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Channel incision, gravel mining and bedload transport in the Rhône river upstream of Lyon, France (“canal de Miribel”)

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Abstract

The Miribel canal is a former arm of the Rhône embanked between 1848 and 1857 over a length of 18 km to improve navigation at low discharges. The impact of this was a hydraulic tilting of the long profile characterised by 4 m of degradation upstream and 4–6 m of aggradation of bedload downstream. This phenomenon increased downstream flooding. Since 1937 a diversion dam has controlled upstream water input, thus reducing the transit of the pebble bedload. However, excessive harvesting of sands and gravels occurred between 1970 and 1980, resulting in a general lowering of the river bed and the accompanying water table with ecological consequences in the alluvial plain and for water supply. This development made it all the more necessary to obtain knowledge about the bedload discharges passing through the Miribel canal, and more broadly about the hydraulic conditions as functions of the varying discharge. Calculation of shear stresses and grain size measurements on the lateral bars after several floods in 1989–90 show that movement of bed-material is initiated at a discharge of 440 $m^3 \cdot s^{-1}$ (equalled or exceeded 40 days $\cdot$ $year^{-1}$), and becomes general at 550 $m^3 \cdot s^{-1}$ (equalled or exceeded 30 days $\cdot$ $year^{-1}$). Transport discharge is thus relatively frequent and involves distal fluvio-glacial deposits composed of fine-grained materials. Potential transport calculated by the Meyer-Peter formula is around 60,000 t $\cdot$ $year^{-1}$ for the range of discharges between 440 and 850 $m^3 \cdot s^{-1}$. For these discharge values, the canal experiences a loss of materials without replacement from upstream; for higher rates of discharge, the floodgates let through an unknown quantity of materials which partially make up the loss. Gravel harvesting ceased in 1991 but the diversion dam will have to be operated in a different way in order to increase the input of bedload into the canal.

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1. Introduction

Bedload mobility and important solids transport are characteristic features of rivers rising in the Alps, particularly in the case of the Rhône (Bravard et al., 1992). Since the end of the 19th century, long term impacts resulting in a shortage of sediment supply from river heads include mountain afforestation, gravel harvesting and embankment construction. Moreover, construction of reservoirs and stream diversion by hydropower schemes are responsible for the entrapment of bedload and the disruption of gravel transport in the by-passed reaches of the Rhône (Klingeman et al., 1994; Bravard, 1994). Long term management of alluvial plains involves preservation of flooding and protection of wetlands and the supply of drinkable water, through the stabilisation of river bed and groundwater levels.

The development of the Miribel canal in the mid-19th century above Lyon is a classic French example of disruption of liquid and solid flows since it caused a large-scale tilting of the long profile. This modification increased both upstream watertable lowering and downstream flooding. Rectification of its effect, which engineers a century later regarded as complete (Winghart and Chabert, 1965), has been called into question following ill-considered extraction of gravel between 1970 and 1980. Two studies (Bravard et al., 1991; Poinsart, 1992) have shown the seriousness of the current development and have encouraged a change of extraction policy. They also provide basic data to update knowledge concerning bedload transport in the Rhône, particularly because direct access to river bedforms and bed material grain size is possible in the Miribel canal.

2. The miribel canal

2.1. Historical background to the development

During the 19th century, the Rhône was developing in a braided pattern (Fig. 1). The Miribel channel had occupied the course of the former main arm along the Dombes by 1726 (Belmont, 1989). This arm was embanked between 1848 and 1857. As early as the 1870s, the effects of embankment were apparent in the hydraulic tilting of the long profile, observed at the time of the 1874 and 1875 floods. Concentration of water and straightening of bends gave rise to upstream degradation. Downstream, aggradation was linked to the slowing of flow as it passed through Lyon where the bridge piers break the current, and also to the high bedload. The hydraulic tilting has subsequently been self-maintaining since the concentration of flow has accentuated upstream, whereas the former arms of the Rhône, notably those closest to the canal, have gradually filled with fine sediment deposits, and have been invaded by vegetation. In total, between 1847 and 1952, mean incision reached 3.5 m with maxima of 4.5 m, and mean aggradation exceeded 4 m, sometimes reaching 6 to 6.5 m.

