ESTIMATION OF SEDIMENT TRANSPORT AND DOMINANT FLOW IN A HYPERCONCENTRATED FLOW

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ABSTRACT

Las Angustias gully constitutes the natural drainage of the Caldera de Taburiente and is placed on La Palma Island (Canary Islands). The Caldera has been declared National Park for the spectacular nature of its morphology, resulting from a strong erosive process.

The use of the flow occurring in the gully during the year has been hampered by the resources concentration in a low number of floods with high flow, high velocity and a high proportion of solid materials transport, whose index per km^2 is a 14% higher than the worldwide registered one.

For flood water capture, two intakes of Tyrolese type are to be built. It was necessary to estimate beforehand the capacity of sediments transport in the gully, so that the following factors constitute basic elements in the analysis: the sampling of the river bed material, the estimation of the drag coefficients for macro-rough flows (both mobile and rigid bed), the application limits of the transport sediment formulae, the determination of the gully dominant flow and its canalisation.

1. INTRODUCTION

Las Angustias gully constitutes the natural drainage of the Caldera de Taburiente and is placed on La Palma Island (Canary Islands). The river basin has an area of around 56 km^2 and receives the greatest volume of water in the archipelago (25 hm^3/year). The Caldera has been declared National Park for the spectacular nature of its morphology, resulting from a strong erosive process (Figure 1).

The rainfall regime, the extension and quality of the land, together with the inhabitants' work have favoured that the Aridane Valley becomes the richest cultivated area on the Canary Islands. More than 2,000 ha, mainly devoted to bananas growth, are of extreme importance on the island and archipelago economy. The inhabitants' wit, tenacity and effort have allowed to capture, lead and distribute the so demanded water, which comes from headwaters, galleries, wells and intakes.

Nevertheless, the shortage of resources and regulations provoke the overexploitation of the coastal wells, from which around 16 hm^3 a year are extracted, causing salinity and pollution problems. Some measurements have been taken against, as the capture of the punctual torrential floods occurring along Las Angustias gully, but it has been possible to capture only a small part despite the attempts made since more than a century.

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Figure 1 Gully of Las Angustias, Port of Tazacorte and Valley of Aridane

The use of the flow occurring in the gully during the year is hampered by its particular problems: (1) Steep morphology, short river bed and steep slopes both on river bed and sides. (2) Irregular rainfall regime, concentrated on few days a year. (3) Different permeability in materials.

These characteristics lead to the resources concentration on few flash floods, with a high proportion of solid materials transport. This problem conditions the gully's use, so it is not possible to utilize the conventional intake systems (dam-reservoir), since this would imply the sedimentation of the reservoir in a few years. Divert flows up to 2.5 m^3/s have been carried out by an intake work called 'tomaderos', which are similar to the well known Tyrolese or Caucasian intake works, but which are less efficient.

As the divert flood requirements are estimated in 20 m^3/s, the construction of two intakes is planned, with a distance of 1,6000 m between one another, 13 m^3/s capture capacity each one and using 3 m^3/s in each one to flush the intake system. The divert flow would be stored in two ponds (height: 18m, storage volume: 0.5 hm^3 c/pond) placed on the left side of the respective gully's channels. The channel design capacity is 1,000 m^3/s, which is superior to the return period of flood of 1,000 years.

2. STUDY OF SEDIMENT TRANSPORT

The eroded material in the Caldera is estimated to vary from 0.90 to 1.25 hm^3/year, assuming a generalised surface loss in the Caldera between 1.5 and 2 cm a year, that is, a mean erosion rate of 16,666 m^3/km^2/year, a value really higher than those registered in the worldwide literature.

Previous studies and works [PYPSA (1984)] have estimated that the annual sediment transport rate (maximum) is around 427,217 m^3 , a value representing a sediments concentration with respect to the interannual flow ($Q_{1,4}$ =121 m^3/s) of 2.72%, if this event took place 12 times a year on average. This sediment transport rate of 7,120 m^3/km^2 would represent around four times the maximum values worldwide registered. Although the real flow of Las Angustias gully is characterised for being flash flood and with great amounts of solid materials, it should have a concentration in weight and a sediment transport capacity really lower than those previously estimated, possibly due to the following causes:

- Sediment size distribution curves distorted by a bad sample.

- Subestimation of Manning resistance coefficients values.
- Use of Einstein-Brown formulae out of validity range.

2.1 Methods for the calculation of sediments transport

Regarding the source of sediments, the transport may be divided into on the one hand, wash load which comprises very fine material and is transported in suspension, and on the other hand, bed load which is transported along the river bed and in suspension, depending on the sediment size and flow velocity.

The main properties of sediment and its transport are: the particle size, shape, density, sedimentation velocity, porosity and concentration. The incipient movement state of the sediment for a flat bed is quantified from the critical value according to Shields, being its assessment less precise with bed forms. Research on sediment transport has been done for decades, without obtaining a really satisfactory equation which properly interrelate the flow and sediment properties. This is mainly due to the complexity of the problem, including the effect of the different bed forms on the fashion and magnitude of the bed transport, the stochastic character of the problem and the difficulty to verify the laboratory research in prototype. Nevertheless, significant advances have been achieved. Most followed approaches can be synthesized into a correlation between the sediment transport parameter Φ and a flow parameter Ψ:

$$\Phi = \frac{q_s}{D^{3/2}\sqrt{g\Delta}} \tag{1}$$

$$F_{rd}^2 = \frac{1}{\Psi} = \frac{U*^2}{\Delta g D} \tag{2}$$

Where: q_s is the total bed transport (m^3/sm); $\Delta = (\rho_s - \rho)/\rho$ =1,65 sediment specific density; D sediment particle size (m); $U* = \sqrt{\tau_0/\rho} = \sqrt{gRS_0}$ sheer velocity (m/s); S_0 river bed longitudinal slope.

