{BS}$$

$$g_{BT} = g_B + g_{BS}$$

Fig 5.1
Kinds of sediment transport

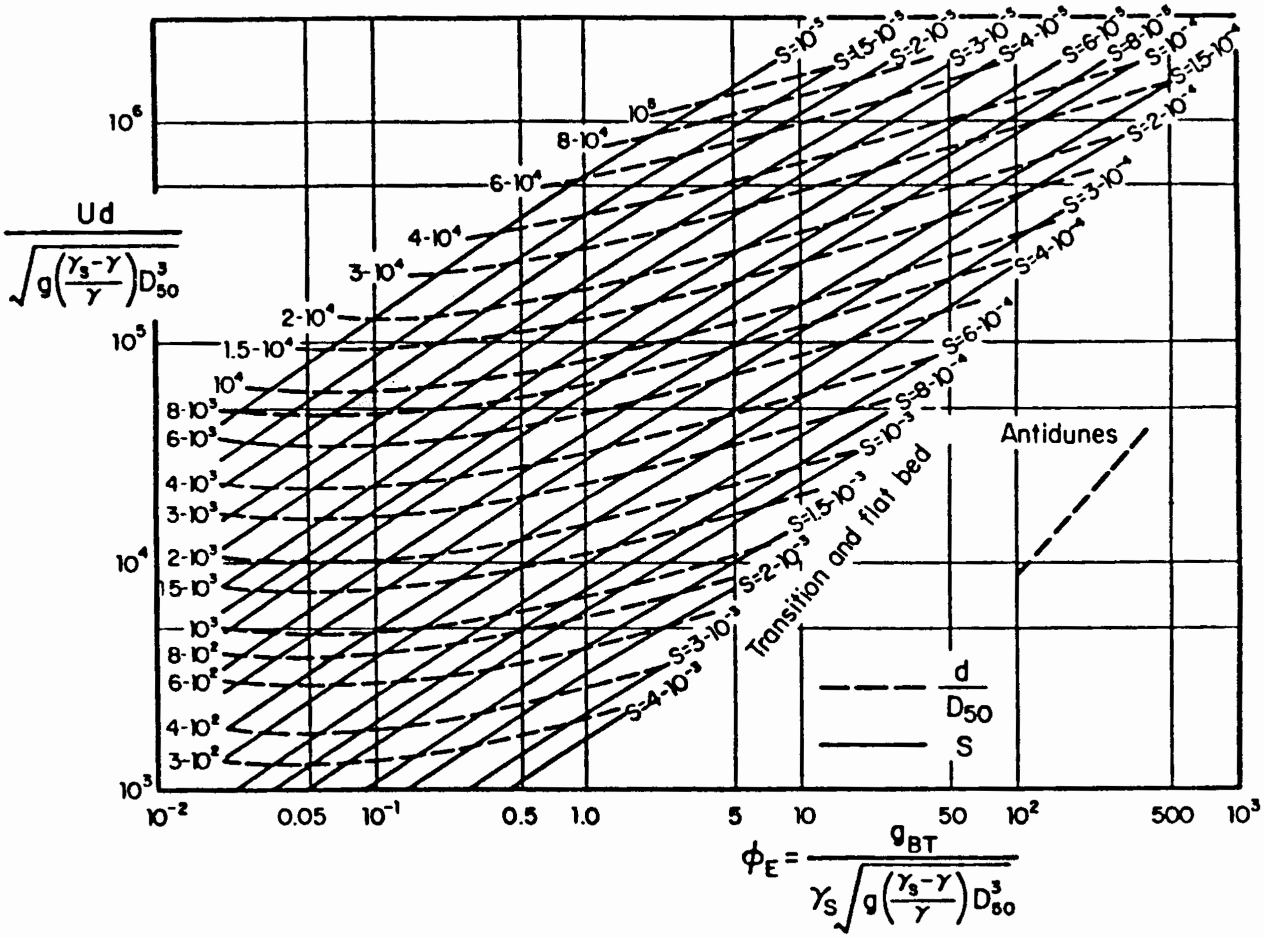


Fig 5.2

Diagram to obtain sediment discharge, after Engelund and Hansen

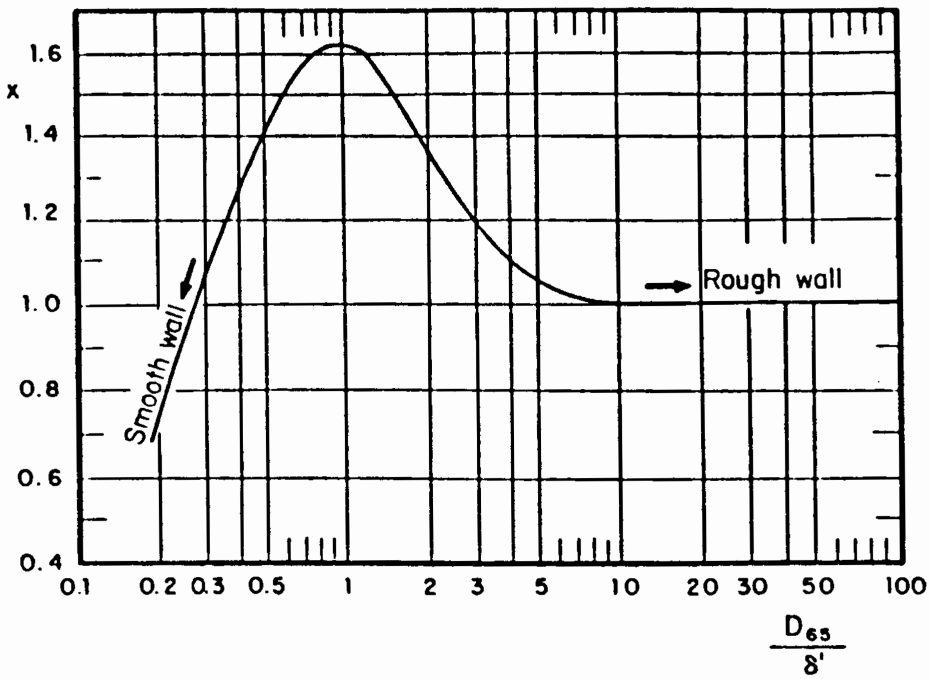


Fig 5.3
Correction factor x. Einstein method

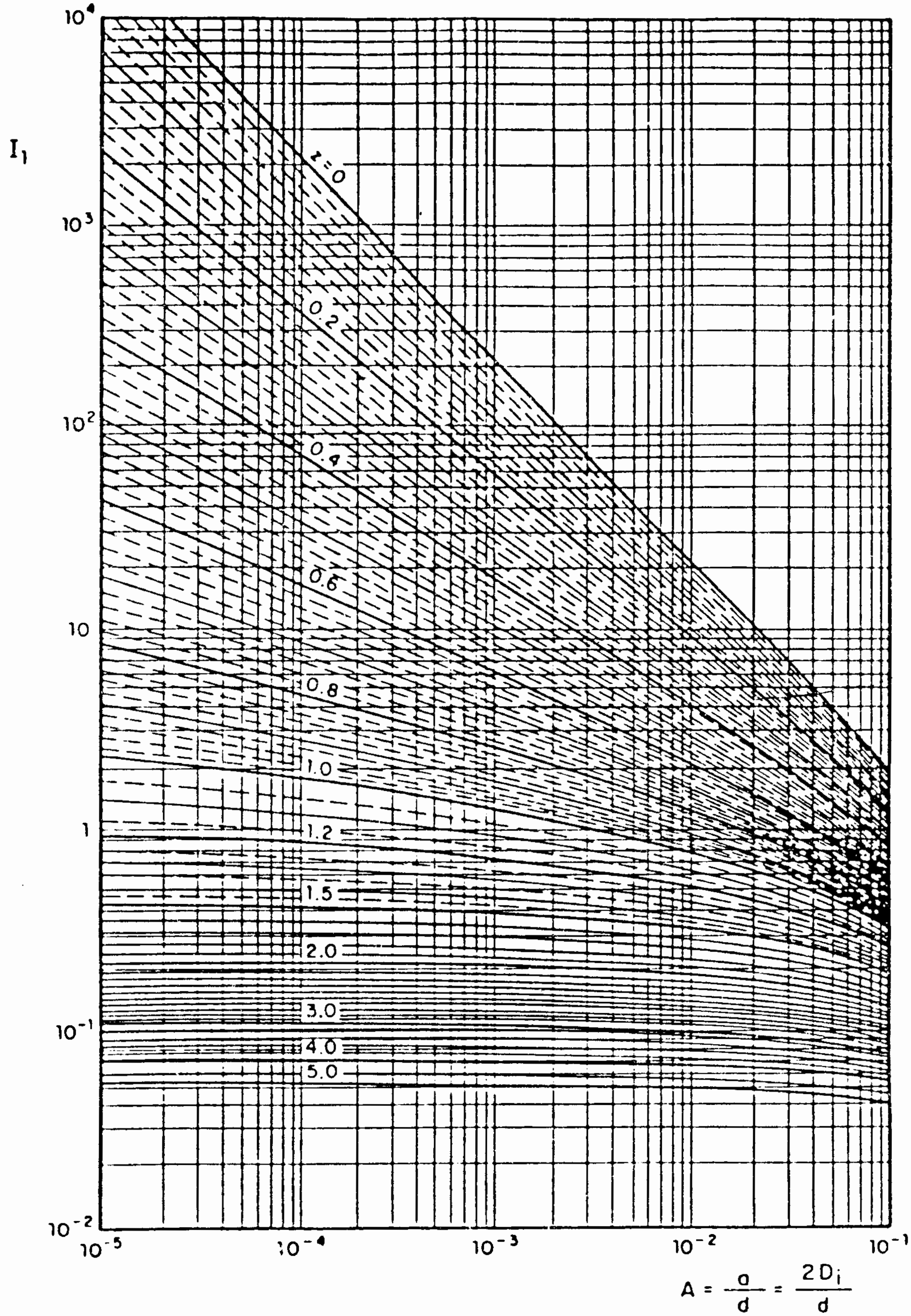


Fig 5.4

I_1 as a function of A and z. Einstein method

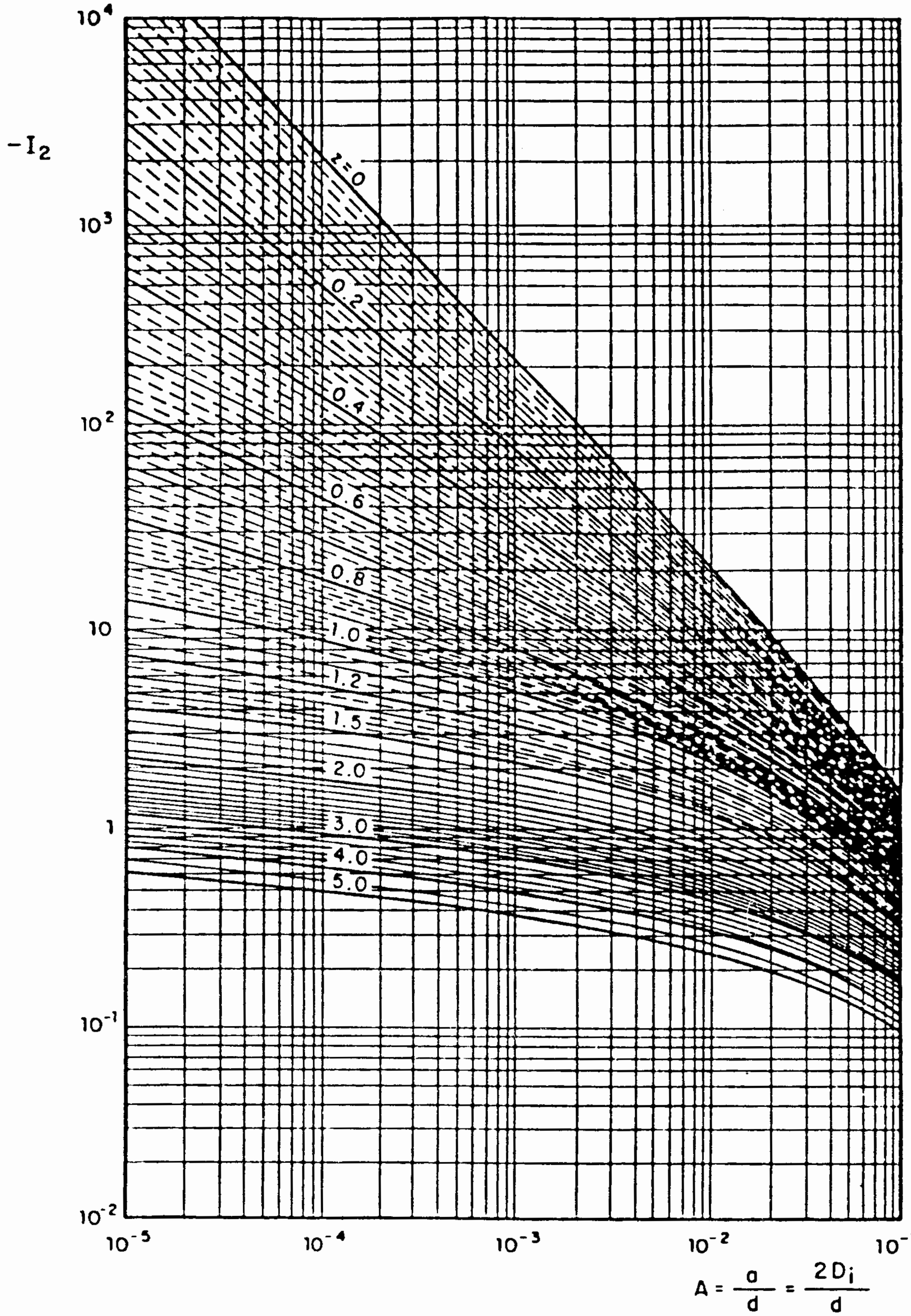


Fig 5.5

I_2 as a function of A and z. Einstein method

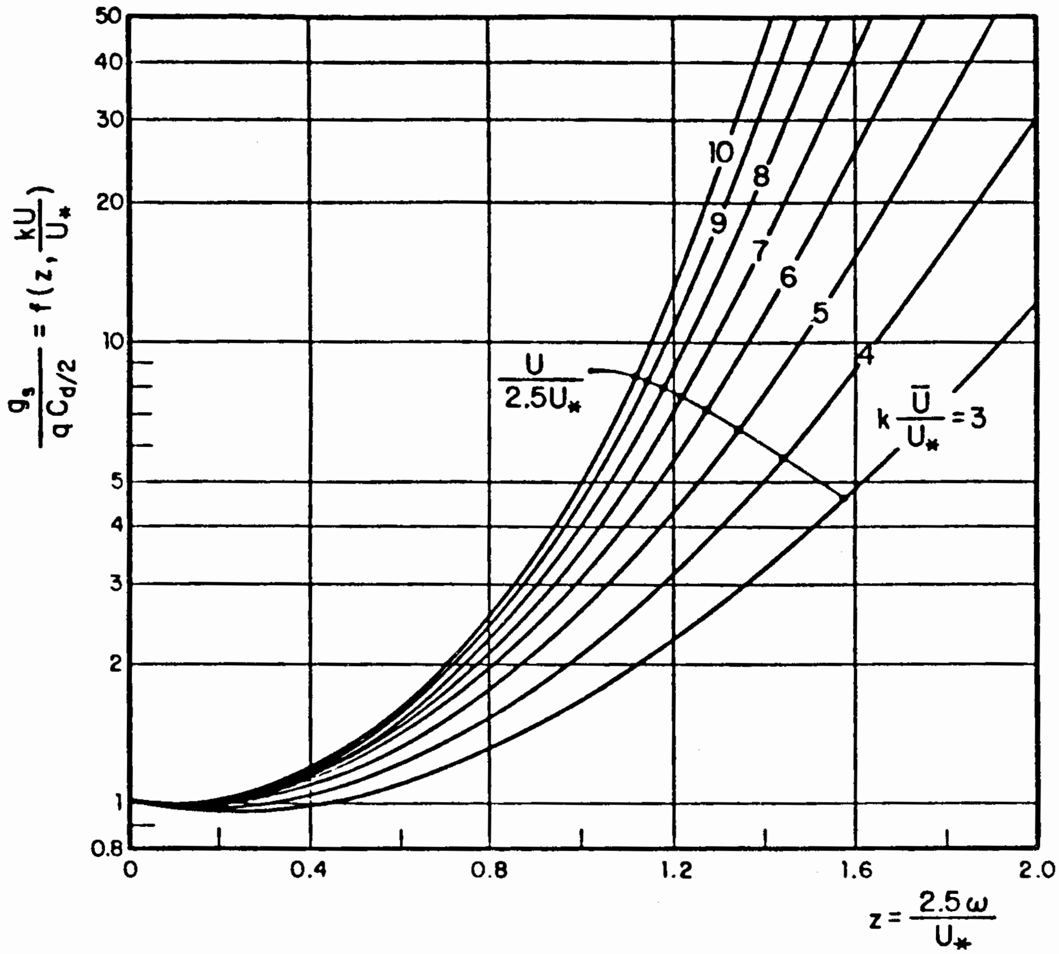


Fig 5.6
Rate of sediment in suspension, after Brooks

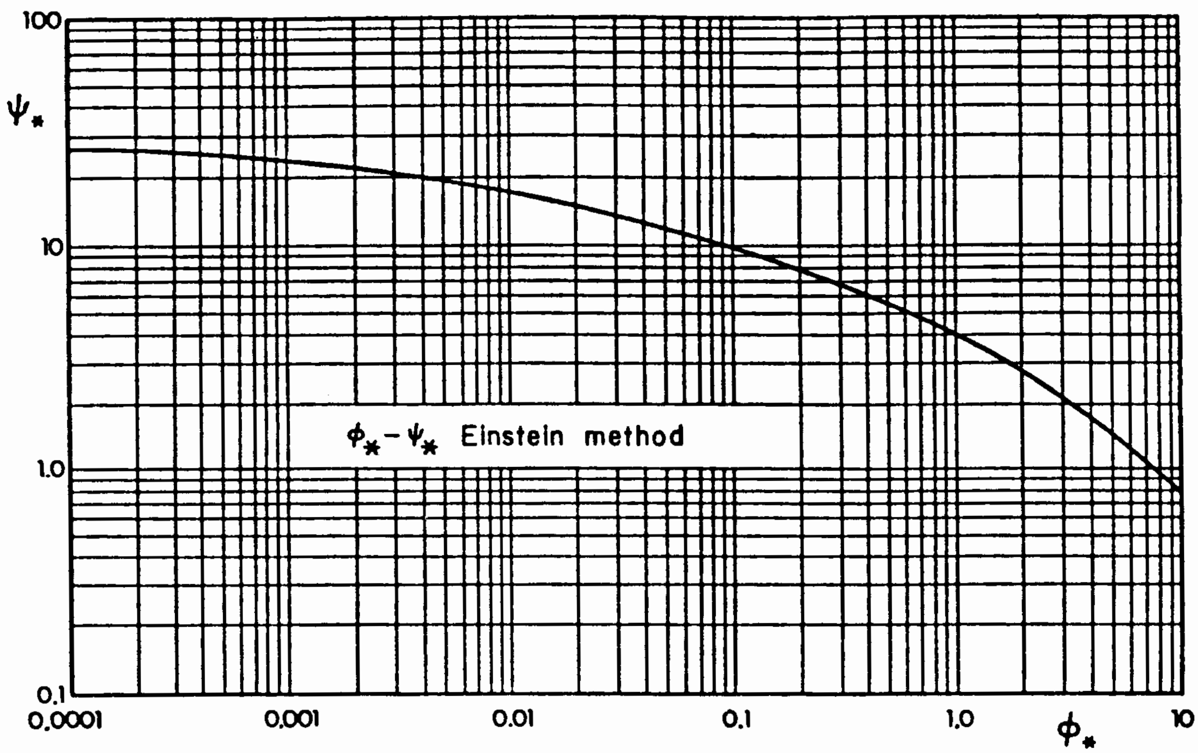


Fig 5.7
Curve of $\phi_* - \psi_*$. Einstein method

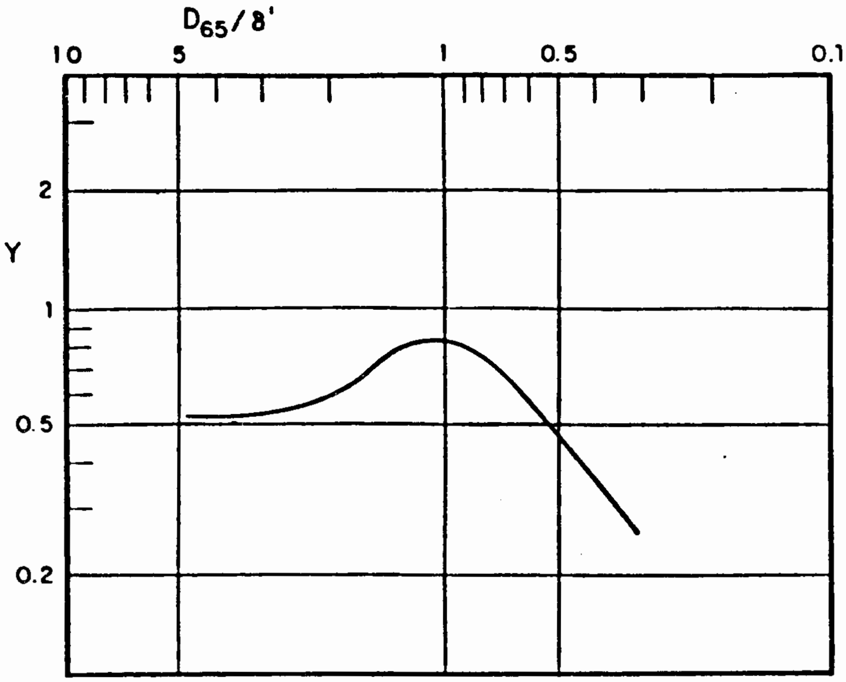


Fig 5.8
Correction factor Y. Eintein method

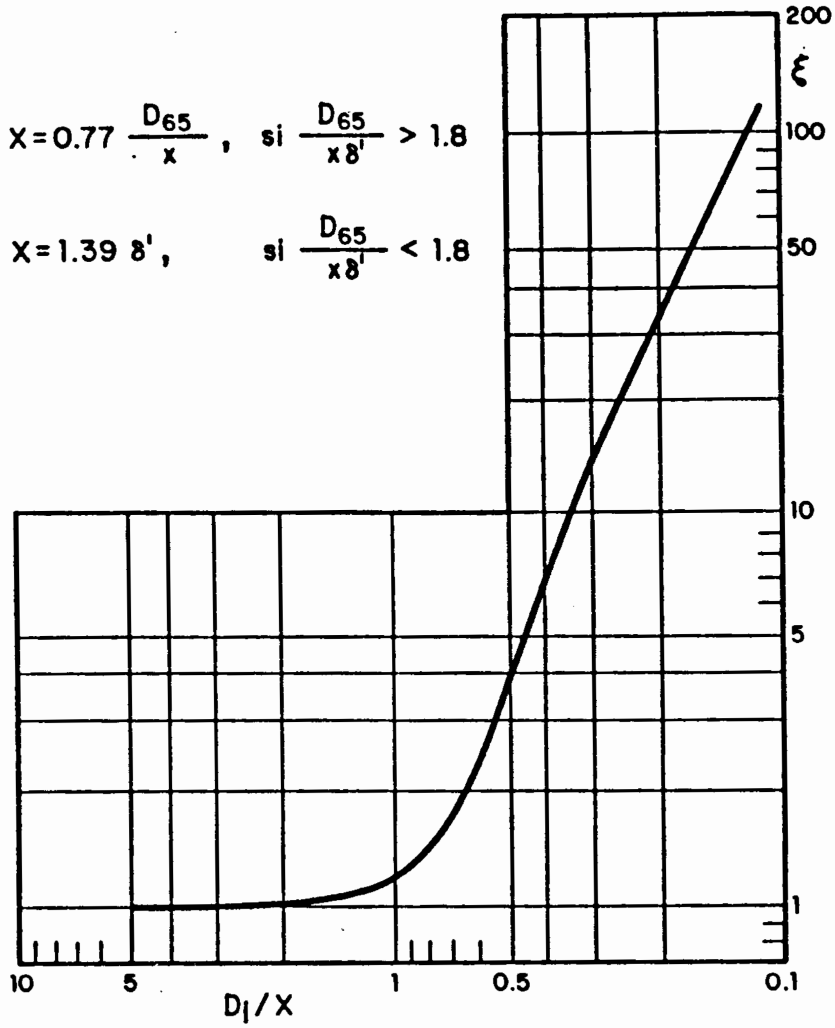


Fig 5.9
Correction factor ξ. Einstein method

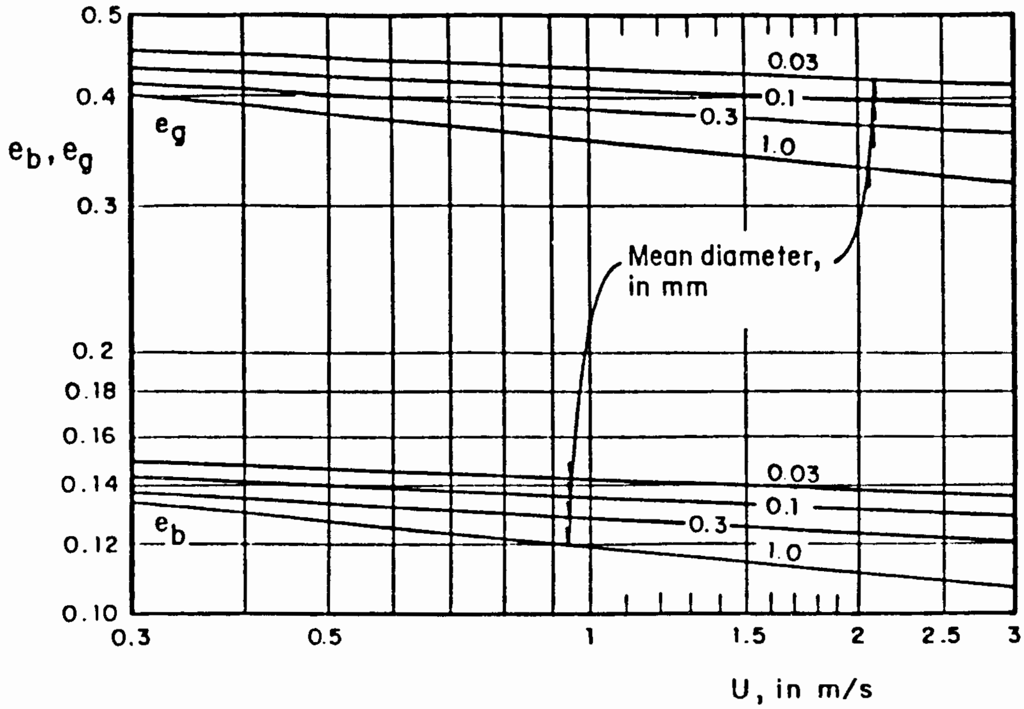


Fig 5.10
Theoretical values of e_b , e_g after Bagnold

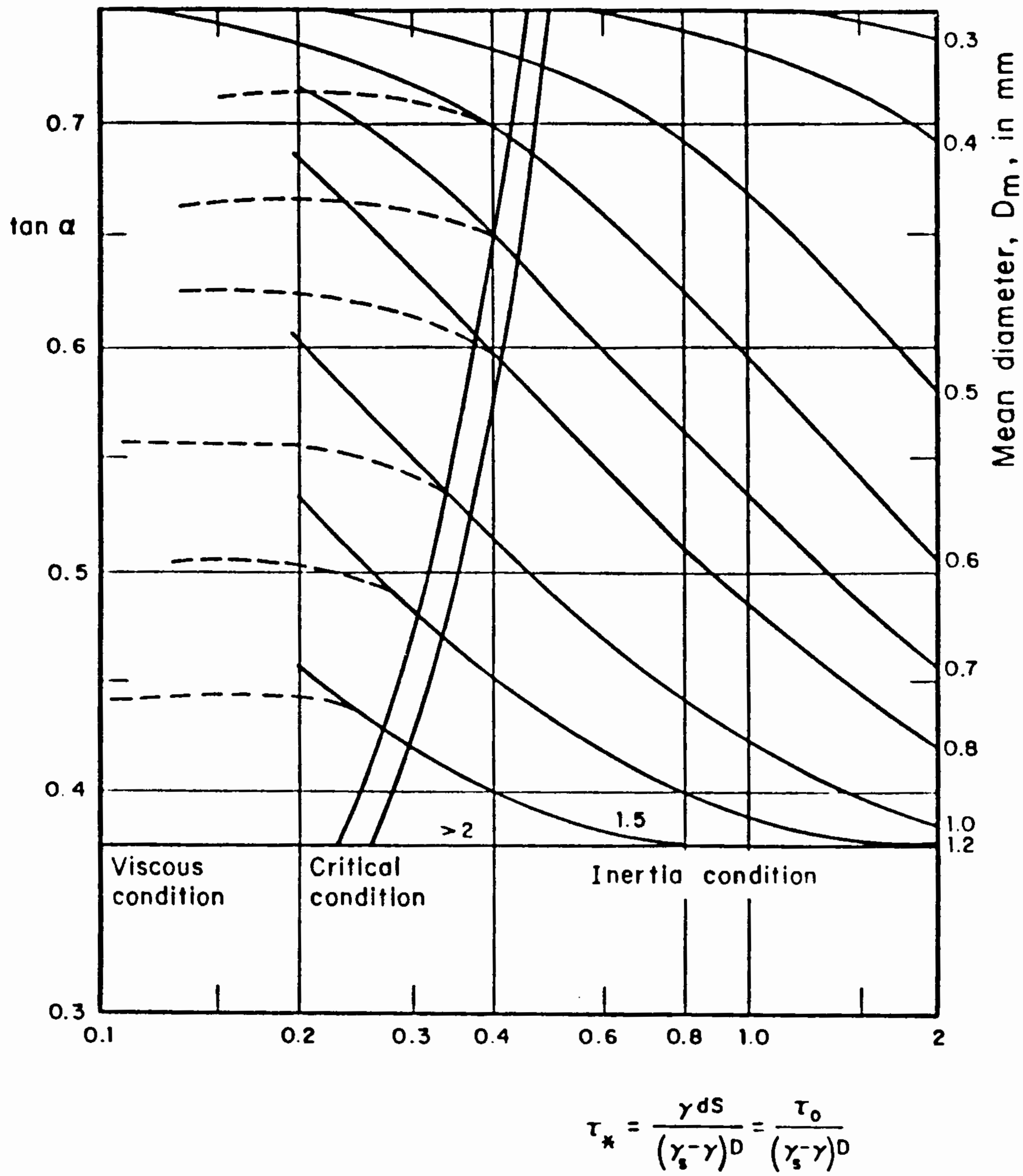


Fig 5.11
Friction factor, after Bagnold

CHAPTER 6

CHANNEL AND RIVER STABILITY

6.1 Introduction

Under normal conditions the stretches of all rivers reach a certain degree of equilibrium; this means that if one or several of the parameters which intervene in that stability condition are not modified, water will continue to flow in the way it used to. If any parameter is naturally or artificially modified, with time and very slowly, the river stretch will change to a new equilibrium condition.

Among the parameters which constitute the equilibrium condition the following can be cited

- Q liquid discharge and its distribution during a year, in m^3/s
- C concentration of the wash load material in kgf/m^3 or expressed in fraction, by weight or volume. It is the suspended sediment smaller than 0.062 mm.
- Q_{BT} total bed sediment discharge, both, the one carried on the bed layer and that in suspension, in kgf/m^3 or m^3/s
- A cross section area, in m^2
- d mean depth. It can be obtained by dividing the hydraulic area by the mean width b of the surface

$$d = \frac{A}{b} \tag{6.1}$$

- S hydraulic slope of the current

D_i representative diameter of bed material, in m. Percentage i of the mixture by weight with a diameter smaller than D_i

γ_s specific weight of the bed material, in kgf/m^3

γ specific weight of water, in kgf/m^3

In general, there is equilibrium among

a) The river or channel discharge

b) The solid discharge that enters into the stretch and which the current can transport into the same stretch

c) Bed and bank material characteristics

d) Longitudinal slope

e) Geometry of the cross section

The modification of any of these parameters influences all the others and will change them until they achieve a new equilibrium.

The causes of modification may be:

I Natural { a) abrupt; b) slow }

II Artificial or provoked by man

I.a Abrupt natural modifications may occur when

I.a.1 Meanders cutoff, rivers change their course or there are floods
I.a.2 Telluric movements that change land configuration or river courses
I.a.3 Ash concentration produced by volcanic eruptions

I.b The most usual slow changes are associated to

I.b.1 Slope changes due to erosion or gradual sedimentation along the river

I.b.2 Granulometric changes of sediments due to bank erosions or land slidings, etc.

I.b.3 Slope changes provoked by an increase in the length of the stretch as results of the sediment deposits which advance into the sea or lagoons. Delta formation

I.b.4 Flow changes due to micro and macroclimate modifications in the natural characteristics of the basin.

These and other causes modify the geometrical characteristics and the slope of rivers and brooks, but many of these changes can not be appreciated during a man's life span. Because they are not frequent and their effects are restricted, earthquakes, for instance, do not have much influence in the modification of natural streams; the same is also true with climatological changes and with the increment in the length of river mouths; on the contrary, meanders cutoff is a common phenomenon in the streams of the plain.

II. Artificial modifications —or manmade— are the main cause of alteration of the equilibrium condition of rivers, up to the point in which there are stretches (especially when close to the river mouth) that no longer exist due to the absolute control of the flow. Among the most important modifications produced by man, the following can be cited:

II.a Dam construction. These works give origin to two main alterations: first, they modify the annual hydrograph and, ocassionally, the annual flow volume which takes place when part of the waters do not go back to the river but are either sent to other streams or they are transported through canals with irrigation purposes. Second, transported sediments are retained in the reservoir up to the point in which even the finest particles are kept in many big reservoirs.

This process diminishes the hydraulic capacity of the river stretch between the dam and the first important affluent, in addition to bed erosion and slope changes in the downstream reach close to the reservoir (see 4.5).

II.b Watershed erosion. Although many different factors produce soil loss in watersheds, two are the most common and provoke at the same time the greatest effect: deforestation and defficient cultivation techniques.

Deforestation has two effects: a) it reduces rain water concentration and may increase flow volume, what may finally lead to changes in the hydrograph for the same rain conditions. b) It may considerably increase the amount of sediments which comes to the rivers because, when the vegetal cover disappears, soil and rock particles remain exposed to the direct impact of the rain.

Of these effects, the increment of solid materials in rivers is the most important, because it modifies its slope completely diminishing its depth and increasing at the same time the possibility of floods produced by the loss of the hydraulic capacity of the river.

Defficient agricultural technics also produce big quantities of sediments towards the rivers, with the already mentioned consequences.

II.c Navigation. In order to guarantee the minimum depths which facilitate the navigation during most part of the year, weirs and locks are built to modify the depth and, consequently, the hydraulic slope and the transport capacity.

II.d Reduction of rivers width. Groins or revetments are built in many stretches to prevent lateral erosion, to direct and fix the flow and, finally, in navigable stretches, to increase the depth. This last procedure normally reduces the width of the river sections.

II.e River beds rectification. To reduce the length of navigable stretches or to avoid erosion in some curves, it is a normal practice to cut off meanders (which increase river slopes) or to construct works like those described in the previous paragraph.

II.f Diversion dams. They momentarily retain sediments, thus modifying the bottom and water level above them and, consequently, the water depth.

II.g Land communications. The roads built in the plains interrupt in greater or smaller degree the flow discharges during floods. Besides, by exploiting borrows and leaving cuts and slopes unprotected, a larger amount of sediments gets into the rivers. Here the effect of bridges is also included, especially those which, due to a defficient design, reduce the hydraulic capacity of the river.

All these processes change and modify the equilibrium of rivers. Of course, those due to human errors ought to be prevented; this is the case for instance of deforestation and of wrong agricultural techniques. However, many alterations are the consequence of very beneficial works like the construction of dams, land communications, navigation facilities, etc. But, by taking into account the changes they produce in rivers, it is possible to avoid or reduce the damages —if any— and to know the river future stability condition.

6.1.1 Stable channel: importance of its study

The study of channel and river stability with sediment transport has a practical aplication for

a) The design and construction of earth channels with no visible modifications during their life span.

b) The prediction of the new slope or geometric characteristics of a river section when one of the characteristics of the river or the watershed change.

At present, the methods developed to relate the already mentioned parameters are only applicable when the discharge is done through a unique section and there are no islands or bifurcations in the stretch. Besides, most of them have been proposed for the design of earth channels.

Before presenting these methods, some concepts or related themes will be explained here for the better understanding of the stable behaviour of channels and rivers.

6.2 Types of stability

As rivers present different degrees of stability, we propose the following classification:

a) Static stability
It occurs when the flow is not capable of carrying bank and bed materials, the sections do not vary, and the river does not undergo lateral slides. Channels with no bed load transport and some rivers during the dry season have this degree of stability. Lined channels have static stability, but this can also occur when rivers flow through very cohesive materials. During the dry season, static stability is also present in rivers where the bottom is covered by cobbles or boulders, and the flows are not so strong as to move or displace them. Immediately downstream of a large dam. The reaches of a river also attain static stability, due to the armoring process on the bed.

b) Dynamic stability

It is present in rivers with a single channel where all the discharge flows through it and there is sediment transport. Although theri cross section can vary, it remains almost the same when measured during the same season year after year.

Dynamic stability supposes the presence of only one channel and its definition is taken from the *regime theory*. This degree of stability is the most widely studied and the methods discussed here are useful to find the geometric and slope characteristics of dynamically stable rivers. Single channel rivers are dynamically stable when no constructions modify them.

c) Morphologic stability

Morphologic stability is a very wide concept. It means that in any natural river, slope, width, depth and the number of branches through which the water flows, depend on flow itself, yearly distribution, sediment characteristics and on the quantity and quality of sediment transport.

All river reaches present morphologic stability, provided that no recent sudden change has occurred like a telluric movement or meander cutoff.

When reaches have a static or dynamic stability they also have morphologic stability, but the converse does not necessarily hold.

Static and dynamic stability represent two extreme conditions when dealing with a river reach in a single channel. If a river is altered, its stability can be modified. The dynamic stability concept considers lateral bank erosion and cross section adjustments due to changes in the liquid flow.

6.3 Degree of freedom

Another important concept for the better understanding of these methods is the degree of freedom of a current. (see fig 6.1).

I. Flows with one degree of freedom. When a flow is made to pass through a lined channel or a fixed width channel and the bed has no bed load transport, water elevation is the only variable factor. Then, it is said that the current has one degree of freedom and one equation is enough to know the flow depth. The best example is represented by a lined channel with permanent flow.

II. Flows with two degrees of freedom. If, through a lined channel a certain quantity of solid material is constantly fed to the liquid flow, its depth and bed slope adjust themselves to allow the liquid flow to carry all the solid material. Then, it is said that the flow has two degrees of freedom because depth and slope can be adjusted. Two equations evaluate these variables, one for flow resistance and the other for sediment transport.

III. Flows with three degrees of freedom. If a channel is formed by alluvial material and it is fed with a combination of liquid flow and sediment transport (like in case II), its slope, depth and width will adjust when the liquid flow carries the solid material in a continuous, uniform way. In this case the flow has three degrees of freedom, and three equations are needed to define its state of equilibrium.

Natural currents have three degrees of freedom. If the liquid and sediment discharge besides the bed material characteristics are known, three equations are needed to define the width and the depth of the cross section and the hydraulic slope.

Even if for Blench and other authors natural channels have four degrees of freedom because they can meander, they all use only three equations to define channel stability.

Meanders develop when a stable river slope is lower than the general plain slope. To reach the lower slope the river increases its length by forming meanders.

6.4 River zones

To make some explanations easier, we propose to divide the total length of the river in three parts or zones, thus following Lebediev. They are

a) *Mountain zone*. It has the following characteristics: steep slopes in river beds, large size sediments and no agricultural areas. Since river beds lie among mountains, when a flood occurs, water levels raise without overflowing.

b) *Intermediate zone*. Here bed slope and sediment size diminish. Sediments consist mainly of sands and gravels and beds are generally found among hills, even if there are plains where agriculture exists. Such areas form through sedimentary processes and are subject to floods whenever the hydraulic capacity of the river is exceeded.

c) *Plain*. In this zone smaller slopes are found, and land consists of fine sediments carried and gradually deposited by the river. These zones are more or less plain thus favouring agriculture over large extensions. They also undergo floods that can last longer than those of the intermediate zones, because their discharges are bigger than the drainage capacity small slopes can provide.

When mountains are very close to the sea, rivers have no time to develop the last zone, being this the reason why plains do not exist. This may also sometimes occur in the intermediate zone.