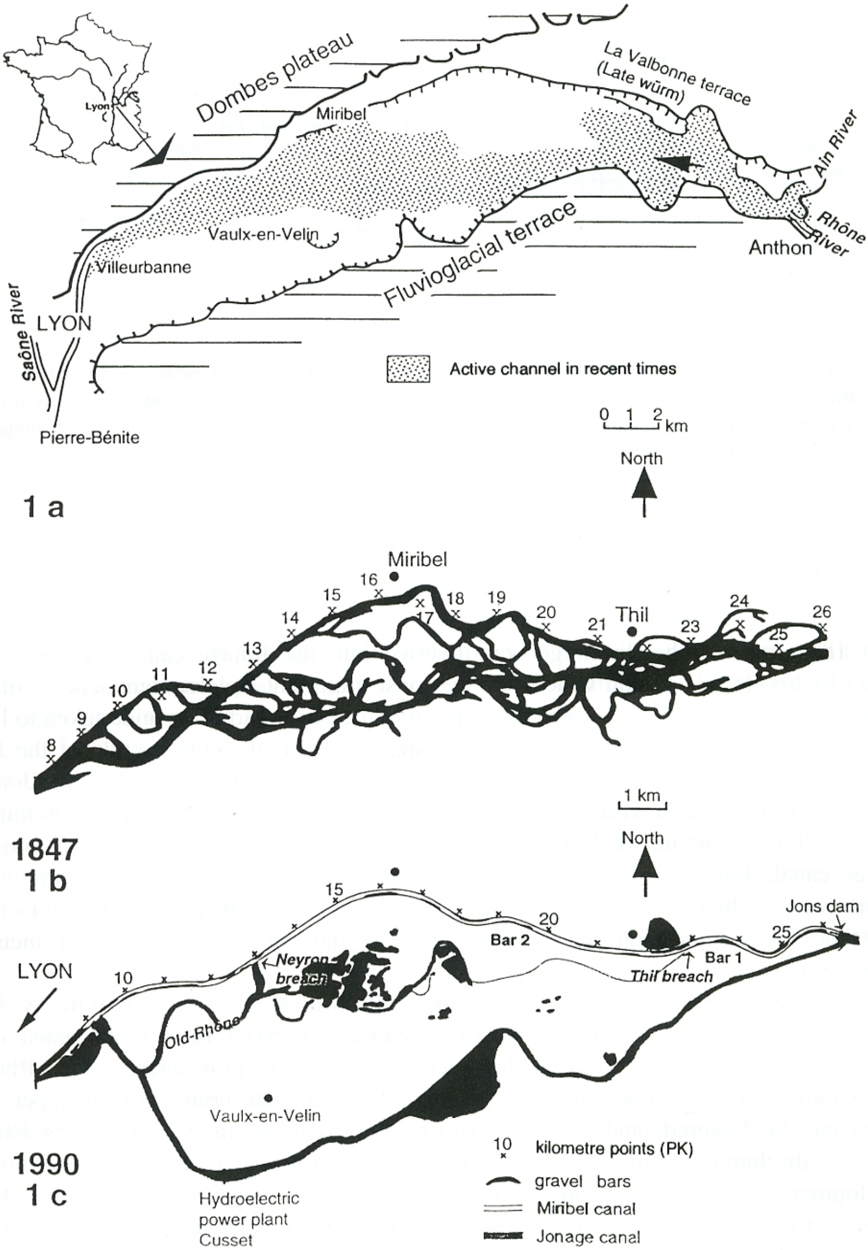

Fig. 1. Framework of the study and evolution of the fluvial pattern from the 19th century. (a) Geomorphological situation with Dombes plateau, fluvioglacial terrace, late Würm terrace and active channel in recent times. (b) The Rhône in 1847, before embankment. (c) The Rhône in 1990: the Miribel channel and the Jonage channel; localisation of the breaches and the gravel bars.