The value F_{rd}^2 is the square of the densimetric Froude number and is equivalent to the inverse of the flow parameter Ψ. In general the river bed transport q_s varies with velocity power V^n, where n may vary between 3 and 6; thus, a good knowledge of velocity field is required.

2.2 Formulae used for the calculation of sediments transport

From the great diversity of formulae available for the calculation of sediment transport, those best fitting Las Angustias gully's characteristics have been selected and always taking into account the so particular geomorphologic conditions. Therefore, regarding Simons and Sentürk (1992) and Graf's (1984) the following formulae have been selected for our analysis: Colby (1964), Meyer-Peter and Müller (1948), Ackers-White (1990), Engelund-Hansen (1967), Yang (1976), Einstein-Barbarrosa (1952). Einstein-Brown's equation (1950) constitutes a simplified procedure from Einstein's general method which is included in the current study for being the main method used in PYPSA's work (1984). Table 1 shows the formulae and the main parameters.

2.3 Basic information for the calculation of sediment transport

Two types of information is required: first, the granulometric bed material (characteristic diameters) and then, hydraulic information from the flow characteristics. Further information to determine the wash load deals with the measurement of the sediments concentration in suspension, but it has not been possible to carry out on this stage of the study.

Table1 Sediment Transport Formulae used in the study

AUTHOR	FÓRMULAE	OBSERVATIONS
Colby (1964)	$g_{BTmax} = 1{,}13U^{3{,}326}$; $g_{BT\,\text{mín}} = 0{,}46U^{3{,}326}$ $\quad 0{,}1mm \le D_{50} \le 1mm$	U – mean velocity d- depth
Meyer-Peter y Müller (1948)	$\gamma(\frac{K_s}{K_r})^{3/2} R_s I = 0{,}047\gamma'_s D_m + 0{,}25\gamma_s'^{\,2/3} \rho^{1/3} (\frac{g_{BT}}{\gamma_s})^{2/3}$; $K_r = \frac{26}{D_{90}^{1/6}}$; $K_s = \frac{B^{2/3} K_m K_w}{\{K_w^{3/2}(B+2d) - K_m^{3/2} 2d\}^{2/3}}$; $K_m = 1/n$; K_w	g_{BT} = Unit total bed transport in weight (T/ms) q_{bv} = Unit total bed transport in volume (m^3/ms)
Ackers-White (1990)	$g_{BT} = \gamma_s K D_{35} U (\frac{U}{U_*})^n (\frac{F_*}{F_{*c}} - 1)^m$ Si $1 \le D_* \le 60$: K= $\exp\{2{,}79Ln(D_*) - 0{,}426(LnD_*)^2 - 7{,}967\}$ $n = 1 - 0{,}56\log D_*$; $F_{*c} = (0{,}23/\sqrt{D_*}) + 0{,}14$; $m = (6{,}83/D_*) + 1{,}67$ $K = \exp\{2{,}79Ln(D_*) - 0{,}426(LnD_*)^2 - 7{,}967\}$ $F_* = (U_*^n/\sqrt{g\Delta D})\{U/(\sqrt{32}\log(10d/D))\}^{1-n}$; $D_* = D_{35} \cdot \left(\frac{g \cdot \Delta}{\nu^2}\right)^{1/3}$	**The method is applicable if:** $D_* \ge 1$ y $F_* \le 8$ Si $D_* > 60$:n=0;m=1,78 F_{*c} =0,17; $K=0{,}025$ $\sigma_g = \left(\frac{D_{84}}{D_{16}}\right)^{0.5}$ $D=D_{35}$ si $\sigma_g > 3$ $D=D_{50}$ si $\sigma_g \le 3$
Engelund-Hansen (1967)	$g_{BT} = 0{,}05\gamma_s U^2 \tau_*^{3/2} (\frac{D_{50}}{g\Delta})^{1/2}$	**The method is applicable if:** $\text{Re}_* = \frac{U_* D_{50}}{\nu} \ge 12$ $D_{50} \ge 0{,}15mm$; $\sigma_g \le 2$
Yang (1976)	**Sand transport**: $g_{BT} = 0.001 \cdot U \cdot d \cdot \exp\left\{11.8929 - 0.153 \cdot L_n\left(\frac{\omega \cdot D_m}{\nu}\right) - 0.297 \cdot L_n\left(\frac{U_*}{\omega}\right) + \right.$ $\left. + \left[1.78 - 0.1563 \cdot L_n\left(\frac{\omega \cdot D_m}{\nu}\right) - 0.2085 \cdot L_n\left(\frac{U_*}{\omega}\right)\right] \cdot L_n\left(\frac{U \cdot S}{\omega}\right)\right\}$ **Gravel transport:** $g_{BT} = 0.001 \cdot U \cdot d \cdot \exp\left\{15.3836 - 0.633 \cdot L_n\left(\frac{\omega \cdot D_m}{\nu}\right) - 4.816 \cdot L_n\left(\frac{U_*}{\omega}\right) + \right.$ $\left. + \left[2.784 - 0.1327 \cdot L_n\left(\frac{\omega \cdot D_m}{\nu}\right) - 0.1228 \cdot L_n\left(\frac{U_*}{\omega}\right)\right] \cdot L_n\left(\frac{U \cdot S}{\omega} - \frac{U_c \cdot S}{\omega}\right)\right\}$	Si $1{,}2 < \frac{U_* D}{\nu} < 70$ $\frac{U_c}{w} = \frac{2{,}5}{\log(U_* D/\nu) - 0.06} + 0{,}66$ Si $70 \le \frac{U_* D}{\nu}$; $\frac{U_c}{w} = 2{,}05$
Einstein-Barbarossa (1952)	$i_B g_{Bi} = \Phi_* i_b \gamma_s (g\Delta D_i^3)^{1/2}$; $g_{BT} = g_B + g_{BS}$ $i_S g_{BSi} = i_B g_{Bi}\{P_E I_1 + I_2\}$; $i_{BT} g_{BTi} = i_B g_{Bi} = \{1 + P_E I_1 + I_2\}$ $g_{BT} = \sum_{i=1}^{n} i_{BTi} g_{BTi}$.; $\Psi_* = \xi Y(\beta/\beta_x)^2 \Psi'$; $\xi = f(D/X)$; $Y = f(D_{65}/\delta')$ $X = 0{,}77\Delta'$ si $\Delta'/\delta' > 1{,}8$; $X = 1{,}39\delta'$ si $\Delta'/\delta' < 1{,}8$; $\beta = \log 10{,}06 = 1{,}025$ $\beta_x = \log(10{,}6X/\Delta')$; $\Psi' = \Delta(D_i/R'I)$; $E = a/d$; $a = 2D_i$; $z = w/(K\beta U'_*)$ **Note**: The integrals are calculated by means of the respective abacus. All the values of the different parameters are not included. For a more complete description see Simons and Sentürk (1992) and Graf (1984).	$P_E = 2{,}303\log\frac{30{,}2d}{\Delta'}$ $I_1 = 0{,}216\frac{E^{Z-1}}{(1-E)^Z}\int_E^1 (\frac{1-y}{y})^Z dy$ $I_2 = 0{,}216\frac{E^{Z-1}}{(1-E)^Z}\int_E^1 (\frac{1-y}{y})^Z \ln(y)dy$ $\Delta' = \frac{K_s}{\chi}$; $K_s = D_{65}$; $\chi = f(K_s/\delta')$ $U' = \sqrt{gR'I}$
Einstein-Brown (1950)	$\Phi = f(\frac{1}{\Psi})$; $\Phi = \frac{q_{bv}}{\gamma_s K\sqrt{g((\gamma_s/\gamma)-1)D_S^3}}$; $\frac{1}{\Psi} = \frac{\tau}{(\gamma_s - \gamma)D_s}$; $\Phi = \frac{g_{bw}}{K\sqrt{g\gamma'_s D_s^3}}$ si $1/\Psi > 0{,}9$ $\Phi = 40(\frac{1}{\Psi})^3$; $K = \sqrt{\frac{2}{3} + \frac{36\nu^2}{gD_s^3((\gamma_s/\gamma)-1)}} - \sqrt{\frac{36\nu^2}{gD_s^3((\gamma_s/\gamma)-1)}}$	**The method is applicable if :** $0{,}19 \le \tau_* \le 1{,}00$ $\tau_* = \frac{\tau_0}{(\gamma_s - \gamma)D_s}$ ν - cinematic viscosity