6.5 Formative discharge

To study river stability, a flow must be fixed to represent the annual hydrograph. This flow, associated with channel stability, is called formative discharge and several different criteria can be followed for its determination

a) Dominant discharge. This is the name given to a formative flow that, when constant through a year, will transport the same quantity of bed sediment as the yearly hydrograph.

Therefore, to find the dominant discharge, bed load is calculated for each day during a year as a function of the mean daily gaged flow. All results are added and then divided by 365, being the result the average daily transport. From this value it is easy to obtain the liquid daily flow capable of such a transport. The liquid flow, in m^3/s, associated with this mean daily sediment transport, is called dominant discharge.

In some cases, this procedure is applied only during the rainy seasons because there is no transport during the dry periods. If these days were considered, the dominant discharge would have a very low value; so, it has to be calculated taking into account only the days when the current carries bed sediments. This is correct, since no bed transport means that the current is not forming its own channel.

The dominant discharge value is generally lower than the other two formative discharges explained below. Better results are obtained when total bed sediment discharge is considered and not only bed load transport.

b) Maximum discharge in the main channel of a river
Some authors consider that the formative discharge for rivers in plains is the maximum flow capable of running without overflowing through the main channel and towards the plain. In most problems, this criterium led to congruent results.

c) Discharge with a return period of 1.4 years
For some authors, including Leopold and Madok, formative discharge has a return period of 1.4 years. If a gage station is close to the river reach under study, formative discharge can be found by analyzing maximum flows by means of the Nash or Gumbel criteria.

In some cases the formative discharge with a return period of only 1.4 years is similar to the maximum discharge mentioned in point 6.

It is always better to apply the three criteria and check the discharges. Generally, an additional factor permits to know which is the most adequate.

6.6 Methods to study channel stability

It has been said that a current running in a single channel reaches an equilibrium among the river hydrograph or formative discharge, the sediment flow which enters into the river reach under study and the characteristics of the cross section and of the sediment.

Once the formative discharge Q, the bed transport which enters into the reach under study Q_B or Q_{BT}, and a representative diameter of bed material D, are known, it is usually better to obtain the slope S, the width b of the surface and the depth d, capable of making the reach under study stable.

To obtain these variables three equations are needed and several methods have been proposed: the regime theory, Altunin's method and that of Maza-Cruickshank. The first includes all the formulae, empirical methods and observations of stable channels, and it is especially useful for the design of channels with cohesive or sandy materials. Even if Altunin's method has been designed for rivers with rough materials like gravels and cobbles, it can also work in case of sands. Finally, the third, although developed for sandy rivers, may be applied for gravels, provided it is carefully done.

6.7 The regime theory

It was first proposed by Kennedy in 1895. As he was in charge of the design of a net of unlined channels, he studied and measured the dimensions of a series of earth channels in operation, because their sections had already adjusted to stable dimensions in relation to their liquid and solid discharge.

From 30 observations carried out in the Bari Doab channel, he realized that the mean velocity was a function of the depth and, based on this relation, he dimensioned his project. The relation is

$$U = 0.548\, d^{0.64} \qquad (6.2)$$

where U is flow mean velocity, in m/s, and d is depth, in m.

Afterwards, and as a result of new observations, he modified the expression to

$$U = c\, d^{m} \qquad (6.3)$$

where

- c coefficient which varies from 0.67 to 0.95
- m exponent which varies from 0.52 to 0.64

It was Lindley who, in 1919, used for the first time the word *regime* with reference to a channel to express that its section and slope are in equilibrium with the discharges transported. Therefore, increments or decrements of the discharge modify the width and depth in relation to these values. That is why, after the annual periods, sections and slopes remain practically constant, being this the principal characteristic of dynamically stable channels.

After Kennedy, many other authors, especially British, proposed other empirical equations to relate the geometry of the section and the slope of the channel with the liquid discharge and the bed and wall material characteristics. Among them, Lane, Lacey, Lindley, Bose and Malhotra, Stebbings, Chitale, Inglis, Joglekar, Blench, Simons and Albertson, can be cited; they obtained their data principally from irrigation channels in India, Pakistan and Egypt, and later in Canada and the United States.

This work describes Lacey, Blench, Simons and Albertson's relations. The first, because it is based on a recopilation of all the previous methods; the second, for being the best known, and the third, because it is one of the latest and also because it is the clearest in indicating the value of the coefficient and exponents according to the bed material and bank characteristics.

The regime theory is applicable to sandy or cohesive channels. Due to the fact that most of the data for these methods was taken from channels with cohesive banks, they are very useful in the design of channels of this material.

6.7.1 Lacey's method

This author followed and enlarged Lindley's studies, who had already observed 4,343 km of channels in India. Lacey was the first in considering bed material with parameter f, named as silt factor. Besides, he used in his formulae the hydraulic radius and the wetted perimeter instead of width and depth. Although this represents more work for the designer, it is more precise.

For the systems of units of 1.1, the three fundamental formulae are

$$P = 4.838\, Q^{1/2} \tag{6.4}$$

$$R = 0.4725\, (Q/f)^{1/3} \tag{6.5}$$

$$S = 0.000302\, f^{5/3} / Q^{1/6} \tag{6.6}$$

where

- P wetted perimeter, in m
- R hydraulic radius, in m; when the surface width b is about 40 times larger than the depth d, the hydraulic radius may be considered equal to the depth
- S hydraulic slope
- Q design discharge, in m^3/s
- f silt factor, its value is obtained from the expression

$$f = 55.66\, D_m^{1/2} \tag{6.7}$$

where D_m is the mean diameter of the bed material, in m. It is obtained from eq 2.24.

Besides, Lacey uses Manning's equation where the roughness coefficient is a function of the silt factor and, with this, the flow mean velocity is calculated

$$U = \frac{R^{2/3} S^{1/2}}{n_a} \quad (6.8)$$

where n_a is the roughness coefficient valuated according to the expression

$$n_a = 0.0225 f^{0.25} \quad (6.9)$$

When substituting in this last equation the value of f given by eq 6.7, it is obtained:

$$n_a = \frac{D_m^{1/8}}{16.27} \quad (6.10)$$

If in eq 6.8 the value of R, S and n_a given, respectively, by eqs 6.5, 6.6 and 6.9, are substituted, then

$$U = 0.2423\, Q^{5/36} D_m^{13/72} \quad (6.11)$$

When the channel is very wide, that is $b > 40\ d$, it is important to remember that $R \simeq d$ and $P \simeq B$ (B, width of the bottom channel).

These fundamental formulae are at the same time design formulae, because the wetted perimeter P, the hydraulic radius R and the channel slope S are obtained as a function of the formative or design discharge. Another parameter to be taken into account is factor f, which is a function of the bed and bank particles.

Once P and R are known, the bottom width B and the depth d must be obtained. Thus, if the section is trapezoidal, the following expressions must be used

$$P = B + 2d\,(1 + k^2)^{1/2} \quad (6.12)$$

$$R = \frac{d\,(B + kd)}{B + 2d\,(1 + k^2)^{1/2}} \quad (6.13)$$

where k is the bank slope given as $k = \cot \alpha$, being α the angle formed by the bank and the horizontal.

If the section is rectangular, something rather frequent because it is the kind of section which is better to consider in very wide channels, the equations of P and R are reduced to

$$P = B + 2d \tag{6.14}$$

$$R = \frac{Bd}{B + 2d} \tag{6.15}$$

Therefore, the dimensions of the section are obtained from the expressions

$$B = P - 2d \tag{6.16}$$

$$2d^2 - Pd + PR = 0 \tag{6.17}$$

Considering that PR is equal to the hydraulic area

$$A = PR \tag{6.18}$$

Once P and R are calculated, the results can be checked by obtaining the mean velocity and the area of the section. From this, the discharge through the section is

$$Q = VA \tag{6.19}$$

which must be equal to the design discharge.

Another verification consists in calculating U given by eq 6.11, and then to obtain the area of $A = Q/U$, which has to be also equal to PR. If the results do not agree, the coefficients of n_a and f can be slightly modified.

6.7.2 Blench's method

Taking into account the observations done after Lacey, T. Blench, in 1939, 1941, and in books and articles of later publication, presented his basic design formulae. His method considers two parameters

Bed factor, F_b

Side factor, F_S

These parameters are a function of the suspended sediment discharge, bed particle diameter and the resistance of the bank to be eroded. The most important formulae to evaluate them are summarized here.

a) Bed factor

It takes into account the bed resistance and it is obtained by means of the expression

$$F_b = F_{bo}\ (1 + 0.012\ C) \qquad (6.20)$$

where C, is the suspended sediment concentration in ppm, by weight (Blench expressed it in parts per hundred thousands).

This formula is applicable to sandy bottoms with sediment transport where dunes have formed.

In the previous formula

$$F_{bo} = 60.1\ (D_m)^{0.5} \qquad D_m,\ \text{in m} \qquad (6.21)$$

When there is little information for F_b, Blench recommends the use of 0.8 for fine material (fine sand $D_m < 0.5$ m) and of 1.2 when it is coarse ($D_m > 0.5$ mm).

It is important to note that this bed factor corresponds to the silt factor previously proposed by Lacey.

b) Side factor

This parameter measures bank resistance and it is obtained by the expression

$$F_s = \frac{F_{bs}}{8} \qquad (6.22)$$

where

F_{bs} is obtained by means of eq 6.20, but substituting in it the mean diameter of the bank material, and only if it is sandy.

For the side factor, Blench recommends the following values

0.1 for material with little cohesion
0.2 for moderately cohesive material
0.3 for very cohesive material like clay

As there is not an exact definition of what little and very cohesive mean, the application of this method is not so simple.

6.7.2.1 Basic equations

The three basic equations proposed by Blench for stable channels are

$$F_b = 3.28\ U^2/d \tag{6.23}$$

$$F_s = 10.76\ U^3/b \tag{6.24}$$

$$\frac{U^2}{gdS} = 3.63\ \left(1 + \frac{C}{2330}\right)\left(\frac{Ub}{\nu}\right)^{1/4} \tag{6.25}$$

where b is the free surface width, in m, and ν is the kinematic viscosity, in m^2/s.

Even if Blench is in favour of the four degrees of freedom, he only proposes three equations to solve the channel stability problem.

6.7.2.2 Design equations

Starting from the basic equations, the design equations are derived, from which the geometric variables of the section and the channel slope are obtained. Those expressions are

$$b = 1.81\ \left(\frac{F_b Q}{F_s}\right)^{1/2} \tag{6.26}$$

$$d = 1.02\ \left(\frac{F_s Q}{F_b^2}\right)^{1/3} \tag{6.27}$$

$$S = \frac{0.56\, F_{bo}^{5/6}\, F_s^{1/12}\, (1 + 0.012\, C)^{5/6}}{K\, Q^{1/6}\, (1 + C/2330)} \tag{6.28}$$

where K groups the principal constants, that is

$$K = \frac{6.6\, g}{\nu^{1/4}} \tag{6.29}$$

Besides, if

$$f'(c) = \frac{1}{(1 + C/2330)} \tag{6.30}$$

the formula for the slope is

$$S = \frac{0.56\, F_b^{5/6}\, F_s^{1/12}}{K\, Q^{1/6}}\, f'(c) \tag{6.31}$$

It is not always possible to obtain the side factor F_s with precision. Therefore, Blench proposed two more equations for the slope which are equivalent to the already given. They are

$$S = \frac{0.64\, F_b^{7/8}}{K\, b^{1/4}\, d^{1/8}}\, f''(c) \tag{6.32}$$

$$S = \frac{0.606\, F_{bo}^{11/12}}{K\, b^{1/6}\, Q^{1/12}}\, f'''(c) \tag{6.33}$$

In the first, the discharge Q does not explicitly appear, even if b and d depend on that value. In the second, neither F_S nor depth d intervenes. f('(c), f''(c) and f'''(c) are obtained with the help of fig 6.2, as a function of concentration C of the suspended material.

6.7.3 Simons and Albertson's method

It was proposed in 1967 and it is one of the latest of the regime theory. It has the advantage above the others of the same type of establishing the differences between bed and side materials.

6.7.3.1 Design formulae

In the metric system the three design formulae are

a) For the mean width, in m

$$b = 1.63\, K_1\, Q^{0.5} \tag{6.34}$$

b) For the mean depth, in m

$$\frac{A}{b} = d = 1.331\, K_2\, Q^{0.36} \tag{6.35}$$

valid when R < 2.13 m and

$$d = 0.61 + 1.023\, K_2\, Q^{0.36} \tag{6.36}$$

valid when R > 2.13 m

c) For the slope the authors proposed a formula similar to the one used by Blench

$$\frac{U^2}{gdS} = K_4 \left(\frac{Ub}{\nu}\right)^{0.37} \tag{6.37}$$

In the previous formula, the meaning of some of the variables is the following

A area of the transversal section, in m^2

K_1, K_2 and K_4, coefficients. Their values are indicated in the following table

Materials	K_1	K_2	K_4
1. Sandy banks and bottom	3.5	0.52	0.33
2. Cohesive banks and sandy bottom	2.6	0.44	0.54
3. Cohesive banks and bottom	2.2	0.37	0.87
4. Banks and bed with rough material	1.75	0.23	
5. Equal to 2 with much transport	1.70	0.34	

6.8 Altunin's method

Altunin's method was developed from the observations done in channels with granular material. This method does not consider sediment transport and analyzes stability with velocities close to the critical. This results in theoretical sections larger than the real ones when applied to sandy channels. In order to make the exposition of the method easier, the concepts and fundamental formulae will be presented first and, afterwards, the design formulae.

Depending on the resistance the banks of a channel offer to be eroded, Altunin classified the sections or river reaches in:

Type a: banks very resistant to erosion, which may consist of very cohesive materials

Type b: banks with little resistance to erosion, like alluvial material without cohesion.

Besides, Altunin distinguishes three principal zones along the river length. They are

1) Mountain zone
2) Intermediate zone
3) Plain or deltaic zone

which have already been explained in 6.4.

6.8.1 Fundamental formulae

As in the other methods, and because a natural current has three degrees of freedom, Altunin also uses three fundamental equations: one which takes into account bank resistance, another, which implies the continuous motion of the bed particles, and the third, referred to flow resistance or friction. It is important to note that Altunin uses two different friction equations when it is considered that he ought to have used only one.

6.8.1.1 Bank resistance formula

This expression is due to Gluschkor and it is of the type of the regime theory. It was obtained in rivers consisting in only one channel with no islands to bifurcate it. The expression relates width to depth:

$$b^m = K\,d \tag{6.38}$$

where

- b width of the flow free surface, in m
- d mean depth. It is the relation between the cross section area and the free surface width.
- k shape coefficient, with a value between 8 and 12 for alluvial channels. As mean value, 10 is recommended. In case of rivers with banks not easily erodable (type a), the value of k lies between 3 and 4; and for those of type b, with very easy to erode banks, the values range from 16 to 20.

  These big values were obtained in hydraulic models where the material had not suffered any kind of consolidation
- m exponent which may be obtained by means of the expression

$$m = 0.72\left(\frac{\Delta D_m}{d\,S}\right)^{0.1} \tag{6.39}$$

- D_m mean diameter of bed material, in m
- S hydraulic slope
- Δ relative density of submerged grains

Exponent m may also be valued by means of the relation

$$m = 0.5 + \frac{U_s - U}{U} \tag{6.40}$$

where

- U mean velocity of the section, in m/s
- U_s surface velocity, in m/s

Altunin does not use the formula 6.38 in this way because he combines it with the following (Manning's equation)

$$Q = \frac{1}{n} b d^{5/3} S^{1/2} \tag{6.41}$$

When combining eqs 6.38 and 6.41, this relation is obtained

$$b = \left(\frac{n Q K^{5/3}}{S^{1/2}} \right)^{\frac{3}{3+5m}} \tag{6.42}$$

where it is better to group all constants in only one by means of the expression

$$A = (n K^{5/3})^{3/(3+5m)} \tag{6.43}$$

which leads to

$$b = A \left(\frac{Q}{S^{1/2}} \right)^{3/(3+5m)} \tag{6.44}$$

After comparing this expression with the data obtained, for practical design problems Altunin recommends the use of the following equation, provided $k = 10$ and $m \doteq 0.7$

$$b = A \frac{Q^{0.5}}{S^{0.2}} \tag{6.45}$$

or, as a function of the unit discharge

$$b = \frac{A^2 q}{S^{0.4}} \tag{6.46}$$

being this equation the first fundamental one used by Altunin. If more accuracy is desired, or if absurd results are obtained, it is necessary to go back to eq 6.42.

6.8.1.2 Formula to guarantee bed material motion

The current mean velocity which guarantees particle movement is obtained with

$$U = a V_{\phi} d^{\alpha} \tag{6.47}$$

where

a coefficient which, for Altunin, has a value of 1 for mountain or intermediate zones, and 1.15 for plains

V_ϕ current mean velocity for 1 m depth which excludes the possibility of erosion but guarantees particle movement. It is obtained with table 6.1, as a function of the material diameter.

α exponent with the following values

1/5, for $d \geqslant 2.5$ m
1/4, for $2.5 > d > 1.5$ m
1/3, for $d \leqslant 1.5$ m

When, during the calculation, d is bigger than the initial supposed value used to choose α, the operation should be repeated with another value of α.

The values of V_ϕ given in table 6.1 are practically the same as those proposed by other authors for the mean critical velocity, that is to say, the minimum mean velocity at which sediment transport starts.

There is also a relation between A and V_ϕ, given by

$$A = 1/(V\phi)^{1/2} \tag{6.48}$$

which is generally used to obtain A.

6.8.1.3 Resistance flow equation

As already said, Altunin uses Manning's formula, which, combined with Gluschkov's, leads to his first fundamental equation, that is eq 6.45. However, he finally proposes a frictional formula of exponential type, that is

$$U = k\, d^z\, S^x \tag{6.49}$$

where k, z and x depend on the bank and bed material. But Altunin proposes the use of his method for coarse material and, besides, he tries to consider other losses produced by bottom irregularities by means of the following values

$$k = 11$$
$$z = 1/2$$
$$x = 1/3$$

6.8.2 Design formulae

Starting from the three fundamental formulae, expressions are obtained to value the unknown characteristics of a channel, either geometrical or hydraulic. Thus, from eqs 6.47 and 6.49 the current depth is obtained

$$d = \left(\frac{a V\phi}{k S^x} \right)^{\frac{1}{z-\alpha}} \qquad (6.50)$$

This equation is applied when both bed material and slope are known, and it will give the maximum depth the river has before it tends to form islands, that is, the maximum for only one branch or channel.

Through this only channel with depth d, the unit theoretical discharge q_t = Ud can pass, and it is obtained by substituting the d resulting from the previous equation and the velocity U given by eq 6.47, thus

$$q = \left(\frac{(a V\phi)^{(1+z)}}{(k S^x)^{1+\alpha}} \right)^{\frac{1}{z-\alpha}} \qquad (6.51)$$

Due to the fact that Q = bq, the theoretical discharge that can pass in the river is

$$Q = A^2 \left[\frac{(a V\phi)^{1+z}}{K^{1+\alpha} S^{\epsilon}} \right]^{\frac{2}{z-\alpha}} \qquad (6.52)$$

where

$$\epsilon = 0.2\,(z - \alpha) + x\,(1 + \alpha) \qquad (6.53)$$

From eq 6.52 it can be concluded that the maximum formative discharge which passes through one channel is bigger when $V\phi$ is bigger too, and this is also bigger for bigger particle diameter. Besides, Q becomes smaller for a bigger S, which increases in natural channels when increasing bed material diameter. Generally, slope effect is greater because it ranges among greater values; that is why to a smaller bed particle diameter corresponds a greater formative discharge through only one channel.

Considering the theoretical discharge obtained with eq 6.52, the theoretical equilibrium slope S is

$$S = \left(\frac{A^{z-\alpha} (a\, V\phi)^{1+z}}{k^{(1+\alpha)}\; Q^{(z-\alpha)/2}} \right)^{2/\epsilon} \tag{6.54}$$

If in the previous formulae the values of k, z and x are substituted by Altunin's values for gravel and cobbles, the following expressions are obtained (they have been already tested with good results for sandy channels)

$$d = \frac{0.728\, V\phi^{10/3}}{(1000\, S)^{10/9}} \tag{6.55}$$

$$U = \frac{0.938\, V\phi^{5/3}}{(1000\, S)^{2/9}} \tag{6.56}$$

$$q = \frac{0.683\, V\phi^{5}}{(1000\, S)^{4/3}} \tag{6.57}$$

$$Q = \frac{7.394\, A^2\, V\phi^{10}}{(1000\, S)^{3.07}} \tag{6.58}$$

$$S = \frac{0.00192\, A^{15/23}\, V\phi^{75/23}}{Q^{15/46}} \tag{6.59}$$

The previous equations were obtained considering the following values: $k = 11$, $z = 0.5$, $x = 1/3$, $a = 1$, $\alpha = 1/5$ and, therefore, $\epsilon = 0.4600$.

Similar formulae are obtained for $\alpha = 1/4$ and $\alpha = 1/3$; they are shown in table 6.2.

6.8.3 Design formulae for specific problems

These formulae depend on the problem to be solved and on the available data. Thus, the following situations may present.

a) First problem

With a given bed material and design discharge, to obtain the section width and depth and the channel longitudinal slope. The design formulae for the methods presented in this chapter were obtained according to this criterion.

Therefore

$$S = \frac{0.00192\, A^{15/23}\, V_\phi^{75/23}}{Q^{15/46}} \qquad (6.60)$$

$$b = \frac{3.494\, Q^{13/23}\, A^{20/23}}{V_\phi^{15/23}} \qquad (6.61)$$

$$d = \frac{0.353\, Q^{25/69}}{V_\phi^{50/69}\, A^{20/67}} \qquad (6.62)$$

The previous formulae were also obtained for $\alpha = 1/5$ and in table 6.3 the equations for the other values of α are given. Although in this table the equations are presented in a more simple form, it is better to show the exponents of Q for each value of α, because it is then possible to compare the exponents that variable may adopt in the other methods presented. Thus, the values of the exponents are

	$\alpha = 1/5$	$\alpha = 1/4$	$\alpha = 1/3$
d	25/69	5/14	15/43
b	13/23	31/56	23/43
S	–15/46	–15/56	–15/86

which are approximately equal to the 0.333, 0.50 and 0.166, respectively proposed by Lacey for a, b and S.

b) Second problem

This problem presents when calculating rivers with several channels or branches, and the corresponding stable characteristics of one of them must be obtained considering that the terrain slope remains the same for all of them. It has to be noted that when branches join, the slope is modified. In table 6.3 the corresponding equations are presented.

c) Third problem

Finally, it may occur that from a known slope —assuming that it will not suffer any modifications—, and from the design discharge that will pass in the channel, it is desired to know the material that will make the channel stable. Therefore, the variables to be calculated are b, d and V_ϕ. Since V_ϕ is a function of bed material, its diameter will be obtained from table 6.1. Besides, table 6.3 shows the expressions needed to obtain b, d and V_ϕ.

6.9 Maza-Cruickshank's method

This method, proposed in 1973, is based on the concept of the degrees of freedom. Then, it takes into account what has been explained in 6.3 with reference to the three equations needed to obtain the width and depth of the section and the slope of a stable channel.

6.9.1 Fundamental formulae

To study the dynamic stability of a channel three equations are needed: one of resistance to flow, another referred to sediment transport, and the third, to bank resistance. Due to the fact that at present most of the formulae used by fluvial hydraulics are empiric, several of each group were tested. The most satisfactory are simple to apply and do not require the use of figures or tables. They are:

a) Flow resistance equations
   a.1) Cruickshank-Maza: eq 3.52
   a.2) Manning: eq 3.13

   Cruickshank-Maza's method distinguishes among the lower regime, the upper one and the transition between them. Because stability studies do not work with maximum discharges but with formative ones, generally the flow has a lower regime. This is why formulae are presented for this last condition.

   With the object of reducing the number of variables and the coefficients and variables with constant values or only depending on bed material, they have been grouped in one variable.

   Thus, when dealing with eq 3.52, the following is used

$$\alpha = \frac{7.85\ \omega_{50}}{D_{84}^{0.634}\ \Delta^{0.456}} \tag{6.63}$$

   So, the continuity equation takes the form

$$Q = \alpha\ b\ d^{\ 1.634}\ S^{0.456} \tag{6.64}$$

Therefore, when considering eq 3.13, Manning's formula remains as

$$Q = \frac{1}{n} b d^{5/3} S^{1/2} \quad (6.65)$$

Eqs 6.64 and 6.65 will be used afterwards.

b) Transport formulae for bed material

For this group there are two equations capable of yielding acceptable results and of extending the application of the method to a greater number of bed materials and transport conditions. Those formulae are

b.1) Meyer-Peter and Müller: eq 5.10

b.2) Engelund: eq 5.18

b.3) Shields (not explained in these notes)

Even if eq 5.10 can evaluate bed load material and eq 5.18 can do the same with total bed transport, both lead to results of the same order of magnitude while

$$\tau_* < 1.5 \quad (6.66)$$

This condition generally holds for the formative discharge of most rivers and channels.

When proceeding in this manner and in order to simplify the expression, these new variables are considered

$$\epsilon = 8(g \Delta D_m^3)^{1/2} \quad (6.67)$$

$$N = \left(\frac{n'}{n}\right)^{1.5} \frac{1}{\Delta D_m} \quad (6.68)$$

$$\beta = \frac{0.04}{\Delta^2 g^{0.5} D} \quad (6.69)$$

Taking this into account, and accepting that $Q_B = bq_B$ and $Q_{BT} = bq_{BT}$, Meyer-Peter and Müller's equation is

$$Q_B = \epsilon \, b \, (N d S - 0.047)^{1.5} \quad (6.70)$$

and that of Engelund's

$$Q_{BT} = \beta \, b \, U^2 \; (d\,S)^{1.5} = \frac{\beta \, Q^2 \, S^{1.5}}{b d^{0.5}} \tag{6.71}$$

When considering eq 6.70 two important simplifications may be done

1) If in a flow, sediment transport is very small or null, the term Q_B is eliminated, only remaining the relation which establishes the critical flow condition

$$N\,d\,S = 0.047 \tag{6.72}$$

Then, the method can be extended and applied to the design of channels without transport.

2) On the other hand, if sediment transport is very important, Nds tends to be much bigger than 0.047, and this constant value may be eliminated, then writing eq 6.70 as

$$Q_B = \epsilon \, b \, (N\,d\,S)^{1.5} \tag{6.73}$$

or

$$Q_B = E \, b \, (d\,S)^{1.5} \tag{6.74}$$

considering that

$$E = \epsilon \, N^{1.5} = \frac{8\,g^{0.5}}{\Delta} \left(\frac{n'}{n} \right)^{9/4} \tag{6.75}$$

It is important to note that when calculating $\epsilon N^{1.5}$ the terms D_m can be simplified. In this case, sediment transport does no explicitly depend on the mean diameter of the bed material.

When considering eq 6.71 it is important to remember that the transport critical condition or particle movement should be previously obtained using what is explained in Chapter 4, because, due to the structure of eq 4.71, a value of Q_{BT} will always be obtained even if that transport does not exist. Below the critical condition, instead of applying eq 6.71, a value of Q_{BT} equal to zero should be asigned, $Q_B = 0$.

c) Relation of bank resistance

After studying different possibilities, especially those of the regime theory, Gluschkov's formula was finally chosen (as already seen when referring to Altunin's method). Eq 6.38 establishes that

$$b^m = K\,b \qquad (6.76)$$

It has to be remembered that m varies from 0.5 to 1, that in case of alluvial channels a mean value of 0.7 may be considered, and that K for the same condition has a mean value of 10.

From the different methods of the regime theory several formulae of the type of eq 6.76 and the following values for m and K are obtained. So from Lacey

$$m = 0.667$$

$$K\ (\text{mean}) = 10.65$$

with Blanch's method m and k have the values

$$m = 0.667$$

$K = 6.76$ to 12.52 when F_B takes the values 1 or 2, respectively

with Simons and Albertson's

$$m = 0.7$$

$$K = \begin{cases} 10.2 & \text{for sandy banks and beds} \\ 6.3 & \text{for sandy beds and cohesive banks} \\ 7.0 & \text{for gravel} \end{cases}$$

Finally, in the laboratory, Stebbings found the following values for sand with $D_{98} = 1.2$ mm and $D_7 = 0.6$ mm

m	=	0.7	in all the sections
K	=	8.2	in the sections with very little sediment transport
k	=	25.8	in the sections with maximum sediment transport (before isle formation)

It can be seen that the values given for all methods are rather similar, being Gluschkov's the one which permits greater variations.

These fundamental equations lead to the following group of equations

Formulae

Group	Friction	Transport	Resistance	Utility
I	Manning	Meyer-Peter & Müller (bed load transport)	Gluschkov	For channels with granular material with or without transport
II	Cruickshank	Engelund (total bed transport)	Gluschkov	Sandy channels with sediment transport
III	Cruickshank-Maza	Meyer-Peter & Müller (bed load transport)	Gluschkov	Sandy rivers with any transport condition

A useful combination for a larger size range of bed materials is that of group I. Even if it works for any kind of material with or without sediment transport, it requires the adjustment of the roughness coefficient. When the solid transport is almost zero or very big, Meyer-Peter and Müller's equation may be simplified, thus facilitating the later handling of the design equations. When this is not possible, the design equations are implicit for b, d and S, and their solution is obtained through trial and error. When $\tau_* > 1.5$, it is better to use only the formulae of group II because part of the transport is in suspension and the Meyer-Peter and Müller formula does not consider this condition. The only explicit equations are those of group II, which are useful in case of sands and when there is sediment transport.

Most channels in the plain may be studied with the equations of group III, because they are useful when sediment transport is reduced and the material is sandy. Even if they may be applied for all transport values they may lead to implicit formulae, unless the transport tends to zero or it is very big.

Groups II and III have an additional advantage over group I, because in this case the friction formula does not require the estimation or assumption of the friction coefficient.

It has to be noted that in case of gravels or big diameter particles and plane bottoms, coefficient n may be obtained from a semiempiric equation (see Chapter 3).

6.9.2 Group I. Design equations for granular material and all sediment transport conditions

When applying the three fundamental formulae of group I (Manning, Meyer-Peter-Müller and Gluschkov's), the following general design equations are obtained: one, for width b; another, for depth d and the third, for slope S. They are

$$b^{(7m+4)/3}[Q_B^{2/3} + 0.047\ (\epsilon\ b)^{2/3}] = (n\ Q)^2\ N\ \epsilon^{2/3}\ K^{7/3} \tag{6.77}$$

$$d^{(7m+4)/3m}[Q_B^{2/3}\ K^{4/3} + 0.047\ \epsilon^{2/3}\ d^{2/3m}\ K^{2/m}] = (n\ Q)^2\ N\ \epsilon^{2/3} \tag{6.78}$$

$$S^{-(7m+4)/(10m+b)}[Q_B^{2/3} + 0.047\ \epsilon^{2/3}\ (nQK^{5/3})^{2/(5m+3)}\ S^{-1/(5m+3)}] =$$

$$= \frac{\epsilon^{2/3}\ N(nQK^{5/3})^{(3m+2)/5m+3}}{K} \tag{6.79}$$

These formulae are implicit for all the variables. But, even if their solution is simple, they have the disadvantage of not showing the way each parameter affects the final result.

6.9.2.1 Group I. Condition with small or null sediment transport

When in the stretch under study $Q_B \rightarrow 0$, the term may be eliminated from the general equations, thus simplifying the final expressions

$$b = (4.613\ n\cdot Q\ K^{7/6}\ N^{1/2})^{6/(7m+6)} \tag{6.80}$$

$$d = (4.613\ n\ Q\ N^{1/2}/K^{1/m})^{6m/(7m+6)} \tag{6.81}$$

$$S = [0.047\ K^{3/(5m+3)}/N\ \ (nQ)^{3m/(5m+3)}]^{(10m+6)/(7m+6)} \tag{6.82}$$

If in the previous equations it is accepted that m = 0.7, which is the mean value for alluvial channels, the exponents may be easily compared with those of the formulae proposed by other authors, thus

$$b = 2.32\ (n\ Q)^{0.55}\ \ K^{0.64}\ \ N^{0.28} \tag{6.83}$$

$$d = 1.8\,(n\,Q)^{0.385}\,N^{0.193}/\,K^{0.55} \tag{6.84}$$

$$S = 0.0261\,K^{0.55}/(n\,Q)^{0.385}\,N^{1.193} \tag{6.85}$$

6.9.2.2 Group I. Condition with high sediment transport

In case that in the stretch under study the sediment transport is high, corresponding to $\tau_* = dS/\Delta\, D_m > 0.5$, the term 0.047 of the Meyer-Peter and Müller's formula can be neglected. Then, eq 6.74 is obtained where, as already said, diameter D_m does not intervene in the value of the sediment transport. This equation can also be written as

$$Q_B = Eb\,(d\,S)^{1.5}$$

All this leads to the following explicit design equations

$$b = [n\,Q\,K^{7/6}\,(\epsilon/Q_B)^{1/3}]^{6/(7m+4)} \tag{6.86}$$

$$d = (1/K)\,[n\,Q\,K^{7/6}\,(\epsilon/Q_B)^{1/3}]^{6m/(7m+4)} \tag{6.87}$$

$$S = [(Q_B/E)^{2/3}/(n\,Q\,K^{1/(6m+6)})^{(3m+2)/(5m+3)}]^{(10m+6)/(7m+4)} \tag{6.88}$$

Assuming that m = 0.7, it is finally obtained

$$b = (n\,Q)^{0.674}\,K^{0.787}\,(E/Q_B)^{0.225} \tag{6.89}$$

$$d = (n\,Q)^{0.472}\,(1/K)^{0.449}\,(E/Q_B)^{0.157} \tag{6.90}$$

$$S = Q_B^{0.974}/E^{0.974}\,K^{0.075}(n\,Q)^{0.921} \tag{6.91}$$

6.9.3 Group II. Design equations for sandy channels and sediment transport

In this group the basic equations are those of Cruickshank-Maza for friction, Engelund's for transport, and that by Gluschkov for bank resistance. From them, the design equations for lower or superior regime can be obtained, even if here only those referred to the lower regime are presented. They are

$$b = (\beta^{0.304}\ Q^{1.608}\ K^{1.786}/\alpha\ Q_{BT}^{0.304})^{1/(1.786m+1.304)} \tag{6.92}$$

$$d = (\beta^{0.304}\ Q^{1.608}/K^{1.304/m}\ \alpha Q_{BT}^{0.304})^{m/(1.786m+1.304)} \tag{6.93}$$

$$S = [Q_{BT}^{(1.224+m)}\ K^{1.388}/\alpha^{(1.224+0.612m)}\ \beta^{(1.224+2m)}\ Q^{(1.224+3.388m)}]^{1/(3.279m+2.374)} \tag{6.94}$$

If in the previous formulae it is accepted that m = 0.7, it is obtained

$$b = 0.308\ D_{84}^{0.247} K^{0.7}\ Q^{0.63}/\ \{\omega_{50}^{0.39}\ (\Delta g)^{0.06}\ (D_{35}\ Q_{BT})^{0.119} \tag{6.95}$$

$$d = 0.439\ D_{84}^{0.174}\ Q^{0.441}/\{\omega_{50}^{0.274}\ (\Delta g)^{0.042}\ K^{0.51}\ (D_{35} Q_{BT})^{0.083} \tag{6.96}$$

$$S = 2.967 Q_{BT}^{0.56}\ K^{.296}\ D_{84}^{0.223}\ \Delta^{1.278}\ g^{0.280}\ D_{35}^{0.56}/\omega_{50}^{0.352}\ Q^{0.767} \tag{6.97}$$

When in the formulae of this group sediment transport tends to zero, absurd results are obtained.

6.9.4 *Group III. Design equations for sandy channels with all sediment transport conditions*

When working with sandy channels the formulae of this group may be applied; they are those of Cruickshank-Maza, Meyer-Peter and Müller and Gluschkov. Their advantage over group II lies in the fact that they can be used even if sediment transport tends to zero.

However, if the value of sediment transport does not permit the following simplifications, implicit equations have to be used; this is not so with those of group II, thus, it is better to work with the equations shown in 6.9.4.1, where $Q_B \rightarrow 0$.

The general design formulae are

$$b^{(2.583m+1.526)}\ (Q_B^{2/3} + 0.047\,(\epsilon b)^{2/3} = E^{2/3}\ K^{2.588}\ (Q/\alpha)^{2.193} \tag{6.98}$$

$$d = b^m/\ K \tag{6.99}$$

$$Q_B^{2/3} = E^{2/3}\ K^{0.0893/W}\ (Q/\alpha)^{(2+3m)/3W}\ S^{(1.178m+0.696)/W} -$$

$$- 0.047\ \epsilon^{2/3}\ (Q/\alpha)^{2/3W}\ K^{1.089/W}\ S^{-0.304W} \tag{6.100}$$

where

$$W = 1 + 1.634\,m \tag{6.101}$$

6.9.4.1 Group III. Condition with null or very small sediment transport

Proceeding in a similar form to that described in 6.9.2.1, we arrive at the following design equations

$$b = [21.277\, N K^{2.583}\, (Q/\alpha)^{2.393}]^{1/(2.58m+2.193)} \tag{6.102}$$

$$d = b^{m}/K \tag{6.103}$$

$$S = [0.047\, K^{1/W}/N\ (Q/\alpha)^{m/w}]^{w/(1.178m+1)} \tag{6.104}$$

Accepting that m = 0.7, it is obtained

$$b = 2.147\, N^{0.25}\, K^{0.646}\, (Q/\alpha)^{0.548} \tag{6.105}$$

$$d = 1.701\, N^{0.175}\ (Q/\alpha)^{0.384}/K^{0.548} \tag{6.106}$$

$$S = 0.0275\, K^{0.548}\, (\alpha\ /Q)^{0.384}/N^{1.175} \tag{6.107}$$

6.9.4.2 Group III. Condition with large sediment transport

From the engineering point of view, when $\tau_* > 0.5$, the term 0.047 of eq 6.74 can be eliminated, and the following equations are obtained

$$b = [(QK/\alpha)^{2.193}\, (E/Q_B)^{2/3}]^{1/(2.583m+1.526)} \tag{6.108}$$

$$d = b^{m}/K \tag{6.109}$$

$$S = [Q_B^{2/3}/E^{2/3}\, K^{0.0893/W}\, (Q/\alpha)^{(2+3m)/3W}]^{W/(1.178m+0.696)} \tag{6.110}$$

Accepting m = 0.7

$$b = (QK/\alpha)^{0.658}\ \ (E/Q_B)^{0.2} \tag{6.111}$$

$$d = (Q/\alpha)^{0.461} \; (E/Q_B)^{0.14} / K^{0.539} \tag{6.112}$$

$$S = (Q_B/E)^{0.94} / K^{0.06} \; (Q/\alpha)^{0.9} \tag{6.113}$$

6.9.5 Summary and comments

When studying the stability of river and channels with granular beds where the particle mean diameter is bigger than 2 mm, the equations of group II may not be applied because the Engelund's formula (eq 6.6.4) is useful only in case of sands. Due to the fact that the eq 3.5.2 has not been tested for materials with mean diameter greater than 8 mm, its application is not recommended in this case.

Depending on the transport intensity the equations of grup II can not be used when $\tau_* < 0.1$. Then only those of groups I or III should be applied because with the Meyer-Peter and Müller's formula the critical condition without sediment transport can be obtained.

When sediment transport is very small, the results of the method described (groups I or III) are very similar to those obtained with Altunin's. This is quite logic because he uses as a fundamental equation a formula for the beginning of particle movement.

6.9.6 Application of stability formulae

In contrast to the theories like that of the regime and Altunin's, the method described in this chapter is the only one which takes into account the upstream sediments that pass through the stretch under study. Besides other aspects, this permits to predict the changes of section and slope a stream will suffer as a consequence of channel modifications and alterations in the quantity and quality of sediment like, for instance, when they accumulate because of deforestation or they reduce due to the presence of a dam.

In spite of the fact that the principal problem is to know the amount of sediments a stream transports, there is another one which may present serious difficulties: it refers to the liquid discharge.

When near the area under study there is a gaging station, Q and Q_B can be easily obtained through a numeric calculation. But if there are no stations, the inversal procedure may be adopted, that is to say, to choose stretches where, near the area under study, the river is formed by only one channel, and from it to obtain d, b, S and a. From these values, the formative discharge Q and the sediment transport Q_B can afterwards be calculated.

When the natural condition of a river is altered, several problems arise, which do not require an exact knowledge of sediment transportation, because the relation between solid and liquid discharges does not generally vary or, at the most, only changes very slightly. Then, eqs 6.92 to 6.97 can predict the alterations in the river geometry produced by the engineering works, thus making it very easy to analize the future behaviour of all parameters when modifying one of them.

The results here presented presuppose certain conditions which may not hold. Besides, and due to the simplifications adopted, the results obtained with the following formulae are only approximate but may give the civil engineer an idea of the future tendencies of the river.