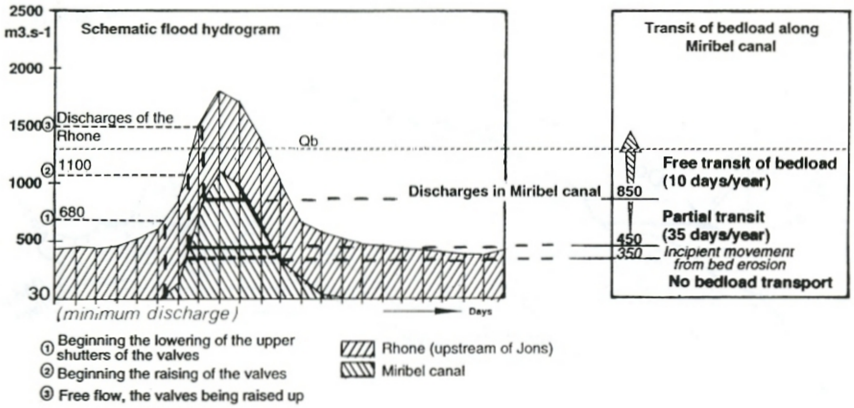

Fig. 2. JONS dam operation and possibility of bedload transit along Miribel canal. Schematic flood hydrograph of the Rhône upstream of Jons dam and in Miribel canal. The number of days per year where a discharge is equalled or exceeded, is calculated from the classed discharges curve established by the C.N.R. (Compagnie Nationale du Rhône) in the period 1920–1988. Qb is the bankfull discharge (1300 $m^3 \cdot s^{-1}$), with a return period equal to 1.4 year (calculated by the C.N.R., from Gumbel adjustment in the period 1900–1987).

2.2. Attempted corrective measures

In 1899, part of the discharge was diverted into the Jonage canal leading to the Cusset hydro-electric station which was the most important development scheme of the by-pass type in Europe (Fig. 1c). Early corrective measures had to be undertaken to limit the concentration of flow into the incising canal. In 1937, the construction of the Jons dam allowed a fairer distribution of flow between the two canals. Since then, the Jonage canal can draw off a maximum of 650 $m^3 \cdot s^{-1}$ and the Miribel canal has a minimum reserved flow of 30 $m^3 \cdot s^{-1}$. It is probable that, by diverting 650 $m^3 \cdot s^{-1}$ into the Jonage canal, Jons dam reduced the flood peak and thereby reduced the duration of discharges at which bedload entrainment occurs in the Miribel canal. Conversely it might be conjectured that, by blocking part of the bedload, it increased incision downstream.

At present the discharge of the Rhône above the two canals exceeds 680 $m^3 \cdot s^{-1}$ on average for 120 days per annum, so the minimum reserved flow is exceeded over approximately one third of the year. For successive increases in discharge in the Rhône, the operation of five flood-gates occurs (Fig. 2). Each gate comprises an upper flap which can be lowered, and a bottom sluice which can be raised. When the Rhône reaches a discharge rate of 950 $m^3 \cdot s^{-1}$, two courses of action are possible according to development company instructions. (i) If the discharge is expected not to exceed 1100 $m^3 \cdot s^{-1}$, full lowering of the flaps is sufficient to ensure clearance of surplus flow (500 $m^3 \cdot s^{-1}$ in Miribel canal $+650$ $m^3 \cdot s^{-1}$ in Jonage canal). Then discharge entering the Miribel canal carries only material in suspension. (ii) If the discharge in the Rhône is expected to go beyond 1100 $m^3 \cdot s^{-1}$, the central sluice-gate is raised, which allows transit of part the bedload. Above 1500 $m^3 \cdot s^{-1}$ all the bottom sluices are raised so that flow is not obstructed, with a discharge exceeding 850 $m^3 \cdot s^{-1}$ in the Miribel canal. At this discharge, bedload transit from upstream is not affected by the operation of the dam.