Flow characteristics interrelate with channel bed material through resistance coefficients, whose coupling with sediment transport classic formulae is not solved for macrorough flows yet.

2.4 Sampling of bed channel's material and obtaining representative sediment size distribution curves

Volumetric sampling techniques have been followed by extracting from the channel a volume of subsuperficial material. First, the superficial layer as thick as the size of the biggest particle observed in the surface (and the flow has been able to transport) is removed.

The sampled volume must be representative of the channel granular material, so the biggest extracted particle must not represent more than 1% in weight of the whole sample. Three samples have been taken in three different places (C-1, C-5 y C-10), whose weight is 51.671 kg, 13.568 kg and 30.519 kg, respectively. Each sample was subjected to quarter method as much as necessary, in order to get the fraction for the standard sieve of the particles smaller than 80 mm. Therefore: C-1=29.132 kg; C-5=29.956 kg and C-10=32.860 kg, and finally the corresponding sediment size distinctive curves are obtained. Given the curves similarity, an only mean curve was taken.

Since the particles size which can be transported according to Shields criterion (corrected by armour phenomenon) is 0.60 m for interannual flow ($Q_{1.4}$=121 m^3/s) and 1.30 for millenary flow (Q_{1000}= 850 m^3/s), an amplified sediment size distribution curve was built to include the particles bigger than 80 mm. For this purpose, the particles up to 2,260 mm found within the sample were taken into account. Finally, only the particles up to a 1,230 mm of equivalent diameter were included.

The three amplified sediment size distribution curves (with the corresponding correction in weight) were averaged to get a mean amplified sediment size distribution curve characteristic of the particles up to 1,200 mm.

The particles bigger than 100 mm (4 inches) were removed from the sediment size distribution curves of PYPSA study (1984), assuming they weight approximately 20% of the sample. However, even considering $D_{máx}$=100 mm from the resulting sediment size distribution curve, if the volumetric sampling criterion is applied, not only should 50 kg be sampled but 100 kg. Besides, when rejecting the particles bigger than 100 mm without measuring or reckoning for their appropriate inclusion in the sediment size distribution curve, the samples are not representative of the river bed.

Table 2 shows the characteristic diameters of the mean sediment size distribution curves (up to 80mm and amplified up to 1,200), together with the characteristic diameters of PYPSA (1984) and the correction proposed by the author. It should be highlighted that the results obtained after the authors' proposal are very closed to the characteristic diameters corresponding to the curve up to 80 mm. The great difference among characteristic diameters, which at first are representative of the same river bed, is mainly due to the sampling criterion.

Table 2 Characteristic diameter of the different sediment size distribution curves

Sediment size distribution curve	D_{90} (mm)	D_{84} (mm)	D_{65} (mm)	D_{50} (mm)	D_{35} (mm)	D_{16} (mm)	D_m (mm)
PYPSA (1984):	-	100	37	11.74	5.35	0.25	17.8
PYPSA corrected:	50	31	14.1	6.5	3.5	0.18	15.6
TYPSA(1998)							
Up to 80 mm:	38	27	14.1	5.9	2.3	0.5	13.8
Up to 1.200 mm:	1,000	870	420	28	9.5	1.3	370

An immediate consequence of this difference is the high impact in the calculation of sediment transport, since it is inversely proportional to the diameter of the characteristic particle raised to three halves: $q_s \alpha (1/D^{3/2})$.