Using the formulae of group II, the following paragraphs show the simplified solution to problems where the values of several parameters are not known.

6.9.6.1 River rectifications

When a river which meanders and curves is rectified, its length diminishes, consequently increasing its slope. In this case, Q, Q_B and K are kept constant before and after the process.

Besides, in many channels it is possible to guarantee that the bed material characteristics also remain constant (W_{50}, D_{35}, D_{84}); however, if channels carry particles of different size like sands and gravels, when S is artificially increased there is a tendency towards increasing the size of the bed material through a process of natural selection.

If the material is rather uniform, remaining then practically constant before and after the rectification which has increased the slope, it can be observed that

a) When the banks are not protected, the river will tend to regain the original slope, developing the already existing meanders or creating new ones. This occurs along the rectification or in the upstream stretch of the river close to it.

b) When the exterior banks of the curve in the zone to be rectified are protected, the river will tend to regain its equilibrium slope, eroding the bottom upstream the rectification and depositing the sediments downstream.

c) Depending on the real resistance of the banks and the bottom to erosion, intermediate situations between a and b may present.

While the equilibrium slope is achieved —process which may imply several years—, b and d are modified. As a function of eqs 6.95 to 6.97, the approximate width and depth values for a new slope can be found, starting from the hypothesis that before and after the rectification the annual hydrograph and the volume of the transported material do not change. Thus

$$b_1 = b_o (S_o/S_1)^{0.21} \tag{6.114}$$

$$d_1 = d_o (S_o/S_1)^{0.148} \tag{6.115}$$

Subindexes 0 and 1 are indicative of the characteristics before and after the rectification.

6.9.6.2 Channels with two degrees of freedom

If a river passes through a very populated area or if it is to be improved with navigation purposes, sometimes both banks have to be protected by means of groins or revetments to avoid erosion or the lateral movement of the river. The width of these works and that of channelling is generally smaller than that of equilibrium; therefore, this river stretch has a smaller degree of freedom; this means that only two equations are necessary to know its slope and depth. One corresponds to flow resistance and, the other, to sediment transport. From them, it is obtained

$$d = \frac{\beta^{0.17}\, Q^{0.9}}{\alpha^{0.55}\, b_1^{0.73}\, Q_{BT}^{0.17}} \tag{6.116}$$

$$S = \frac{b_1^{0.426}\, Q_{BT}^{0.611}}{\alpha^{0.185}\, \beta^{0.611}\, Q} \tag{6.117}$$

where b_1 is the new width artificially fixed. It can be seen that when b decreases, depth increases and the slope tends to diminish. That is why, upstream of the channelling, the bottom descends while, at the downstream end, it tends to go upwards. Even if the last effect is harmful because it reduces the hydraulic capacity of the river, it may be eliminated by the adequate dredging of the portion downstream of the protection.

6.9.6.3 Modifications due to big dams

When building a dam of great reservoir capacity, the passing of sediments towards the downstream reaches is interrupted and the hydrograph of the current is altered. As a function of the alteration produced, downstream of the dam two reaches of the river may be distinguished

a) The first locates immediately downstream of the main dike: the bottom descends and the slope diminishes because the particles carried by the water are not substituted by others, remaining all of them in the reservoir; consequently, with time, the channel will acquire a static stability. With every discharge of the spillway, the river becomes more stable, beginning from the sections nearer to the dam. Both, the behaviour of the river in this zone and the calculation of the bottom erosion are explained in point 7.5.

b) The second stretch, which comprehends up to the point where the first important affluent converges downstream of the river, corresponds to the reach where the sediment transport acquires new stability and there are no alterations in the levels of the bottom, existing, however, a variation in the annual hydrograph. Of course, the solid discharge changes as a function of the liquid discharge. If there is more than a dam devoted to irrigation and not to the generation of electricity, the discharge alterations will be even greater.

In a big dam downstream of the stretches, where there is not the bed erosion explained in point a, it may be supposed that the bed material characteristics and K remain constant before and after the construction of the dam, and that only Q and Q_{BT} vary.

With eqs 6.95 to 6.97, the width, the depth and the slope of the new channel can be obtained as a function of the stable characteristics of the river before the dam is built

$$b_1 = b_0 \left(\frac{Q_1}{Q_0} \right)^{0.627} \left(\frac{Q_{BT0}}{Q_{BT1}} \right)^{0.118} \qquad (6.118)$$

$$d_1 = d_0 \left(\frac{Q_1}{Q_0} \right)^{0.439} \left(\frac{Q_{BT0}}{Q_{BT1}} \right)^{0.083} \quad (6.119)$$

$$S_1 = S_0 \left(\frac{Q_0}{Q_1} \right)^{0.56} \left(\frac{Q_{BT1}}{Q_{BT0}} \right)^{0.768} \quad (6.120)$$

In these expressions, subindexes 0 and 1 indicate, respectively, the conditions before and after the construction of the dam.

It is to be noted that, when Q diminishes, the relation Q_1/Q_0 is less than one; however, the relation Q_{BT0}/Q_{BT1} reaches values bigger than one but, as it has smaller exponents, the decrement Q_1/Q_0 is predominant; therefore, the depth and the width decrease after the river alteration. This reduces considerably the river hydraulic capacity, being this a fact which has to be taken into account to avoid serious floodings when discharging the spillway, above all if this is done many years after the dam is built.

Slope alterations may take place especially if the river follows a new and rather sinous course, generally between the banks of the original flow

a) Constant slope before and after the construction of the dam. If this is the case, in the previously mentioned stretch S_1 and S_0 are equal and, therefore, from eq 6.120 it is obtained

$$Q_{BT1} = Q_{BT0}(Q_1/Q_0)^{1.37} \quad (6.121)$$

Taking this result into account, the width and the depth of the flow will tend towards the following values

$$b_1 = b_0 \ (Q_1/Q_0)^{0.465} \quad (6.122)$$

$$d = d_0 \ (Q_1/Q_0)^{0.325} \quad (6.123)$$

6.9.6.4 Watershed deforestation

When deforesting a watershed, the volume of the sediments which arrives to the river increases. To know the geometric characteristics of the channel, considering that there is an equilibrium between the volume of sediments which gets into the river and the volume which it is capable of transporting, eqs 6.95 to 6.97 can also be used. Assuming that $Q_1 = Q_0$, they adopt the form

$$b_1 = b_0 \, (Q_{BT0}/Q_{BT1})^{0.218} \tag{6.124}$$

$$d_1 = d_0 \, (Q_{BT0}/Q_{BT1})^{0.083} \tag{6.125}$$

$$S_1 = S_0 \, (Q_{BT1}/Q_{BT0})^{0.56} \tag{6.126}$$

The assumption that the material which gets to the river is similar to the already existing on its bottom before the deforestation may, in some cases, result in a very coarse simplification.

It has to be remembered that Q_{BT1} is always bigger then Q_{BT0} and, therefore, S_1 will always be bigger then S_0. That is why some reaches of the river may become completely clogged with sediments.

6.10 References

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TABLE 6.1 VELOCITY V_ϕ AS A FUNCTION OF MEAN PARTICLE DIAMETER WHEN DEPTH IS 1.00 m

Diameter, in mm	V_ϕ in m/s	Diameter, in mm	V_ϕ in m/s
		46	1.44
		48	1.47
		50	1.50
1.0	0.60	52	1.54
2.5	0.75	54	1.56
5	0.80	56	1.59
10	0.83	58	1.62
15	0.86	60	1.65
20	0.90	65	1.69
25	0.98	70	1.73
30	1.04	75	1.76
32	1.11	80	1.80
34	1.17	85	1.84
36	1.24	90	1.88
38	1.29	95	1.91
40	1.35	100	1.95
42	1.38	150	2.40
44	1.41	200	2.60

TABLE 6.2 FORMULAE TO DESIGN STABLE CHANNELS WITH SANDY OR GRAVELY BEDS ($D > 1$ mm)

Variables	$\alpha = 1/5$	$\alpha = 1/4$	$\alpha = 1/3$
Mean depth, d	$\dfrac{0.732\ V_\phi^{10/3}}{(1\,000\ S)^{10/9}}$	$\dfrac{0.685\ V_\phi^{4}}{(1\,000\ S)^{4/3}}$	$\dfrac{0.565\ V_\phi^{6}}{(1\,000\ S)^{2}}$
Mean velocity, U	$\dfrac{0.939\ V_\phi^{5/3}}{(1\,000\ S)^{2/9}}$	$\dfrac{0.909\ V_\phi^{3}}{(1\,000\ S)^{1/3}}$	$\dfrac{0.835\ V_\phi^{3}}{(1\,000\ S)^{2/9}}$
Discharge per unit width, q	$\dfrac{0.686\ V_\phi^{5}}{(1\,000\ S)^{4/3}}$	$\dfrac{0.623\ V_\phi^{6}}{(1\,000\ S)^{5/3}}$	$\dfrac{0.471\ V_\phi^{9}}{(1\,000\ S)^{8/3}}$
Discharge in one branch, Q	$\dfrac{7.46\ A^2 V_\phi^{10}}{(1\,000\ S)^{3.07}}$	$\dfrac{6.20\ A^2 V_\phi^{12}}{(1\,000\ S)^{3.73}}$	$\dfrac{3.50\ A V_\phi^{18}}{(1\,000\ S)^{5.73}}$
Slope, S	$\dfrac{0.00192 A^{0.653}\ V_\phi^{3.26}}{Q^{0.326}}$	$\dfrac{0.00163 A^{0.563}\ V_\phi^{3.21}}{Q^{0.268}}$	$\dfrac{0.00123 A^{0.31}\ V_\phi^{3.15}}{Q^{0.154}}$
Non erodible velocity, V_ϕ	$\dfrac{6.85\ Q^{0.10} S^{0.307}}{A^{0.2}}$	$\dfrac{7.40\ Q^{0.083}\ S^{0.312}}{A^{0.175}}$	$\dfrac{8.45\ Q^{0.049}\ S^{0.317}}{A^{0.10}}$

TABLE 6.3 DESIGN FORMULAE FOR STABLE CHANNELS WITH GRAVELLY RIVER BEDS*

Variable	α = 1/5	α = 1/4	α = 1/3
	First problem	Data: Q, V_ϕ, A	
S	$\frac{0.00192\ A^{0.653} V_\phi^{3.26}}{Q^{0.326}}$	$\frac{0.00163\ A^{0.563} V_\phi^{3.21}}{Q^{0.268}}$	$\frac{0.00123\ A^{0.31} V_\phi^{3.15}}{Q^{0.154}}$
B	$AQ^{0.5}/S^{0.2}$	$AQ^{0.5}/S^{0.2}$	$AQ^{0.5}/S^{0.2}$
q	Q/B	Q/B	Q/B
d	$(q/V_\phi)^{5/6}$	$(q/V_\phi)^{4/5}$	$(q/V_\phi)^{3/4}$
	Second problem	Data: S, V_ϕ, A	
d	$\frac{0.732\ V_\phi^{10/3}}{(1\ 000S)^{10/9}}$	$\frac{0.685\ V_\phi^{4}}{(1\ 000S)^{4/3}}$	$\frac{0.565\ V_\phi^{6}}{(1\ 000S)^{2}}$
q	$V_\phi d^{6/5}$	$V_\phi d^{5/4}$	$V_\phi d^{4/3}$
B	$A^2 q/S^{0.4}$	$A^2 q/S^{0.4}$	$A^2 q/S^{0.4}$
Q	q·B	q·B	q·B
	Third problem	Data: Q, S. A	
V_ϕ	$\frac{0.817Q^{0.1}(1\ 000S)^{0.307}}{A^{0.2}}$	$\frac{0.855Q^{0.083}(1\ 000S)^{0.312}}{A^{0.176}}$	$\frac{0.943Q^{0.049}(1\ 000S)^{0.317}}{A^{0.10}}$
B	$AQ^{0.5}/S^{0.2}$	$AQ^{0.5}/S^{0.2}$	$AQ^{0.5}/S^{0.2}$
q	Q/B	Q/B	Q/B
d	$(q/V_\phi)^{5/6}$	$(q/V_\phi)^{4/5}$	$(q/V_\phi)^{3/4}$

* In some cases these equations are also applicable to sandy river beds. V_ϕ values are taken from table 6.1

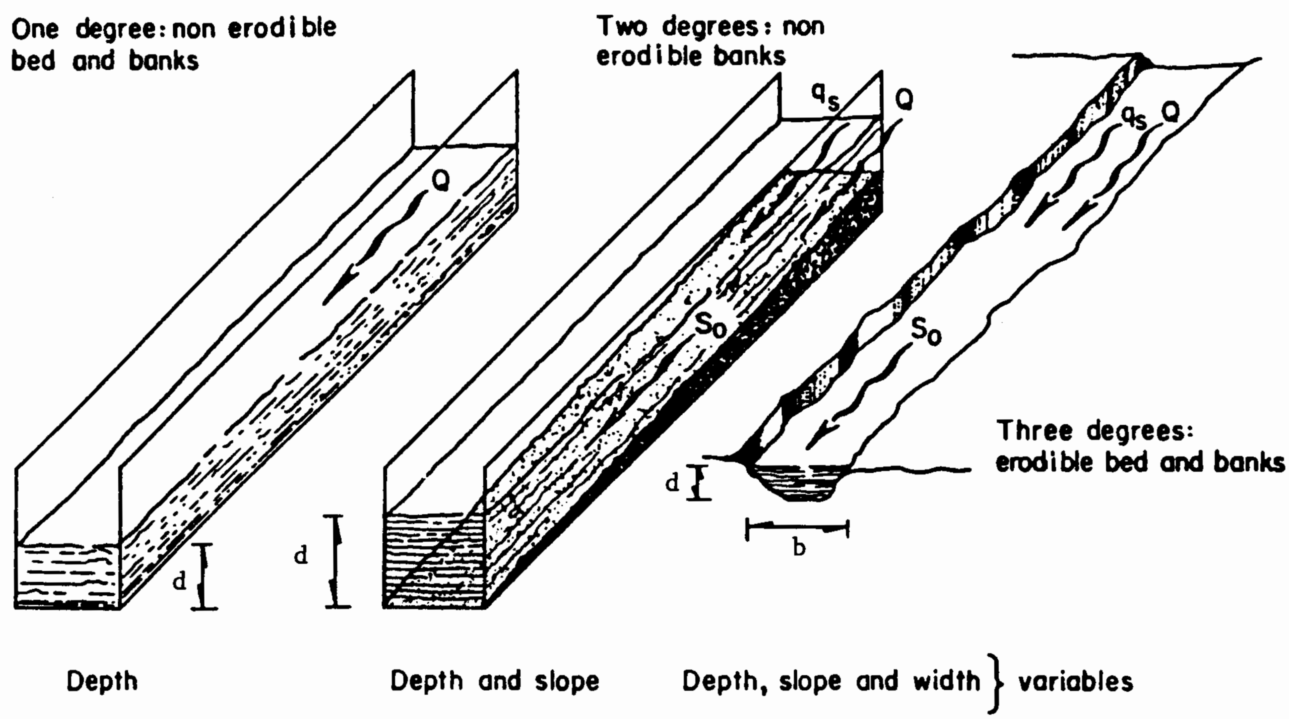

Fig 6.1
Degrees of freedom

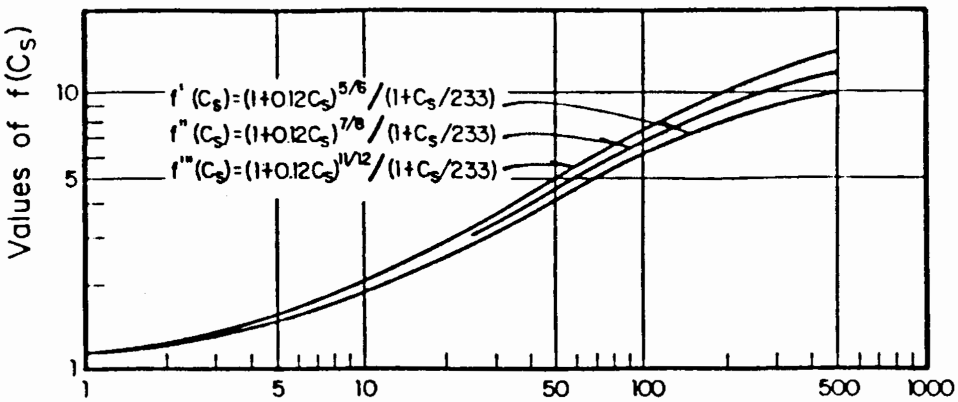

Fig 6.2
Values of $f'(C_S)$, $f''(C_S)$ and $f'''(C_S)$ as a function of the concentration C_S

CHAPTER 7

SCOUR IN RIVER BEDS

The fluvial and the civil engineer who design works in rivers like bridges, pumping stations, inlet works, etc., need to know and evaluate the possible scours and erosions capable of affecting their structures. The following are some of the most important

a) General scour

b) Transversal scour

c) Scour in bends

d) Local scour
   d.1 in piers
   d.2 in abutments

e) Erosion downstream of big dams

f) At the end of spillways and of the discharges of hydraulic works

g) Scour below pipes

Each of them is described here and, afterwards, an empiric method for their evaluation is presented.

a) General scour is produced by bottom descent as a consequence of the greater capacity of the flow to carry suspended bed material during floods; to the different capacity every reach has of transporting sediments during floods and to the amount of sediments which enters in each of them under those circumstances. Therefore, human factors do not take part in the process.

b) The transversal scour is produced when the cross section of a natural bed is in part reduced naturally or as a consequence of an engineering work like, for instance, the accesses to bridges and abutments. Width reduction is balanced with an increase in depth up to the point in which a liquid and sediment discharge continuity is attained.

c) The scour in bends is produced close to their exterior bank due to the helicoidal flow which forms when the river changes its direction. Even if it is not produced by man it may increase when protecting a bend by means of a revetment.

d) The local erosion takes place at the foot of structures interposed to currents like, for instance, at the foot of piles, abutments, groins, etc. In this kind of erosion, two different types may be distinguished: structures surrounded by flow (d.1. scour in piers) and structures close to the bank (d.2. scour in abutments).

e) The erosion downstream of large dams consists in the gradual descent of the bed produced by the constant particle transport in the downstream direction. These particles are not replaced because most sediments remain in the reservoir.

f) The erosion at the foot of discharge works is produced by the great energy the flow has in those zones, which is afterwards dissipated by the resulting turbulence, making particles raise. As the erosion develops, a thick layer of water forms between the falling water and the bed, until particles are no longer lifted.

g) The scour under pipes will be considered at the end. Even if this type of erosion is also local, it must be dealt with in a different way.

Generally, transversal scour and scour in bends exist in nature, and are independent of manmade structures in river beds. The other previously mentioned erosions, including transversal scour, are a consequence of the works which affect bed stability.

Here, some calculation methods to quantify the different types of erosions are presented, being most of them empirical.

7.1 General scour

The determination of the bottom descent produced by the general scour becomes a very important value when a bridge, an aqueduct, oilduct, etc., are to be built under the bottom. Due to the fact that the most usual of these works is bridge construction, the calculation procedure explained afterwards includes all the intervenent parameters and coefficients of the section under study.

For this process the application of Lischtvan-Lebediev's method is recommended because it determines the equilibrium condition of the flow between the mean flow velocity and the mean velocity required to erode a material of a given density and diameter. This method can be applied whether the subsoil material distribution is homogeneous or heterogeneous, considering that this last condition takes place when the material is formed by layers of different materials.

Subsoil material distribution is considered homogeneous when there is only one type of material in all the depth produced by the general scour. It is heterogeneous when more than one material forms layers in the depth reached by the scour.

The general equilibrium condition is given by

$$U_r = U_e \tag{7.1}$$

where

U_r real mean flow velocity, in m/s

U_e mean velocity needed for the flow to erode bed material (incipient motion point), in m/s

To apply these methods the following data are required:

- Maximum design flow, Q_d.

- Water elevation of the previous flow in the cross section of the river under study.

- Cross section of the reach or section under study obtained during the previous dry season.

- If soil is granular, bed material granulometry is required since mean diameter D_m is calculated from it.

— If the soil is cohesive, the volumetric weight γ_v of a dry sample is obtained by dividing the dry sample weight between the non altered volume it had when it was gathered.

7.1.1 Calculation of U_r

The basic hypothesis is that a unit discharge passing by any unitary width of a section remains constant during the erosion process.

The variation in the mean velocity of flow U_r as a function of depth and for each point of the section may be obtained through the analysis of a vertical strip of the cross section (see fig 7.1). To perform the calculation, the hypothesis that during the erosion process the unit flow through each strip is constant is postulated.

According to Manning the flow through a section of thickness ΔB is

$$\Delta Q = V \Delta A = \frac{1}{n} S^{1/2} d_o^{5/3} \Delta B \qquad (7.2)$$

Where S is the hydraulic gradient. Since a constant roughness across the whole section has been considered, $(1/n)\, S^{1/2}$ is constant for any point and is designed as α

$$\Delta Q = \alpha\, d_o^{5/3} \Delta B \qquad (7.3)$$

The value of α may be expressed in a general form as a function of the mean depth of the flow before scour d_m, the mean velocity in the section U_r, and the design flow Q_d, since

$$Q_d = \frac{1}{n} S^{1/2} d_m^{5/3} B_e \qquad (7.4)$$

Since the flow causes turbulence close to piers and abutments, it is necessary to modify the value of Q_d, by a contraction coefficient μ, which is shown in table 7.1

$$Q_d = \frac{\mu}{n} S^{1/2} d_m^{5/3} B_e \qquad (7.5)$$

$$Q_d = \mu\, \alpha\, d_m^{5/3} B_e \qquad (7.6)$$

in which

$$\alpha = \frac{Q}{\mu\, d_m^{5/3}\; B_e} \tag{7.7}$$

Now, in the strip under study, when d_o increases to any value d_s, the velocity decreases to a new value U_r, and Q in the strip of thickness ΔB is expressed as a function of the velocity and depth as follows

$$\Delta Q = U_r\, d_s\, \Delta B \tag{7.8}$$

so that, substituting in eq 7.3

$$U_r\, d_s\, \Delta B = \alpha\, d_o^{5/3}\; \Delta B \tag{7.9}$$

from which the actual flow velocity may be derived

$$U_r = \frac{\alpha\, d_o^{5/3}}{d_s} \tag{7.10}$$

In eqs 7.2 to 7.10 the meaning of the variables is as follows (see fig 7.1)

d_o initial depth existing in a predetermined vertical line of the section between the water level when the flood passes and the bottom level registered during the dry season, in m

d_s depth after bed scouring occurs. It is measured from the water level when a flood passes to the eroded bottom level, in m

α coefficient deduced from the data at hand

B_e effective channel width, substracting all obstacles, in m. To find B_e (see figs 7.2 and 7.3), a perpendicular line to that of the current is plotted. All obstacles are projected on this line and B_e is reached by adding all free spaces. It is very important to know B_e when there is a bridge in the section under study. When there are no obstacles, $B_e = B$

d_m mean channel depth as the result of dividing the hydraulic area by the effective width, in m

$$d_m = \frac{A}{B_e} \tag{7.11}$$

μ coefficient which takes into account the contraction effect produced by bridge piers; its value is found in table 7.1 as a function of mean flow velocity and the span between piers when there is a bridge.

The proposed value of U_r (eq 7.10) is fulfilled whenever the section width remains constant during the time a flood lasts, and the bed descends uniformly over the section width. The first condition is generally fulfilled, while the second one holds only in river reaches where the main bed width is completely covered by water during the dry season.

For rivers in which the channel width during the dry season is smaller than that of the rainy season, islands or uncovered zones appear and vegetation grows on them, thus increasing roughness and decreasing velocity. As results, during the flood season the main discharge tends to run on the channel of the dry season. When this happens, flow velocity increases in that part of the section, and diminishes everywhere else. Zones with higher velocity produce more scour than expected.

Because of all this, where bridges stand, it is important to keep the flood channel free of vegetation in a length equal to one or two times the river width upstream and one half downstream.

7.1.2 Calculation of U_e

The minimum velocity needed to move bed material depends on the nature of the materials the load is composed of.

For non cohesive or granular soils

$$U_e = 0.68 \beta D_m^{0.28} d_s^x \tag{7.12}$$

For cohesive soils

$$U_e = 0.60 \beta \gamma_d^{1.18} d_s^x \tag{7.13}$$

Flow velocity U_e, which is calculated with eqs 7.12 and 7.13, is very similar to that required to start particle transport.

In the previous equations

β coefficient which takes into account the return period for the design flow. Its value is found in table 7.2

x variable exponent with a different value for each equation. In the equation for non cohesive soils, its value depends on D_m, in mm; in that for cohesive soils it depends on volumetric weight γ_v, in ton/m³. Its values are given in table 7.3.

7.1.3 Calculation of scour d_s for homogeneous soils

Once the soil type at the site is known, and assuming that the roughness is constant for all the section, the scouring depth is obtained by making U_e and U_r equal. So, the following equations are obtained.

For granular soils, using eqs 7.10 and 7.12

$$d_s = \left| \frac{\alpha\, d_o^{5/3}}{0.68\, \beta\, D_m^{0.28}} \right|^{1/(1+x)} \tag{7.14}$$

For cohesive soils, using eqs 7.10 and 7.13

$$d_s = \left| \frac{\alpha\, d_o^{5/3}}{0.68\, \gamma_v^{1.18}\, \beta} \right|^{1/(1+x)} \tag{7.15}$$

When a reach is small and has no vegetation or when, due to the absence of flood channels, it is used both for dry and flood seasons, it is said that the section has uniform roughness.

Eqs 7.14 or 7.15 should be applied in several vertical lines of a section. In each of them a depth d_s is obtained as a function of the initial depth d_o. All obtained points are joined to find the theoretical profile of the scoured section (see fig 7.4).

7.1.4 Calculation of scouring d_s, for heterogeneous soils

Heterogeneous soils are made of two or more materials (which may be very different) and are found in layers. They may probably consist of a mixture of strata some of which are cohesive and others non cohesive, in different distributions. Sometimes only granular materials are found, being the differences in the diameters, densities, etc. No matter the type of stratification, the equilibrium depth may be analytically obtained either by a trial and error method or a semigraphical one.

a) Analytical or trial and error method
If the stratified material distribution under a vertical line is available, we can choose the upper layer and, according to the nature of the material, eqs 7.14 or 7.15 is applied (see fig 7.5). If the resulting depth d_s is under the layer, the second stratum is chosen and the trial is repeated with the equation which corresponds to the soil type of this second stratum. In the first trial, where calculated depth d_s lays within the stratum under study, the searched d_s has been found. This calculation should be made in order, that is, starting from the top of the stratum down to the deepest. When the calculated d_s is over it, the material is very resistant to erosion. In these cases, the erosion depth to be chosen is the upper boundary of the stratum under consideration.

b) Semigraphical method
Consider any point P_i at an initial depth (see fig 7.6) of which its geological features are known. Once the depth of the boundaries of the different strata are known, U_e is found by using eqs 7.12 or 7.13 at the boundaries between them for each stratum. Next, the value of U_r is determined from eq 7.10 for various arbitrarily chosen depths.

After these steps, the values are plotted on a system of coordinate axis, with velocity along the horizontal one and depth d_s along the other. Then, the curves for U_e and U_r are obtained. The point of intersection gives the equilibrium depth for scour and the corresponding mean velocity.

7.1.5 General scour calculation when roughness is not uniform in the section

When in the section width two or more zones have different roughness, the calculation procedure is similar to the previous. The only difference lies in the fact that calculations have to be performed for one zone at a time, because to each one the corresponding α_i has to be found

$$\alpha_i = \frac{Q_{d_i}}{D_{m_i}^{5/3} \, B_{e_i} \mu_i} \tag{7.16}$$

Generally, this problem arises when a wide flood channel is covered with vegetation. In eq 7.9, the subindex i given to the variables refers to the value of each variable for every division of the cross section under study. To value the flow discharge which passes through a sectional zone, the following equation is used

$$Q_{d_i} = \frac{Q_d \, A_{e_i} \, C_i \sqrt{d_i}}{\sum_{i=1}^{n} (A_{e_i} \, C_i \sqrt{d_i}\,)} \tag{7.17}$$

where

Q_{d_i} discharge running through each sectional zone, in m^3/s

A_{e_i} actual hydraulic area before erosion in each span studied, that is, total hydraulic area minus the area of obstacle projections on a plane perpendicular to the flow, in m^2

Q_d total design discharge, in m^3/s

C_i Chezy's roughness coefficient in each zone or span. It is calculated by the following eq

$$C_i = \frac{d_i^{1/6}}{n_i} \tag{7.18}$$

or using some of the equivalent equations shown in Chapter 3.

d_i mean depth in each zone considered, in m

n_i Manning's roughness coefficient in each zone

7.1.6 Effect on scour when the flow transports much sediment in suspension

When due to upstream conditions the flow transports clayey or silty materials along non cohesive channels, there is a reduction in scour depth for a given mean velocity. This may be due to the fact that a certain amount of turbulence is needed to lift particles. This turbulence, in turn, is a function of the flow velocity divided by the kinematic viscosity of the fluid. When this fluid carries much silt or clay in suspension, its specific weight and viscosity increase, which in turn reduces the degree of turbulence of the flow. Therefore, if for a given flow depth the same degree of scour as for clear water is desired, the mean velocity must be increased. This is done by introducing coefficient ψ into eq 7.12. This coefficient depends on the value γ_m of the specific weight of the water with suspended sediment and it is found in table 7.4

$$V_e = 0.68\,\beta\,\psi\,D_m^{0.28}\,d_s^x \qquad (7.19)$$

Hence, for homogeneous beds, eq 7.14 becomes

$$d_s = \left|\frac{\alpha\, d_o^{5/3}}{0.68\,\beta\,\psi\,D_m^{0.28}}\right|^{1/(1+x)} \qquad (7.20)$$

TABLE 7.4. COEFFICIENT ψ AS A FUNCTION OF γ_m

$\gamma_m (kgf/m^3)$	1050	1100	1150	1200	1250	1300	1350	1400
ψ	1.06	1.13	1.20	1.27	1.34	1.42	1.50	1.50

7.1.7 Conclusions and recommendations

With this method not only the value of the general scour in any reach along the river can be calculated, but also the scour due to contractions in those sections where the area has been reduced. In section 7.2, Straub's criterion is presented, but it can only be applied in the determination of the scour produced by contractions.

It has already been noted that the proposed method requires data relatively easy to obtain:

a) The design flow Q_d chosen for a certain frequency or return period, which may be obtained by means of a statistical procedure.

b) The cross section at low water, which is the most easily obtained.

c) Properties of the bed materials, γ_s and D_m, as well as their distribution in the subsoil, for which some soundings are required.

The fundamental hypothesis is that flow per unit width remains constant during the whole scouring process for every strip chosen, neglecting the possible existence of a transverse flow. This is always true, except in the case of the exterior parts of a bend. When working with bridges the abutments do not permit lateral flows.

When considering the basic hypothesis of the conservation of the unit flow, an inconvenience may arise related to the fact that the bed material of a region may be more resistant to scouring than the rest of the section.

In the less resistant region the bed will lower faster. The result will be that after some time the unit flow will increase here and decrease in the region where the material is more resistant. To a less resistant material corresponds depths greater than estimated, while to more resistant material corresponds smaller ones.

In taking into account the resistance of the material, no consideration was given to the time needed for scour to take place in a particular one.

The theoretical computed value for scour may be eadily reached if the material is granular or non cohesive; however, for cohesive materials a certain time is required for the flow to do its work, which may be longer than the duration of the flood. Because of this, the scour may be smaller than the calculated for these materials, even if it may have a greater scouring capacity. It is not easy to evaluate the degree of accuracy of the proposed formulae and criteria because they have only been applied in a few cases and, above all, because no observations have been made in bridges recently built.

The simplest way to carry out further observations is through a series of boreholes at low water level, and afterwards by filling them with a material different from that of the bed, for example, pieces of brick. In the following dry period, the boreholes should be drilled again until the depth at which the material had not been transported is reached. This depth indicates the level reached by the bed during the maximum flood of the previous year. To locate these wells easily a pipe or a 3/4" metal rod is introduced in them leaving a projection of 1 or 2 m above the actual bed level. With these wells, observations can be made in the following years, provided that the maximum future flows are greater than those first measured. This method is only one of a large variety any engineer can design.

7.2 Transversal scour or scour due to contractions

It can be calculated with the methods described in section 7.1, taking into account in this case the effective width B_2 of the contraction.

However, to obtain an approximate value of the transversal scour, Straub's equation can be used

$$d_2 = \left(\frac{B_1}{B_2}\right)^{0.642} d_1 \tag{7.21}$$

Subindex 2 is for a reduced section, 1 for the values of a non altered upstream section, d = depth and B = width.

Straub's formula can only be applied for homogeneous material distribution and sandy soils. Because of its simplicity, it is better to use it before a definite calculation is made with the method indicated in paragraph 7.1. This is useful when calculating erosion in a reduced section, since one of its hypothesis supposes that the width of the free surface B remains constant, that is, that lateral erosion can not occur.

Basic in Straub's method is the assumption that the transportation between a non altered section upstream and the section with the width reduced is continuous. We have obtained similar formulae using different sediment transport equations.

7.3 Scour in bends

There are several ways of solving this problem. If the cross section for the dry season is available, the method for general scour, showed in 7.1, can be applied since the dry season profile indicates greater depths for the exterior bank of the bend. Besides, this method provides a knowledge of the maximum depth and the approximate shape of the scoured cross section.

If the dry season profile is not available, maximum depth is calculated from the characteristics of the bend plan view: its curvature radius r measured at the middle of the width, and the width of the free surface B. The maximum depth d_{max} is, according to Altunin

$$d_{max} = \epsilon\, d_r \tag{7.22}$$

where

- ϵ coefficient that depends on the relation r/B; its value can be found in table 7.5
- d_r maximum depth in the straight reach upstream the curve, in m

There are other methods to value mean depth in curves and meanders. The maximum depth can occur in any zone along a curve, but not in all of it at the same time; but, most frequently, it appears at the downstream end of the curve. The value given by eq 7.22 takes into account the probable general scour.

7.4 Local scour

Two kinds of local scour are of interest: the one which takes place at the foot of the obstacles surrounded by the current and, the other, which is produced by the obstacles joined to the banks that deflect the flow. An example for the first case is that of bridge piers and, of the second, bridge abutments and groins.

7.4.1 Local scour at the foot of bridge piers

This case has been studied by many researchers like Romitta (1953), Laursen-Toch (1956), Bata-Todorovic (1959), Neill (1960, De Souza Pinto (1961), Maggiolo (1961), Jarocki (1961), Maza-Sánchez (1963), Breusers (1967), Shen-Schneider-Karaki (1969), Melville (1976) among others. Here only the Maza-Sánchez method is presented and discussed.

Maza-Sánchez obtained the three diagrams shown in fig 7.7 to 7.9, which make possible the determination of the local scour as a function of the ratio between the pier width and the water depth and the square of the Froude's number of the flow.

This method is only useful for beds covered by sand and gravel. The diagrams have been developed for rectangular, rectangular with round fronts and circular shapes, but they are also useful when the axis of the pier forms an angle with the direction of the flow. For slanted piers, the Froude's number should be corrected by a factor f_c if its value is above 0.06; the value of f_c is indicated in figs 7.7 and 7.8 as a function of the angle ϕ formed by the flow direction and the axis of the pier. To apply the method, it is not necessary to know the mechanism by which scour develops, nevertheless, a brief description is done below in order to give a clearer picture of the formation and development of scour at the foot of bridge piers. These observations were made in a flume at the hydraulic laboratory of the Engineering Institute of the National University of Mexico.

The parameters which affect scour depth at the foot of a bridge pier may be grouped, according to their nature, in five different classes:

a) Hydraulic parameters
   1. Mean flow velocity
   2. Depth of the flow at the face of the pier
   3. Velocity distribution
   4. Direction of the flow with respect to the pier axis

b) Parameters which depend on the bed material
   5. Grain size (or diameter)
   6. Granulometric curve
   7. Grain shape
   8. Degree of cohesion
   9. Submerged specific weight
   10. Thickness of the subsoil layers

c) Geometric parameters
   11. Pier width
   12. Relationship between length and width of the pier
   13. Cross section of the piers

d) Characteristics which depend on bridge location
   14. Contraction at the bridge section
   15. Curvature radius of the reach
   16. Works to control the flow, either upstream or downstream (dikes)

e) Time parameters
   17. Duration of flood peaks
   18. Time required to remove bed material and to reach a stable condition

Because of the way the studies were carried out, factors 10, 14, 16, 17 and 18 were neglected; however, they have been discussed in previous chapters. Experiments were performed with three granular materials, four types of piers and different angles of attack. In every case, for a chosen material, pier and pier location, only the flow depth and the velocity varied.

7.4.1.1 Rectangular pier aligned with the flow

To understand the mechanism of scour a summary is presented of what happens under ideal conditions, that is to say, with a constant flow depth in the model while progressively increasing velocity. When water reaches a certain mean velocity, scour starts at the two corners of the upstream face and the scoured material deposits to the sides. As velocity increases, scour at the corners continues until the same depth is reached across the width of the front (see figs 7.10 and 7.12). Increasing velocity further, bed material transport begins with the corresponding contribution of this material to the area of the scour hole. With more velocity, the scour increase continues until a certain velocity is reached at which scour proceeds no further while the depth of the flow remains the same. The scoured material, which at the beginning was found at the sides, is displaced downstream due to the increase in velocity; this increment persists until it reaches the far end of the pier when the bed material begins to be transported. At great velocities there will always be a piling up at that end and, in some cases, there will be another deposit below the pier, a little further downstream.

Since scour depth varies, the stabilization of the scouring process is disturbed by the formation of ripples and dunes. When a dune crest approaches the scour zone, depth increases reaching a maximum when the crest is at the boundary of the scour zone. This value keeps constant until half the dune has been displaced downstream, and then decreases until the next dune passes. The experiments done at the Engineering Institute showed that between two to five hours are needed to achieve scour equilibrium.

7.4.1.2 Circular and rectangular piers with round fronts

Even if in these cases scour develops in a way similar to that of rectangular piers, it shows certain peculiarities: scour begins in two regions at approximately 65° to each side of the pier axis (see figs 7.11 and 7.12). The rest of the process is the same except for the differences registered in the velocities at which scour cones meet, and at which the same degree of scour occurs at the pier and leads later to maximum scour. Downstream of circular piers, velocities increase to a greater degree than in rectangular ones and form a vortex with much suspended material.

7.4.1.3 Slanted piers

The scouring processes in rectangular and in rectangular with round fronts piers are nearly identical when comparing the edges of the rectangular pier and the region between 60 and 65° to each side of the second pier (see fig 7.12). Assuming low velocity and a low depth of flow, scour begins at edge A as results of a vortex with a vertical axis which is found there.

At the beginning the material is transported towards D, but after a slight increase in velocity, transport expands towards B and C with a resulting increase in scour in that direction. The funnel-shaped depression so formed moves slightly towards D, but soon becomes deeper along the face AB of the pier and then expands towards C and D. When velocity produces transport in the bed, there is an instant when there is practically the same depth along AB and BC (see fig 7.12).

For velocities slightly higher than that which produces maximum scour, the same scour is found around corner C. The events here described were observed with α equal to 30°. If this value is compared to the case of a non slanted pier, the really important difference lies in the fact that not only the degree of maximum scour is much greater but that it is also found downstream of the structures. It is important to mention that the curves given were mainly derived for materials with mean diameters of 0.17 and 0.56 mm and that for obtaining the scour for particles with a mean diameter of 1.30 mm, the graphs always gave greater values than those experimentally measured. In other words, particle diameter has a direct influence on scour value. The influence decreases as $\mathbb{F}^2$ increases. In general, when this parameter is more than 0.1, the effect of the diameter is negligible.

7.4.1.4 Limitations of the method

Anybody willing to use the graphs may find two limitations. The first is that they have been derived for only three types of piers, the rectangular one being exclusively of theoretical interest. But it was included because it is mentioned by most of the researchers who have studied this problem and because its results may be easily correlated to ours, However, it is useful because it gives maximum scour, thus illustrating extreme conditions.

The other limitation is that particle diameter of the material has not been considered. This problem is not very significative in the range of sands for model studies and it is even less significant for prototypes, even if it may be important when dealing with coarser materials. But this is not a serius limitation, since scour is mainly found in sandy and silty channels.

7.4.2 Local scour at the foot of abutments

The method here discussed was proposed by Artamonov, and it permits to determine the depth of the scour at the foot of abutments and groins end. This type of scour depends on the flow theoretically intercepted by the structures on the slope at the end side of the abutment and in the angle between the longitudinal axis of the structure and the flow. The scour at the foot of an abutment, measured from the free surface of the flow, is given by

$$S_T = P_\alpha P_q P_k d_o \qquad (7.23)$$

where

- α angle between the abutment axis and the flow (see fig 7.13)
- d_o depth of the flow in the area near the abutment before scouring
- P_α coefficient which depends on α; its value is found in table 7.6
- P_k coefficient which depends on the slope of the sides of the abutment; it is found in table 7.7
- P_q coefficient which depends on Q_1/Q, where Q_1 is the theoretical flow that would pass through the area occupied by the abutment if it were not there, and Q is the total flow in the river. The value of P_q is found in table 7.8.