2.3. Alternative management of the sedimentary supply during the second half of the 20th century

Planning and development since the 19th century have sought to achieve stabilisation of form and concentration of flow in a single channel. In the Miribel canal the stability of the long profile was supposed to be effective in the early 1960s. However in the Rhône catchment, other factors added to the impact of channel embankment on the reduction of sediment transport: afforestation in the Alps, depopulation and land dereliction, changes from cereal cultivation to pasture land, all of which have increased stability of slopes and reduced sediment supply (Bravard, 1991; Bravard and Peiry, 1993). Indeed, most of the bedload discharge of the Rhône in this reach originates in the Ain watershed, which has been dammed since 1932 and whose wandering has been reduced mainly by afforestation of the active tract. Management since 1950 seems to have completely ignored the possibility of complex delayed impacts, particularly changes in the flow and especially in the bedload discharge.

Since the 1950s, the development of motorway infrastructure and urban building has given a commercial value to the bed material of rivers. However, construction of the hydro-electric power station at Pierre-Bénite (1965; Fig. 1a) raised concern about accumulation of sands and gravels, particularly in the reservoir. Basically their extraction appeared to provide both a ready supply of raw materials and indirect protection against floods. Industrial gravel extraction began in 1957 immediately upstream of Lyon [1.5 millions of tons (Mt) between 1974 and 1991] and extended into Lyon (2.5 Mt between 1974 and 1979). From 1979, extraction began in the bed of the Miribel canal itself (Fig. 3). Alarming erosion of river beds and banks began after the decennial flood of February 1990. A legal inquiry (requested by the Service de la Navigation du Rhône et de la Saône) culminated in the cessation of extraction at the end of 1991 (Bravard et al., 1991). Recent extraction caused a rapid and severe deepening of the canal. Comparison of longitudinal profiles for 1979 and 1985 shows erosion between the kilometric points (PK) PK 23 and PK 12 (Fig. 3). The mean gradient of the Canal has thus increased from 0.48×10^{-3} $m \cdot m^{-1}$ in 1952 to 0.58×10^{-3} $m \cdot m^{-1}$ in 1985. Comparison of series of cross-sectional profiles from 1979 to 1985 enabled quantities lost between PK 26 and 13.2 to be assessed at 244 157 m^3, which implies an average removal of 24 cm.

Longitudinal profiles from 1985 and 1990 show incision of the channel which varies considerably from site to site (Fig. 3). Upstream the profile is stable in those reaches not subjected to extraction, and where the bed is an outcrop of fluvio-glacial deposits (early Würm) (Caclin and Poinsart, 1987). Where the canal is incised, extraction concessions exist. For the 1990 profile, the effects of the 10-year flood of 1990 have been identified. Taking into account the size of the extraction pools, notably between PKS 21.5 and 24.5, one can suppose that the bedload input from the flood was modest, or that extraction after this flood was very substantial (declared extraction for 1990 was about 70 000 m^3 between PKS 21.5 to 23.15 and 24 to 25.2). In the part of the canal which