2.5 Estimation of Manning resistance coefficient in mobile bed

The calculation of the flow characteristics depends mainly on the resistance coefficient, apart from the cross section and the longitudinal slope. Given the great quantity of sediment transport and the considered size, we are facing a macroroughness problem. Therefore, in the case of a 1,000 years return period the main geometric values of the stretch of gully under study (lower width =b=33 m; left side=1:0,49 (V:H); right side=1:1,74 (V:H); longitudinal slope=0,0392) and the characteristic mean curve amplified up to 1,200 mm, then: $D_{84} \cong$ 0,870 m; Q=1.000 m^3/s; y=3,86 m; R=3,20 m. So we find a macroroughness problem, since y/D_{84}<50 (*3,86/0,870=4,44<50*).

The bed form resistance is not explicitly taken into account because R_h/D_{50}<2.000 (*3,20/0,028=114,29<2.000),* or a possible rise of resistance for the variation in flow density and viscosity, because the sediments concentration is really inferior to 10% in weight (limit between hyperconcentrated flow and mud flow) [Wang (1994)].

Nevertheless armour phenomenon does take place since the sediment size distribution typical deviation is extended or graduated (σ_g>3); for the two characteristic curves:

Up to 80 mm

$$\sigma_g = \left(\frac{D_{84}}{D_{16}}\right)^{0.50} = \left(\frac{0,0130}{0,0005}\right)^{0,50} = 5,10 > 3;$$

Aplified up to 1200 mm

$$\sigma_g = \left(\frac{D_{84}}{D_{16}}\right)^{0,50} = \left(\frac{0,87}{0,0013}\right)^{0,50} = 25,87 > 3$$

The resistance coefficient is quantified from the new characteristic sediment size distribution curve extended to 1,2000 mm, since at first, this curve best represents the river basin characteristics. There exist different formulae for the estimation of the roughness coefficient in case of macrorough flows, whose foundations are based on Keulegan integration (1938), from Prandtl-Von Kármán's law of the mean distribution of turbulent, permanent and uniform flow velocities, in straight channels with rigid and rough limits. These equations are generally expressed as:

$$C* = \frac{V}{V*} = \left[\frac{8}{f}\right]^{1/2} = \frac{2,3026}{\kappa}\log\left[a\frac{R}{Ks}\right] \tag{3}$$

Where: $C*$ is Chézy nondimensional coefficient ($C* = C/\sqrt{g}$); g gravity acceleration; V mean flow velocity; $V*$ velocity associated to sheer stress ($V* = \sqrt{\tau_0/\rho}$); ρ and γ density and water specific weight; τ_0 mean sheer stress produced by the flow into the channel's walls and bed ($\tau_0 = \gamma R S_0$); R hydraulic radius in the transversal section of the channel; S_0 bed slope (in uniform flow it is equal to hydraulic gradient S_w and energy line S_f); f Darcy-Weisbach friction factor; κ Von Kármán constant in clean water (≅0,407); a form coefficient (depending on the geometry of the channel transversal section); Ks roughness equivalent to Nikuradse grain of sand.

In mobile bed channels, beside gravity action and surface resistance, sediment transport and form resistance (waves or coarse grains on the surface bed) also influence on the resistance to flow. However, in straight rivers constituted by rough material where sediment transport does not produce considerable waves on the river bed, we can use Keulegan equation.

The equivalent roughness is usually expressed in terms of a river bed granulometry characteristic diameter D_n; so, $Ks = \alpha_n D_n$, where α_n is the nondimensional factor of texture or relative equivalent roughness, which depends on flow conditions and disposition and representative size of the river bed's roughness. If the main values are replaced in Keulegan equation, the following formulae are obtained:

$$C* = \frac{V}{V*} = \left[\frac{8}{f}\right]^{1/2} = 5{,}657\log\left[\frac{R}{D_n}\right] + A_n \quad (4)$$

$$A_n = 5{,}657\log\left[\frac{a}{\alpha_n}\right] \quad (5)$$

Next, the formulae used for the calculation of the resistance to flow in rough river beds of steep slope are presented, indicating the validity range of relative immersion. Where there are no explicit expressions for the calculation of Manning coefficient, Strickler relation has been used: $n = R^{1/6} / C$.

Table 3 Formulae of resistance coefficient in permanent river bed of macrorough flows

AUTHOR	FÓRMULAE	OBSERVATIONS
Limerinos (1970)	$C* = 5{,}657\log\left[\frac{R}{D_{84}}\right] + 3{,}281 \quad 0{,}90 \le \frac{R}{D_{84}} \le 68{,}55$	$n = \frac{0{,}1129 R^{1/6}}{2\log\left[\frac{R_b}{D_{84}}\right] + 1{,}160}$
	$C* = 5{,}657\log\left[\frac{R}{D_{50}}\right] + 0{,}990 \quad 1{,}90 \le \frac{R}{D_{50}} \le 177$	R–Total hydraulic radius
Bathurst (1985)	$C* = 5{,}62\log\left[\frac{d}{D_{84}}\right] + 4 \quad 0{,}3 \le \frac{d}{D_{84}} \le 50$	$0{,}4\% \le S_0 \le 4\%$
	$C* = 5{,}657\log\left[\frac{d}{D_{50}}\right] + 1{,}333 + 0{,}737\left[\frac{1}{d/D_{50}}\right] \quad 0{,}3 \le \frac{d}{D_{50}} \le 77$	d- Depth $0{,}001\% \le S_0 \le 6{,}55\%$
Fuentes y Aguirre (1991)		
García Flores (1996)	Supercritical Regimen: $C* = 5{,}756\log\left[\frac{d}{D_{84}}\right] + 3{,}698 \quad 0{,}3 \le \frac{d}{D_{84}} \le 100$ $C* = 5{,}756\log\left[\frac{R_b}{D_{50}}\right] + 1{,}559 \quad 0{,}6 \le \frac{R_b}{D_{50}} \le 200$	$n = \frac{0{,}111 d^{1/6}}{2\log\left[\frac{d}{D_{84}}\right] + 1{,}2849}$ Rb – Bed total hydraulic radius
	Subcritical Regimen: $C* = 5{,}756\log\left[\frac{d}{D_{84}}\right] + 2{,}2794 \quad 0{,}3 \le \frac{d}{D_{84}} \le 100$ $C* = 5{,}756\log\left[\frac{R_b}{D_{50}}\right] + 0{,}2425 \quad 0{,}6 \le \frac{R_b}{D_{50}} \le 200$	$n = \frac{0{,}111 d^{1/6}}{2\log\left[\frac{d}{D_{84}}\right] + 0{,}7919}$