As may be observed, when the bridge crossing is straight, $\alpha = 90°$, and if the abutment slope is vertical, eq 7.23 reduces to

$$S_T = P_q d_o \qquad (7.24)$$

It is also worth to note that when the slope is 3:1 at the side of the abutment, scour reduces from 50 to 100 per cent.

This applies in a similar way to groins, but it is necessary to include if the groins are built on both banks and opposite to one another. In this case, scour S_T may be reduced up to 75 per cent

$$S_T = 0.75 P_\alpha P_q P_k d_o \qquad (7.25)$$

Even if there is no way to determine the scour when the groin is covered by water, since the method above gives the maximum possible values, it is better to consider Q_1 as the maximum theoretical flow intercepted by the abutment at its crest, but d_o must be taken to the free surface in eq 7.23.

When water passes over the structure (normally in groins), the downstream side should be protected because it is also exposed to scour.

7.5 Erosion downstream of large dams

What is generally known with the generic expression of "erosion downstream of large dams" refers to the gradual descent of the river bottom along a reach where its length increases with time because all the discharges of the dam are almost free of sediments. Therefore, the bottom particles transported downstream are not replaced by others coming from upstream.

This degradation process or erosion is greater in the reaches immediately downstream of the discharge and goes on diminishing downstream until there is a section which remains "non-altered" or stable with reference to this kind of erosion (see figs 7.14 and 7.15).

It is considered that downstream of this first non-altered section the bed material and the slope vary with respect to their original condition as a result of morphologic changes produced by modifications of the formative discharge (see 6.5). When this discharge changes the first thing that modifies is the depth and afterwards the width. Slope changes very slowly through a very long time period. This second reach, where the mentioned erosion does not occur, is called the "second reach" of the river downstream of the dam.

This erosion takes place slowly but continuously along the life of the dam as long as the flow which produces the river discharges can transport the bed material. Therefore, the reach affected by this erosive process, or first reach, increases its length in continuous and gradual form while the bottom of the affected reach descends (see figs 7.14 and 7.15).

The erosion downstream of large dams mainly depends on the discharges, their variation and permanence, on the physical characteristics of the bed material and, consequently, on the sediment transport that takes place in the non-altered reach or second reach.

Although complex, the calculation of the scouring process may be done easily as long as the material in the bed and in the depth affected by the erosion are the same in all the first reach.

This process becomes much more complicated, and consequently, its analytic determination, not only when the erosion uncovers materials with different granulometries, but also when the material of the bed becomes armored. If in all the reach and its depth the material is the same, but the granulometry is extended or well gradel with $\sigma_g > 3$ (which means that it can get armored), the maximum critical condition must be considered because only the discharges that exceed it can destroy the armor coat and produce sediment transport. In some rivers where this happens, there is only sediment transport and, therefore, erosion, when there are great discharges through the spillway. In case of small discharges of the spillway or the intake work, the armored bottom does not permit sediment transport or bottom scouring.

7.5.1 Methods for solution

The calculation method presented in this work offers several advantages. The first is that both the erosive process and the morphologic changes of the channel are considered in the same reach; that is to say, the section width varies simultaneously with the slope, the depth and the sediment transport. In the second place, it does not present the instabilities of the numerical methods. Third, it is considered that it is more easily used by the design engineer than the simultaneous solution of the following three differential equations: continuity, energy and sediment transport continuity. For the quantification of the erosion downstream of dams this method uses the equations Maza and Cruickshank proposed in 1973 to study channel stability.

With reference to the eqs 6.75 or 6.99, it is important to note that, according to Stebbings and Friedkin studies in laboratory channels where they observed the changes in width and depth in sections with different sediment transport, they found out that the exponent m is in all cases equal to 0.7, while coefficient K varied from 8.22 to 25.82 between the minimum and the maximum transport. Therefore, when studying the erosion downstream of large dams and if there is no data to calibrate the eq 6.76, the following values of K are recommended: $K_c = 10$ for the section immediately downstream of the dam where sediment transport is almost null, and $K_r = 18$ for the last section downstream of the reach subject to erosion, because there is no certainty that the transport in it is maximum. For the intermediate sections several variations of K were tested and the most congruent results were obtained when K varied in elliptical form according to the relation

$$K_i = K_c + (k_r - k_c)\sqrt{1 - (\frac{X_i}{L})^2} \qquad (7.26)$$

considering the system of axis and numeration of the section shown in fig 7.16.

In eq 7.26 the meaning of the variables is

K_c coefficient of the first upstream section of the stretch under study, immediately downstream of the dam, $K_c = 10$

K_r coefficient of the last downstream section of the stretch subject to erosion. It corresponds to the portion of the river which is not subject to scouring or second reach

L length of the first reach subject to erosion. Distance between the already mentioned sections which increases with time

X_i distance between the initial section and that from which K_i is to be obtained, see fig 7.16

Before presenting both the equations and the procedure, it is important to remember that the exact solution to the problem is difficult because of the following

1. There are no systematic and complete measurements taken during several years to compare the analytic results with reality.

2. In the quantification of erosion and of morphologic change different variables take part. Even if some of these variables must be obtained from real data, when these do not exist, the analytic value of these variables is accepted without later adjustments. This is the case with exponent m and coefficient K.

3. To quantify the scouring downstream of dams the existence of the same material is supposed in the bed, in the banks and all along the channel section, all which is not necessary true. Besides, it is also accepted that the material is the same in all the scoured depth.

4. When there is a limited transport of sediments, they tend to form an armor, process which increases when the standard geometric deviation of their sizes also increases. Neither the changes of this armoring along the first reach have been observed in nature, nor those that take place in the armoring from the middle of the reach towards the banks until they arrive at the water surface, mainly in the first upstream sections.

The object of this method is to know, for the first reach, the erosion suffered by the channel bed, the changes suffered in the river sections because of the formative discharge dûe to the presence of the dam, the slope alterations provoked by the erosion and the time during which they take place.

7.5.2 Preliminary calculations

The calculations indicated will be performed taking into account the hypothesis shown in 7.5.3 and figs 7.16 and 7.17.

1. It is important to know the formative discharge, the slope and the transversal section of the river reach downstream of the future dam up to an approximate length of 50 km. In this way, K, m and α for this reach are obtained without the need of supposing them. With these parameters adjusted to real observations, the scouring calculation is more precise.

2. At the end of the first reach, that is to say, at a certain distance from the dam which varies in length with time, the channel is not affected by this scouring. Downstream of this point, the river hydraulic slope remains equal to the original S_r. Therefore, $S = S_r$.

3. The dam new formative discharge is calculated. When there is a hydropower plant the dayly maximum average discharge may be obtained. If the spillway discharges during long periods, the formative one, associated to those discharges can also be used.

4. The width b_r and the channel depth d_r for the reach non affected by the erosion or second section are obtained with S_r and the new formative discharge Q. Of course these parameters correspond to the last section of the first reach. To do this

   4.1 Q_B is calculated with eqs 6.100, given $S = S_r$

   4.2 Once Q_B is known, b_r is obtained with the help of eq 6.98

   4.3 Once b_r is known, d_r is obtained with the help of eq 6.99

5. For the first section immediately downstream of the intake or spillway non af-

fected by local scouring, the sediment transport is considered to be null, thus the critical hydraulic slope $S = S_c$ will present. It is calculated with eq 6.100, accepting that $Q_B = 0$, which takes the form

$$S_c = \left[\frac{0.047}{N} K^{\frac{1}{w}} \left(\frac{\alpha}{Q}\right)^{\frac{m}{w}}\right]^{\frac{w}{1.178m+1}} \quad (7.27)$$

6. To perform the calculation, the first reach is divided into n equal reaches. In each of them eqs 6.98 to 6.100 are applied. It can be observed that the unknown quantities are four: Q_B, S, b and a for each of them, but as there are only three equations, some of the hypothesis of point 7.5.3 will be accepted

7. If in the first reach of the river the banks are made of rock or they are very resistant, one degree of freedom is lost because the width b is considered as constant during all the scouring process; then only two equations, the 6.64 and the 6.70 are needed to solve the problem. Form them it is obtained

$$S = \left(\frac{\alpha_b}{Q}\right)^{0.849} \frac{1}{N^{1.387}} \left[\left(\frac{Q_B}{\epsilon b}\right)^{2/3} + 0.047\right]^{1.387} \quad (7.28)$$

$$d = \left(\frac{Q}{\alpha b}\right)^{0.849} \left[\frac{N}{(Q_B/\epsilon b)^{2/3} + 0.047}\right]^{0.387} \quad (7.29)$$

There are two equations with three unknown quantities: Q_B, d and S; then the procedure to be followed is similar to that explained for three degrees of freedom.

7.5.3 Hypothesis

As thre are more unknown quantities than equations the following hypothesis are established

a) The sediment transport varies according to a pre-established law from zero in the first section up to Q_{B_r} in the last section of the first reach. These sections are separated a distance L_r (measured along the river) which grows with time. Initially, the linear variation was chosen for this work, but any other may work as long as it is continuous. Afterwards, and in order to verify which is the right one, the principle of sediment continuity must hold for each of the reaches.

Once the variation Q_B is accepted, b, d and S are obtained for all the reaches into which the length L_r is divided, using eqs 6.98 to 6.100.

b) Instead of this hypothesis, a known variation of S may be accepted, from which b, d and Q_B can be obtained within the first reach subject to scouring.

c) It has been seen that coefficient K also depends on sediment transport, due to the fact that for only one bed material and in experimental form a value of K = 8.22 was obtained for Q_B = 0 and of K_{max} = 25.82 for Q_B maximum. Therefore, K will vary from K_c = 10 in the first section up to $K = K_r$ in the last. As apparently this variation is not linear, to start the calculation the use of a formula of the eq 7.26 type is recommended

$$K_i = K_c + (K_r - K_c)\sqrt{1 - (\frac{X_i}{L})^2} \quad \text{(eq 7.26)}$$

If due to the lack of data, K is unknown, it is recommendable to start with K_r = 18 to 20.

7.5.4 Scouring calculation

These steps are to be followed (see figs 7.16 and 7.17):

1. Consider figs 7.16 and 7.17. A length L_r is chosen for the reach subject to erosion. A coordinated system is selected where the origin has the same position of the last section of the eroded reach (at the end of L_r, measured along the river); the x axis is horizontal and directed towards the dam and the y axis is vertical and directed upwards. The horizontal distance from the dam to the origin of the axis is L (see fig 7.16), therefore

$$L = L_r \cos \alpha \quad (7.30)$$

$$S_r = \tan \alpha \quad (7.31)$$

$$\alpha = \tan^{-1} S_r \quad (7.32)$$

2. L is divided in n reaches, for instance, 20; n = 20

3. For every section between reaches R_{B_i} is supposed, which, as first option, varies

linearly from zero to Q_B. Then, it holds that

$$Q_{B_i} = (1 - \frac{X_i}{L}) Q_{B_r} \tag{7.33a}$$

or

$$Q_{B_i} = (1 - \frac{L_{r_i}}{L}) Q_{B_r} \tag{7.33b}$$

Remember that downstream of the origin (see fig 7.16) it holds that $S = S_r$ and $Q_B = Q_{B_r}$.

4. With eq 7.26 K is obtained for every section for i = 0, 1, 2,..., n.

5. Once Q_{B_i} and K_i are known and using eqs 6.98 and 6.100, b_i, d_i and S_i are obtained for every section.

6. The profile of the energy line is calculated starting from $X_o = 0$ and finishing with $x_n = x_{max} = L$

$$H_i = (\frac{S_i + S_{i-1}}{2}) \Delta x + H_{i-1} \tag{7.34}$$

where

$$\Delta X = L/n \tag{7.35}$$

for i = 1, 2, 3, ..., n.

For the condition $x = x_o = 0$, H_o takes the value of

$$H_o = d_o + \frac{U_o^2}{2g} + Z_o \tag{7.36}$$

where d_o and U_o are respectively the depth and the mean velocity of the last section downstream of the first reach and equal to the theoretical uniform condition of the second reach. Z_o may either be taken as the bottom elevation in the indicated section or it may be given a value of zero.

Since $S_o = S_r$ and $S_n = S_c$

$$H_1 = \left(\frac{S_1 + S_0}{2}\right) \frac{L}{n} + H_0 \qquad (7.37)$$

$$H_2 = \left(\frac{S_2 + S_1}{2}\right) \frac{L}{n} + H_1 \qquad (7.38)$$

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$$H_n = \left(\frac{S_c + S_{n-1}}{2}\right) \frac{L}{n} + H_{i-1} = H_{max} \qquad (7.39)$$

for $i = 1, 2, 3, \ldots, n$.

7. The profile of the eroded bed is calculated, measuring its elevation over the x axis

$$Z_i = H_i - d_i - \frac{U_i^2}{2g} \qquad (7.40)$$

$i = 0, 1, 2, 3, \ldots, n$.

Therefore, $Z_o = 0$, or $Z_o =$ elevation of the river bed in the zero section, and

$$Z_n = Z_{max} = H_{max} - d_n - \frac{U_n^2}{2g} \qquad (7.41)$$

When the erosive process advances fast and L is big (more than 10 km), d_n and U_n have the values of d_c and U_c, respectively, in uniform regime.

8. Once every H_i is known, the scour depths E_i are measured from the original bottom

$$E_i = y_i - Z_i \qquad (7.42)$$

where

$$y_i = L_i S_r + Z_o \qquad (7.43)$$

for $i = 1, 2, 3, \ldots, n$.

To the original condition $x_o = 0$ to Z_o a value of zero may be asigned or the elevation of the river bottom at that point. Then it also holds that $Y_o = Z_o =$ bottom elevation, in $Z_o = 0$.

9. The eroded volume is obtained (see fig 7.17)

$$\forall_e = \sum_{i=0}^{n} \left(\frac{b_i + b_{i-1}}{2}\right) \left(\frac{E_i + \bar{F}_{i-1}}{2}\right) \Delta x \qquad (7.44)$$

10. The time during which the volume $\forall_e$ was eroded is calculated, considering that the volume of sediments that passes through section 0, during the time t, without considering the voids, is

$$\forall_{B_r} = t\ Q_{B_r} \qquad (7.45)$$

$\forall_{B_r}$ is the solids volume only. If the original material in the river bed has a void relation e, or a porosity $\bar{n}$, the total volume $\forall_r$ occuppied by the sediments is

$$\forall_T = (1 + e)\ \forall_{B_r} \qquad (7.46)$$

or

$$\forall_T = \frac{\forall_{B_r}}{(1 - \bar{n})} \qquad (7.47)$$

Since $\forall_T = \forall_e$, taking into account eqs 7.44 to 7.47, the time needed to scour $\forall_e$, from the moment the dam was closed, has the value of

$$t = \frac{\forall_e}{(1 + e)\ Q_{B_r}} \qquad (7.48)$$

or

$$t = \frac{(1 - \bar{n})\ \forall_e}{Q_{B_r}} \qquad (7.49)$$

Remember that in a soil sample the following relations hold

$$\bar{n} = \frac{e}{1 + e} \qquad \text{and} \qquad e = \frac{\bar{n}}{1 - \bar{n}}$$

11. The same procedure is followed for other lengths of L

12. Once the former is done for several lengths of L up to about 50 km, the following is shown

a) Depth profile for every length of L

b) Curve of L_j versus time

c) Curve of $Z_{max\,j}$ versus time

d) Curve of $H_{max\,j}$ versus time

e) If it is necessary to know the evolution of the scouring process in a given river section, with the help of the first two figs a and b, the bottom depth and the free surface elevation are obtained, both versus time for that section. At any point the value of the free surface level is

$$S_{level} = H_i - \frac{U_i^2}{2g} = Z_i + d_i \tag{7.50}$$

4.4 Other comments

I. With the calculation sequence already presented and as long as there are no preliminary data about the evolution of erosion during the first years, the use of the following values is recommended: m = 0.7, constant all along the reach; $K_r = K_o = 18$; $K_c = 10$; k_2 = according to what has been expressed in eq 7.26.

II. It is necessary to check if there is sediment continuity in every reach. To do this consider fig 7.18 and any reach between sections i and (i + 1), at two different times: j and (j + 1). The bottom elevations at any point and given time have a value y_{ij}. When the Y_{ij} were given as elevation with respect to a common reference horizontal line, it holds that

$$[V_e]_{i,i+1}^{j,j+1} = \left(\frac{b_i + b_{i+1}}{2}\right)\left(\frac{L_{r_j} + L_{r(j+1)}}{2n}\right)\left[(Z_{i+1}^{j} - Z_{i+1}^{j+1}) + (Z_i^j - Z_i^{j+1})\right]^{1/2} \tag{7.51}$$

If when calculating every profile it was accepted that $Z_o = 0$, when applying eq 7.51, the level of reference will be considered at axis x and instant (j + 1); therefore, for instant j, all the values must be increased in $S_r\,[\Delta L]_j^{j+1}$, which is the difference in elevation between two axis x, from instant j to instant (j + 1). Take into account that

$$[\Delta L]_j^{j+1} = L_{j+1} - L_j \tag{7.52}$$

Instead, all the $Z_{i,j+1}$ keep their value.

In this way a volume $[\text{¥}_e]_{i,\,i+1}^{j,j+1}$ and the following continuity equation for the sediment must be kept

$$(Q_{B_i} - Q_{B_{(i+1)}})\,\Delta t\,(1 + e) - [\text{¥}_e]_{i,\,i+1}^{j,\,j+1} = [\Delta \text{¥}_e]_{i,\,i+1}^{j,\,j+1} \tag{7.53}$$

where Δt is the time elapsed between j and j + 1.

As this is done for the n reaches, all the $\Delta \text{¥}_e$ will be known. Consider that while the hypothesis a) of 7.5.3 given by eqs 7.33 holds, it has to be written as

$$(Q_{B_i} - Q_{B_{(i+1)}}) = \frac{Q_B}{n} \tag{7.54}$$

When the linear variation of Q_B explained in point a) is correct, all the $(\Delta \text{¥}_e)$ must be zero. If this is not true another variation of Q_B has to be chosen, for instance, exponential, elliptical or parabolic, up to the point in which all the $\Delta \text{¥}_e$ keep the condition $[\Delta\, \text{¥}_e]_{i,\,i+1}^{j,\,j+1} = 0$.

Even if up to the moment the sediment transport variation that scrupulously keeps this condition has not been found, the linear variation of the examples studied almost keep it.

III. It is important to note that the fulfillment of the energy equation expressed by the Bernoulli's equation between sections has not been explicitly considered, instead, the permanent flow associated to every section was used; this means that the results are not correct when L is too small. But as L grows, the depths obtained hold with the equation. In order to satisfy the reasonable doubt that may arise at this point, it is advisable to obtain the free surface profile by means of the methodical application of the Bernoulli's equation, which is first written as

$$Z_{i+1} + d_{i+1} + \frac{Q^2}{2gd_{i+1}\,b_{i+1}} = Z_i + d_i + \frac{Q^2}{2gd_i^2\,b_i^2} + \frac{1}{2n}\left(\frac{Q}{\alpha}\right)^{2.193}\left[\frac{1}{d_{i+1}^{3.584}} + \frac{1}{d_i^{3.584}}\right] \tag{7.55}$$

where d is the unknown quantity, if the already obtained b_i are accepted; α is obtained from eq 7.55.

7.6 Erosion produced by bottom gate discharge

For a gate placed as shown in fig 7.19, where the discharge is free and the hydraulic jump forms immediately after the outlet on an erodible bed, the depth of erosion can be obtained by means of the method proposed by Valenti. However, he does not say at what distance from the gate the maximum erosion occurs. Fig 7.19 includes the values of all the parameters needed to plot it, with the exception of $\mathbb{F}_r$ and D · $\mathbb{F}_r$ is the Froude's number associated to section 1.

The erosion produced by sky jumps and deflectors should be studied by means of hydraulic models with mobile bed, because the jet is seldom continuous, and its shape and the air mixed in it may vary greatly from one structure to another. The best are those by Froude, because in his models the scale of particle fall velocity is equal to that of flow velocity.

7.7 Scour under pipes

When pipelines for oil, gas, water, etc., must cross a river, they are placed under the bed. If a flood causes general erosion, the pipe may become partially exposed causing, perhaps, local scour under it. The value of this scour may be estimated with the aid of fig 7.20 —suggested by Maza—, being the non dimensional parameters here, a/D, $\mathbb{F}_r$ and S/D. Where a is the distance from the bed to the lowest end of the pipe (it can be negative), in m; $\mathbb{F}_r$ is equal to $U/\sqrt{gd}$, where U and d are mean velocity and depth, in m/s and m, respectively; and S is the scour under the pipe, measured from the original bottom level, in m.

The curves in fig 7.20 provide the value of the maximum under-pipe erosion. However, the most important point in the problem is not really the extent of the erosion, but whether it takes place. Scour occurs when more than half the diameter of the pipe remains uncovered and the Froude's number is above 0.1.

Under pipe erosion may be prevented and, consequently, pipe failure too, by placing the pipe below the bottom, at a depth where it may not be uncovered by a passing flood.

7.8 References

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TABLE 7.1 CONTRACTION FACTOR μ

Mean velocity in m/s	Span between two piers, in m												
	10	13	16	18	21	25	30	42	52	63	106	124	200
> 1	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
1.00	0.96	0.97	0.98	0.98	0.99	0.99	0.99	1.00	1.00	1.00	1.00	1.00	1.00
1.50	0.94	0.96	0.97	0.97	0.97	0.98	0.99	0.99	0.99	0.99	1.00	1.00	1.00
2.00	0.93	0.94	0.95	0.96	0.97	0.97	0.98	0.98	0.99	0.99	0.99	0.99	1.00
2.50	0.90	0.93	0.94	0.95	0.96	0.96	0.97	0.98	0.98	0.99	0.99	0.99	1.00
3.00	0.89	0.91	0.93	0.94	0.95	0.96	0.96	0.97	0.98	0.98	0.99	0.99	0.99
3.50	0.87	0.90	0.92	0.93	0.94	0.95	0.96	0.97	0.98	0.98	0.99	0.99	0.99
4.00 or higher	0.85	0.89	0.91	0.92	0.93	0.94	0.95	0.96	0.97	0.98	0.99	0.99	0.99

TABLE 7.2 COEFFICIENT β

Return period in years	Coefficient β
1	0.77
2	0.82
5	0.86
10	0.90
20	0.94
50	0.97
100	1.00
500	1.05
1 000	1.07

TABLE 7.3 VALUES OF X AND 1/(1 X) FOR GRANULAR AND COHESIVE SOILS

COHESIVE SOILS						GRANULAR SOILS					
γ_s, in kgf/m^3	x	$\frac{1}{1+x}$	γ_s, in kgf/m^3	x	$\frac{1}{1+x}$	D_m, in mm	x	$\frac{1}{1+x}$	D_m, in mm	x	$\frac{1}{1+x}$
0.80	0.52	0.66	1.20	0.39	0.72	0.05	0.43	0.70	40.00	0.30	0.77
0.83	0.51	0.66	1.24	0.38	0.72	0.15	0.42	0.70	60.00	0.29	0.78
0.86	0.50	0.67	1.28	0.37	0.73	0.50	0.41	0.71	90.00	0.28	0.78
0.88	0.49	0.67	1.34	0.36	0.74	1.00	0.40	0.71	140.00	0.27	0.79
0.90	0.48	0.67	1.40	0.35	0.74	1.50	0.39	0.72	190.00	0.26	0.79
0.93	0.47	0.68	1.46	0.34	0.75	2.50	0.38	0.72	250.00	0.25	0.80
0.96	0.46	0.68	1.52	0.33	0.75	4.00	0.37	0.73	310.00	0.24	0.81
0.98	0.45	0.69	1.58	0.32	0.76	6.00	0.36	0.74	370.00	0.23	0.81
1.00	0.44	0.69	1.64	0.31	0.76	8.00	0.35	0.74	450.00	0.22	0.83
1.04	0.43	0.70	1.71	0.30	0.77	10.00	0.34	0.75	570.00	0.21	0.83
1.08	0.42	0.70	1.80	0.29	0.78	15.00	0.33	0.75	750.00	0.20	0.83
1.12	0.41	0.71	1.89	0.28	0.78	20.00	0.32	0.76	1000.00	0.19	0.84
1.16	0.40	0.71	2.00	0.27	0.79	25.00	0.31	0.76			

TABLE 7.5 VALUES OF THE COEFFICIENT ε AS A FUNCTION OF r/B

r/B	∞	6	5	4	3	2
ε	1.27	1.48	1.84	2.20	2.57	3.00

TABLE 7.6 VALUES OF CORRECTION OF COEFFICIENT P_α, AS A FUNCTION OF α

α	30°	60°	90°	120°	150°
P_α	0.84	0.94	1.00	1.07	1.188

TABLE 7.7 VALUES OF CORRECTION OF COEFFICIENT P_k, AS A FUNCTION OF k

Slope k	0	0.5	1.0	1.5	2.0	3.0
P_k	1.0	0.91	0.85	0.83	0.61	0.50

TABLE 7.8 VALUES OF COEFFICIENT P_q, AS A FUNCTION OF Q_1/Q

Q_1/Q	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80
P_q	2.00	2.65	3.22	3.45	3.67	3.87	4.06	4.20

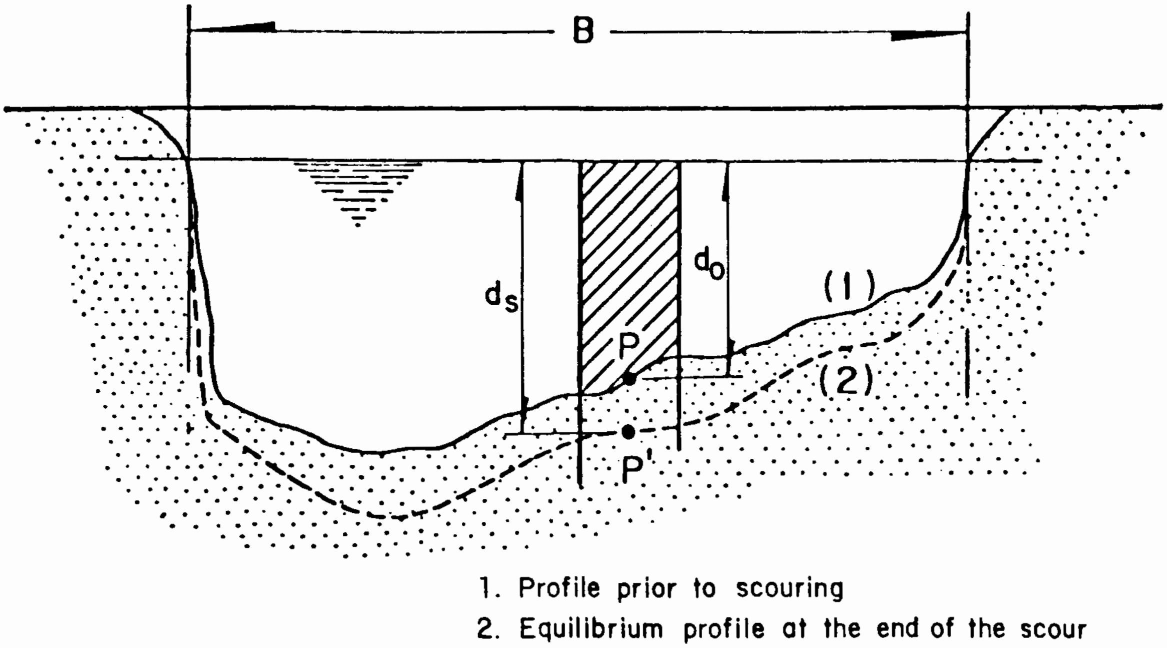


Fig 7.1
Variables considered to determine v_r

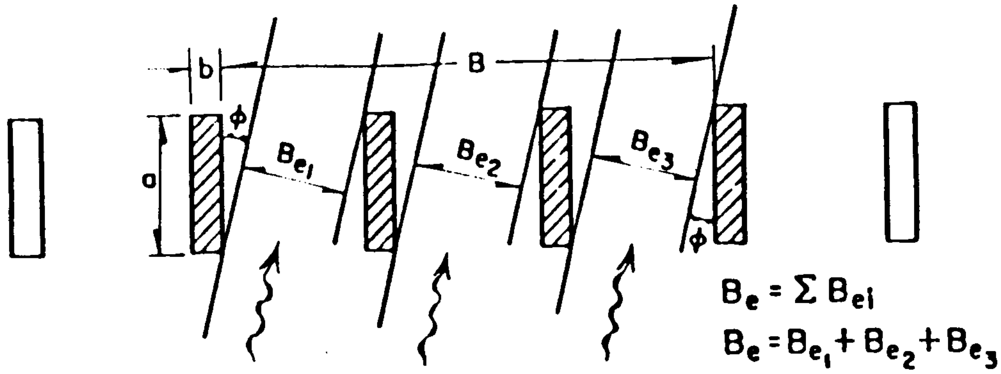


Fig 7.2
Meaning of B_e

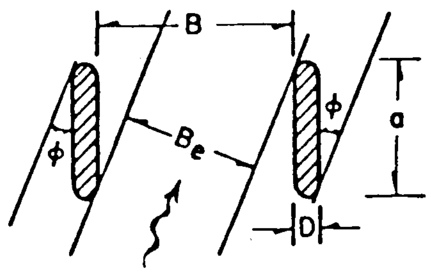
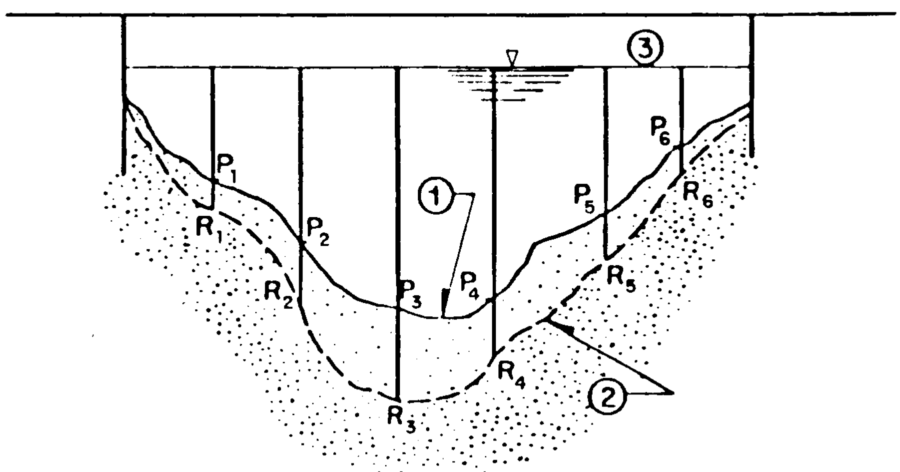


Fig 7.3
Graphical method to obtain B_e

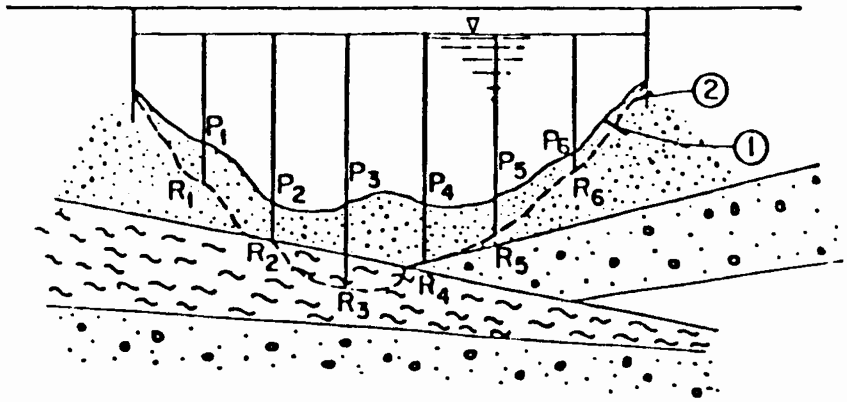


P_i Points under study, prior to scour

R_i Theoretical points reached during the scour

1. Cross-section before stouring occurred
2. Cross-section after scouring
3. High water level

Fig 7.4
Scour in a homogeneous bed



PP_i Points under study

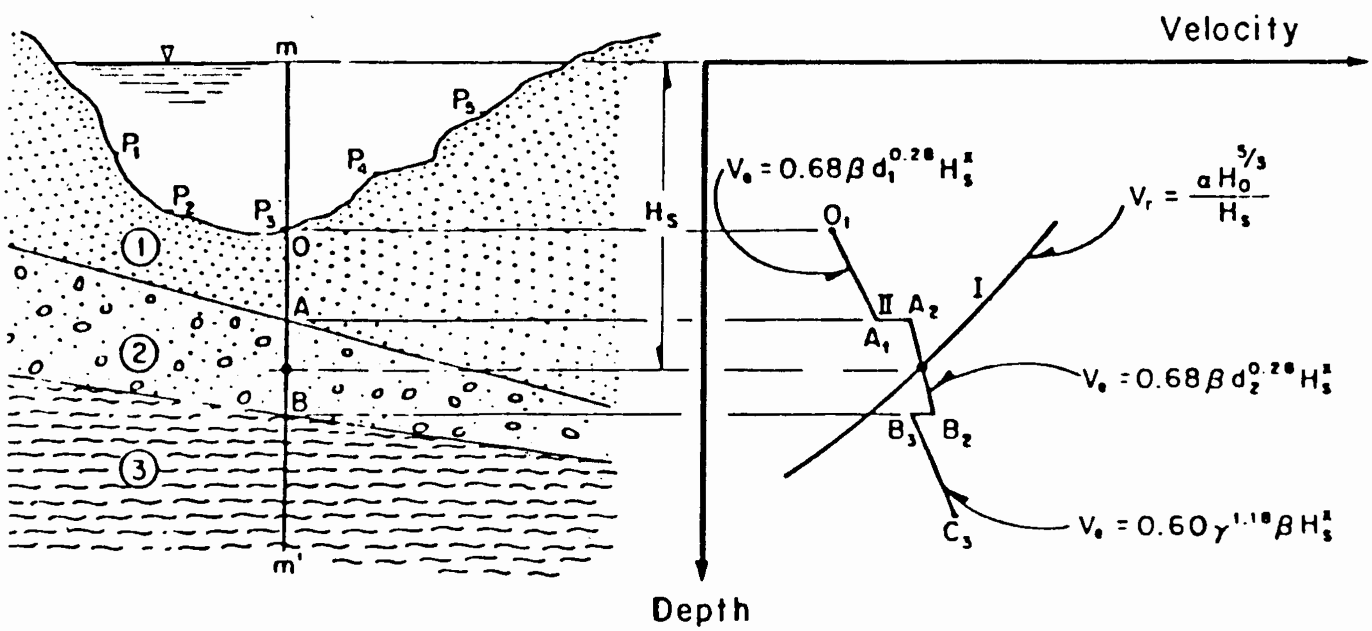
1. Cross-section before scour
2. Cross-section after scour

Sand

Sand with gravel

Silt

Fig 7.5
Scour in heterogeneous beds



a) Cross-section showing the different layers

b) Curves for V_e and V_r versus H_s for point P_3

Fig 7.6
Semigraphical method

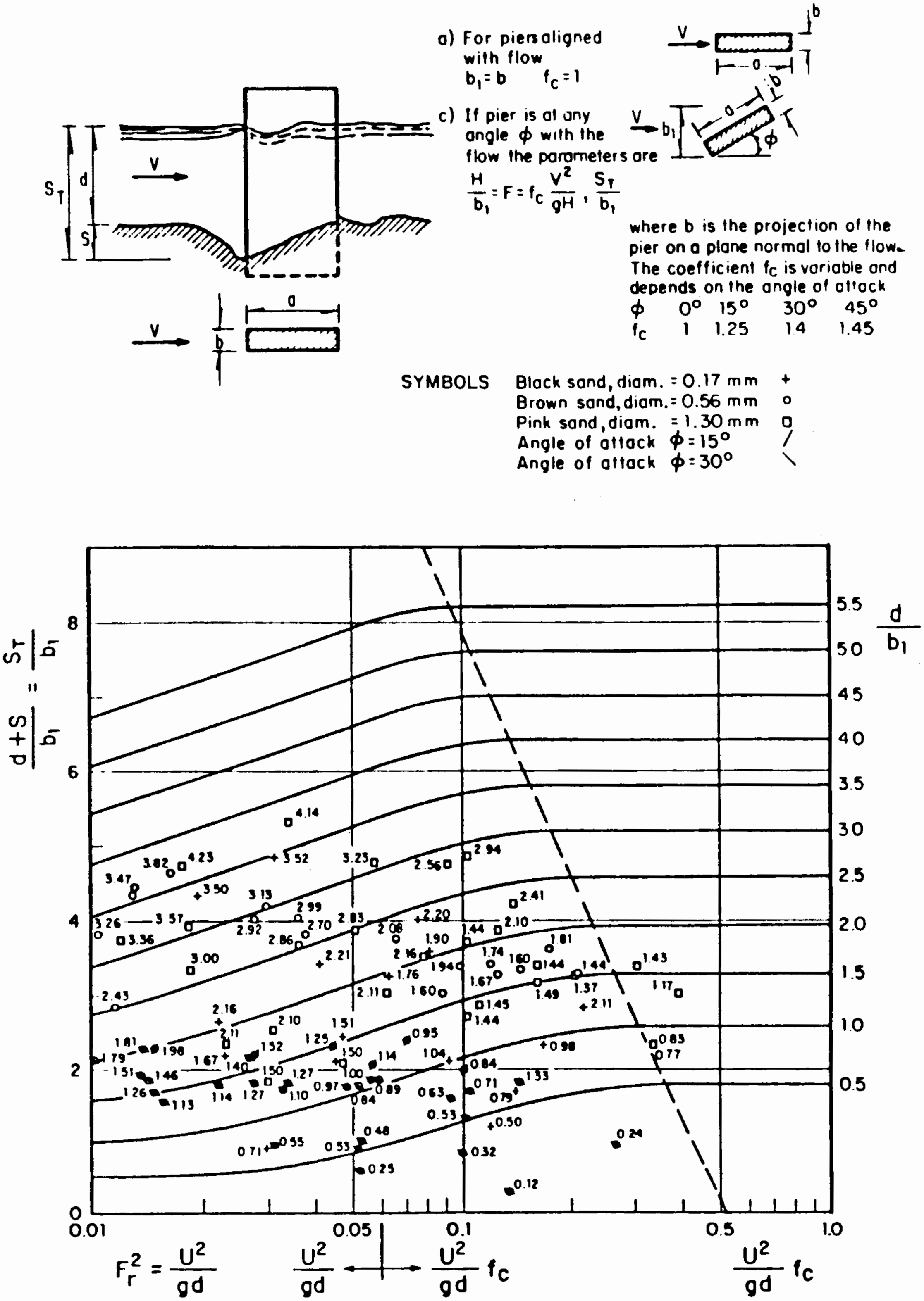


Fig 7.7

Scour around rectangular piers (after Maza and Sánchez)