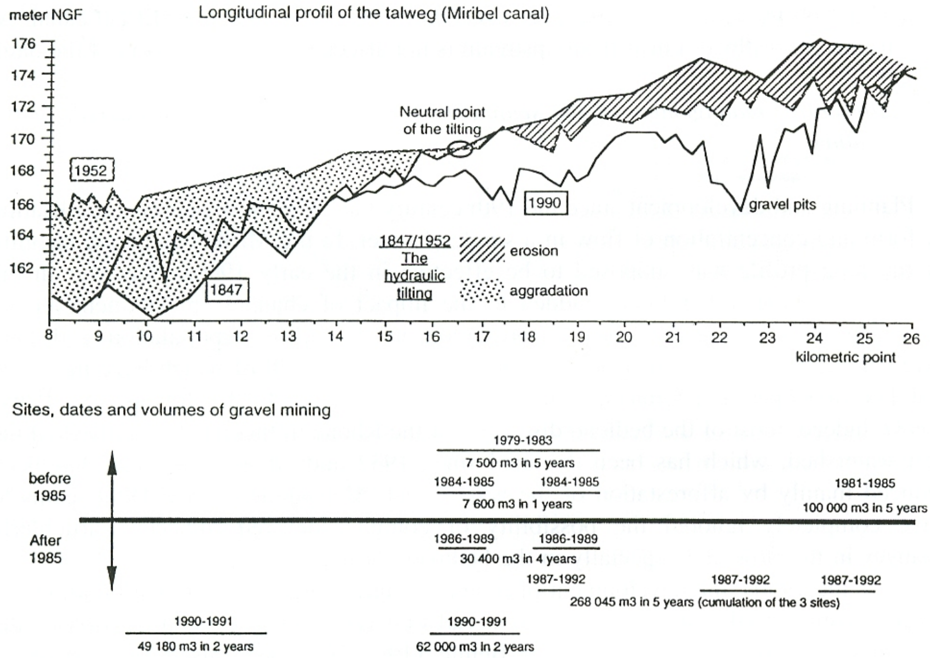

Fig. 3. Change of the long profile of the thalweg (Miribel Canal); historical tilting (1847/1952). Since 1980, general erosion of the thalweg has occurred because of gravel mining. Profiles of 1847 and 1952 redraw from Winghart and Chabert (1965). The surveys of 1965 and 1990 have been performed every 200 m by the C.N.R (Compagnie Nationale du Rhône). N.G.F. (Nivellement Général de France) elevation in meters above sea level.

was formerly raised (PK 13 to 8), the development of the bed between 1985 and 1990 shows an average incision of 80 cm which can be, almost exclusively, attributed to the extraction of gravel (Fig. 3).

However, after extraction stopped, successive floods will readjust the profile by filling the deep extraction pools and lowering the riffles in between. It is possible that degradation of the bed is slowed locally by progressive paving and the exhumation of fluvio-glacial deposits (Poinsart et al., 1989). Indeed, since the ten-year flood of 1990, sites of fluvio-glacial outcrops have become more numerous. Instability in the Miribel canal is also apparent in its attempt to create an entrenched meander in the embanked bed, and the displacement of riffles observed by comparison of profiles and marking of sediments (Poinsart, 1992). New ''breaches'' have begun to appear on the left bank, the embanking rocks are destabilised, the banked-up sandy sides are heavily eroded and trees are uprooted. Thus the encouragement of extraction has merely served to exacerbate flow concentration. This development made it all the more necessary to obtain knowledge about the solid discharges passing through the Miribel canal, and more broadly about the hydraulic conditions as functions of the varying discharge.

3. Hydraulic parameters and bedload transport in the Miribel canal

3.1. Methodology — general equations

All formulae which enable bedload discharges to be calculated rely on hydraulic parameters such as shear stress. As a first step, its was therefore necessary to evaluate these parameters and to observe their variation with discharge. Experiments with marked pebbles have not only enabled determination of the flow required to initiate movement of the bed material, but also testing of the adequacy of different values of critical shear stress. With this in mind, shear stress was calculated using two classical approaches.

First, the total shear stress (τ) was determined from the product of the energy slope (S) and the hydraulic radius (R):

$$\tau = \rho g S R \tag{1}$$

with ρ the volume fluid mass and g the acceleration due to gravity.

In view of the large width-depth ratio of the Miribel canal one can consider $R = d$ (d = depth). Furthermore, in view of the absence of significant longitudinal variations in the cross sections, the energy line gradient can be considered equal to the water level slope.

In accordance with work of Einstein and Barbarossa (1952), total shear stress calculated using Eq. 1, must however be split into two components:

(i) Grain shear stress (τ') which alone is responsible for transport of the bedload and which alone is necessary in the Meyer-Peter formula; and