Table 4 shows Manning resistance coefficients in mobile river bed, calculated as the arithmetic average of the values obtained from the macrorough formulae, for the different flows in Las Angustias gully, in a stretch immediately upstream of the future intake installation. The Manning coefficient n=0,088 corresponding to interannual flow ($Q_{1,4}$=121 m^3/s), is really higher than the value estimated by PYPSA (1984) for a similar flow (Q=145 m^3/s), where n=0,0205; a value completely undervalued for the real physical conditions of Las Angustias gully.

Table 4 Resistance coefficients and main hydraulic parameters

Q (m^3/s)	n	v (m/s)	y (m)	A (m^2)	P (m)	R (m)	Froude Number
50	0,104	1,68	0,87	29,70	35,73	0,831	0.58 Subcritical R.
121	0,088	2,61	1,35	46,41	37,20	1,248	0.73 Subcritical R.
500	0,068	5,14	2,70	97,23	41,43	2,347	1.00 Critical R.
1000	0,062	6,93	3,86	144,20	45,06	3,20	1.19 Supercritical R.

2.6 Analysis of the results obtained from the calculation of sediment transport

Table 5 summarises all the results in (T/s), obtained from the seven calculation methods, for the two characteristic sediment size distribution curves and the four flows under analysis. It is necessary to remember that the presented calculation methods do not quantify the transport of wash load.

Table 5 Sediment transport from different methods and characteristic sediment size distribution curves

FORMULAE	Curve up to 80 mm				Curve up to 1200 mm			
	Q=50 m^3/s	Q=121 m^3/s	Q=500 m^3/s	Q=1000 m^3/s	Q=50 m^3/s	Q=121 m^3/s	Q=500 m^3/s	Q=1000 m^3/s
1. Meyer-Peter y Muller	0,13	0,49	2,74	5,54	-	-	0,70	6,43
2. Ackers-White	0,03	0,13	0,99	2,34	0,01	0,07	0,73	1,77
3. Engelund-Hansen	1,54	7,21	79,06	245,67	0,33	1,52	16,66	51,77
4. Einstein-Brown	206	697	4634	11745	19,99	67,70	450,28	1141
5. Einstein-Barbarrossa	0,05	0,32	1,38	6,66	1E-05	6E-05	0,10	0,43
6. Yang	0,89	2,51	11,75	23,59	1,75	4,26	16,07	29,67
7.1. Colby (máximos)	0,21	0,95	8,63	23,33	0,21	0,95	8,63	23,33
7.2. Colby (mínimos)	0,08	0,39	3,51	9,50	0,08	0,39	3,51	9,50
8. Average (T/s)	**0,23**	**0,80**	**4,83**	**11,83**	**0,08**	**0,35**	**2,74**	**8,29**

The results obtained from Einstein-Brown y Engelund-Hansen methods are really higher than those from the other methods, because the calculation values go out of the application range of these formulae. Furthermore, Yang method also presents some problems in its application range in the curve amplified up to 1,200 mm. That is the reason why these methods are not taken into account when working out the average. As PYPSA (1984) used the Einstein-Brown method, its results are not valid. Meyer-Peter and Müller method does not quantify the suspension-bottom load. At any case, the values obtained from this method is placed around the average value of the considered method. This formula is not applicable to the two first analysed flows in the case of the curve amplified up to 1,200 mm, as shown in the table. In the case of the characteristic curve to 80 mm, the sediment transport average falls within Colby's maximum and minimum transport limits, what means that these results would be reasonable.

Regarding the characteristic curve amplified to 1,200 mm, the arithmetic average falls slightly under Colby's minimum transport value, because Ackers-White and Einstein-Barbarossa methods undervalue the calculation of sediment transport for the unusual big size of the characteristic diameters and possibly, these calculation methods are not adequate for the prediction of transport on this size scale.

In any case, Colby method provides the maximum and minimum transport capacity in function of mean velocity and based on transport envelopes to D_{50}=1 mm. In this case D_{50} are equal to 6 mm and 28 mm respectively, from which maximum and minimum transport limits would be assumed to be lower than those proposed by Colby. Therefore, we can state that the sediment transport for the different flows would be between the values calculated from the two characteristic sediment size distribution curves to 80 mm and amplified to 1,200.

3. DETERMINATION OF THE DOMINANT FLOW

From a geomorphologic perspective, the dominant or formative flow is the one determining the flow geometry in high water level, whereas the regimen theory perspective seeks to substitute the flow annual regimen variability for an equivalent flow. The volume of sediments transported in a year is supplied not only by high water level flow, for its high solid flow transported (despite its short length of time), but also by mean flow, for its long length of time (despite its low solid flow transported). Given that river bed sediment transport does shape the river bed, the dominant flow is defined as the one which runs as constant flow during the whole year, and transports the same volume of total river bed sediments as well. In general, the dominant flow corresponds to the interannual flow, that is, a flood with a return period between 1and 2 years (Richard, 1982). However, in the case of torrential and unstable rivers from a hydrologic perspective (as Las Angustias gully), the dominant flow could even correspond to a return period of 7 years, as it usually happens to some Mediterranean rivers (Martín Vide, 1997).