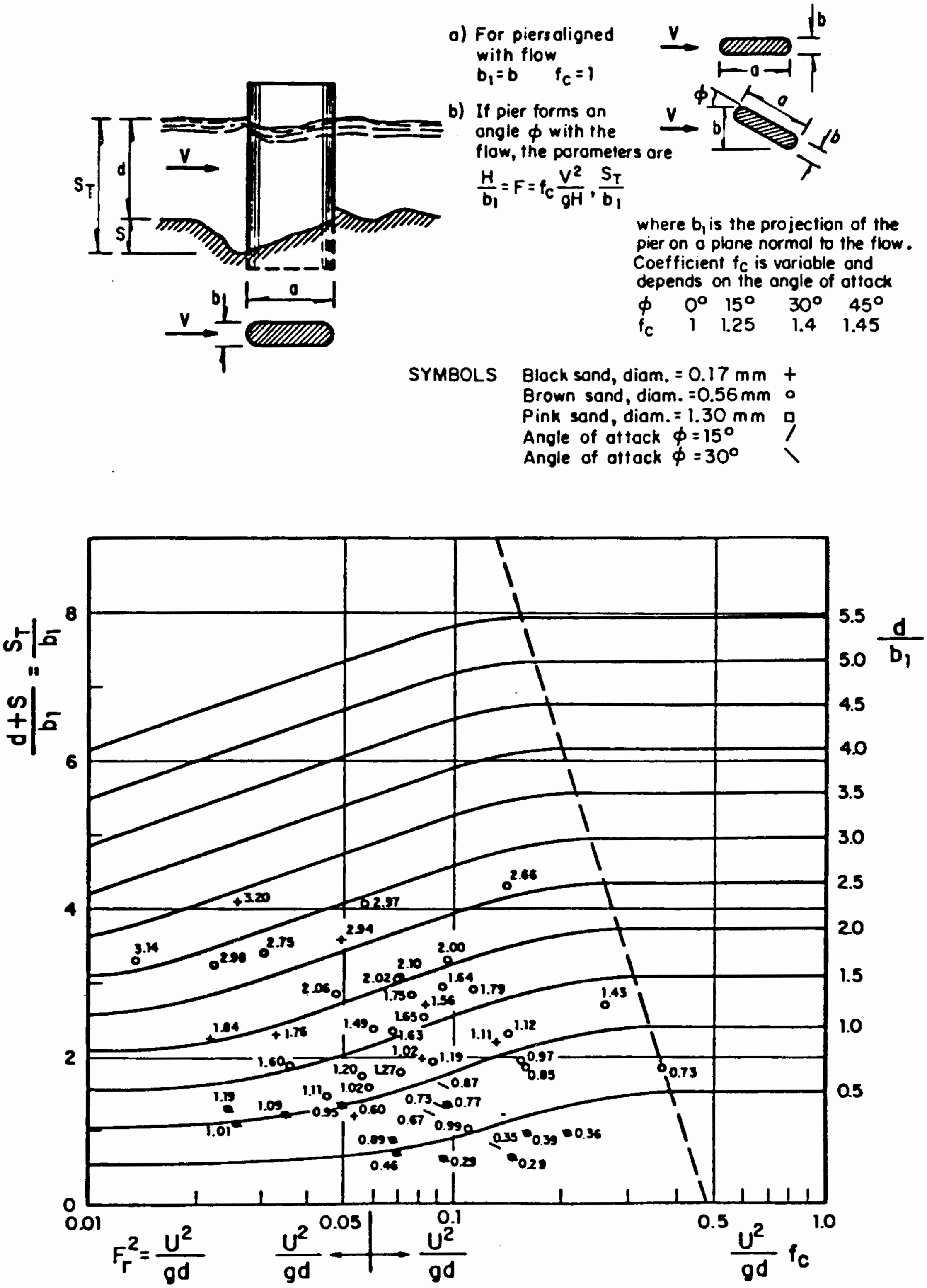


Fig 7.8

Scour around rounded piers (after Maza and Sánchez)

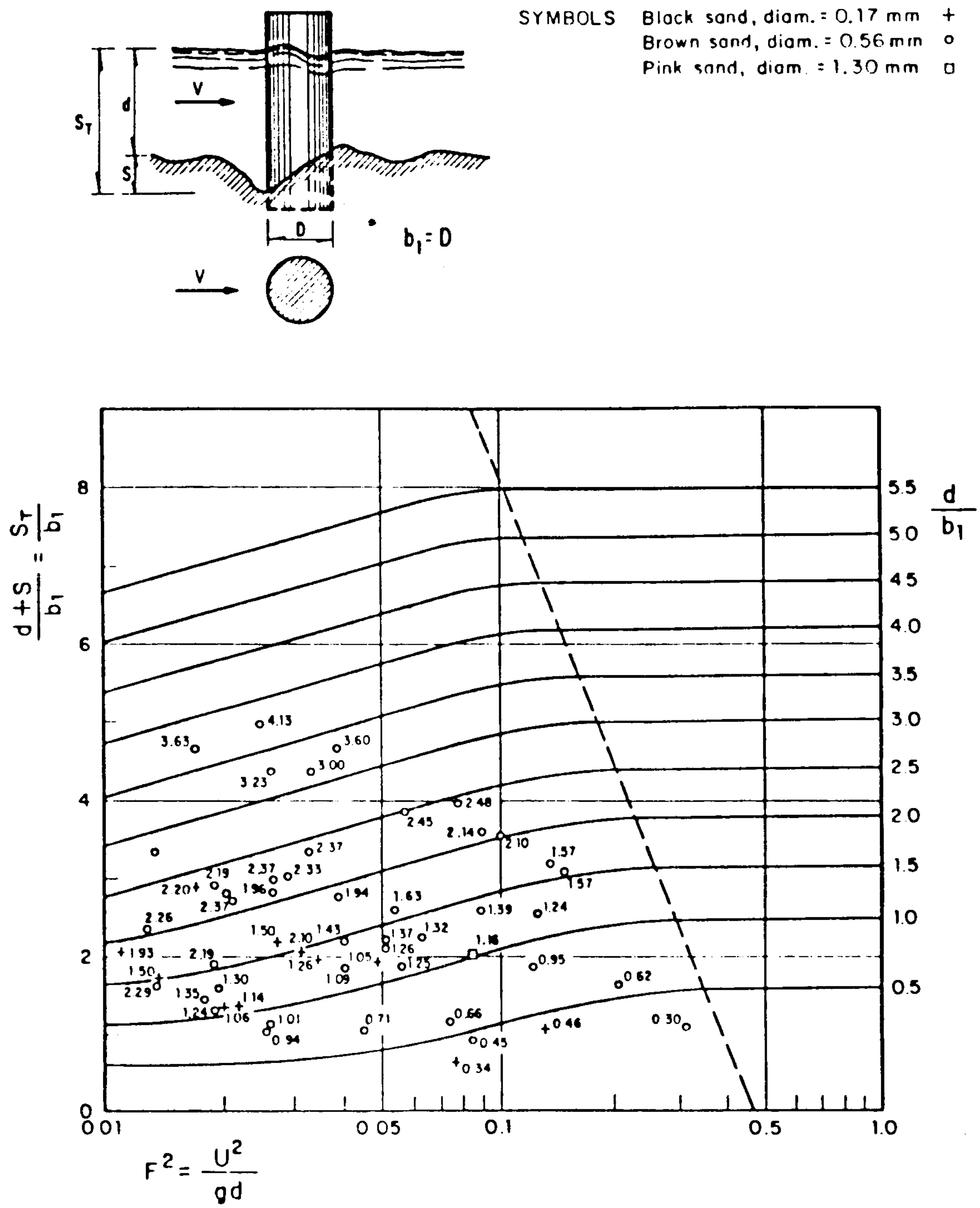


Fig 7.9

Scour around circular piers (after Maza and Sánchez)

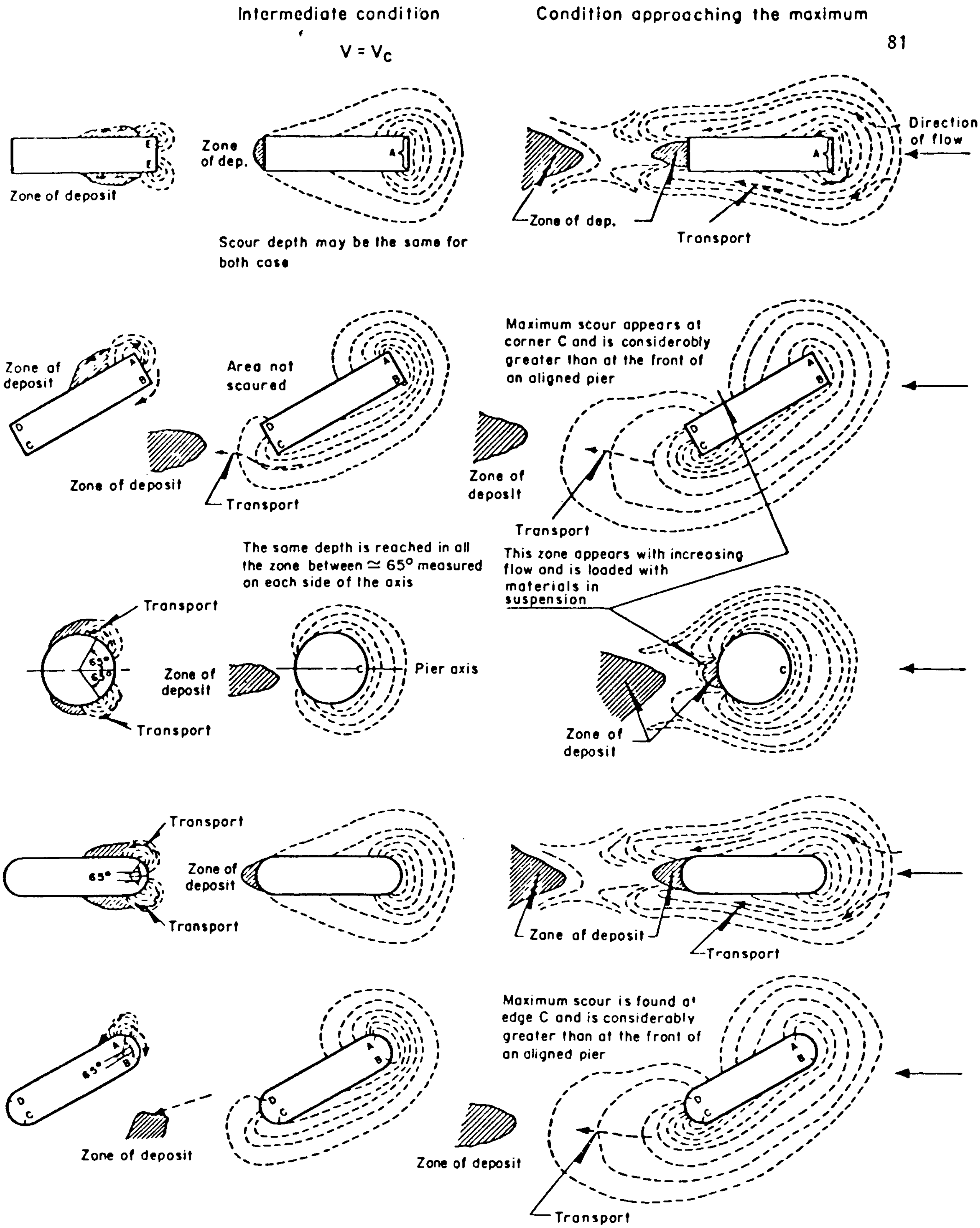


Fig 7.12
Diagrams which show different stages in the scour process (After Maza and Sanchez)

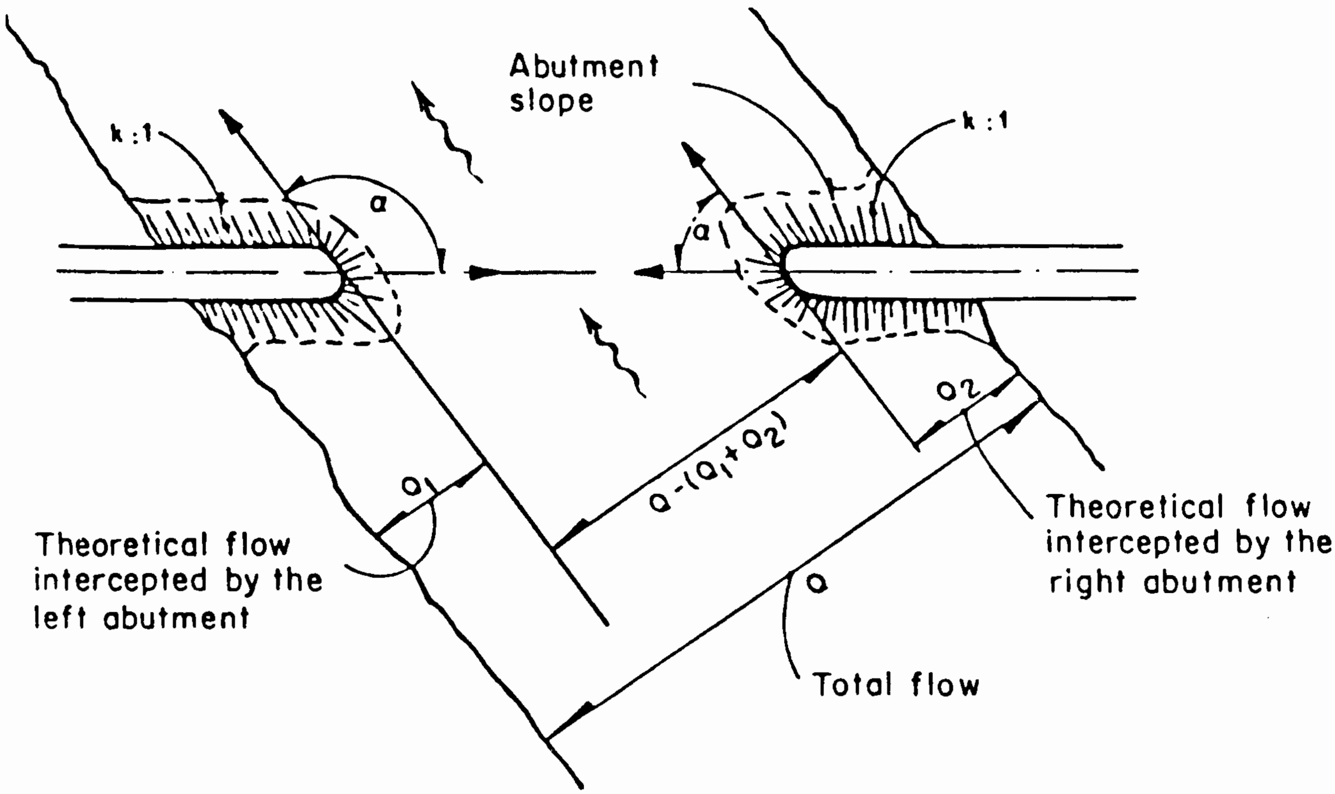


Fig 7.13
Layout of abutments

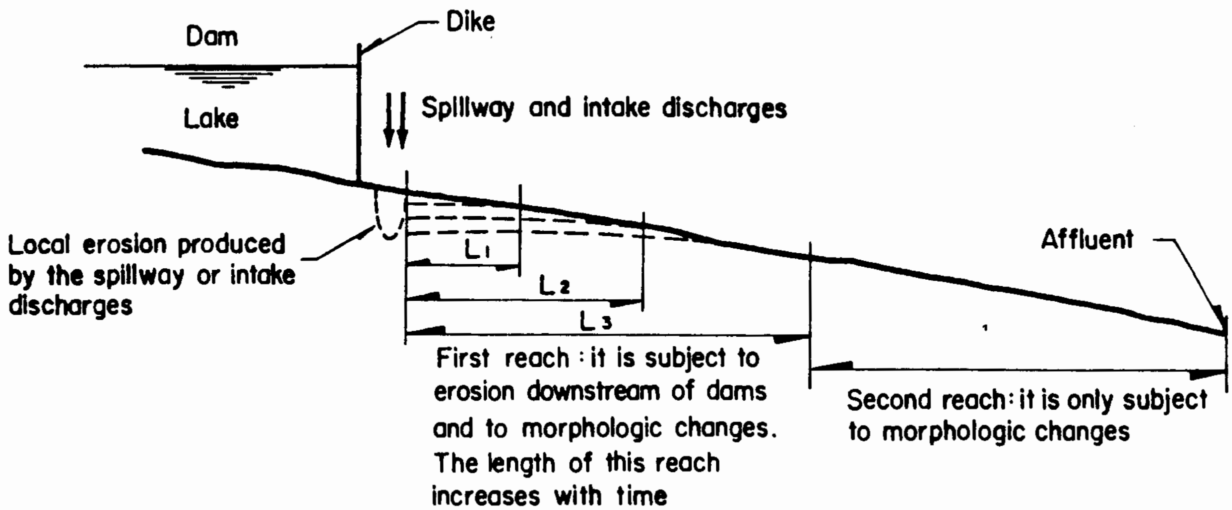


Fig 7.14
Changes a river undergoes downstream of dams

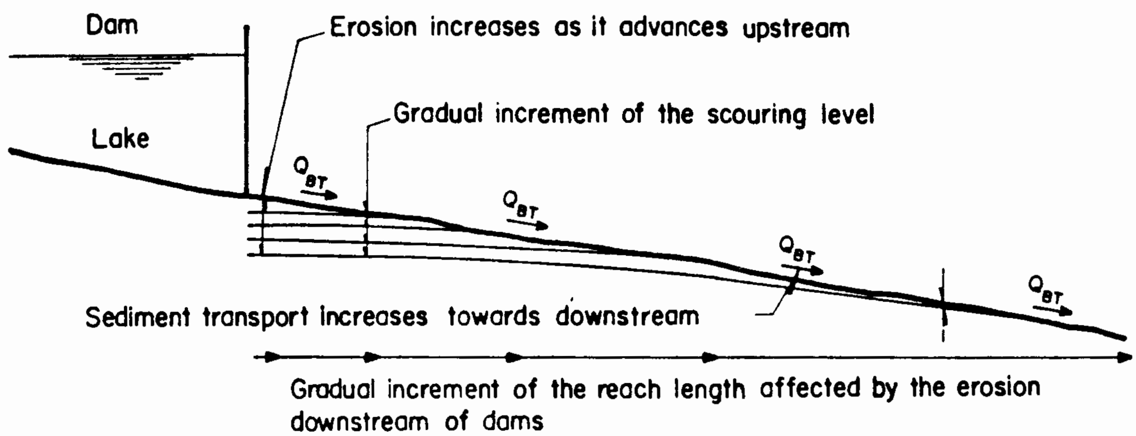


Fig 7.15
Effects of erosion downstream of dams

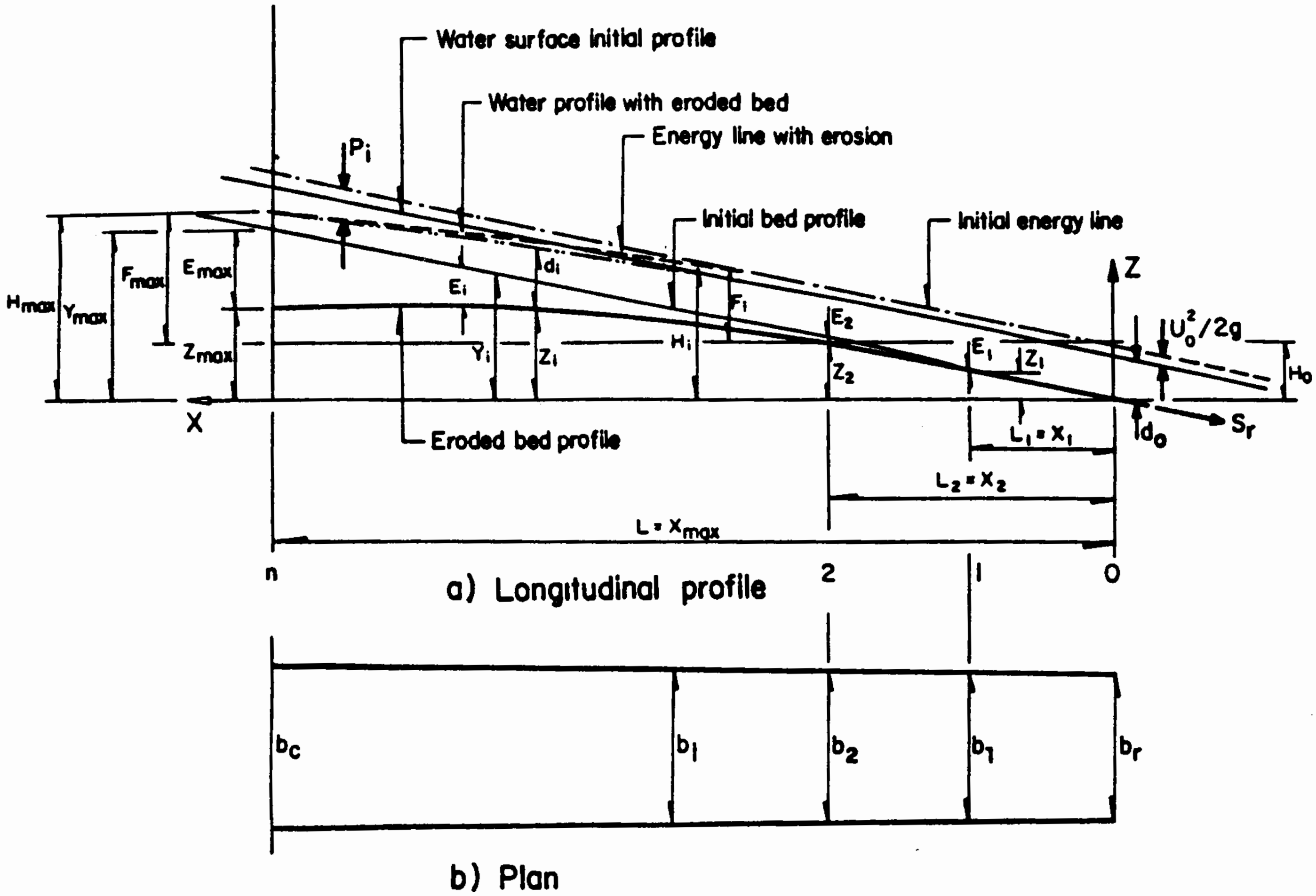


Fig 7.16
Erosion downstream of dams. Variables with the axis x as reference

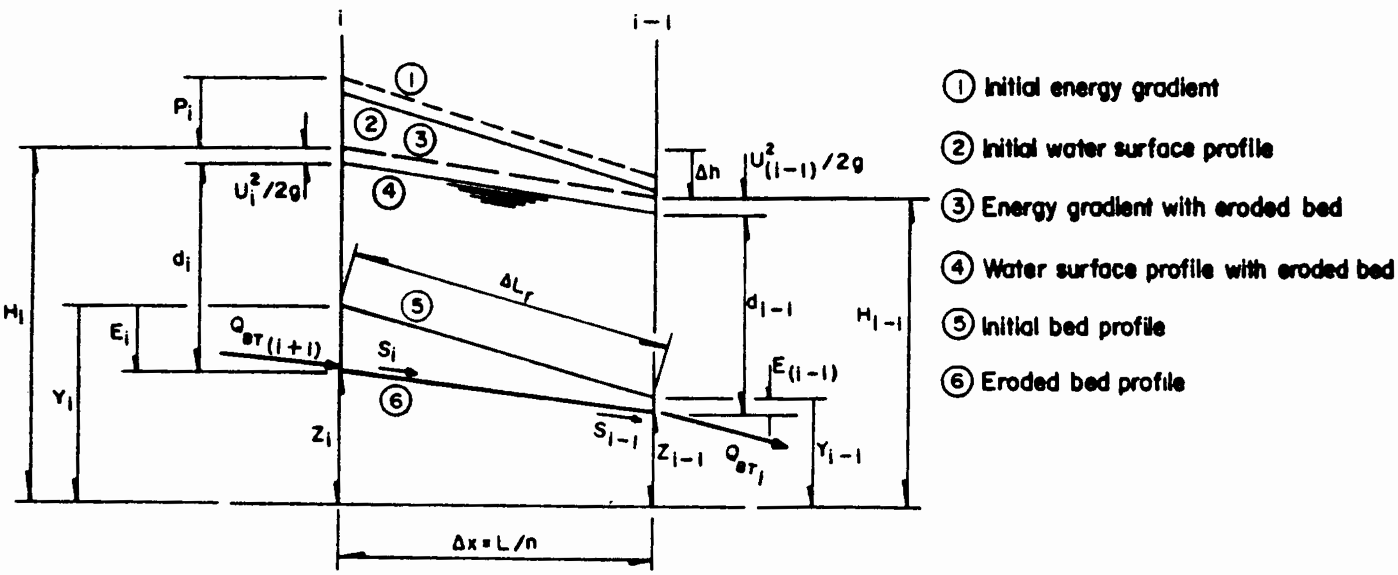


Fig 7.17
Calculation element

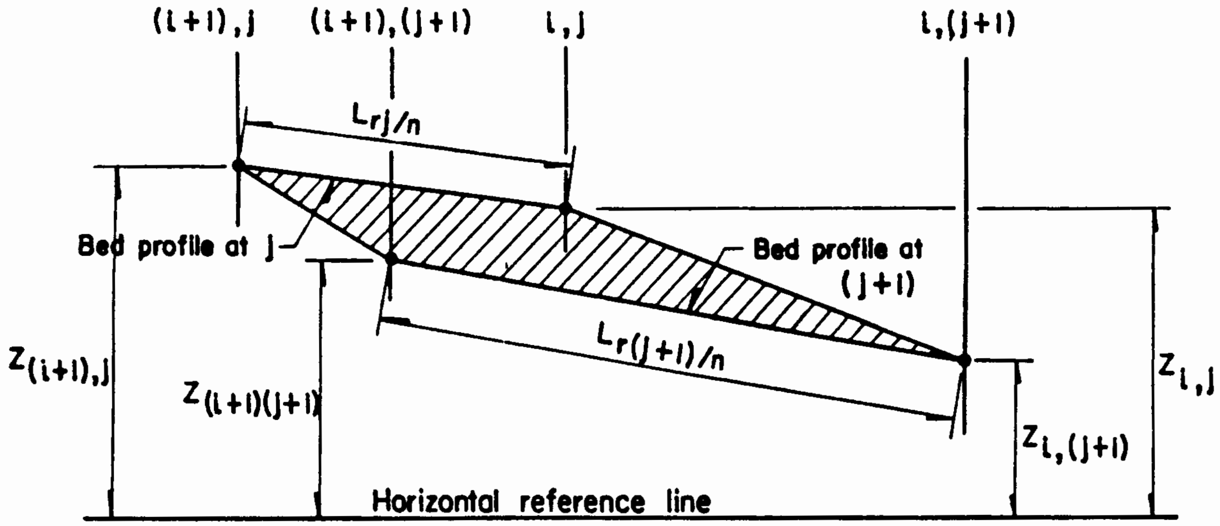


Fig 7.18
Eroded volume between the sections $i, i+1$ and the instants $j, j+1$

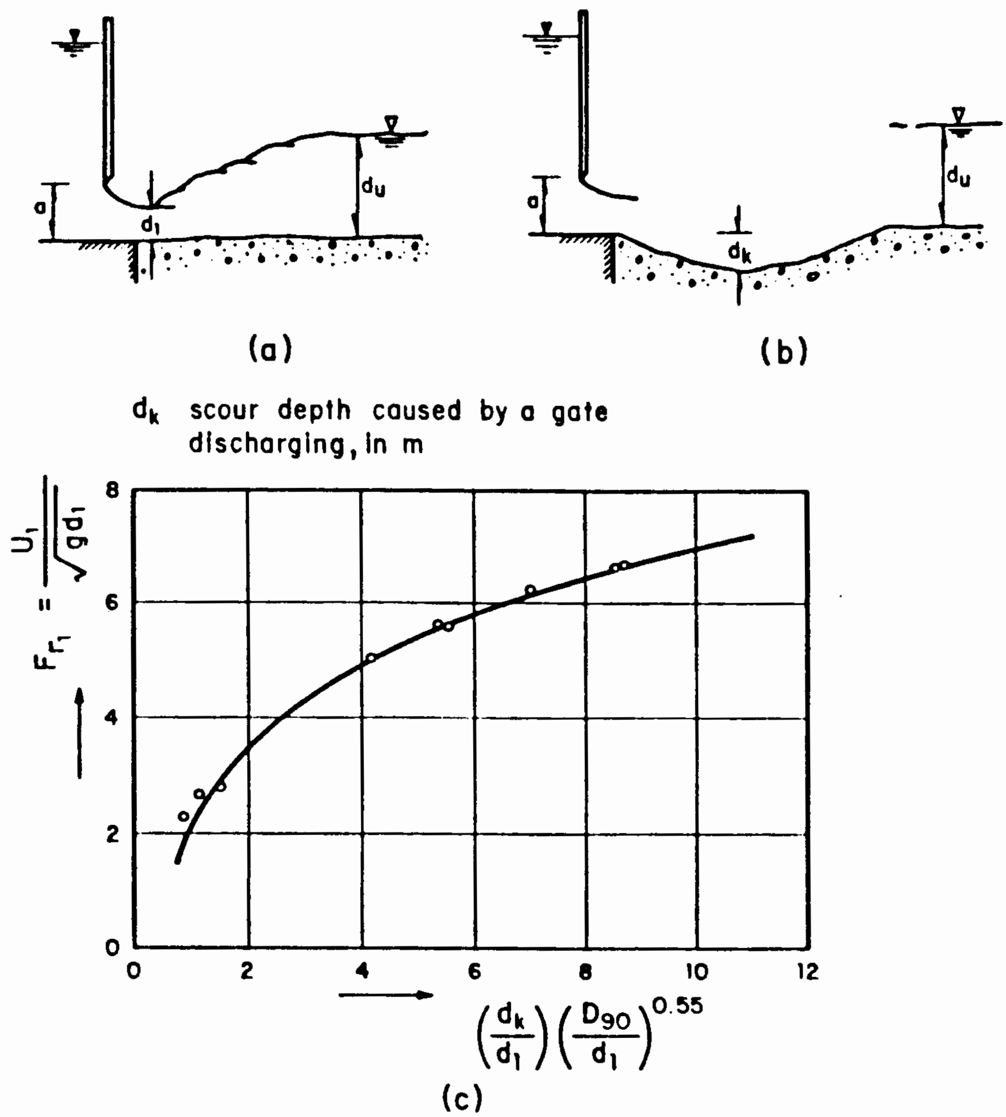


Fig 7.19

Scour downstrean of a gate (After Valenti)

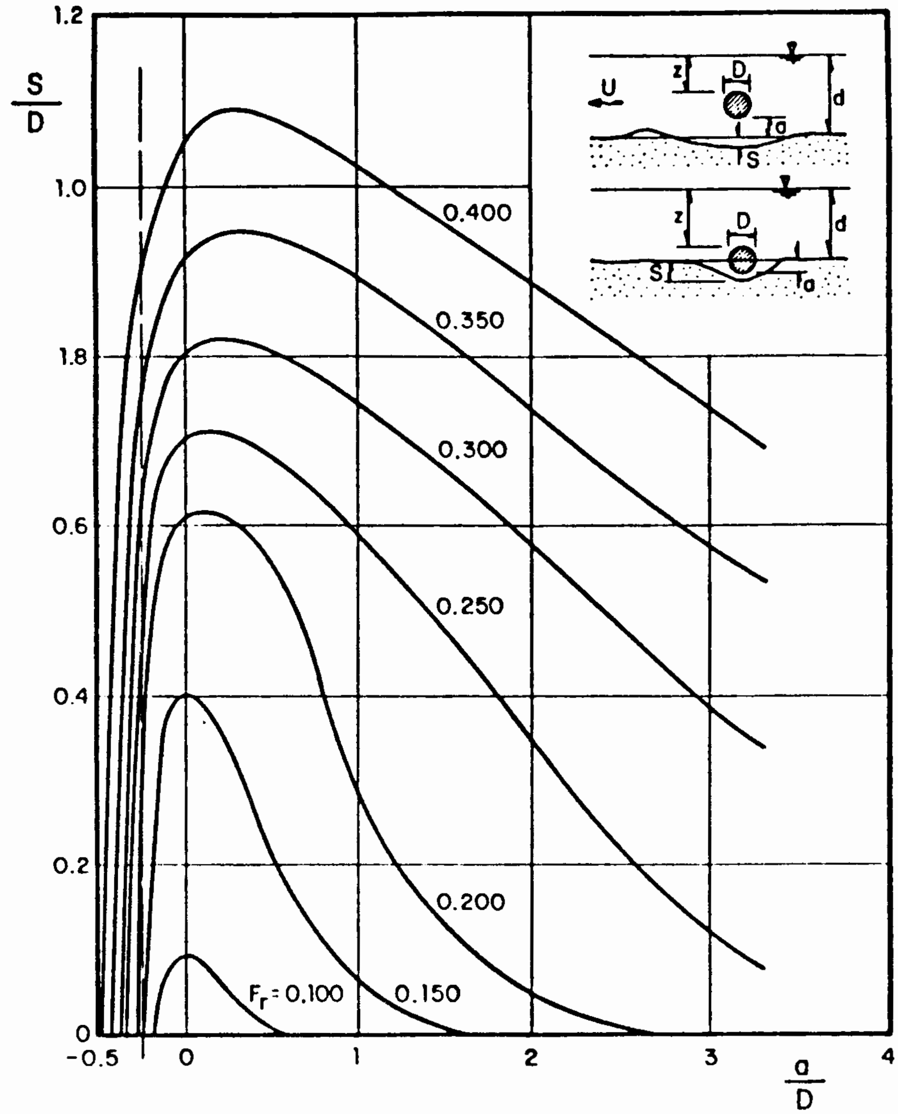


Fig 7.20

Local scour under a pipeline, as a function of $\frac{a}{D}$ and F_r

CHAPTER 8

PROTECTION WORKS

This chapter is about two kinds of protection works: a) those used to fix river banks and to avoid lateral flow displacements, and b) those meant to protect bridges; this is the most common structure among those built to interfere with the river discharges.

Protection works against floods are dealt with separately in Chapter 9.

8.1 General remarks

Most of the river stretches are stable. That is, for a given annual hydrograph, bed sediment transport, and the characteristics of the materials which constitute the river bed and banks, a river adjusts its slope, width and depth.

Different types or levels of stability have been mentioned in Chapter 6: static, dynamic and morphological.

The first one is reached by a river stretch when the current practically can not wash away the material which forms the river bed.

The other two stability levels do not imply that the river does not move; in fact, all rivers undergo a certain lateral displacement, mainly when they flow through alluvial soils. The lateral displacements occur on the exterior banks of the curves and can be of importance even in case of only one flood. In the inner side of a curve the bank also undergoes a displacement provoked by the deposit of alluvial material.

It has been stated that a stable river has a rather constant slope, width, and depth for the formative discharge. The lateral displacements generally provoke a decrement of the river slope, but in another nearby stretch a meander may be cutoff, thus resulting in a slope increase. So that, the general slope remains practically the same.

The major lateral displacements occur in curves. Due to the centrifugal force developed in these zones, the elevation on the exterior side rises, producing a bottom current from the exterior bank towards the inside. The addition of this bottom current to the river normal one results in a helicoidal current in the curves, that washes the bottom materials towards the interior bank. In this way, there is erosion in the extrados of the curve and a deposit in the intrados, which form a deeper channel close to the exterior bank. On the other hand, due to the existence of greater depths at the curves, the velocity increases close to the exterior bank, thus facilitating the remotion of materials from the bank.

When erosion takes place, the bank slope tends to become vertical until the material fails and the upper part slides into the river current. Then, the bank forms a gentle slope, but as the current washes away the bottom particles, the cycle starts again.

When the lower part of the side slope (zone 1, fig 8.1) is protected, the upper part (zone 2) is eroded and only the slope decreases. Nevertheless, the curve remains stable and the river channel does not displace laterally. But, if only the upper zone is protected, the current goes on washing away the material. The undermining produces a cavity until the upper part and the protection structures collapse.

8.2 Methods to protect the banks

The most common procedures for the direct protection of the exterior bank of curves are groins, revetments and longitudinal dikes. Their purpose is to avoid the contact of the high velocity stream lines with the bank materials so that they are not washed away.

The revetments, built with materials the current can not erode, directly the bank slope and the bottom of the river bed. Between these materials and those of the river bed a filter is usually placed to prevent the fine particles to pass through the openings of the protection. These particles are removed by the turbulence and vortexes of the current and the rapid fluctuations of the water level, especially during the flood seasons.

When revetments can not be placed against the bank material, longitudinal dikes are generally built; these, due to the quantity of material used in their construction, are the most expensive of their kind. Sometimes they are called spur dikes and are mainly applied to direct the flow towards the bridges (see 8.4.2).

Groins are structures supported or embedded in the banks and standing in the current. Besides moving the current lines away from the banks, they make the sediments to deposit among the structures.

8.2.1 Advantages of each type of protection

Revetments have two main advantages: first, they set the river bank definitively, thus preventing future displacements along the river bank configuration and, second, they do not reduce the original hydraulic area.

One of their most important disadvantages lies in their very complicated construction which highly increases the cost. Besides, they need a very careful maintenance, because a failure in even a small portion endangers the whole work.

On the other hand, groins are of easier construction and maintenance, what makes them cheaper and, in addition, the cost of maintenance diminishes with time. Besides, even if the end point of a groin is eroded, the rest of the structure continues to work and the destruction of one of them does not seriously endanger the rest. These dikes are also easier to repair.

However, they do have a few disadvantages: they reduce the hydraulic area and increase bank roughness. Also, they can not be used in curves with a small radius and they do not set the bank in a definite way.

Besides the works mentioned, there are others which deviate sediments to zones where they do no harm. They consist of small not very wide, screens placed on the bed or attached to floats; with a height up to 1/3 of the normal depth, variable length, and the longitudinal axis forming a $12°-20°$ angle with the flow direction.

Another type of antierosion work consists of low screens or dikes placed all along the bed of a cross section, thus favouring sediment deposit in the upstream bed. Its height is variable and depends on the problem to be solved, but generally 1.0 to 1.5 m above the bed is enough. This work is used to protect underground pipes, or bridges because it causes the bottom to raise.

The following paragraphs deal with groins, revetments, longitudinal dikes and local protections at the foot of piers, abutments and under pipes.

8.3 Groin design

Groins are structures with the form of dikes or screens interposed and are placed in the current, joined to the banks. They deviate the current away from the bank, preventing bank particles from being eroded.

The most important aspects to be considered when designing a groin protection are:

a) Layout, curve radius, length of tangents and stable width of the river.

b) Length of the groins.

c) Separation between groins.

d) Longitudinal slope of the groins crest.

e) Orientation angle with respect to the bank.

f) Permeability of the groins. Construction material.

g) Scour in curves and local scour at the end of the groin.

The first five points will be dealt with here, but with reference to the last two, only some brief comments will be included.

8.3.1 Layout

When a rectification is needed, a protection structure can be designed either for a natural bank or for a new one. In both cases, in the ground plan, an almost parallel line to the river axis must be drawn along the banks with the ends of the groins reaching it (see fig 8.2), being the length of the groins equal to the distance from the bank to the line.

The distance between the new river banks (width B) will be given by the study of the channel stability, taking into account the slope changes, if the river stretch is navigable and if the river channel is being rectified.

When the rectification of a sandy and silty river channel is considered, it is better, whenever possible, for the radii of the curves to the river axis to have the following length r

$$2.5\,B < r < 8\,B \tag{8.1}$$

where B is the mean width of the free surface in the straight stretch.

When the curve to be protected is uniform, all the groins have the same length and orientation angle and, therefore, the distance between them is equal (see fig 8.2a).

With the former radii, the protection will work efficiently. If the radius of the curve is smaller, the distance between the groins decreases and, from the economic point of view, it is better to build a revetment on the bank. If the radii are longer, the river tends to form a channel with smaller radii within the curve and, consequently, not all the groins will work efficiently.

When only the actual banks of the river are to be protected and no rectification works can be done, the line that joins the end of the groins should be as uniform as possible, even if it does not have a unique radius. These kinds of projects are the most common during the first phase of the development a region because the main purpose is to stabilize the banks with a minimum cost (see fig 8.2b).

The imaginary line which joins the end of the groins influences their length. The distance between them is then determined by this length and the orientation of the groin. That is why several possible locations for that line have to be carefully studied (see fig 8.2).

When protecting a single curve or a complete stretch, the first three groins on the upstream side should have a variable length: the first, the smallest possible one (equal to the depth), and the others will increase their length uniformly, so that the fourth attains the planned length (see fig 8.3). The longitudinal slope of the crest must be uniform for all of them and, consequently, equal to the remaining groins.

Finallly, it is important to note that even though the imaginary line that joins the ends of the groins may have different radii curvature, all the radii of that line should be measured towards the same side, that is, towards the inside of the curve.

When a curve is to be formed with several radius like, for instance, three, their radii must decrease from upstream towards downstream. This means that the segment of the circle with longest radius should be upstream, the intermediate in the middle part, and the smallest, at the downstream lowest end. Straight stretches among them should not exist, but placed one after the other, until leaving the curve and entering into the next straight stretch.

8.3.2 Length of groins

The total length of groin is formed by the embedded and the working or active length. The first one is inside the bank and the other lies in the current.

The working length of the groin measured on the crest is independently choosen within this already proven limits

$$d \leqslant L_T \leqslant B/4 \tag{8.2}$$

where B is the mean width of the channel and d is the mean depth of the flow, both for the dominant discharge.

The groins may be built without an anchoring length, that is, without penetrating into the bank, but only resting at its edge. The maximum recommended length is equal to $L_T/4$. So the total length is (see fig 8.5a)

$$L = 1.25\, L_T \tag{8.3}$$

It has been mentioned that the working length of a groin is independently choosen. However, and as already stated, all the points of the groins should reach a line which may depend on a preselected groin length.

For economic reasons it is better to make the anchoring length as short as possible. The technique followed consists in finishing the groins directly against the bank, and that is why some of them are flanked (approximately 4 per cent). However, repairing the damages that may occur in a few groins is much more economical than to embed them. The repairs are made during the following dry season, and they consist of lengthening the groin until it is connected to the bank of the eroded edge. The groins usually fail during the first flood season but, once repaired, they work adequately almost without any further maintenance. When a failure must be absolutely prevented in a reach of special interest, it is convenient either to reduce the distance between the groins or to embed them all into the bank at a maximum anchorage length of 0.25 L_T.