(ii) Bed-form shear stress (τ'') due to the resistance offered by irregularities in the channel and the banks (i.e. bed-forms). Thus the equation can be written

$$\tau = \tau' + \tau''$$

In rectilinear channels with homogeneous beds and without variation in depth, the bed-form shear stress is negligible so that, as demonstrated in a flume (Petit, 1989), the grain shear stress represents the total shear stress ($\tau' = \tau$). However in a natural river, this is far from being the case and it is necessary to assess τ'. Different methods exist to ascertain the divergence between τ' and τ'', but the one which is among the most commonly used, and which produces satisfactory results (Richards, 1982; Petit, 1989, Petit, 1990), is based on the relationship between roughness due solely to resistance of particles (n_o), known by means of the Strickler formula (1923), and the total roughness (n_t) in Manning's formula:

$$\tau' = \tau (K)^{3/2} \tag{2}$$

with

$$K = \frac{n_o}{n_t}$$

where

$$n_o = 0.048\, D_{50}^{1/6} \tag{3}$$

and

$$n_t = \frac{R^{2/3} S^{1/2}}{V} \tag{3b}$$

with D_{50} the median diameter of the material forming the bed, and V the mean velocity.

Second, shear stresses were also calculated using friction velocities (u^*) as in

$$\tau = u^{*2} \rho \tag{4}$$

with

$$\frac{u}{u^*} = 2.5 \ln \frac{y}{y_o} \tag{4b}$$

where u is the velocity of the current measured at a depth y from the bottom, y_o is the roughness height of the D_{50} of the material constituting the bed. With regard to the latter, the relationship $y_o = 0.2 D_{50}$ is generally adopted for gravel bed rivers (Hey, 1979; Hammond et al., 1984; Petit, 1990).

Additionally, in experiments carried out using marked pebbles, observations of initiation of or absence of movement were related to shear stresses evaluated by these two methods so as to calculate the values of the critical shear stresses. The entrainment relation most often used is Shields' function (θ_c), a dimensionless relationship between the density of the sediment (ρ_s) and fluid (ρ_f), the diameter of particles (D), the kinematic viscosity (ν), the acceleration due to gravity (g) and the shear stress exerted on one grain, where

$$\theta_c = \frac{\tau}{(\rho_s - \rho_f) g \cdot D} Fct(u^* D/\nu = Re^*) \tag{5}$$

According to Shields' diagram, $\theta_c = 0.060$ when the particle Reynolds number (Re^*) is greater than 10^2, which is generally the case in natural rivers. However, as will be subsequently seen, this value of $\theta_c = 0.06$ has been questioned by various authors.

Bed load transport was determined by means of the Meyer–Peter equation because it is recognised as giving good results: studies noted by Larras (1977) have shown that the Meyer-Peter formula fits observations well over the central third of the Rhône, as well as in the French part of the Rhine. This concurs with the conclusions of Graf (1971) for the Danube. These authors recommend this particular formula from among the different methods of determining bedload, while at the same time acknowledging that it can lead to appreciable errors. These errors consist mostly of overestimates of the bedload, as Frecaut (1972) observed for the Moselle, because the formula gives the "potential" bedload discharge, implying that movable material must be available in sufficient quantity, and on a permanent basis, to guarantee such transport. Another cause of over-estimation by the Meyer-Peter formula results, as shown by Parker et al. (1982), from the occurrence of paving of the bed which tends to protect the material and delay movement, requiring a higher critical shear stress. The importance of such protection has been clearly confirmed by Reid and Frostick (1984, Reid and Frostick (1986), but always in dealing with relatively heterogenous material ($D_{90}/D_{50} > 3$). The Meyer–Peter formula occasionally underestimates bedload transport, which has justified adaptation of some of its parameters (Smart, 1984), although mainly in the case of steeply sloping rivers.

The Meyer–Peter equation in its initial formulation is

$$\left(g\rho_f(K)^{3/2}Sd\right) - 0.047g(\rho_s - \rho_f)D_{50} = 0.25\rho^{1/3}(g_b)^{2/3} \qquad (6)$$

where the first element on the left side in Eq. 6 actually represents the shear stress multiplied by the factor K which enables the grain shear stress to be defined (cf. Eqs. 1 and 2 above), the second element represents the critical shear stress (Eq. 5) with $\theta_c = 0.047$ and the right hand side of the equation represents the transport as submerged mass per unit of time and width (g_b), multiplied by $\rho^{1/3}$ in such a way as to counterbalance the terms on the left and to make the equation dimensionless (Richards, 1982). Thus, after transformation, Eq. 6 can be expressed as follows.

$$g_b = 0.253(\tau' - \tau_{cr})^{3/2} \qquad (6b)$$

In order to achieve accurate estimation of solid transport it is necessary therefore to estimate the values of τ' on the one hand, and to check the equation $\theta_c = 0.047$ on the other. This was done by the following method.