In order to characterize the number of equivalent events in such a way that, the times that an event occurs in a characteristic year could be estimated to adjust the corresponding solid transport, we proceeded to reckon the events from the known hydrologic data. Likewise, the days in which an event took place were reckoned and the equivalent hydrograms events in 6 h time base were obtained (3 hours of concentration time in the channel). The obtained results are shown in table 6:

Table 6 Number of equivalent events corresponding to different flows

Return period	**FLOW (m^3/s)**	**Nº of days in which the event takes place**	**Nº of years of the sample**	**Days/years average in which the event takes place**	**6 h equivalent events**
1.07	50	51	27	1.89	7
1.4	121	80	27	2.97	12
2	172	47	27	1.75	7
5	277	34	27	1.25	5
7	300	23	27	0.85	4
10	350	11	27	0.40	2
25	447	5	27	0.74	1

The number of events in a year is estimated to vary among the 7 events corresponding to flow $Q_{1,07}$=50 m^3/s, 12 events of the interannual flow $Q_{1,4}$=121 m^3/s and strict events from the 25 years return period. Figure 2 shows the relations ‘liquid flow - Nº of events’ and ‘liquid flow - solid flow’, and the resulting product curve. We can observe that the flow most frequently presented corresponds to interannual flow $Q_{1,4}$=121 m^3/s, whereas the dominant flow corresponds to the flow of 5 years return period Q_D= Q_5=277 m^3/s, because it presents more transport (Qs=3,210 m^3/s), with a rate of 34.668 m^3.

This is perhaps a more representative value of the annual sediment transport rate from Las Angustias gully to La Viña (A=49 km^2) intake area, leading to a 867 m^3/km^2 index which is 14% higher than this registered by Ven Te Chow (1966): 762 m^3/km^2 for river basins between 26 and 260 km^2. Therefore, we can conclude that from the cubic metre million (25.000 m^3/km^2/year) considered as the annual total erosion rate, only a part is transported by the gully, depending on the flood flow which occurs. As the sediment transport capacity of the gully is lower than the river basin erosion rate, then the river basin is under a sedimentation or increase process where the future intake of La Viña is to be placed.

PR (AÑOS)	Q (m3/s)	Qs (m3/s)	F N°EVENTOS	FxQs (m3/s)
1,07	50	0,058	7	0,406
1,4	121	0,218	12	2,616
2	172	0,377	7	2,639
5	277	0,642	5	3,210
7	300	0,750	4	3,000
10	350	0,906	2	1,812
25	447	1,245	1	1,245

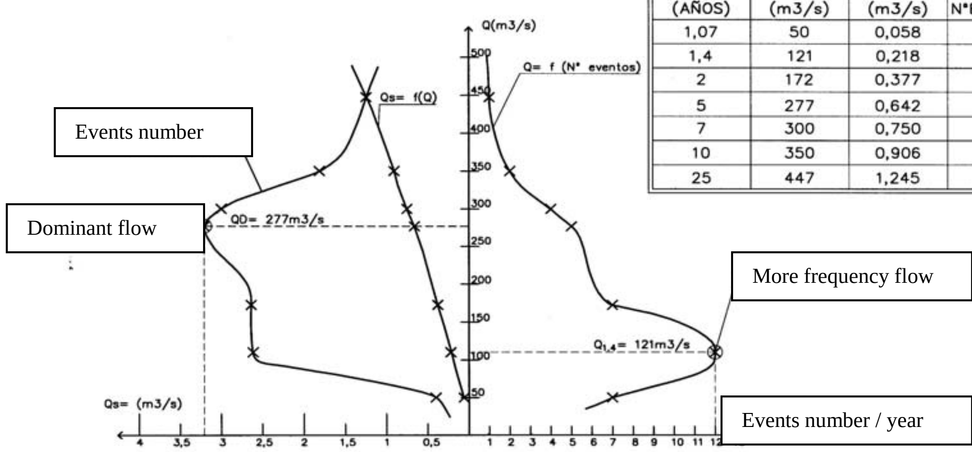

Figure 2 Liquid flow - Nº of events and liquid flow - solid flow

Both the river bed transport and the suspension-bottom load have been obtained by Einstein–Barbarossa method. Figure 3 shows that, for the lowest flows, the river bed transport is really higher than suspension-bottom load (84% al 6% for a flow $Q_{1,4}$=121 m^3/s, proportion which increases until 61% and 39% for the flow $Q_{1.000}$=836 m^3/s); proportions according to macrorough flows (García, 2000).

Table 7 shows that sediment concentration in weight in Las Angustias gully varies from 0.48% for the interanual flow ($Q_{1,4}$=121 m^3/s), 0.61% for the dominant flow (Q_5=277 m^3/s), to 0.93% for the millenary flow (Q_{1000}=836 m^3/s).

4. CANALISATION

As aforementioned, the hyperconcentrated flow of Las Angustias does not allow to have a conventional dam-reservoir system, since it would be silted in around 6 years as the useful volume in La Viña is 610.575 m^3. Therefore, the proposed solution is to derive the flows (Q=13 m^3/s, making use of 3 m^3/s for a continuous flushing of the system) by means of intake works of Tyrolese type, depositing the divert flow through a Dufour-Bieri sand trap for their later incorporation to the storage pond; the rest of the flows are allowed to pass through a channel. The beginning of the channel is placed on 237 level, that is, the top of the dam as the control section of the channel. This control dam or step is 18 metres high and 2 metres wide at the top. The upstream face is buried in such a way that the top is leveled to the channel and the water runs along downstream side toward the channel. The upstream slope is 1(V):0,3(H) and downstream slope is 1(V):1,3(H). The river bed width at this point is 57.50 m. Next to the step, at 222 level, the canalisation begins, where it is provided with a constant slope of 3,88%.