8.3.3 Distance between groins

The distance between groins is measured at the river bank between their starting points and primarily depends on the length of the upstream groin, its orientation, and the bank location. For its calculation it is necessary to consider not only the angle between the groin and the downstream bank, but also the theoretical expansion of the current when passing by the end of the groin, usually at 9° to 11° (see fig 8.4).

The recommended separation for groins in straight stretches with no bank anchorage is

α	Separation S_P	
90° to 70°	(5.6 to 6.8) L_T	} Normally 5.5 L_T
60°	(5.4 to 6.6) L_T	

The recommended separation S_P between groins placed in curves should be graphically obtained as indicated in fig 8.4. If the curve is regular and with only one curvature radius, it is

$$S_P = (2.5 \text{ to } 4)\ L_T \tag{8.3}$$

For curvature radius of more than 4B, separations of $4L_T$ have been used. If the curve is irregular or it has a small radius of curvature, the separation between the groins will have to be found graphically (see fig 8.4). At the same time, their lengths and orientation angles are chosen.

When groins are built with no anchoring length within the bank, the recommended tested separations are slightly smaller the theoretical ones obtained from fig 8.5 and eq 8.4. If groins are anchored with 0.25 L_T length, the separation can be the theoretical one indicated in the above mentioned figure. For a more economical structure, groins can be separated by 8 L_T in the straight reaches and 6 L_T in the curves. But the following year, intermediate groins of shorter length must be built on the upstream reach of the failed ones or of those that have threatened to fail.

8.3.4 Crest elevations and slopes

Groins were constructed without a longitudinal slope ($S = 0$) towards the center of the river channel and with slopes from 0.02 to 0.25. Besides, experimental groins with horizontal crests and slopes from 0.1 to 0.5 and 1 were also tested.

According to these tests, groins should slope towards the center of the river, beginning at the bank elevation or at that of the water level of the formative discharge.

The maximum height of the extreme end within the channel should be 0.50 m above the actual river bed, with slopes from 0.05 to 0.25. The groins constructed with longitudinal slopes of 0.1 or more favor sediment deposition between them and are more economical (see figs 8.3 and 8.6).

Groins with so great longitudinal slopes towards the center of the river channel have the following advantages

a) There is practically no local scour at the end of the groin.

b) When groins have vertical walls (sheet piling), only slight erosion takes place at the foot of the upstream side.

c) When groins have tilted sides (rockfill) with slopes of 1.5:1, deposition takes place adjacent to the foot of the downstream face which protects the groin.

d) Each groin nly requires between 40 and 70 per cent of the whole material needed to build groins with horizontal crests, but the greatest savings are achieved with rockfill or gabions.

e) With these slopes the deposition of sandy material between the groins takes place faster than when the crests are horizontal.

f) There are no flanking problems with any of the groins built with these slopes and separated four times the working length, taking into account that they were only tested.

In all the observed structures, the elevation of the crest at the starting section was the same as that of the bank.

8.3.5 Groin orientation

Groins can be oriented downstream, upstream or perpendicular to the water current. The orientation is measured by the angle the groins longitudinal axis forms with the downstream tangent of the bank at the starting point (see figs 8.3 and 8.5).

In the regular curve of a straight stretch, it is better if groins form a 70° angle with the direction of the current. For an irregular curve, and more so if its radius is less than 2.5B, the orientation angle should be smaller than 70° and may even reach values of about 30° (see fig 8.2b).

Orientations which exceed 90° need smaller distances between groins, consequently, more of these structures will be needed to protect the same channel length. 120° angles were tested, but they did not work satisfactorily: when one of these groins failed, there was much more bank erosion than when the spur dike inclination was between 70° and 60°.

For angles between 70° and 90°, groins had practically the same length (see fig 8.5). Since water current is not parallel to the bank for all discharges, allowance should be made for an angle of 70°, instead of placing perpendicular groins.

In a curve with a very small radius of curvature ($r < 2.5B$) and with α less than 40°, it starts to be better to build a revetment for protection.

8.3.6 Permeability, construction materials for groins

A large variety of materials can be used in groin construction: wood, tree trunks and branches, rockfill, mortar, concrete, steel prefabricated elements, wire, etc. The most usual in our country are built with woodpiling or rocks as rockfill or gabions (boxes made of wire-mesh).

If groins are supposed to remain permanently inside the main water channel, it is advisable to make them impervious to isolate as effectively as possible the current from the bank. But when groins are used to reduce the current velocity in a zone that is to be afterwards filled up with the sediments transported by the river (for instance, to form a new bank), it is better to make the groins permeable to let the water current pass through them and deposit its sediments.

Construction materials should be strong enough to resist the current of water and the impact of tree trunks and of the floating objects the river may transport. For this reason, and also because of the loss of resistance due to rotting, groins made of tree trunks and branches are usually destroyed.

8.3.7 Local scour in spurs

During construction, local scour is important at the end of groins when loose elements such as rocks, gabions, etc., are employed and, besides, water current velocity is of more than 50 cm/s; then, it is advisable to cover the bottom of the river channel where the groin will rest with a layer of riprap 30 cm thick, and afterwards to build the spur dike from the bank towards the center of the channel (see fig 8.6a). If that layer is not placed in advance to avoid local scour during construction, greater quantities of material will be needed.

Local scour at the end of groins is not so important an influence if the structure has a large longitudinal slope.

8.4 Revetments and longitudinal dikes

Revetments are structures which rest directly on a river bank to prevent water flow from touching it.

When the bank can not be directly lined because the protection has to be placed away from the actual bank, or because it is so irregular that it needs a continuous regular protection, dikes have to be built with the slope in contact with the flow duly protected.

Plan geometry of revetments and dikes is similar to that of groins. Anyway, it is always better to use the banks because revetments are not so expensive as dikes.

Revetments can be made of concrete slabs, gabions, prebuilt concrete elements and rock. The building procedures vary according to the material chosen, machinery available, place, water levels in the river and their periods of permanence.

Due to the fact that bank material must not protude from joints or holes of the protection walls or dikes a filter must be placed between both. It can be made of synthetic or bituminous materials or duly graded stone.

If rock is used for the revetment, rock size must be obtained through the application of the critical shear stress criterion. Artificial materials generally resist current flow and are not carried away.

The most destructive process takes place at the foot of the slope or in the revetment starting point. For protection, one of the following two solutions may be applied: a) if the construction is built on dry ground, either a ditch 1-2 m deep can be dug and filled with gabions or rocks to support the wall or a sheetpile can be set, b) the second solution consists in a rock carpet of width equal to the formative depth, or at least 2 m, and 40 to 70 cm thick. When due to floods the bed lowers, the protection lowers too over the scoured bottom, thus preventing the protecting layer of the revetment from sliding.

Upstream, the revetment end or starting point should be embedded towards the inside of the bank.

The simplest kind of revetments is made of rockfill or riprap. It is better if stones have different sizes because the smaller work as filters.

Revetments should be periodically checked, especially after the flood season to repair scoured or eroded parts. It must be kept in mind that when part of the revetment fails, all the structure is in danger.

If the curvature radius of a curve is smaller than 2.5 times the width of the free river surface, revetments instead of groins should be installed as protection.

When river banks are very high, protections should cover a section from the river bed to levels of the formative discharge or of the mean flow in the flood season. If such section is protected, the bank will not recede because erosion at the bank foot is avoided; however, some can take place above the protection but only leading to a reduction of the bank slope until it stabilizes. It is advisable to cover the upper, unprotected part with vegetation.

Revetments should be flexible, that is, built with rock, riprap or gabions, because, if erosion or bank slumps occur, revetments can settle without failing. Rigid revetments of concrete plates of masonry can not move, that is why their construction requires great control and the bank to be carefully consolidated for good support.

Revetments are placed over river banks, but also on earthfill slope, roads or flood protection dikes when these structures may come in contact with running water that erodes them.

Besides, it is even better if some protection or rectification works combine groins and revetments.

When a navigation canal is built, its banks are generally lined, not really because of the flow, which may be very slow, but because of the waves produced by the boats.

8.5 Protection works against local erosion

The design of these kind of protections differs according to the works to be protected: piers or bridge abutments, groins, pipes, etc. Those suggested here are mainly useful for sandy and gravel soils, and are only recommended for piers, bridge abutments and under pipes placed transversally to the direction of the current. The main problems provoked by local erosion occur in the intermediate and plain zones or rivers.

The protection works discussed here may be unnecessary if, at the very beginning of a project, all possible scours are considered and evaluated, and if the future lateral upstream movements of the banks are forseen.

8.5.1 Protections against scour at the foot of piers

The protections here mentioned are useful for all kinds of structures in river beds and completely surrounded by flow.

Two different methods are recommended: the first one, suggested by Levi and Luna, can be used when the longitudinal axis of the pier is aligned according to the direction of the flow. In this case, a screen is set in front of the pier, at a distance 2.2 times its width. The width of the screen is equal to that of the pier (see fig 8.7).

If the screen is set during the construction of the pier, it reduces erosion in 70 per cent, but it also works when scouring has already occurred because it tends to refill ditches. The screen height can be of one third of the maximum flow depth, noting that screens are no longer useful when the axis of the pier forms an angle with the flow direction.

The second protection method consists in rockfill or riprap placed at the foot of the pile. In this case, the size of the stones to be used can be obtained from table 8.1, where stone diameter —given in centimeters—, is a function of flow velocity, depth and the specific gravity of the stone.

If there is no doubt that the pier will always be aligned with the direction of the flow, the protection is placed in front of it; if not, it is better to surround the pile with the protection (see fig 8.8).

It is also advisable to dig so deep as to have the maximum level of the protection at the level of the general scour (see fig 8.8).

The material must form a minimum of three layers to prevent the finer bottom materials from being carried away by the vortexes usually originated at the corners of a rectangular pier, or in an area of 65° at each side at the front of a circular pier or of one with rounded corners.

Sometimes, it is necessary to protect the actual bottom without digging in front of the piers, and in these cases the general scour must be considered. The protection should be placed during a flood, due to the fact that there is not only some general scour but also a strong erosion at the pier foot; this can be filled with stones (see figs 8.8 and 8.9) by letting them fall through a pipe to prevent the current from carrying them away. It must be remembered that the main cause of bridge failures is scour at the foot of piers produced rather by general scour than by local erosion. However, local scour is easiest to avoid or to protect from.

8.5.2 Protection against scour at the foot of abutments

Scouring at the foot of abutments can be prevented by means of two different procedures: 1) by substituting the erodable bottom material by a riprap of similar characteristics to those described in paragraph 8.5.1, or 2) by placing at each abutment end a spur dike (see fig 8.10).

The plan geometry of a spur dike corresponds to a portion of an ellipse. To avoid any erosion at the abutment, another dike is placed downstream with a length approximately equal to a third of the first one.

For the first approach, the plan geometry can be fixed by means of the criterion proposed by Latuisckenkov, according to which the magnitude of the semiaxis of the ellipse is a function of the ratio Q/Q_m, where Q is the total flow which runs in the river during floods, and Q_m is the theoretical flow which runs in the width of the bridge span (see fig 8.11). To do this, the magnitude of X_o and Y_o must be known

$$X_o = \lambda B_m \tag{8.5}$$

$$Y_o = Z X_o \tag{8.6}$$

where

- B_m bridge span between abutments
- λ coefficient which depends on ratio Q/Q_m, and on whether there are one or two spur dikes. Its value is found in fig 8.11
- Z is a function of Q/Q_m. Its value is given in the following table

TABLE 8.2. VALUES OF $Z = X_o/Y_o$

Q/Q_m	1.175	1.19–1.33	1.34–1.54	> 1.55
Z	1.5	1.67	1.83	2.00

- Y axis always parallel to the desired direction the stream lines flow under the bridge section (see fig 8.12).
- X axis perpendicular to Y; its direction is towards the bank.

Once X_o and Y_o are known, the remaining points are given by

$$X = X_o \left(1 - \sqrt{1 - \frac{Y^2}{Y_o^2}} \right) \qquad (8.7)$$

The downstream dike is symmetrical to the first third part of the upstream one, but it should be built only up to $Y = -(1/3)\, Y_o$.

The upstream end of the dike is extended by means of a segment of a circle with a radius of $0.2\ B_m$, covering an angle of 30° (see fig 8.12).

These protections not only prevent scouring at the foot of abutments but induce the current to follow the longitudinal pile axis. That is why they are most often used to guide the flow under a vridge than to avoid local scouring at abutment ends.

As already recommended, protection by spur dikes has the advantage of making flow uniform under the bridge section and of keeping the flow direction against the piles constant. The main disadvantage of this solution compared to the riprap protection lies in its cost. Due to this fact, riprap is generally used when it is not necessary to guide the flow but to protect an abutment foot.

As already said, spur dikes to direct the flow through the section under the bridge are generally very long and, therefore, very expensive.

To obtain more economical solutions, studies have been carried out with hydraulic models and it has been observed that it is convenient both to keep the elliptic shape of the dikes and to reduce the semiaxis size. This is done by lowering λ and Z up to 30 per cent of their original values, what still provides safe protection and performance. This kind of reductions should be tested in hydraulic models.

8.5.3 Protection against erosion under pipes

A protection made of a stonefill or riprap has been proved, with the minimum diameter indicated in fig 8.13.

It was observed that if the pipe is partially exposed, it is better to place over it and at its bottom a volume of material capable of supplying enough refill, since some scour will be undoubtly produced. The size of the rock elements is given in table 8.1.

8.6 References

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TABLE 8.1 MINIMUM DIAMETER OF THE STONES THAT FORM THE PROTECTIVE ROCK AS A FUNCTION OF γ_s AND U FOR A DEPTH OF FLOW EQUAL TO 1m*, in cm

Mean velocity U_1, in m/s	Specific gravity of the material in kgf/m^3 1 600	1 800	2 000	2 200	2 400
1	8	8	7	6	6
1.3	15	13	12	11	10
2.0	18	16	13	13	12
2.5	27	24	21	19	18
3.0	38	34	31	28	26
3.5	53	46	42	38	35
4.0	68	60	54	50	46
4.5	86	77	69	63	58
> 4.5			85	77	70

* If depth, d, is different from 1.00 m

$$U = U_1 \, , \; d^{\alpha}, \text{ where } \alpha = \frac{1}{2 + d}$$

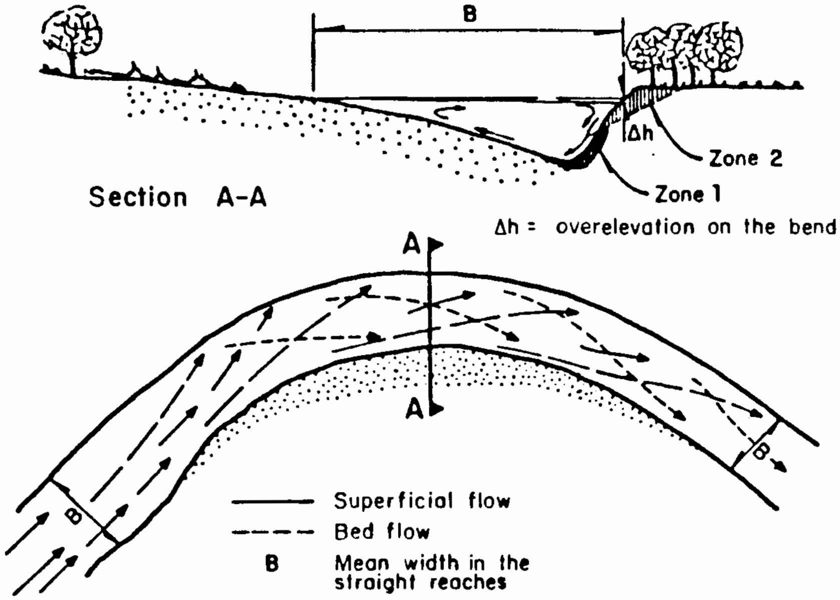


Fig 8.1
Scheme of flows in river bends

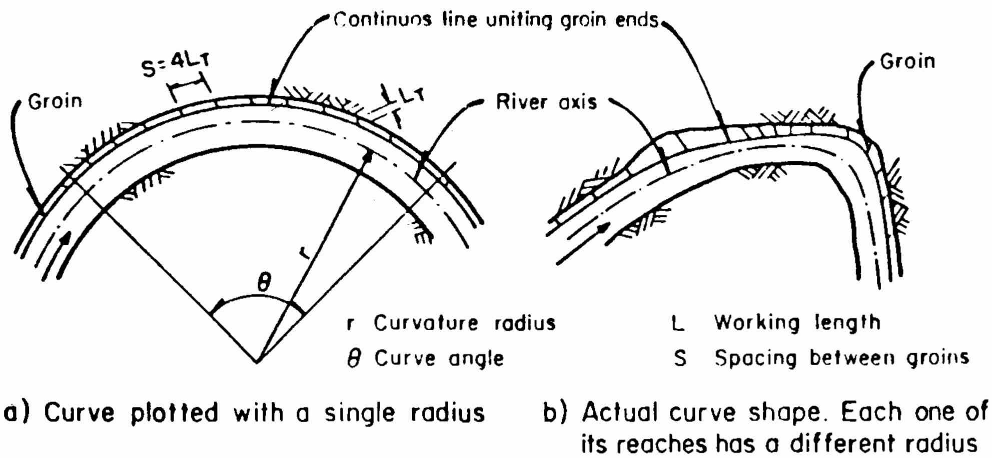


Fig 8.2
Layout of a protection with groins

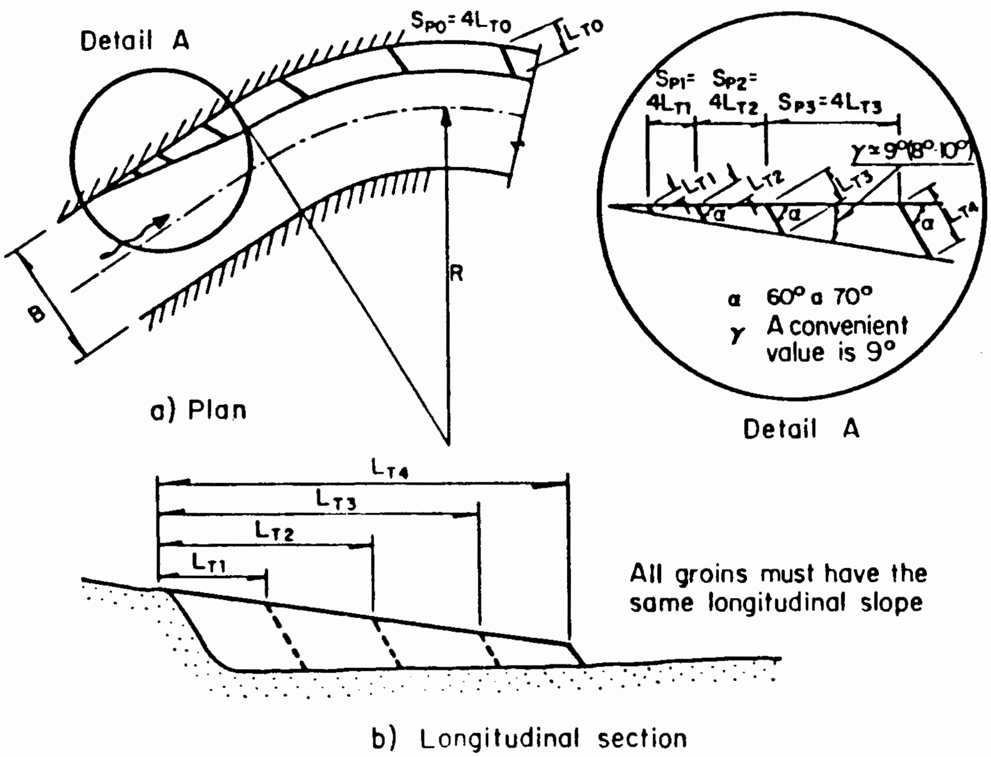
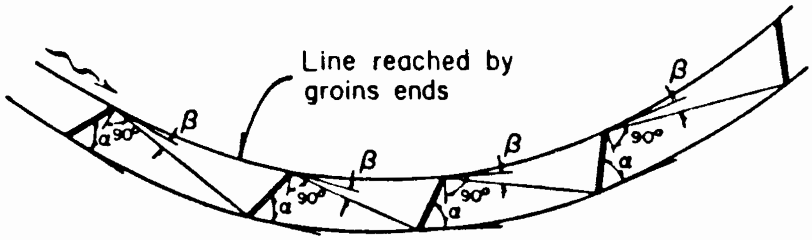
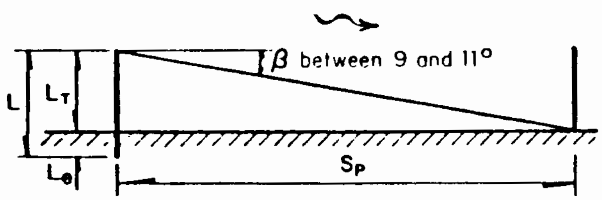


Fig 8.3
Layout of the first groins for a protection work



Note: If the curve is circular all lengths and spacings are equal

Fig 8.4
Layout and separation of groins in a curve



$5.1 L_T \leq S_P \leq 6.3 L_T$

(the biggest possible spacing is convenient but if groins are not embedded, S_P should be $5 L_T$)

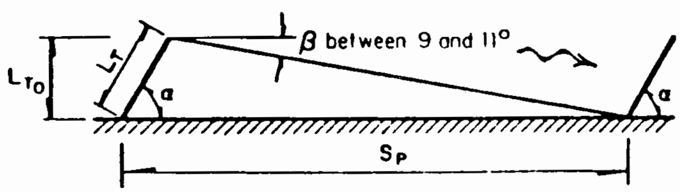
L_T Working length

L_e Embedded lenght

L Total lenght

a) Groins perpendicular to the current (plan)

$S_P = L_T(\cos\alpha + \text{sen}\,\alpha \cot\beta) = L_{T_0}(\cot\alpha + \cot\beta)$

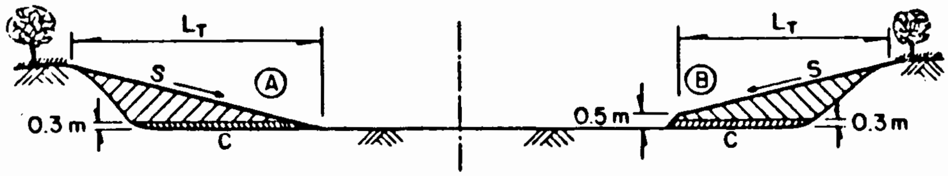


α	L_T	S_P	S_P Embedded yes	no
60°	$1.06 L_{T_0}$	$(5.4$ a $6.6) L_T$	$6.5 L_T$	$5.5 L_T$
70°	$1.15 L_{T_0}$	$(5.6$ a $6.8) L_T$		

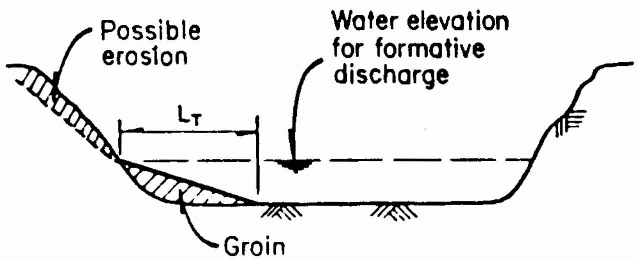
b) Downstream oriented groins

Fig 8.5
Layout of groins in straight banks

Slope S must be uniform. Desing (B) also had a good performance. The foundation C must be built first to avoid local erosion



a) Construction of groins when height between bed and plain is small



b) Construction of groins when height between bed anc plain is great

Fig 8.6

Groin construction as a function of the height between bed and plain

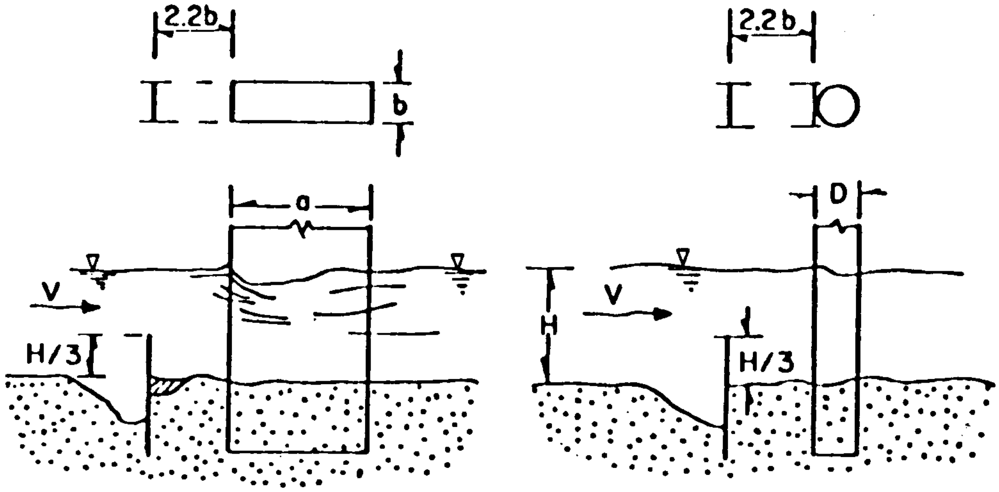
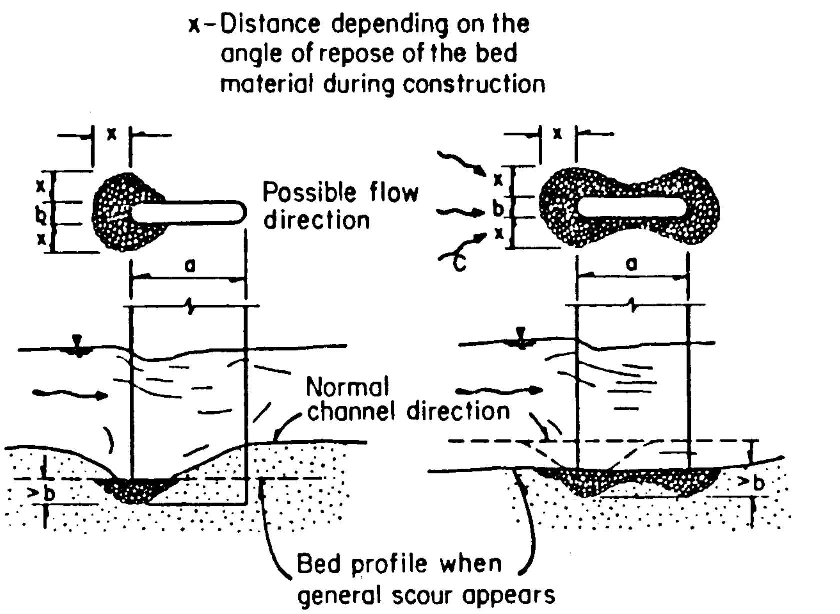


Fig 8.7
Placing of protective screens according to Levi–Luna



a) Angle of attack of zero degrees. The thick line of the bottom shows that the scour is local and general scour conditions do not appear

b) Variable angle of attack. The thick line of the bottom shows maximum general scour conditions

Fig 8.8
Methods of placing riprap to avoid local scour (After Maza and Sánchez)

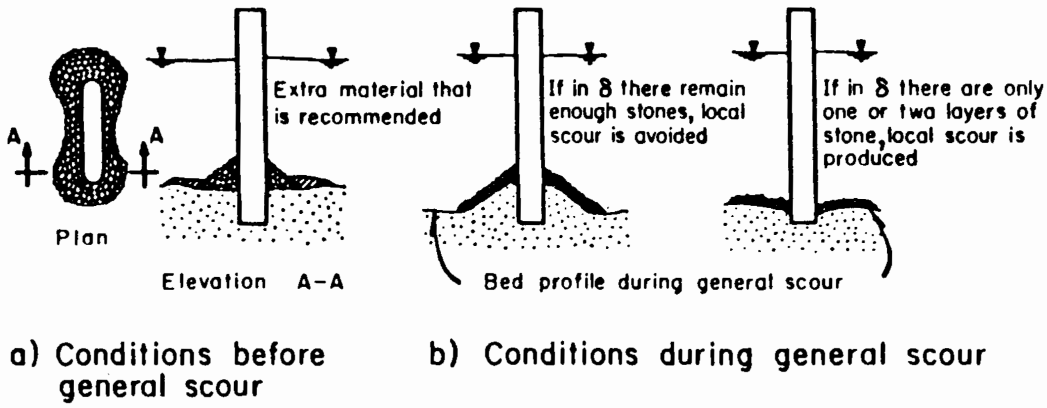


Fig 8.9

Behaviour of a mound placed on the actual bed with the aim of protecting the pier

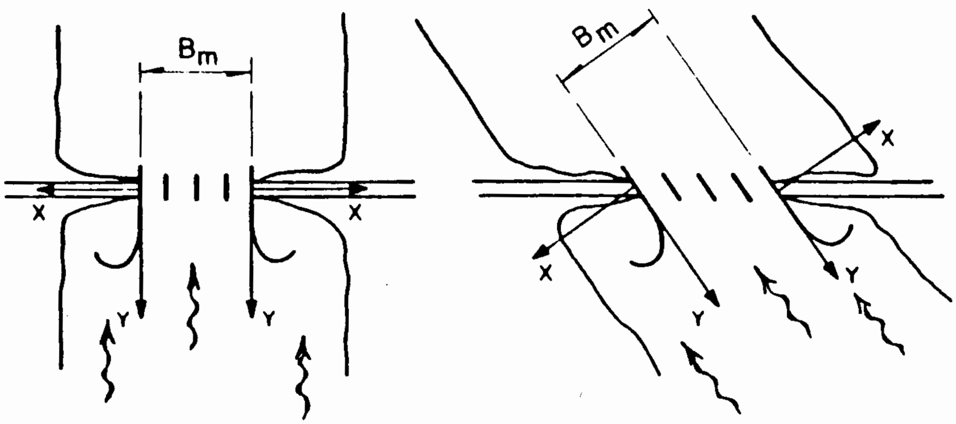


Fig 8.10

Placing of channelling dikes with respect to the longitudinal axis of the bridge and the flow

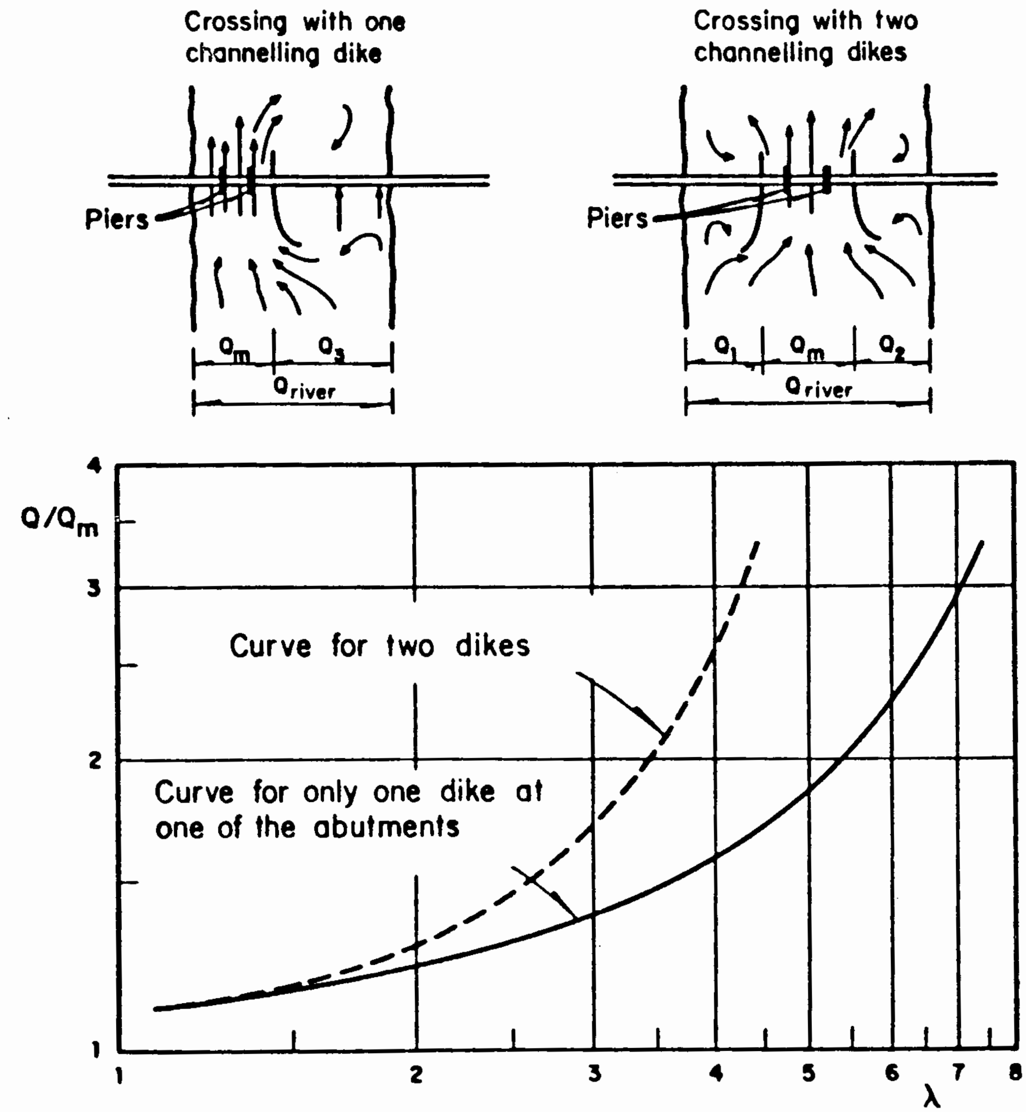


Fig 8.11
Values of λ *as a function of* $\frac{Q}{Q_m}$

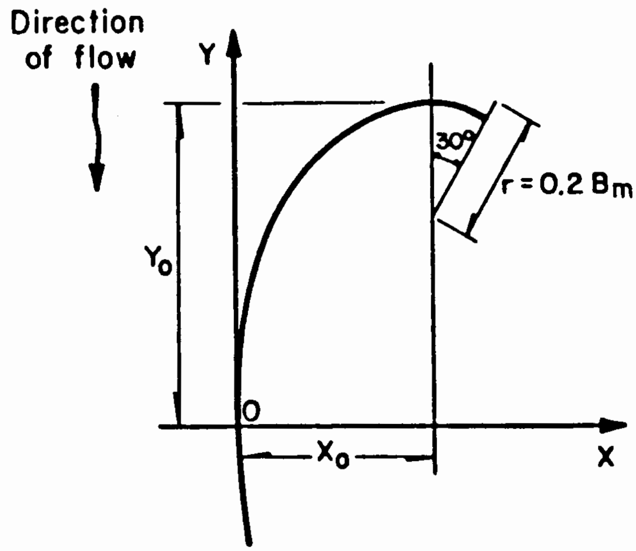
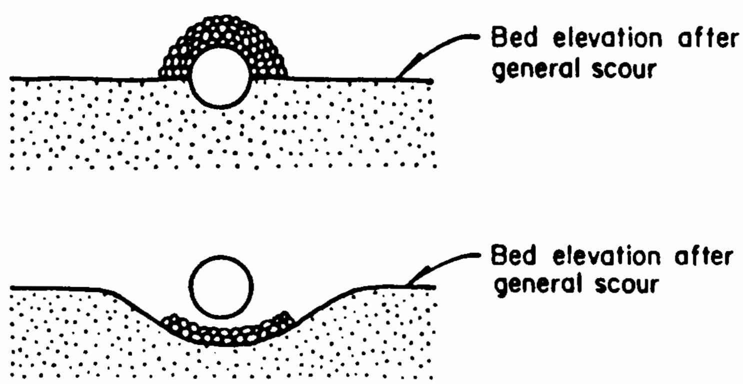
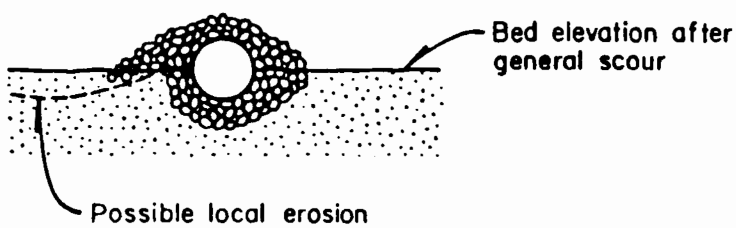


Fig 8.12
Geometry of a spur dike



a) Wrong protection. Erosion is produced if rock protection is placed only over the pipeline



b) Correct protection placing

Fig 8.13
Rock protection for a pipeline

CHAPTER 9

FLOOD CONTROL

History shows that man has always settled in places where water provision is guaranteed. These places are found at river banks and at the sides of lakes. In fact, the great civilizations of the past developed along rivers like the Ganges, the Tigris, the Euphrates, the Nile, etc., where there were fertile lands and intensive agriculture could take place.

For practical ends, in this work we shall distinguish three zones along a river: mountain, intermediate or piamonte and plain (see 6.4).

People living in mountain zones almost never suffer from floods because they can build their houses in the highlands. However, in intermediate areas, agricultural surfaces undergo them, even if the losses of houses or animals are not very significant because they may be higher than the areas the water covers. But almost all plain zones are subjet to floods. Because their potentialities for development are greater, it is here where most important flood protection works must be built.

The variation of a river flow depends on the watershed rainfall distribution in time and space. When flows are high, they often tend to exceed the river main channel capacity, causing the water to overflow and flood the adjoining lands. Its topography will determine whether the overflowed water runs on the plains in the general direction of the river, thus returning to it when the water level descends, or remains in lower zones until it evaporates, infiltrates, or finds other rivers. In general, overflowed water can (see fig 9.1)

— Infiltrate
— Evaporate
— Run over a plain and then return to the river
— Run over a plain and find another river
— Feed existing lakes
— Remain in lower zones until infiltrated, evaporated or used.

The volume of the overflowed water is substracted from the river main channel flow, lowering the maximum river levels downstream. In consequence, big floods may appear in reaches upstream and not in zones close to the sea. When a river plain zone is short or it does not exist, as in the Pacific slope of the Americas, floods almost cover all the river length in the plain zone. But since slope is strong and flows and watersheds are small, floods only last a short time, partly because of the high velocities the water can reach. On the contrary, in the American Atlantic, plains have smaller slopes and are fully developed; in it, floods cover big extensions of land and last longer, also because both maximum discharge floods and watershed areas are greater.

9.1 Sediments

In a natural current there is a water discharge as well as a sediment load transport.

Due to the different fluvial behaviours, two different kinds of sediments can be distinguished (see Chapter 5).

a) Sediments at river beds and banks

b) Wash load

The first one can be transported either on the river bed or in suspension. The transported volume is a function of the hydraulic characteristics of the current and those of the material, specially diameter and specific gravity. As shown in Chapter 5, a number of methods may be used to predict this transport.

The second one consists of very fine material in suspension, which does not come from the bed (when there are no data about this material, a diameter under 0.062 mm is supposed). To evaluate this solid discharge, there is no other procedure but that of measuring it directly, since watershed contribution varies with rain, ground composition, vegetal cover, etc. Measurements must be taken daily or as periodically as possible.

When an increase in liquid discharge causes overflow, greater amounts of sediments are carried out from the river channel. The coarser particles deposit as soon as water velocity diminishes, being this the reason why river banks may present higher elevations than the adjoining terrains. On the plain, solid particle sedimentation goes on; the heaviest deposit occurs in lower zones or where vegetation density is greater, thus producing a decrement in flow velocity. Overflows tend to level plains by filling the lower parts with sediments and raising at the same time its entire level; these elevations are bigger upstream and diminish slowly towards the sea, causing an effect which can be clearly seen during the dry season; near the sea, the difference in level between the surface of the water and the plain may range from a few tenths of centimeters to one or two meters at the most (in this case there is also the influence of the tides). Upstream, such difference increases little by little up to several tenths of meters, about 200 to 300 km away from the river mouth. This is valid only for plain zones, where the river is the only land shaping agent.

Very little of the sediment transported by overflowed water returns to the main current —and generally in the form of wash load— because it is too fine to be deposited even at very low speeds.

9.2 Consequences of floods

9.2.1 Damages

The damages produced by floods, both because of water elevation and high velocity, can be

a) Loss of human lives

b) Loss of cattle and other animals

c) Destruction of crops

d) Damages to property, including houses, furniture, stored food, etc., up to the destruction of houses and urban facilities

e) Interruption and/or destructuion of communication facilities

f) Interruption of electric, telephone, water supply and sewerage services

g) Spreading of deseases.

With reference to economics and the severity of the above listed damages, three situations may be distinguished

1) When a river floods yearly or very often

2) When the return period of a flood is long or the river rarely overflows

3) When a river seldom has water. This is the case of rivers in semiarid zones. But when they do flow, normally a flood is produced.