3.2. Measured-pebble method

Two bars were chosen, those least subjected to human influences. On each bar, four transects were chosen in different geomorphological units: at the downstream end of the riffle, upstream and downstream of the point of maximum convexity, and at the end of the bar (Fig. 4).

Using a one metre square frame, three to five areas were demarcated on each transect. The pebbles located in the inner surface of the frame were painted, then photographed, using a tripod, from a height of 1.70 m (Ibbeken and Schleyer, 1986). A 28 mm lens was used. In the photographs the *b* axis of the particles corresponds to the shortest visible axis (Kellerhals and Bray, 1971; Adams, 1979). From each one-square-metre patch, a sample of 100 particles was measured. The choice of 100 individual elements was dictated by the division into a hundred compartments by strings attached at 10 cm intervals along the side of the frame (Kellerhals and Bray, 1971).

The marking and photography of the quadrats was carried out during periods of flow at the reserved discharge (30 $m^3 \cdot s^{-1}$). After each flood, each quadrat was located and again photographed to obtain a post-flood record. In order to assess the effect of the next flood, the surface was repainted. The two bars studied had 16 and 13 quadrats respectively, in other words 29 samples comprising 100 elements each for each phase of the investigation. Four phases were involved: (i) the initial state, (ii) after a flood of 440 $m^3 \cdot s^{-1}$, (iii) after a flood of 580 $m^3 \cdot s^{-1}$, (iv) after the ten year flood (2200 $m^3 \cdot s^{-1}$).

Comparison of photographs of the same surface before and after a flood enables the sample to be subdivided into two groups the size of which can vary between 0 and 100, and which are complementary. The first group is the one in which the material has remained stable and is the no-motion D_i sample (it occurred once only, for the 440 $m^3 \cdot s^{-1}$ discharge). The second is that which is removed, which is the motion D_i