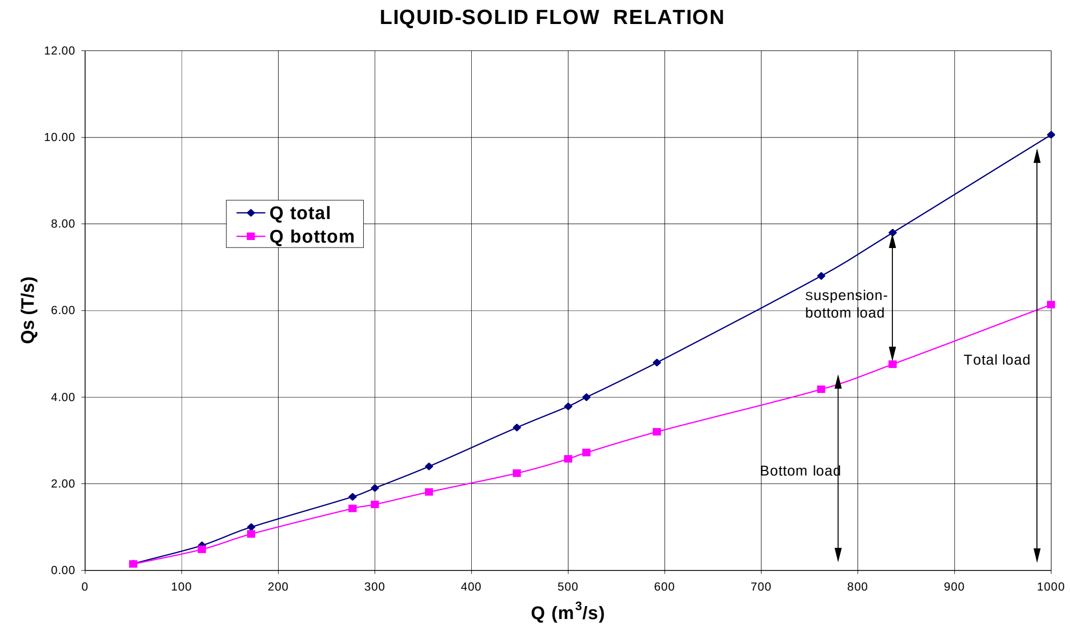

Figure 3 River bed transport and suspension-bottom load

The first stretch is straight, 59.25 m long and 57.50 m wide. The stretch leads to a transition where the channel is narrowed to 16.00 m wide, which is the width value of the channel along most of its development. This change in width in the section is of 107.36 m and starts at 219.70 to finish at 215.52.

Table 7 Main results from the calculation of sediment transport

RETURN PERÍOD	LÍQUID FLOW (m^3/s)	SOLID FLOW IN WEIGHT Qs (T/S)			CONCEN-TRATION IN WEIGHT (%)	ANNUAL TOTAL VOLUME	
		BOTT.	SUSPENS.	TOTAL		(m^3/s)	(T)
1,4	121	0,484	0,092	0,577	0,48	28.194	74.714
5	**277**	**1,428**	**0,272**	**1,700**	**0,61**	**34.668**	**91.870**
10	350	2,016	0,384	2,400	0,69	19.562	51.840
50	519	2,720	1,280	4,000	0,77	16.302	43.200
500	762	4,148	2,652	6,800	0,89	27.713	73.440
1.000	836	4,758	3,042	7,800	0,93	31.789	84.240

From this point on, the channel maintains a constant section, 16 m bed wide and trapeze form. The left slope is adapted to the pond occurring on the porous concrete wall, whose height is always higher than the channel water depth. The right slope is more vertical, it fits the gully topography and maintains an average value of 1(V):0,7(H). This stretch is 527.73 m long, following always the pond's shape until 195 level where it rejoins the river.

The flow profiles have been determined by the known model of the gradually varied and one-dimensional regime of Engineers Department of the United States Army HEC-RAS (1998). The main hydraulic variables (velocity, sheer stress and depth) and the type of flow are fundamental to define the type of protection and height of the channel; at the same time, the determination of these variables depends on the estimation of the resistance coefficient.

As the flow to turn up is hyperconcentrated and with high velocity and sheer stress, the channel has a 1m-thick concrete revetment (HA-25), and its external part has a concrete mask of high resistance of silicon smoke (HAR-55), 0.30 m-thick on the sides and 0.50 m-thick on the bottom.

The estimation of the hyperconcentrated flow resistance coefficient on rigid bed has been realized by the formulae proposed in Nalluri (1992):

$$\lambda_s = 0{,}851\lambda_0^{0{,}86} C_v^{0{,}04} D_{gr}^{0{,}03} \tag{6}$$

Where: λ_s is Darcy-Weisbach's resistance factor on rigid bed with sediment transport; λ_0 Darcy-Weisbach's resistance factor on rigid bed with clean water; C_v volumetric sediment concentration; D_{gr} grain size non-dimensional factor. For the application of the formulae it is required to estimate first λ_0 and C_v. Manning resistance coefficient for the protection of the channel with concrete would correspond to an average value n=0,014, so the resistance factor with clean water would be: $\lambda_0 = \frac{n^2.8.g}{R^{1/3}} = 0{,}0118$.

The energy supplied by the solid phase, for a volume unit and a distance unit downstream and in non-dimensional way according to Wang and Wang (1994), for the value of C_v=0,0036 (C_p=0,0093 in weight) and the slope corresponding to the stretch under study(S=0,388) is: $\frac{E_d}{\gamma} = C_v S = 0{,}00014 < 0{,}004$; a value really lower than the limit between the hyperconcentrated flow and the mudflow.