The regions with annual floods suffer practically no human losses, and those of animals are scarce because the people are used to floods and take the necessary precautions when they occur. The same happens with furniture and family belongings which, in the flood season, are lifted to the roofs or to the second floor of houses, if there is one. Crops are only lost if floods come before harvest time. In these regions what is more important than the loss of crops is the impossibility of raising a second or third one. Very frequent damages in those zones are the interruption of surface communications and the deseases spread by drinking water.

One the other hand, when the return period of a flood is long or it rarely overflows, all kinds of damages may occur, since the people who inhabit such regions are not used to floods or have never seen one.

In semiarid regions, where rivers almost never have water, things go as far as having some people settled in the river channel; in this case, everything is destroyed by the first flood which comes by.

9.2.2 Advantages

The main advantages produced by floods are

a) They moisten and fertilize the land for the following agricultural cycle

b) There is an intensive recharge of aquifers

c) Water stored in lower parts forms small deposits that help animal life to survive, especially in semiarid regions

d) They prevent downstream overflowing in zones where population is generally larger or where agriculture, cattle raising, industry and services are probably more important.

As a region develops and populates, damages increase and benefits diminish: then, floods have to be stopped or reduced. Even if in the first case, the advantages floods can bring about disappear, with appropriate works the first three points in the list can persist. The most difficult to keep and most often lost refers to the recharge of aquifers, which then only depends on the local rainfall and on the infiltration along the river.

9.2.3 Actions to reduce or prevent damages due to floods

In order to prevent, to reduce or to diminish the damaging effects of floods, two kinds of actions may be taken

a) Structural or direct

b) Non structural or indirect

The first group includes those works which directly interfere with the flow like the structures built in the channel or in the banks to store, deviate or confine the water or make it flow with more velocity and without obstacles.

The second group corresponds to those works which, while not intefering or modifying the flow, permit to know beforehand the volume of the hydrographs, the maximum discharges and the maximum levels of water which can either pass or attain in certain places. Therefore, the inhabitants of the area may be warned thus saving lives and, possibly, reducing other damages.

This group includes the protective actions and all the measures taken to maximize the operation of the works or structures built along the river. They will be discussed later.

9.4 Maximum discharge

From the data of the hydrometric and climatological stations located in the channel, hydrographs are obtained of the floods which may take place in the areas to be protected. Those hydrographs are then associated with their corresponding frequency or return periods.

Taking into account the safety of the population and its belongings, the ideal thing to do is to design and build protection works for the maximum discharge possible to occur. However, when considering the economic aspect, this never or only very seldom can be done, due to the high cost of these works.

Therefore, all the designs for structural protection works are effective for a given discharge in relation with its return period. That is why there is always the chance that the area is flooded if a greater discharge occurs.

Due to this limitation, when a zone is to be protected, there are two actions of vital importance.

a) To make the people in the area know the way the protection work built there functions and its limitations.

b) To elaborate a protection plan (non structural action) that takes into account, at least, what to do and how in case a bigger discharge than the designed one presents, and then to inform the people about the plan.

Among other factors the maximum discharge contemplated by every protection work depends on: the benefit attained by the work, economic possibilities, politic and social consequences a failure could have not only at the national but also at the international level, number of safe people and possible human losses for every probable maximum discharge, etc.

Here we show some possible return periods in relation with the characteristics of the area to be protected, taking into account that these values are only general and preliminar. In fact, the final choice and, in consequence, that of the maximum discharge, depends on the local factors of every particular site.

Characteristics of the area to be protected	Return period, in years (tentative)
— Isolated agricultural parcels with no possible human damages	5
— Risky distincts without human risks	25
— Agricultural areas with little population	50
Agricultural areas with population	100
— Industrial and urban zones	500
— Densely populated areas	1000
Cities	>1000

For every particular area different protective schemes must be studied; each of them with its different return period and maximum discharge. It must be noted that this analysis can not take into account neither the human losses nor the sociopolitical factor because they can not be quantified in terms of material costs.

Based on the costs of the different alternatives, the expected benefits, the social and political consequences of a failure, the evaluation of the economic resources, the protection plan adopted for a maximum discharge and on the local organization available to carry out the plan, then, the maximum discharge can be chosen.

It is advisable to include in the design group the local and political authorities after explaining to them the advantages, disadvantages and cost of the principal possible alternatives.

9.5 Structural or direct actions

These actions interfere directly with the river flow by storing, deviating or confining the waters.

The main structural actions which can be taken to reduce or prevent floods, in a preselected area, include at least one of the following works:

a) Perimetrical levees around towns or important structures

b) Longitudinal levees along one or both river banks

c) Permanent diversion by means of diversion channels

d) Temporal diversions to lagoons, low plain flood areas or to artificial reservoirs limited by levees

e) Meanders cutoff or channel rectifications

f) Big dams with capacity to regulate floods. There may be only one or a series of them

g) Breaking-peak dams. There may be only one or a series of them

h) Dredging of the main channel

i) Remotion of the vegetation in the main channel or in the flood channel between the banks of the river

j) Drainage to remove the rain water, especially in the areas protected with levees

k) Basin forestation.

Except in very particular situations only one of the works indicated solves completely the flood control problem in a predetermined area. When extended areas are to be protected, the adequate and successful combination of two or more of these measures is necessary.

The structural or direct actions may achieve the complete prevention of river overflows, and of the rain from flooding a well drained area. The principal disadvantage of these works lies in the cost, which is so high that it is quite difficult to cover it entirely, even if their beneficial effects may excede the costs of the damages produced by floods.

Another fundamental aspect that it is necessary to consider when designing these kinds of works is the fact that protected people consider themselves safe and forget to fight or to protect themselves against the flood. That is why it is often better to do no work at all than doing it without the previous adequate studies and with the necessary economic means to guarantee a secure and reliable construction.

Here a brief summary of the structural or direct measures is presented with some relevant points to be considered during the design of the project.

9.5.1 Perimetrical levees

When a zone reaches a certain degree of development, towns and important constructions subject to frequent floods should be protected. It has been mentioned before that there is a natural tendency to build towns close to rivers.

At the beginning, people accept the danger and do not worry about living in such a dangerous situation because of the advantage of having water to survive.

But towns grow and they need to be protected against periodic floods. The most common and clear solution is to build a levee around the town. A levee can completely surround a town when it is built in a plain zone. If part of it is in a high area, and provided the topography of the place permits its construction, the levee can be built only in the lower part, fixed to the higher one (see fig 9.2).

Perimetrical levees have the advantage of being the most economical control solution to be found; besides, they do not change the water level because their effect on it is either very small or null, thus eliminating the need of complicated hydraulic calculations.

During a flood, both the conditions of the levees and the levels of the river flow should be constantly checked, evacuating the inhabitants to higher zones at the first signal of danger. It is most important to take these actions when there is no certainty about the levels the water can reach and there are not enough funds to give the structures adequate maintenance.

9.5.1.1 Height of levees

The maximum registered flood levels determine the height of levees. Then, the traces left by water have to be studied, because in undeveloped regions there is not enough available data and only few traces of recent floods can be found. If a gaging station has been established near by, its data will be considered in the studies. If a project to protect an area against floods is started, gages must be placed close to towns, and the gaging of the current must be done at least in one station. All gaging data is useful to calibrate a model. Whenever possible, a flow will be associated with all the maximum water levels observed. The height of the levees is determined by adding a freeboard of 1.00—2.00 m to the maximum water elevation observed. Naturally, it will also depend on the funds available and on the reliability of the data at hand.

9.5.1.2 Crest width

Levee crest width should at least allow the passage of a vehicle, that is a minimum of 3.00 m.

9.5.1.3 Slopes

Levee stability must be planned taking into account the materials available and the soil where it will rest. If it is sand or clay, 2:1 slopes may be considered, but pure silty materials should be avoided. In case of sands, filtration should be determined and, if necessary, slopes should be lowered and the crest widened.

9.5.1.4 Drainage of the protected zone

Due to the fact that levees are borders between the river and the town, adequate drainage should be provided to evacuate the rain water which falls on the town, especially when the water level in the river is higher than the street level. One of the following procedures may be considered

a) The construction of a reservoir with a capacity equal to the rainfall volume expected. This water storage should be connected to the river by means of a pipe placed under the levees with a manual or automatic gate or valve to discharge the stored water when the river flow is under a certain level. This valve should be closed when the river flow exceeds that level (see fig 9.3).

b) If the rainfall volume is important, thus rendering the water storage reservoir too expensive, a reduced tank with a pumping station can be built with a capacity determined according to the maximum measured runoff, after regulating the tank (see fig 9.3).

Before every rainy season the pumping equipment should be properly checked and tested.

9.5.2 Longitudinal levees

They are built along the banks of a river with the purpose of protecting several towns simultaneously, big agricultural and cattle-raising areas, developed regions with communication facilities or factories and service facilities.

Longitudinal levees can be built at one or both river banks to keep water within the main channel. For a given flood, levees height will depend on their length and most of all on their spacing. Since the construction of this type of levee starts when an area is already developed, there will be lots of buildings very close to the river banks, specially in towns.

Longitudinal levees modify completely the conditions of the flow during floods both along the protection structures and over the downstream and upstream areas adjoining them. In general, and for a given flood, the water levels in the main channel are higher than those existing before the protection was built. This happens because, while preventing overflows, all the flow is forced to run through a reduced area. If the protected reach is very long, levees height should be increased as they advance downstream; however, in most cases, levees have a constant height all along the plain.

Immediately upstream of the levee protection water levels are higher, since over-elevation —as explained in the previous paragraph—, produces a backwater effect which influences the upstream area (see fig 9.4).

Downstream, the water elevation is also higher than when no protection structures exist: levees compell all the flood to pass between them, thus increasing the river flow at the end of the structure (see fig 9.4).

Lack of proper studies before very long levees are built can result in serious consequences to river reaches downstream of the protection end. Floods can then occur in areas never affected before.

When only a bank is protected, damages upstream and downstream are not noticeable because the unprotected bank continues to overflow, causing great water volumes to be deviated from the hydrograph. Although water levels in the flooded bank are higher, they do not intensify the damages produced to agriculture. However, they do affect human and animal life, and may also flood areas never flooded before.

The importance of the damages and the extension of the flooded areas depend in large measure on its topography and on the length of the protection built on the opposite bank.

Longitudinal levees are built to guarantee safety, since

a) People feel secure with this kind of protection and do not take the usual precautions against floods.

b) People build more expensive structures because they are protected against floods.

c) If a levee breaks, floods will occur faster and will reach higher levels than before, at least close to the breakage.

The cost of longitudinal levees is a function of river water elevation. In turn, that elevation depends on

a) Maximum discharge of the selected flood

b) Spacing between levees

c) Levees length

The selection of the design flood should be done through an economic study that compares the benefit of preventing the damages produced without levees with the cost of the levees. This can be calculated for several floods with different return periods.

Because of the high cost of levees and because of the danger implied by a too high water level in a river, protections should not consist of only this type of solution, but in a combination of one or several of the possibilities discussed ahead.

The design flood chosen depends on the benefits which result from the protection structure and its cost. When a zone is to be protected by longitudinal levees, it is not reasonable to build them with a height capable of protecting a zone against any possible flood. That is to say, the selected design flood can only have a return period up to 25, 50 or 100 years. Since floods may exceed the design capacity or the registered rainfall may indicate that a coming flood is greater than the designed capacity, a plan of action must be developed for these cases. Some of these measures include evacuation, where and how to break the levees, etc. Of this, more will be said in point 9.6.2.2.

9.5.2.1 Dimensions of the protection structure

A plan view of the zone to protect is essential when placing the levees, which should be as separate from the banks as the existing constructions permit.

Once the initial layout is selected, water levels should be obtained for the present condition as well as for the new one when the protection is already built. From the hydrological study, one or several possible design floods should be chosen, or the range within which the design flow works. And from the flood hydrograph, the period should be obtained during which the maximum flood remains more or less constant.

Besides, the water elevations along the main channel and at different plain localizations can be determined by means of the following variants

a) Without levees (natural condition)
A mathematical or physical model is used to know the levels reached by the waters at any point of the zone under study and at any instant during a flood. Such a model should be able to analize the flood transit along the river reach studied and in its corresponding plain.

The physical model mentioned is a fluvial distorted model with a fixed bed but with a different vertical scale smaller than the horizontal one. The roughness calibration and the measures to account for the effect of the obstacles in the plain are made simultaneously. These models require the detailed topography of all the zone that can be covered by water. In the topographic maps all the obstacles which interfere with the current must be marked, such as dikes, walls, channels, roads and all kind of constructions.

Besides, direct observations of water levels of previous floods are also required. To obtain them, gages should be set in as many places as possible, covering all the flooding area and the whole length of the studied reach. If water elevations are known only for some river reaches, they should be complemented by aerial photographs taken during a flood. Through these, the areas flooded at different times can be established.

With the mentioned data the model used is calibrated by reproducing in it the results observed in nature. The roughness coefficient and that of the head losses due to obstacles are generally adjusted.

Once calibrated, the model can be applied to the study of the selected floods. For one of these, the maximum levels attained by the waters along the river reach will be obtained, as well as the land extension covered by water and the flow velocity at selected points.

b) With a single levee

The water levels in the plain opposite to the levee, as well as upstream and downstream of this structure can be known using any of the mentioned models. Once elevations are obtained for the different floods at the boundary marked by the levee and a freeboard is determined, the construction cost can be calculated for each one of these floods. This cost is later compared with the benefit the protection structure can produce.

In a broad plain with a small levee length, water levels with and without protection vary little. Knowing the water levels before the protection is built helps to determine the levee height, for this knowledge makes more complex studies unnecessary.

c) With levees in both banks

As previously mentioned, the study can be done by means of a physical model but with calibrations carried out only for the zone between levees. Model roughness will be the most important aspect to calibrate. Level calculation can be made analytically in two different ways depending on the time duration of the flood maximum discharge.

If the duration of the flood is longer than that the wave requires to pass through all the protected length, the flow can be considered as permanent. If shorter, the flow is transient or non permanent (see fig 9.5).

The second fact to consider is levee spacing. If the levees are built just over the banks, it can be considered that water shall run in a single section of similar roughness all along. If the bank dikes are far from the banks a variable section will exist that can be divided in two: one corresponding to the main channel and the other to the plain zone (see fig 9.6).

Consequently, and with regards to the calculation, several conditions may present:

Permanent flow	—	main channel or single section
Permanent flow	—	compound section
Transient flow	—	main channel or single section
Transient flow	—	compound section

9.5.3 Permanent diversion

This solution consists in deviating a certain water volume from the river and directing it towards the sea or to another watershed through a channel.

As already said, protecting a zone with longitudinal levees presents the disadvantage of its cost, specially if the protection is long, which also implies levees higher than normal in the downstream reaches. Besides, when they are more than 4 m high, this kind of protection becomes dangerous. As noted, the height of dikes mainly depends on the maximum flow selected and, to a lesser degree, on its volume. To reduce the height of the longitudinal levees, diversion channels can be built provided the topography and the geography allow to do so. The purpose of this solution is to reduce the flow of the design flood. Thus, diversion flow will be substracted from the maximum flow (see fig 9.7).

Diversion channels are generally built on the plain, limited by two longitudinal levees. No excavation is made except for a small central channel and the extracted material is used to build the levees. Because their height depends on the separation between them, it is convenient to space them as much as possible. As previously said, the natural plain soil forms the bottom of the diversion channel. Sometimes it is necessary to excavate a channel, but it is a very costly solution.

For operating these works, the two longitudinal levees along the river are considered as well as the diversion channel in one of the plains. At the deviation point, the corresponding river levee is opened and both ends join the levees of the diversion channel.

As long as the river flows are low and do not exceed the main channel discharge capacity, all the water will run through it.

When flows increase, levels go up in such a way that water covers all the section between the longitudinal levees. As the level at the entrance of the diversion channel is similar to that of the natural terrain, water starts running through it. Thus, from this point on, the river flow downstream is equal to the upstream flow minus the diverted flow.

When the flood peak passes through the entrance of the deviation channel, the selected length of the entrance should let a flow go into the diversion. That, in turn, will make possible for the remaining discharge to run safely through the river, thus rendering more economical the costs of the levees downstream.

Even if the land between the diversion channel levees can be used, they mark the boundaries where nothing should be built and obstacles should not be placed.

In case that the flow downstream of the diversion is not desired to exceed a given discharge, even when a flood is bigger than the design one and, consequently, there will be damage along the diversion channel, then, a limiting flow structure is built downstream of the diversion, within the river. This structure can have holes, short pipes or gates. In the occurrence of a flood, the gate opening is adjusted as to let only the desired discharge out into the river (see fig 9.7).

9.5.4 Temporary diversions

Temporary diversions are used when at one or both sides of the river to be protected there are low zones or lakes that can be temporarily overflown while a flood lasts. Even if those zones are devoted to agriculture or cattle, flooding damages are not serious because they have been assigned to that purpose. When choosing a zone to receive part of a flood volume, it should be remembered that no constructions must be there (see fig 9.8).

As with the diversion channels, this kind of solution should be combined with longitudinal levees. The main difference is that diversion channels are capable of deviating very important water volumes as long as the maximum flow capacity of that channel is not exceeded. On the other hand, to lakes or low zones, only a prefixed volume equal to the storing capacity of the lake or low zone can be diverted.

Another difference lies in the fact that the water stored in such places must return to the river when the flow diminishes, since their useful capacity must be available for the following flood. The possibility of several floods one after the other and the intervals between them must be determined by means of a hydrologic study.

This solution can only be applied when there are low zones adjacent to the river to be protected that can be closed, thus excluding the possibility of flooding other areas. These zones should be limited when they are too broad or when there is a connection to other areas. For this, low levees are built.

As the water elevation of a river depends on its discharge, when the peak of a flood passes, it is convenient that the available volume in the lake discharges through a diversion channel the biggest possible amount of water.

Let us assume a hydrograph like the one shown in fig 9.9a. If water is deviated to the lake before the convenient time, the lake may be filled in advance and the maximum discharge (Q_B) passing downstream could be very large.

The best use takes place when the substraction of water from the hydrograph is made almost horizontally; that is, when the hydrograph peak lowers as much as possible (see fig 9.9).

For the best operation of a temporary diversion channel the deviation must start at point K of the hydrograph. Therefore, the level at the entrance notch of the channel between the river and the lake must have the level corresponding to flow $Q_K = Q_B$, which will be the flow passing downstream. Once the height of the spillway crest is chosen, its length is calculated in such a way as to let flow Q_D pass at instant T_D.

The danger of the previous solution is that a flood may have the same maximum discharge but with a greater volume (see fig 9.9c).

The inflow channel which connects the river and the lake is similar to a diversion channel, and may or may not have levees all along depending on the local topography. If the water runs towards the lake as soon as it is deviated, a notch can be made and water will run over the plain with no levees. If the plain soil has an intensive agriculture, levees are built to direct the water and avoid flooding other surfaces.

To empty the lake and make it available for future use, it is necessary to dig a channel from the lake center towards the river. That channel evacuating capacity is a function of the channel dimensions and of the difference between the surface water level in the lake and in the river. The channel cross section will depend on the time available to discharge the water, that is, the time interval between floods.

If only during a year an important flood occurs, the return or outflow channel may not be built, provided that both infiltration and evaporation dispose of the water before the storage capacity is again required.

The return channel should connect the river downstream of the diversion zone and, since the untimely filling of the lake must be avoided, the channel should be provided with a gate system placed close to the river. This gate is closed when the flood rises and is opened again when, as the flood diminishes, a lower level than in the return channel presents itself in the river.

This solution can be adopted in different places along a river and it will result in a considerable reduction of water levels. The setback is that it requires topographic conditions not always present.

The storage capacity of a single lake can be small if compared with the hydrograph volume. In such a case, this solution is recommendable if several lower zones exist to which water can be deviated, and the flood peak is diminished by stages.

Even when this solution is not chosen, it must be taken into account that floods bigger than the capacity of a protection made with longitudinal levees can occur. In consequence, it is always advisable to keep low empty or less productive areas apart to be flooded in case a bigger flood than the design one comes by and breaks the levees. Levee openings should be made in front of these zones, previously intended for this purpose. This can help to prevent large collapses in front of towns and damages to cattle lands, etc.

9.5.5 Meander cutoffs and rectifications

Another way to reduce overflows in a limited reach of a river is to increase its hydraulic capacity by rectifying it. This capacity increase works only for the rectified reach and the immediatly upstream zone. Conditions stay the same in the remaining length of the river, thus keeping the probability of floods unchanged.

A reach with meanders can be rectified by means of a channel which, if made with the same transversal section of the river, will have a greater hydraulic capacity produced by a bigger slope. For instance, if given the same depth and width, the length of the reach is four times the length of the rectification channel, the hydraulic capacity of this rectification may almost double. Because of the diminished length, slope increases and so does the hydraulic capacity of the rectificated reach (see fig 9.10).

Since in most rivers the regime is subcritical, the rectification levels in the reach downstream remain equal for equal flows, both before and after the rectification has taken place.

In the reach immediately upstream of the rectification the hydraulic capacity increases due to the backwater effect resulting from the rectification; besides, the bed is eroded due to a higher bed load discharge compared with that of the original river. This increase in capacity will extend upstream with time (see fig 9.11).

First, the lowering of the bottom takes place very quickly. Afterwards, it slows down until a new equilibrium slope is reached. This depends on the dominant flow and on the materials exposed when erosion takes place.

The main disadvantage of this solution is that the eroded material tends to deposit in the reach immediately downstream of the rectification, thus diminishing its hydraulic capacity. The backwater produced by this hydraulic capacity reduction has an upstream effect in the rectification itself. The only means available to control this is to dredge the river bed in that reach, trying to keep the same section and slope existing before the rectification was made.

9.5.5.1 Construction of the rectification

The rectified reach should have a cross section similar to that of the river. The rectification can be made first by building a pilot channel that will later increase its width due to bed load discharge and erosion. The pilot channel dimensions depend on the flow and on the physical characteristics of the material of its walls and bottom.

9.5.5.2 Pilot channel dimensions

The pilot channel can be dredged until it reaches the river bed level. The slope will be uniform while connecting the bed elevation of the end section; therefore, it is excavated up to the imaginary line which joins the river bed elevation in the end sections of the rectification channel.

The minimum width of the pilot channel should be equal to twice the height from the pilot bed to the natural terrain level. In case of a landslide caused by erosion at one of the slopes, this prevents the section from being obstructed thus interrupting the water runoff.

Given the minimum width of the diversion channel

$$B_{min} = 2 \text{ (terrain level-bottom level)}$$

the depth and velocities for different flows can be obtained. If for the mean dry season flow, the flow velocity in the channel is higher than three times the mean velocity needed to start sediment motion, the minimum width is that of the project. This guarantees that the material will be carried away in larger quantities than those coming from upstream and eroding the section. First, the broadening will take place towards the sides, although there will be some at the bottom, specially in the first section of the rectification. When the section is broadened, its hydraulic radius increases, thus increasing velocity and the erosive process. What has been mentioned up to this point generally takes place when both bed and wall materials are sandy.

If because of bigger size the material is more resistant, but specially because of its cohesion, as in the case of clays, a wider section should be dug so that the bigger hydraulic radius produces flow velocities capable of the necessary erosion.

9.5.5.3 Downstream dredging

If the material removed and transported from the walls and bottom of the pilot channel is sandy or has a greater diameter, it will tend to deposit in the reach downstream of the rectification, since velocity diminishes here because of the smaller slope. These deposits reduce the hydraulic section and the water surface raises producing a backwater effect which modifies part of the pilot channel. To avoid this, dredging is required in the first reach downstream of the rectification.

9.5.6 Large dams

Dams consist of a main dike or curtain built on the river to block the passage of water and store it and of auxiliary dikes when they are needed. These elements conform the reservoir where the water is stored. Other important works in a dam include the spillway, which discharges the water that is not used, and the intake, through which the water to be used goes out. At a given time, the available capacity of the reservoir may be smaller than the volume of the flood. In such cases, the exceeding water is let out by the spillway, which may be of free discharge or provided with gates. As the inflow increases, so does the outflow; the spillway discharge capacity increases when the water elevation increases over its crest. To increase water level at the reservoir, part of the flood volume remains stored. This process should be controlled by the continuity reservoir equation, which makes the water volume entering during a time period equal to the water volume leaving the reservoir during the same period, plus the volume that remains stored at the reservoir (see fig 9.12).

9.5.7 Peak breaking dams

Generally, peak breaking dams consist of a low curtain and a spillway with a crest elevation (also called control section) almost at the level of the river bed. The spillway width is small so that it does not allow the passage of great flows. The flow is selected as a function of the hydraulic capacity of the river downstream of the dam. In other cases, the discharge structure contains holes or is formed by short pipes (see fig 9.13).

This dam operates as follows: at river normal flow all the water passes through the discharge structure and the flow is not discharged by the dam since upstream there is almost no backwater. In the occurrence of a flood, and due to the fact that the spillway has a limited capacity, less water leaves the dam than enters it, forcing part of it to remain stored in the reservoir. In the presence of the design flood, the spillway lets out the maximum design flow while the reservoir is filled to its full capacity.

If a costly structure can not be built and the design flood has a long return period, the dam curtain can be a spilling one or, after a certain level, the spillway may be broadened. Another solution consists in building a second spillway in such a way as to have all the exceeding flow pass downstream without damaging the curtain. After a flood passes, water keeps going out through the discharge structure until the reservoir is empty.

When the river slope is steep and, therefore, the reservoir does not have enough capacity as to regulate floods up to the desired volume, several dams of similar design should be built one after the other.

This kind of structure is usually constructed on small streams or rivers and upstream of towns.

9.5.8 *Main channel dredging*

An increment in the hydraulic capacity of a channel may also be achieved by dredging its bottom or by increasing the width of some stretches, in other words, by increasing the cross section area.

This solution, which is commonly used to guarantee a definite depth during most of the time in some reaches of navigable rivers, should be applied with great precaution for controlling floods because it only works when the sediment transport capacity is reduced upstream of the dredged area. If this is not so, permanent dredgings are needed to prevent the river from returning to its original form in a short time.

This solution also includes the artificial broadening of some sections with rocky banks and bottoms, mainly when there is a geological dike. These sections may be converted into sections for hydraulic control when the yieldings do not exceed a certain value. The deepening of these sections, more than their lateral widening, produces greater capacity increments while diminishing the level of the water surface.

9.5.9 *Removal of vegetation from the channel*

One of the main factors which increase river depth is its roughness. This is greatly favoured by the existence of obstacles like vegetation or refuse materials. Of them, the most common is vegetation, which must be removed before the flood season.

This problem does not exist in rivers where water flows all the year, simply because vegetation can not grow at least in the main channel. But if they have levees detached from the banks, the remotion reduces to the portion between the levees and the banks.

On the other hand, when rivers run through semiarid regions with a minimum flow during the low water season, greater vegetation can grow because of the subsurface water level. The process is largely intensified when, during two or three successive years, no floods destroy it. When this happens, vegetation becomes so strong that it can even resist the impact of moderate floods, favouring in this way the appearance of islands and of diversions in the main channels. It is precisely in these rivers where the removal of the vegetation is vital because it noticeably increases the water level.

When, during floods, it becomes necessary to control the water levels, the cleaning process is done downstream and upstream of bridges to prevent water concentrations greater than foreseen.

9.5.10 Forestation

In order to prevent the disastrous effects of floods, it is sometimes necessary to forest all or part of the basins because forestation not only delays flow concentration, but especially hinders the deposit of big amounts of sediments which, as they are not carried away, reduce the hydraulic capacity of the river.

To forest an area there must be enough humidity as to guarantee the growth and development of the artificially placed vegetation. But, most of all, it requires care and supervision until it develops and follows its natural process and, besides, a control of the newly formed brooks and ditches so that new erosions will not occur.

9.5.11 Drainage of the protected areas

This theme has been already discussed when dealing with perimetral levees. Two possible solutions were presented to solve the problem of the dislodgement of the rain water which falls over the protected areas, a process inhibited by the presence of the protection structures (see fig 9.3).

This problem is still increased by the longitudinal levees, because there are times when they cross brooks or small affluents which flow towards the main channel. When the affluent is important or carries water all the year round, it is advisable to interrupt the levee to follow the river at both sides until it is embedded in an elevated area.

The solutions to this problem can vary so much as the plain topography, the form of the river, the number of affluentes, the length of the levees, the intensity of the rains, the use of the soil, etc. However, it is included among the flood protections, because, if it is not kept in mind, the protected area can also be flooded, if not by the river, by the local rains.

9.6 Non structural or indirect actions

9.6.1 Measures to prevent disasters

Indirect or non structural actions are those not included in the structural ones already seen in the previous chapter. Their fundamental object is to prevent human losses and, in general, to prevent or reduce damages produced by floods.

From a general point of view, every action taken to prevent damage or provide help during a flood, or when it has already occurred, takes the name of *measures to prevent disasters*. Among them, those related to meteorological, hydrological and hydraulic aspects are called non structural or indirect actions, because they do not imply the construction of works to interfere with the stream of brooks and rivers.

In order to place the *non structural actions* in the general context of the *disaster-preventing measures*, some of these are mentioned in this subchapter. Properly non structural actions are dealt with in 9.7. Disaster-preventing measures are all those which are related to the planning, organization, coordination and execution of the remaining indicated measures.

1. Allow or tend to give —when possible— timely warning of the possible occurrence of a disaster

2. Evacuate the threatened population in a secure, orderly way

3. Help people in danger

4. Give medical and social assistance

5. Prevent plundering

6. Restore in the shortest possible time all the facilities interrupted by the disaster.

It is necessary to take into account that these measures

a) are of very different types and cover a great variety of aspects;

b) are undertook or coordinated by different federal, state or local institutions or organizations;

c) require to be performed at different stages during the occurrence of a flood.

a) When the disaster is caused by a flood, among many others we can name the following lists of measures, where the asterisk refers to non structural actions, that is to say, those associated with hydrological and hydraulic aspects

1.* Organization of an alarm system
2.* Establishment of central, regional or local operation centers
3. Planning
4. Coordination among different institutions
5.* Formulation of laws and regulations
6.* Training of the personnel in charge of the actions here enumerated
7.* Training of the people who could be affected
8. Surveillance to avoid plundering
9. Sanitary actions
10. Construction and operation of camps and first aid centers
11. Evacuation and removal of populations
12.* Repair of interrupted land communications and destruction of those that work as dams or levees
13.* Establishment of radio and phone communications
14.* Repair of damaged hydraulic works
15.* Adequate operation of hydraulic works
16.* Supervision of levees and their immediate repair in order to avoid failures
17.* Obtaintion of data and their timely transmission
18.* Construction of telemetric networks (see fig 9.14)
19.* Hydrological and hydraulic calculations
20.* Studies of cause and effect
21.* Design of mathematical models and programs to predict effects.
22. Construction beforehand of all the necessary structural actions, dams, levees, etc.

b) Among the institutions or organizations generally involved in case of a flood, and which, in consequence, need to be mutually coordinated, the following are the most important

1. The federal or central political power
2. The state or regional political power
3. The municipal or local political power
4. State departments in charge of the construction of the hydraulic works and in charge of the water management and its control

5.* State departments in charge of the construction and operation of land communications in general
6.* Meteorological and hydrometrical services when they do not depend on point 4
7. The army, the navy, and the air force
8. The Red Cross
9. Organisms which provide services like electricity, telephone, drinking water, drainage, air transportation, and so on
10.* Construction companies and those which rent equipment.

c) Finally, and in relation to the moment the measures are taken during the occurrence of the flood, the following actions can be enumerated

1.* Organization and coordination before the rainy season begins
2.* Construction of hydraulic works to prevent or reduce floods (structural action)
3.* Those starting when the rains begin up to the moment when the flood appears. Forecasts, alarm and decision to evacuate
4.* Control during the flood including supervision of works and levees and emergency repairs
5. Actions in the zone under the influence of the waters, including rescue and help
6. Actions after the flood has passed to restore normal conditions
7.* Evaluation actions to feedback point 1.

In every particular case, the measures to prevent disasters depend on different factors including the possible cost of the damage, the number of possible victims a flood may cause, the frequence and magnitude of the floods, financial resources, human resources, available equipment and, besides, political and social conditions which change with time and place.

9.6.2 Non structural actions

The non structural actions mainly consist of hydraulic and hydrological studies. They are dealt with below and can be summed up according to their final and common object in

a) Alarm actions
b) Actions of vigilance and emergency works

The alarm actions may differ depending on the conditions upstream the area to be protected. These are

a.1) Unaltered streams where non structural actions have been practiced

a.2) Altered streams especially with reservoirs

When in the streams there are no control works that modify the normal field, the indirect actions are reduced to the possibility of predicting the floods which could occur in the areas to be protected. But, when reservoirs exist in the streams, it is necessary to conjugate two apparently opposite objectives: on the one hand, to allow the passing of the smallest discharge into the rivers to empty the dams before the rainy season begins. On the other, it is convenient to keep in the reservoirs a big quantity of water to be used afterwards or to attein the highest levels for power generation. In other words, this is a two-sided optimizing problem where, at the same time, it is necessary to minimize the damage produced by the discharges and to maximize the productivity of the waters. This has to be done taking always into account the need of giving the warning of flood as soon as possible.

Besides, the shorter the time the lands are covered by water, the smallest the losses in the agriculture will be, provided that the minimum time to cause permanent damages is not exceeded.

9.6.2.1 Alarm actions

Among the most important alarm actions, there are the following

1. To design a plan of action considering the conditions, resources and magnitude of the possible damages

2. To install pluviographs in the watershed and the scales, both in the gaging stations and in the upstream dams

3. To install radars and equipment for the reception of satellite images and meteorological information for the detection and observance of hurricanes and tropical storms. This is also useful to give timely alarms in relation to heavy local rains and to the possible occurrence of hurricanes.

4. To install radios along with pluviographs and scales of point 2. The data are

transmitted to a central where they are processed.

5. To design hydrological models, previously calibrated, to determine the form of the flood in a definite part of the river or in front of the zone meant to be protected or alarmed. These models will be computer programmed.

6. Instead of point 4, it is advisable to install a telemetric net for the direct transmittance of the data from the pluviographs and scales to a central register station. It is even better if those data enter directly in a computer where, with the programs of the previous section, the maximum discharge and the form of the flood can be obtained (see fig 9.14).

7. Analytic or experimental determination of the water levels of different floods along the river and of the flooded plain. This is the most difficult point to achieve because the mathematical or physical models need a special calibration. To be able to attain it, it is necessary to have reliable data about water elevation and flooded areas during past floods. Besides, it is very important to consider that the results change when new areas are cultivated and highways, railroads, or other hydraulic works, mainly reservoirs and channels, are built. This point considers the possibility of having a mathematical model to simulate the overflown areas obtained in point 5, immediately after its determination.

8. Determination of the areas adjoining the river that can be covered by the waters. This can be correctly obtained if the previous section is done. If that is impossible, as usually happens, the flooded areas will have to be determined over the basis of aerial photographs taken during previous floods. In other words, it is necessary to know the areas affected by different discharges, because this will permit to take the decision to evacuate the zones according to the magnitude of the forecasted flood.

9. Obtaintion of hydraulic-fluvial data. To adjust correctly the program of point 7 and to carry out point 8, it is necessary to design a plan to obtain at least in a minimum of places the water elevation along the river and in the plain during the floods.

   It is essential to install the necessary scales and rain-gages and to have specially trained personnel to perform this task. If possible, it is important to gage the streams in some of these sections along the river and in the affected zones.

10. Design of methods and computer programs for the optimization of flood control if there are one or more dams upstream. This permits a better control of flood (which implies less damage due to the water allowed to pass through the spillways) and to achieve the maximum subsequent profit from the use of water.

11. Establishment of methods and of organized groups to take timely and adequate decisions before and during the occurrence of the floods, on the base of the calculations indicated in points 6 and 7.

It is always necessary that the alarm for protection and evacuation is given with the maximum possible anticipation before the flood comes. Non structural actions of alarm are in general designed to warn people and organisms responsible for the execution of the measures to prevent disasters. These actions will not prevent floods and, if there are no dams, they want even control them. But, because of their cost, which is much lower than that of the structural or direct actions, they are more within the reach of the undeveloped countries; however, for these actions to be effective they need an organization which, paradoxically, those countries do not have. From a certain point of view this happens because development is a consequence of organization.

9.6.2.2 Surveillance and repair actions

These measures are to be carried out in all zones where there are structural actions mainly perimetrical or longitudinal levees, permanent or transient diversion channels, peak breaking dams and small capacity storage dams.

Among the actions to be carried out, there are the following

a) Coordination with the central register station or with the organism in charge of determining the flood magnitude and the alarm.

b) Inventory of the personnel and of the equipment available in the zone.

c) Establishment of vigilance during the 24 hours of the day until the end of the flood. This has to be done on foot, because the bank slopes hinder the detection of failures from vehicles travelling over the top of the levees.

d) Supply of radios, lamps, vehicles and food to the vigilance personnel.

e) Adequate selection and distribution of the closest places to the most critical zones to localize the available equipment for the emergency work. Besides, it is necessary to establish rock deposits in accesible areas of the critical zones because the trucks will have to be loaded with rock. In the same way, a good number of sacks must be ready to be filled with sand when an incipient failure is detected.

f) Coordination with the organisms in charge of disaster prevention to give the timely warning of an inminent failure in a construction or levee.

Finally, when the zone is duly protected with perimetrical or longitudinal levees, the organism responsible for the accomplishment of non structural actions or of the prevention of disasters will have at its disposal a plan with the adequate measures for a flood greater than the one chosen in the design of the levees.

Among others, these measures can be: the selection of those sections of the levees to be destroyed to permit the flooding of less productive zones, the destruction of irrigation channels or land communications which interfere with the free flow of the overflown water; the reinforcing of other levees, irrigation channels and roads where it is better to retain or store the overflown water in an effort to prevent it from reaching more populated or productive areas; finally, we will consider the breakage of levees in those zones where it is possible and even desirable that water goes back to the river once it has returned to its normal level.

9.7 Obstacles in rivers or natural channels

During floods many manmade structures built across rivers turn into obstacles to the flow. These structures increase the water levels upstream of them in reaches which go from some meters to kilometers, depending both on the size of the work and on that of the flood.

Some of those works are

a) Bridges
b) Dams to retain sediments and peak breaking dams
c) Diversion dams and structures
d) Large dams
e) Navigation locks
f) Groins and bank protections

There are some non hydraulic works built outside the river channel which cause devastation during the flood season mainly because it is ignored that those structures provoke floods and, therefore, no measures are taken to prevent or reduce their effect. They are

h) Roads perpendicular to the flow, which act hydraulically as dams or dikes
i) Roads parallel to the flow, which work hydraulically as longitudinal levees.

Although we have said "roads", the term includes all surface communications.

9.7.1 Roads perpendicular to the flow and bridges with scarce hydraulic capacity

A road running transversally to natural runoffs with insufficient drainage can work as a dam, with its earth-fill working as a dike, and the bridge as an inefficient spillway. In this case, the following can occur: during the dry season the flow can pass under the bridge with no problems; however, during the rainy season, the area may not have enough capacity to let the maximum flood through. Under these circumstances, and for equal flows, levels upstream are higher than those existing before the road was built. These increases in water elevation provoke backwaters which widen the size of the flooded areas and result in bigger depths in the originally flooded areas. On the other hand, when the bridge hydraulic capacity is less than necessary, more time is required to let the water volume through and, therefore, floods last longer upstream of the bridge. If the bridge is small, water stores can reach the level of the road and spill over, destroying it.

As it was said before, a flood can produce overflowing of both river banks. Overflown water runs over the flooded valley until it encounters a road. If it does not have enough culverts, it will arrest it. Water will store upstream the road producing the effects discussed above, or else it will run along it until it finds the bridge across the main river.

Thus, a road with insufficient drainage and running perpendicular to a river runoff, produces the following effects

a) It increases water elevations upstream of the road and in flooded areas

b) It increases the duration of floods in upstream areas

c) It can destroy the road for water may run over it

d) It can destroy bridges and culverts due to the water velocity increase under those works as a consequence of the greater water level difference at one and the other side of the road.

Points a) and b) provoke the following effects upstream: damages in the crop raising areas, loss of cattle, destruction of constructions and houses and sometimes loss of human lives. On the other hand, points c) and d) work as a relief of the previous situation, since the partial destruction of the road tends to restore the conditions existing before its construction. However, once the road has caused water storage, its sudden breakage will produce an artificial flood that, in its way downstream, will have a more important destructive effect than the original flood. Therefore, the possibility of devastation caused by a badly drained road exists both upstream and downstream of it along the river.

A road perpendicular to a natural runoff should have structures to let maximum flows and volumes pass during floods without much increasing depth. Several alternatives should be studied to weigh the effects of bridges of different length and to estimate the damages each one of them could produce. In selecting the most economical one, steps should be taken to pay for property affected and to avoid loss of human lives. Usually, damages will be a function of the flood volume plus its maximum flow.

9.7.2 Surface communications parallel to river beds

When a road is built parallel to a river bed, it seems that no bridges are required and only culverts are built in those streams running towards the river. But by not building bridges what it is really built is not only a road but also a dike protection against floods. The effects of that road dikes have been already mentioned when longitudinal dikes were discussed. However, the following items should be highlighted

a) For equal flow, they raise water at levels that were not previously reached. Then, a dike (road) breakage can have worse consequences than those which would occur without it. It should be considered that the road engineer usually does not know that the road will work as a dike and, therefore, does not take such a fact into account in either his project design or in the quality control of the road; then, breakages will often produce the already mentioned damages plus the road destruction.

b) Although they protect zones detached from the river, they move the flood from upstream towards downstream areas. Both effects are greater if there is a road parallel to each bank.

c) Also, as the road designer does not consider the over elevations his project produces on water levels, then the road will be more often overflown and destroyed.

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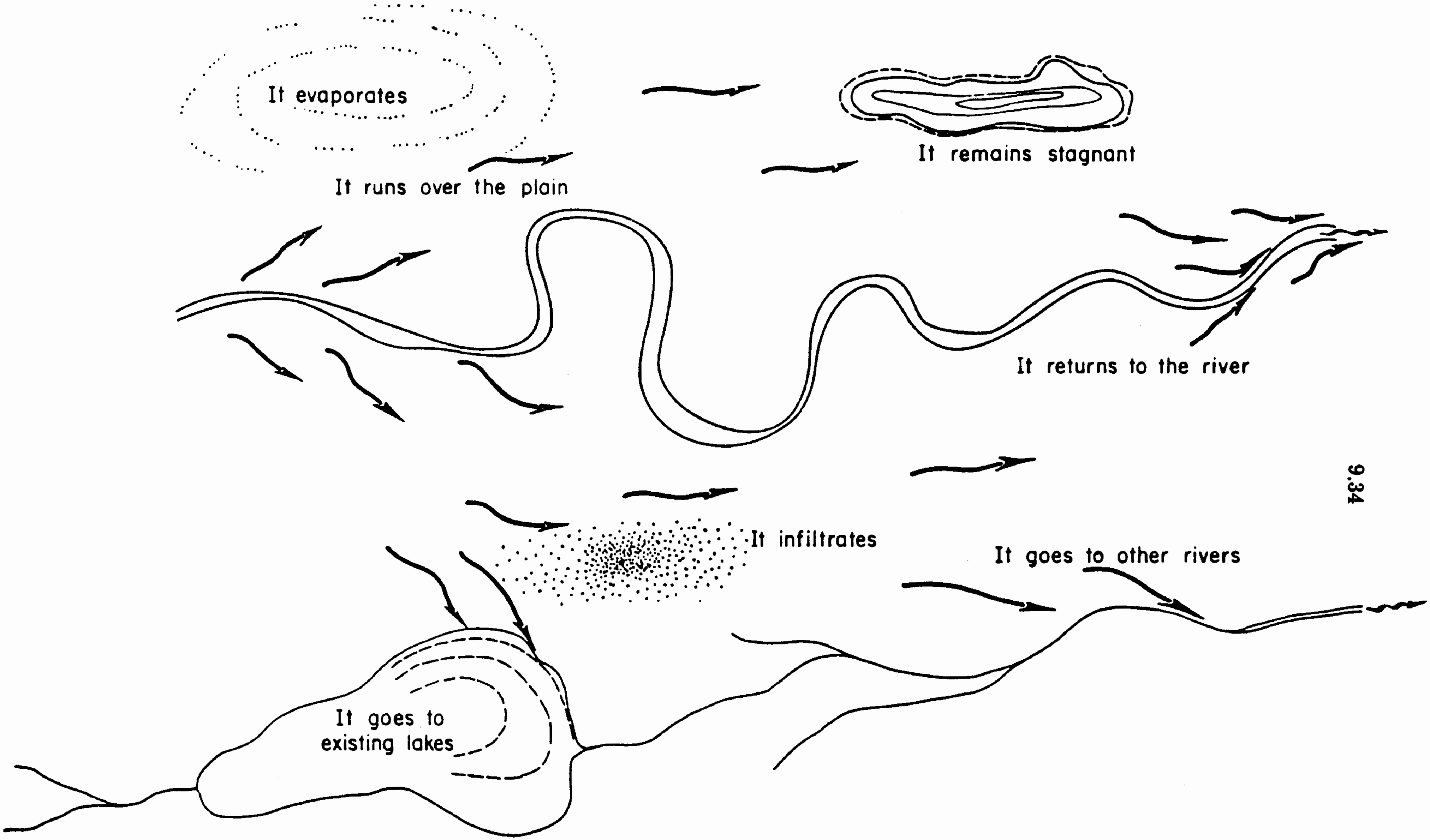


Fig 9.1
Overflown water

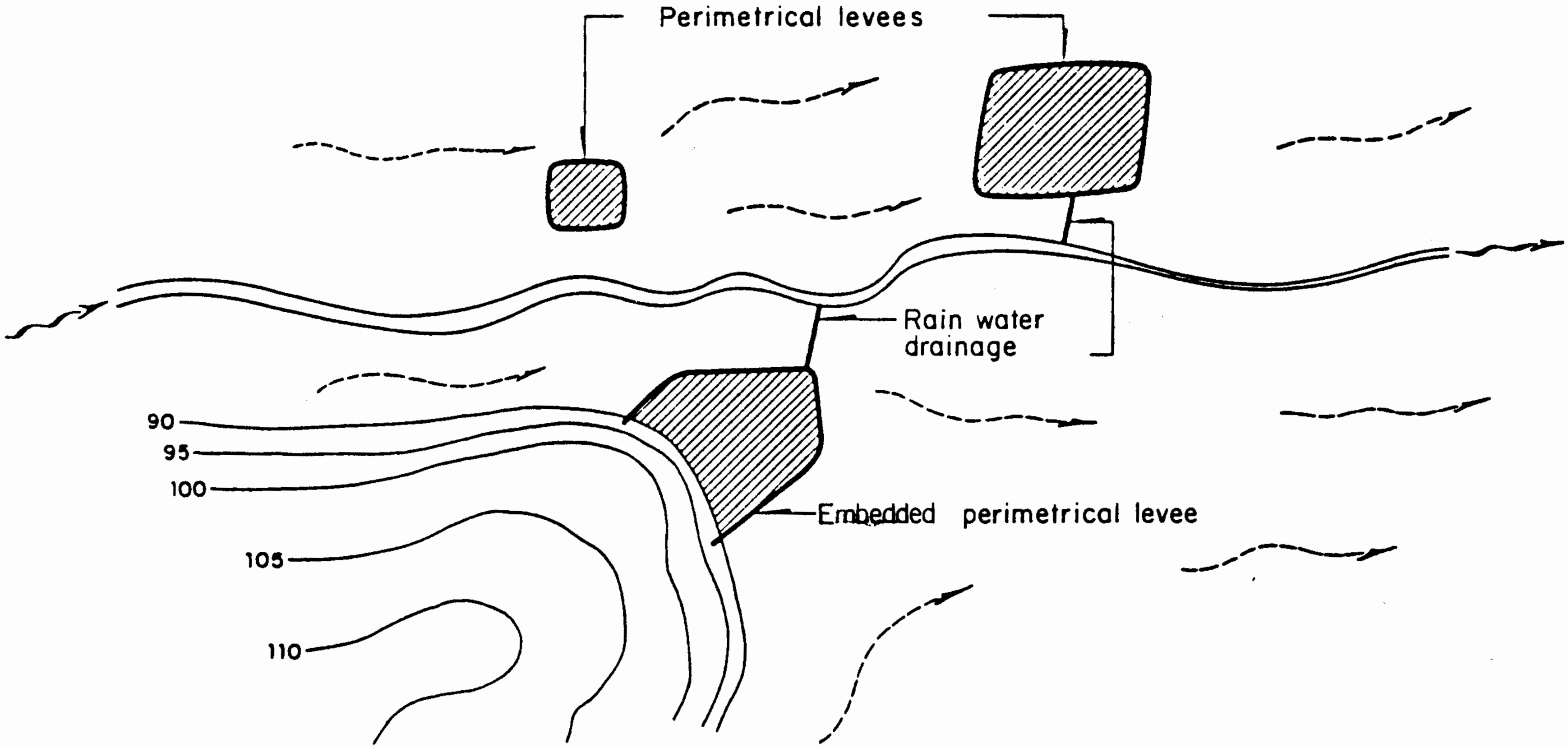


Fig 9.2
Perimetrical levees

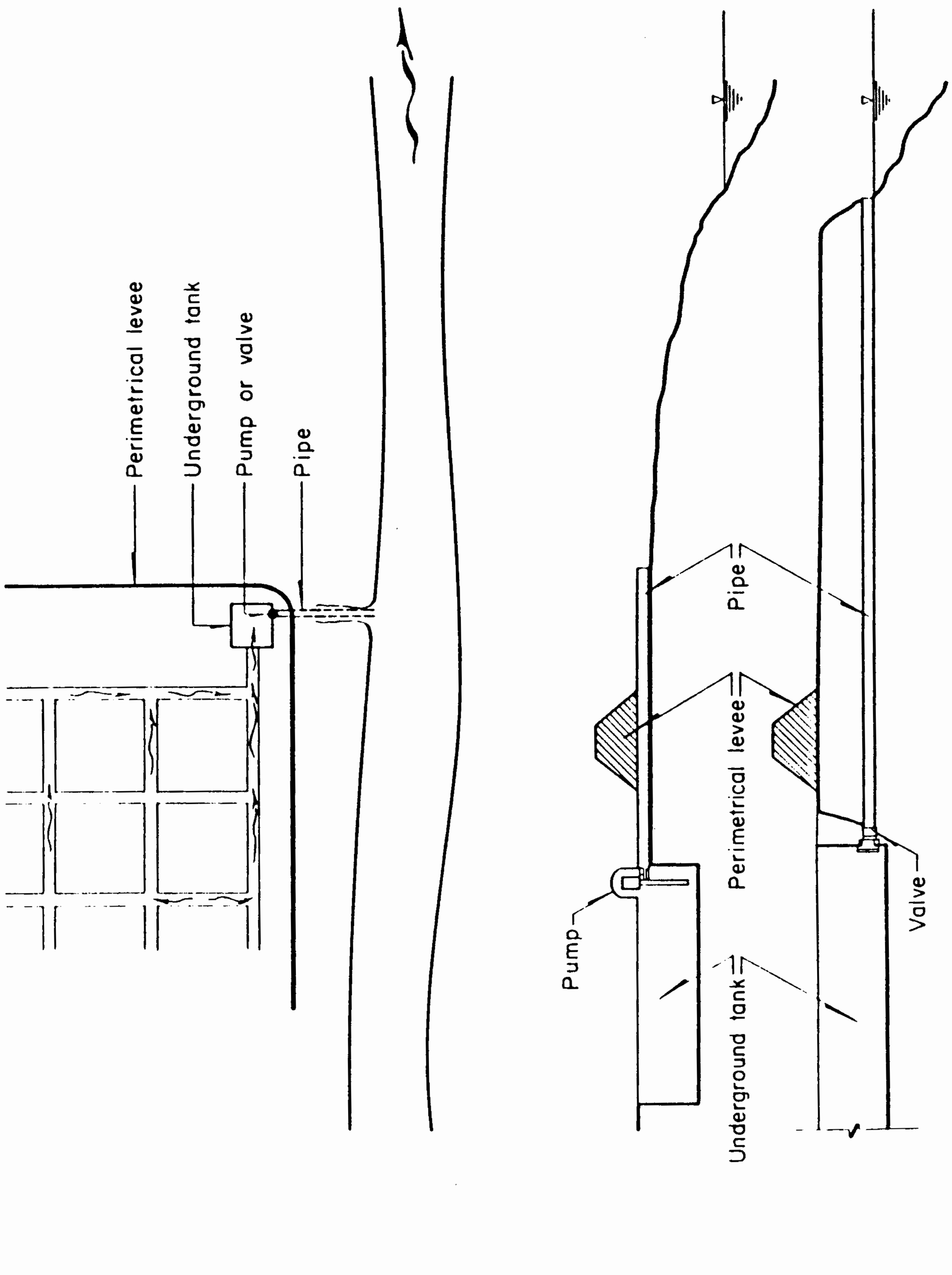


Fig 9.3
Drainage of the areas protected with levees

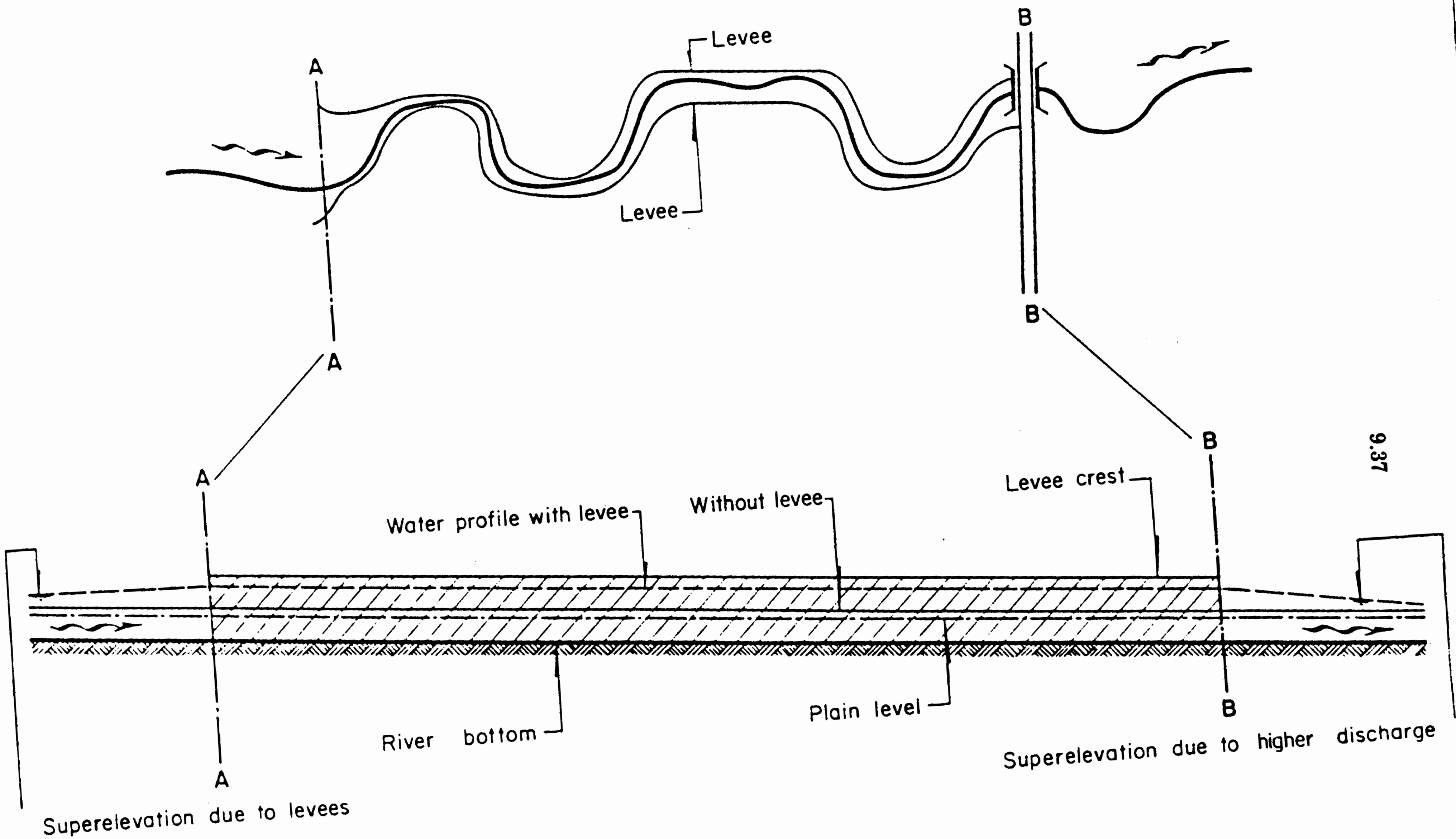


Fig 9.4
Effect due to longitudinal levees

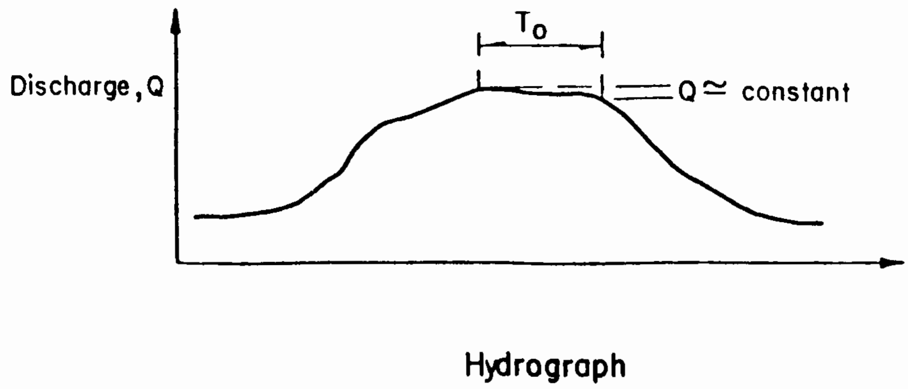
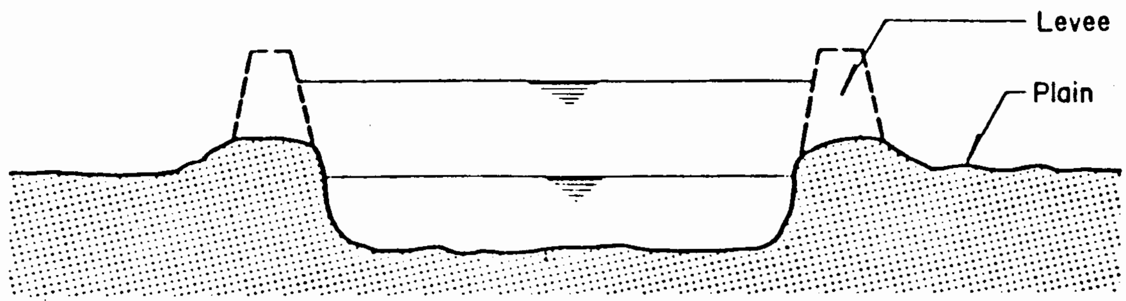
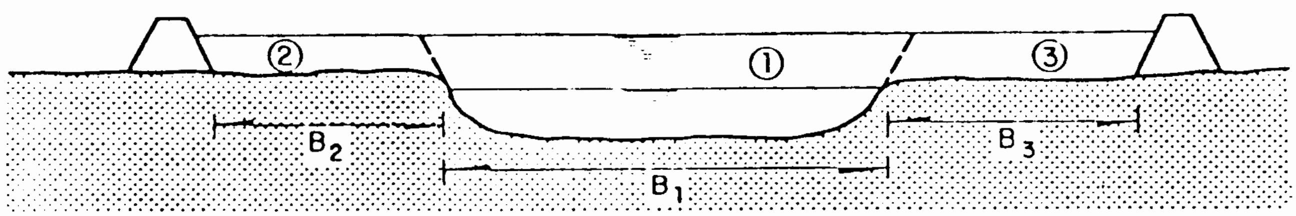


Fig 9.5
Kind of flow as a function of peak duration



a) Simple section of the principal channel



b) Compound section

Fig 9.6
Kind of sections according to leeves separation

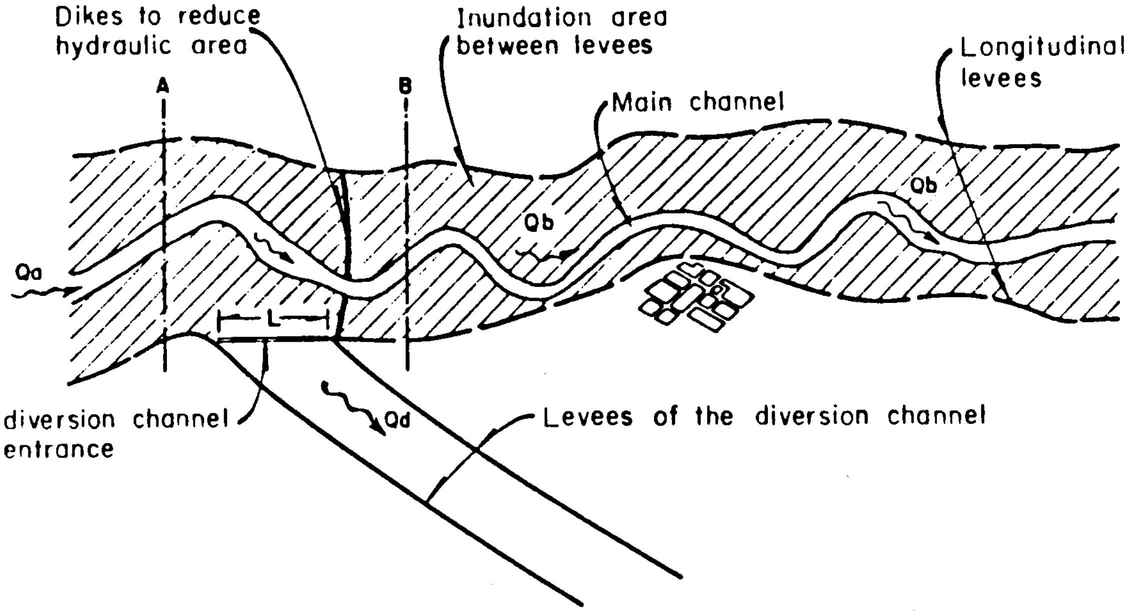


Fig 9.7
Longitudinal levees and diversion channel

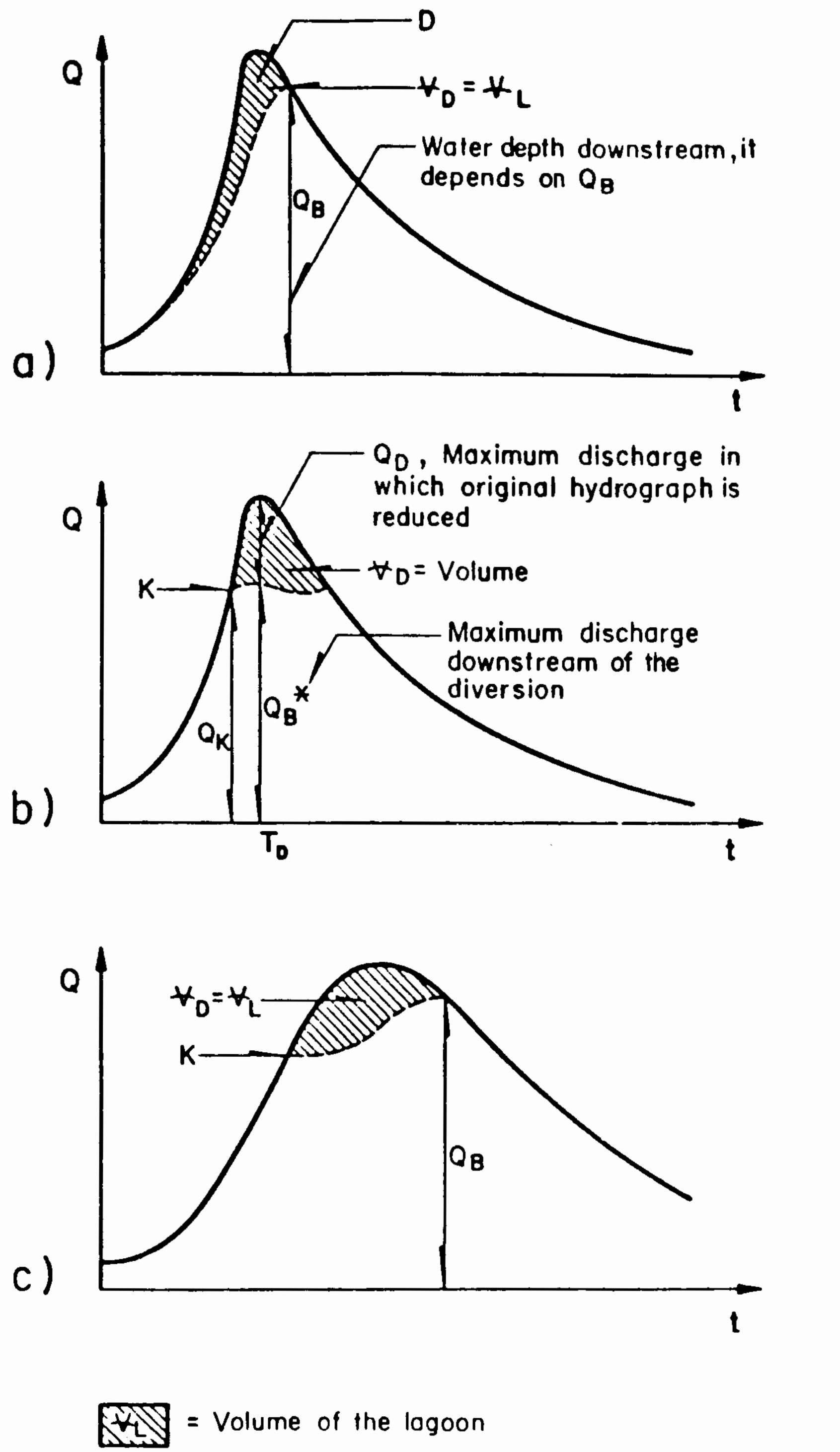


Fig 9.9
Different ways to deviate a small volume of water to a lagoon or zone with lower level

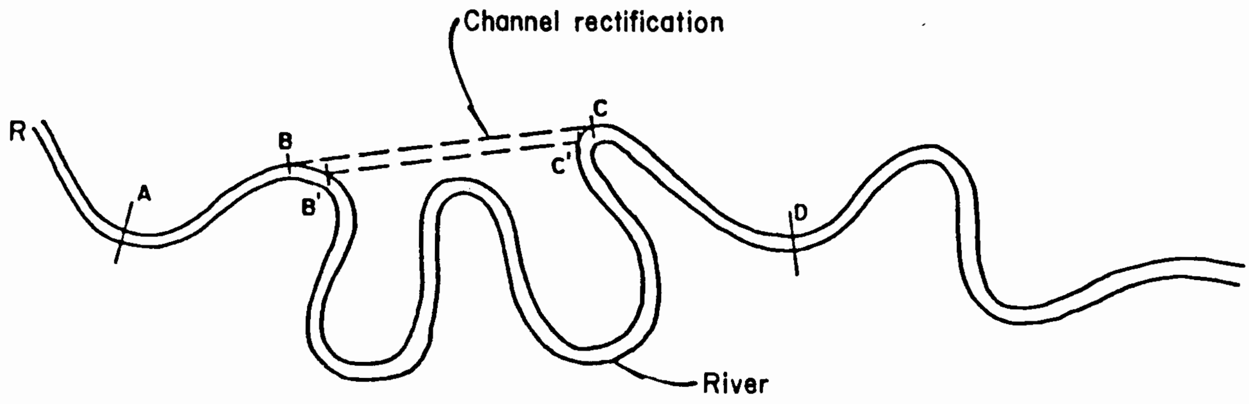


Fig 9.10
Channel rectification

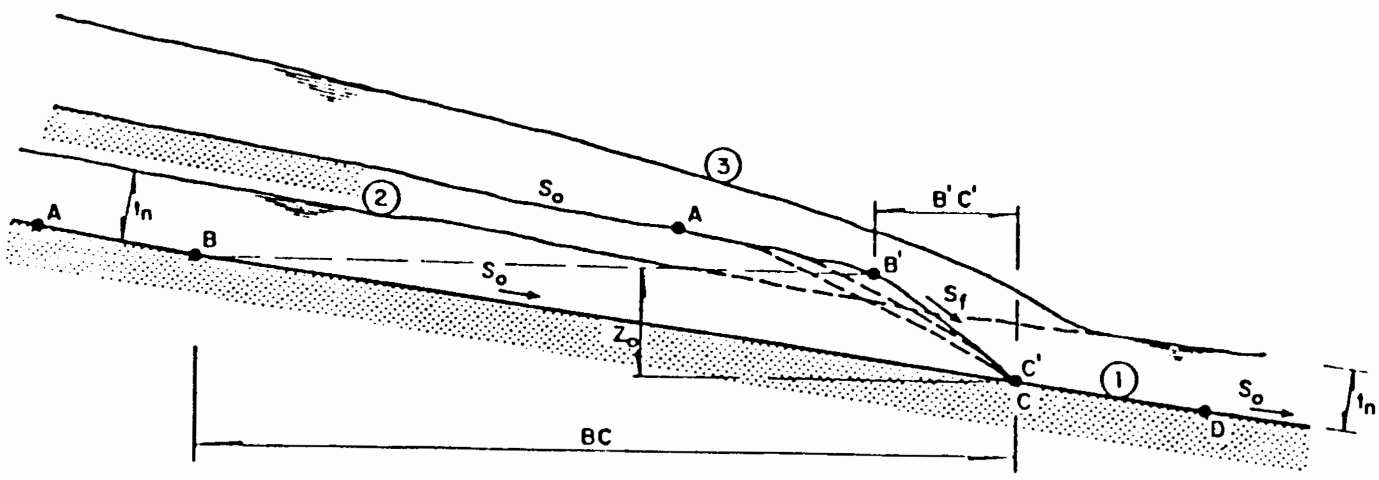


Fig 9.11
Channel rectification. Longitudinal profile

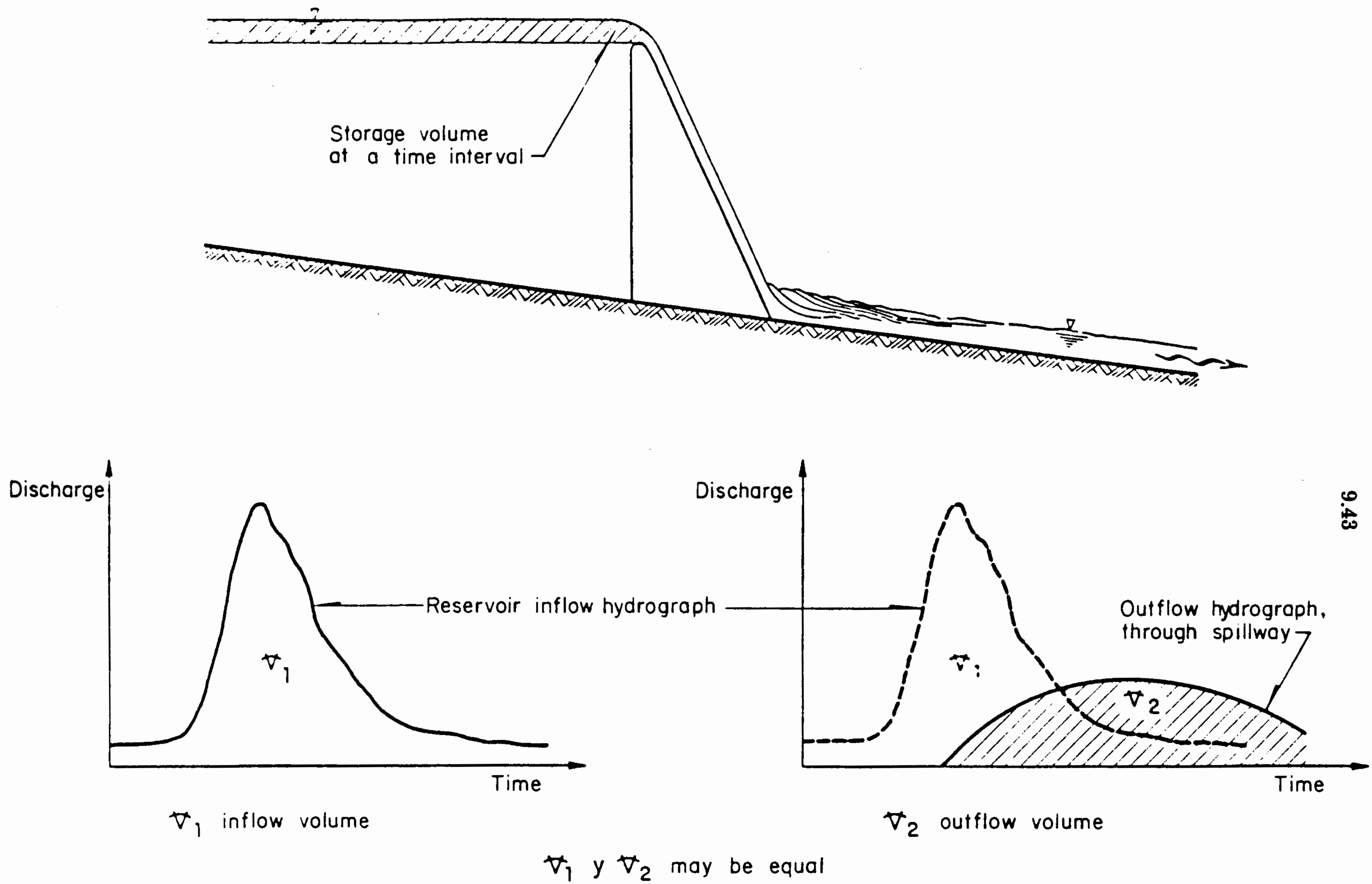


Fig 9.12
Regulative effect of large reservoirs

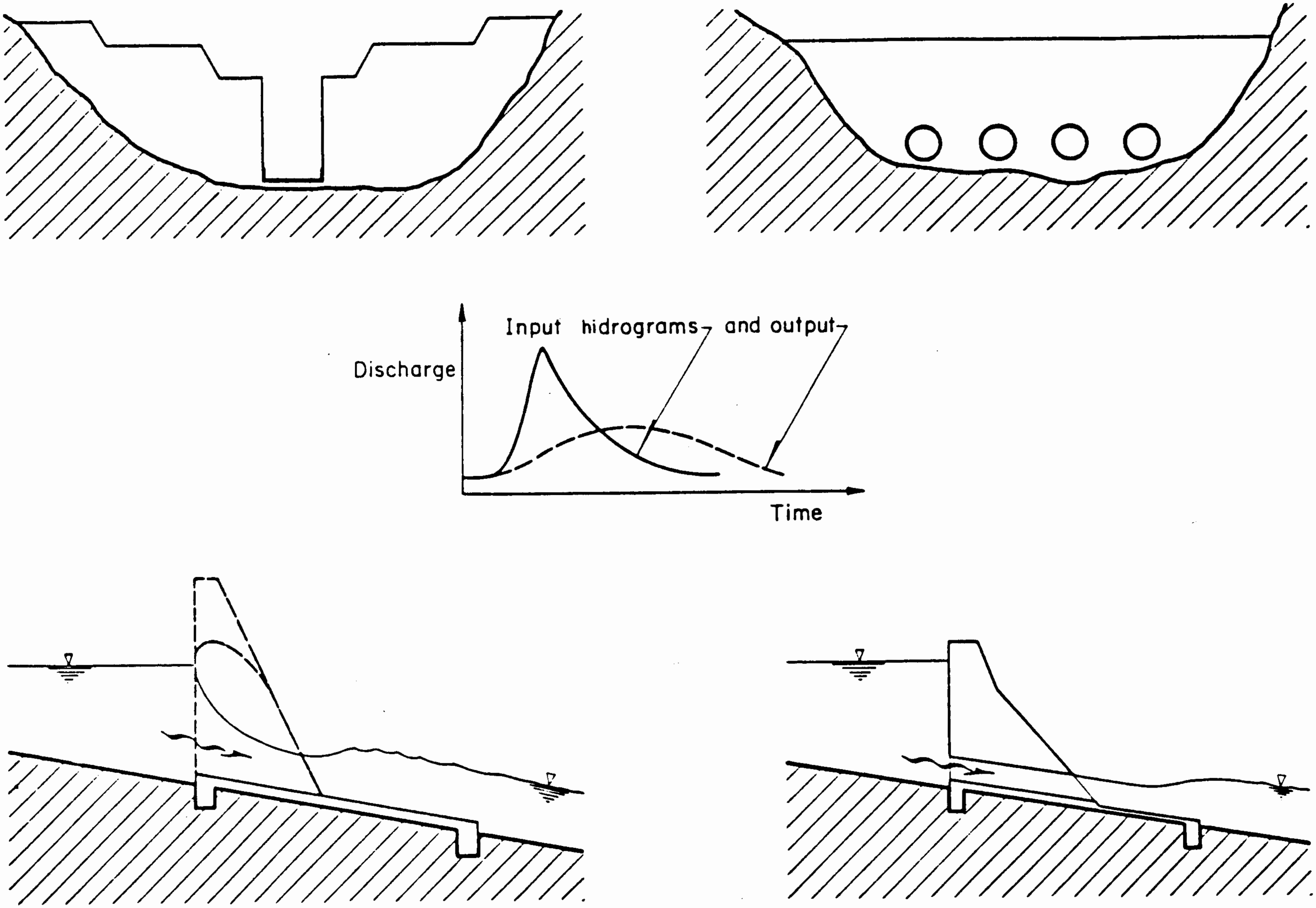


Fig 9.13
Peak-breaking dams

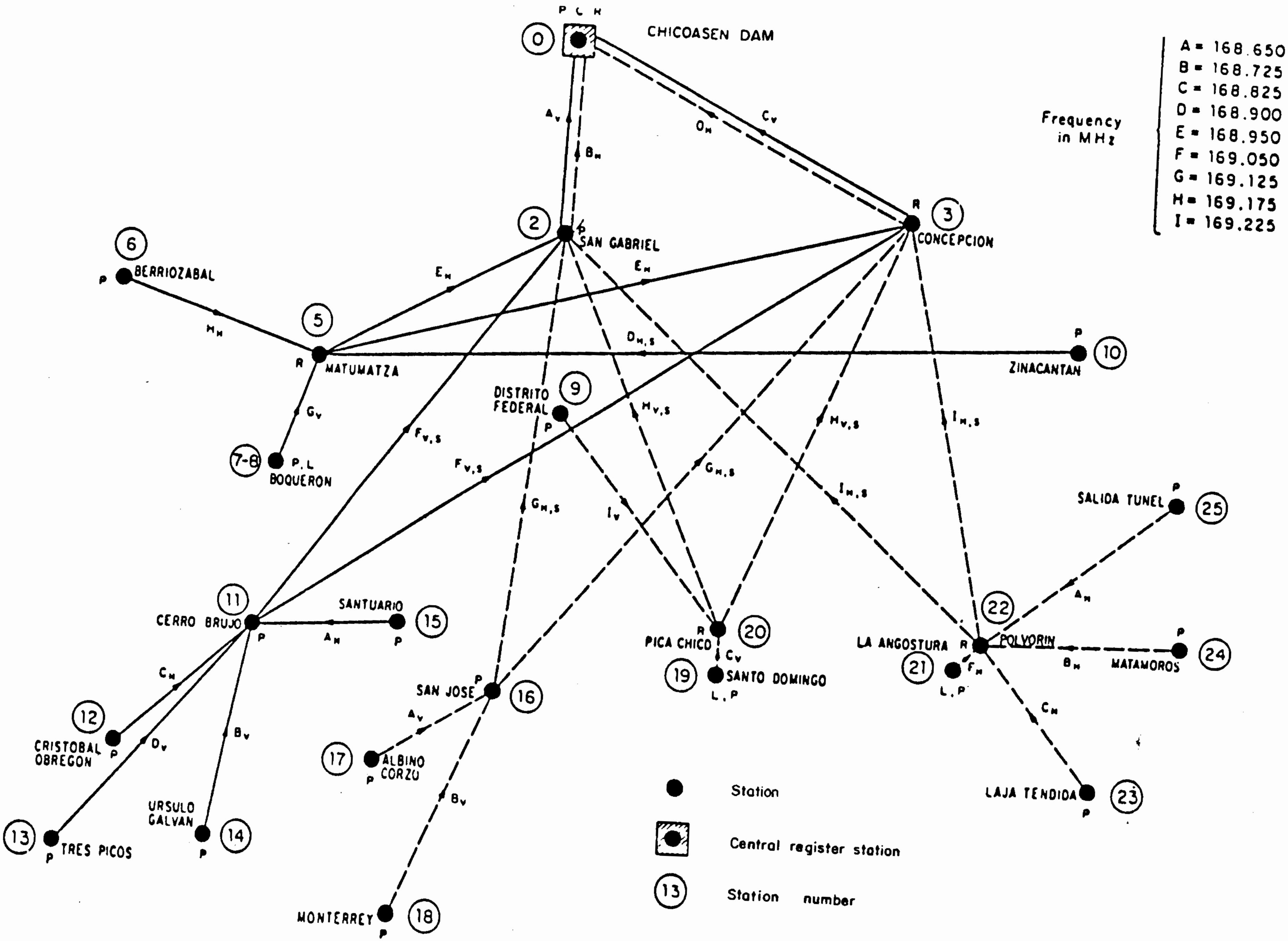


Fig 9.14

Signal distribution. Telemetric hydrologic net of Chicoasen, Chis.