The coefficient of cinematic viscosity of water with sediments concentration has been estimated from the formulae in Graf (1984):

$$\frac{\nu_s}{\nu} = 1 + K_e C_v + K_2 C_v^2 \tag{7}$$

Where ν is the coefficient of cinematic viscosity of clean water (for T=20 ºC $\nu \cong 1.27\text{x}10^{-6}$ m^2/s); K_e Einstein viscosity constant ($\cong$ 2,5); K_2 particles interaction coefficient ($\cong$ 2). Therefore $\nu_s = 1{,}282.10^{-6}$ m^2/s, a value that does not change for the level of sediment concentration. The non-dimensional size of the grain corresponding to D_{50}=0,028 m is:

$$D_{gr} = \left[\frac{(s-1).g}{\nu_s^2}\right]^{1/3} .D_{50} = 600{,}183 \text{. Finally } \lambda_s = 0{,}0228 \text{ y } n_s = R^{1/6}\sqrt{\frac{\lambda_s}{8.g}} = 0{,}020 \text{.}$$

Due to the uncertainty factor existing in all the calculation parameters and from a conservative perspective, a value of Manning average resistance coefficient on rigid bed n=0,025 has been adopted for the flow of the channel design(Q=1,000 m^3/s). The value turns up to be really lower than the value corresponding to a mobile river bed (n=0.062).

Figure 4 shows the following profiles: water depth, critical depth and energy line for the design flow of the channel(Q=1.000 m^3/s). In the first stretch a backwater of strong type S2 appears from the critical conditions to reaching the supercritical depth y_1=0.95 m. Then the depth rises by a strong curve S3 until reaching the conjugated supercritical depth y_2*=2.31 m corresponding to the subcritical depth y_2=6.60 m. In this moment a hydraulic jump around 20 m long takes place. Next, the depth decreases again by a strong curve S2 until reaching the supercritical depth y=3,49 m at the channel exit. The velocities vary between 4,37 m/s (stretch in slow regimen, around 36 m) and 18 m/s in the supercritical depth at the bottom of the steep.

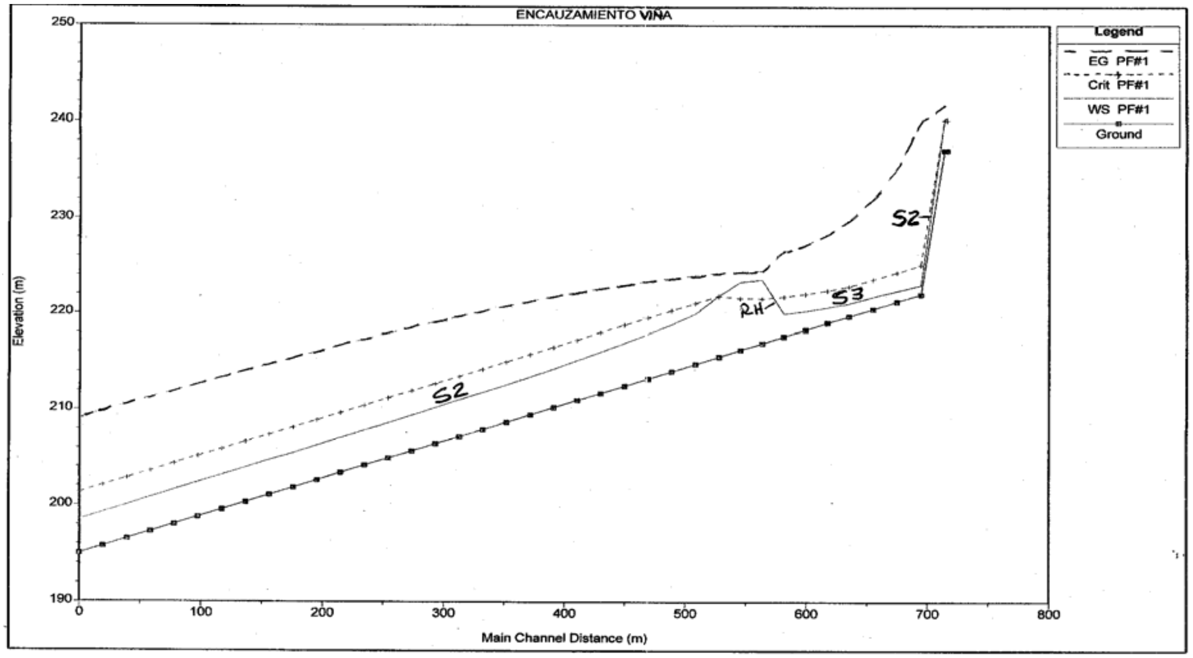

Figure 4 Flow profiles in de La Viña channel for Q=1,000 m^3/s and n=0.025

5. CONCLUSIONS

- This paper puts strong emphasis on the importance of the sampling in the calculation of the sediment transport. So, the characteristic diameters showing the sediment size distribution curve of a certain stretch of a particular flow, will overvalue or undervalue the estimation of its transport capacity according to the real representativity of the sample, as this transport capacity is inversely proportional to the sample characteristic diameter raised to three halves. Likewise, the estimation of the resistance coefficient in macrorough flows has been discussed, since the undervaluation of this coefficient will lead to the overvaluation of the calculation of the sediment transport.
- The validity range of the different formulae is also discussed together with their limitations for the application to flows with big characteristic diameters, as so does in Las Angustias gully.
- The calculation of the dominant flow is an important factor for the estimation of the river bed transport and therefore, of the channel forming.
- For similar flow conditions, the resistance coefficient on rigid river bed is around 2.5 times the resistance coefficient on mobile bed. This difference has profound economic effects on the channel design since the minimum height for the porous concrete wall and high resistance concrete is established in function of the channel water depth.
- Finally, we recommend to carry out samples of the bed transport, and to gage liquid and solid flow, with the aim to contrast the results obtained from this stage of the study.

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