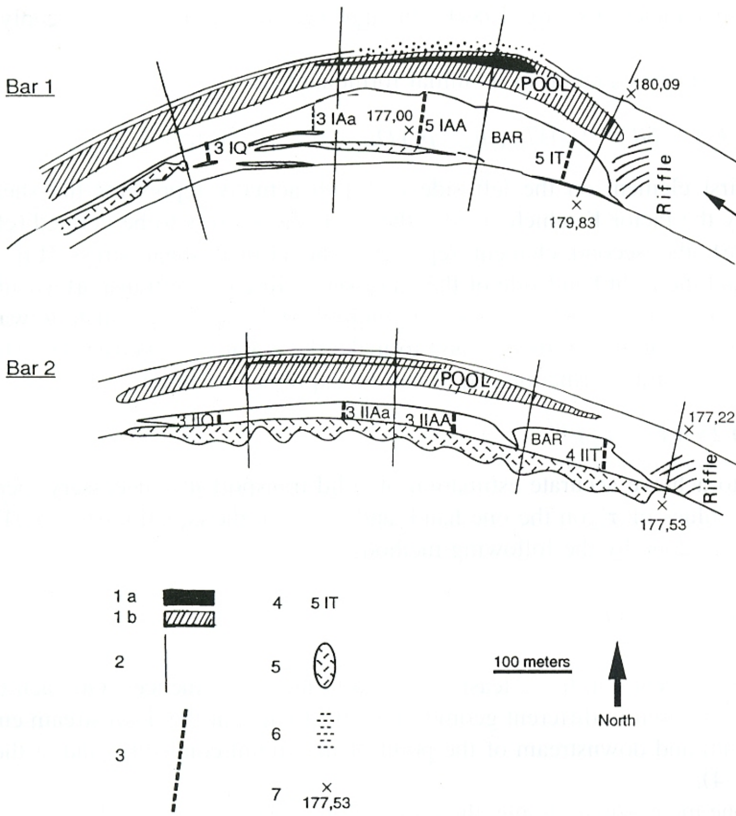

Fig. 4. Location of the quadrats. (1) Water depth for 30 $m^3 \cdot s^{-1}$ in 1985 (1a: 2.5 m depth; 1b: 1 m depth); (2) reference cross-section; (3) transects with 3 to 5 plots (1 m^2); (4) five squares plots along IT transect; (5) tree vegetation on bank levee inside the embankments; (6) Concave bank erosion; (7) NGF benchmark (Nivellement général de la France) in m above sea level.

sample. The D_{50} is represented by the 100 elements which are sampled on the same surface after the flood and considered representative of the bed material.

3.3. Results

The relationship between total shear stress (Eq. 1) and discharge for different sites on different dates is recorded in Fig. 5. The correlation is very good ($r = 0.977$, $n = 32$) and variations from site to site are slight. Furthermore total shear stresses do not reach exceptionally high values even for the ten year flood (max. 60 $N \cdot m^{-2}$).

With regard to the evaluation of τ', it is necessary at the outset to note some aspects of the Manning coefficient of roughness, which features in the calculation of τ'. Mean values of Manning's roughness are relatively low ($n_t = 0.032$), which is justified by the fact that this reach of the Rhône was straightened and embanked and that there are few obstacles and little vegetation to impede the flow. Variations of n_t are not only small

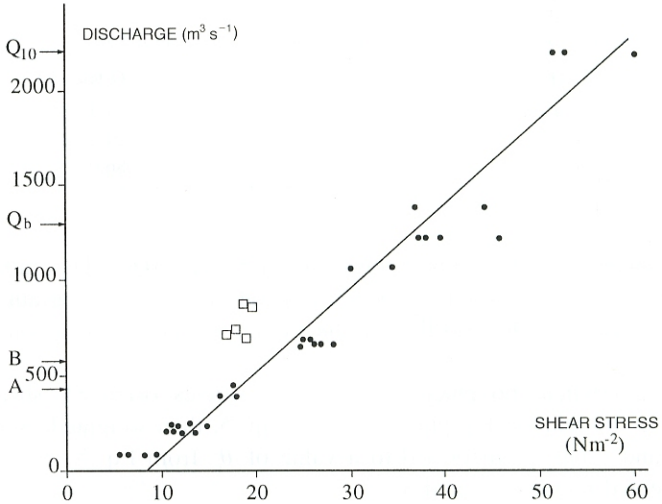

Fig. 5. Shear stress (in $N \cdot m^{-2}$) as a function of discharge; the black points are shear stresses evaluated from the depth/slope product (Eq. 1), the white squares the shear stresses evaluated from friction velocities (Eq. 4). On the y-axis, A is the discharge at which initiation of bed load movement occurs, B the discharge at generalized bedload movement, Qb the bankfull discharge and Q10 the decennial flood.

from site to site, but also in relation to discharge (extremes between 0.024 and 0.045). However there emerges a very slight tendency to an increase in roughness with discharge, which appears to contradict the generally accepted pattern. However, this is only a weak tendency, the correlation being far from significant ($r = 0.59$). Moreover, n_t is slightly greater in pools than on riffles (Table 1), which does correspond with a generally accepted variation.

The ratio τ'/τ established on the basis of Eqs. 2 and 3 is on average 0.71. It varies closely with the Manning coefficient, lacking systematic variation from site to site and appreciable variation with discharge. The ratio τ'/τ is relatively high (compared with small rivers), implying that bed-form shear stresses are relatively minor.

Shear stresses were also evaluated from friction velocities (Eq. 4). These were calculated from velocity profiles measured by the C.N.R. for five discharges ranging between 700 and 800 $m^3 \cdot s^{-1}$. For each discharge, the estimation of point shear stresses was successfully carried out at nearly 20 points on the section. Lateral differentiations in shear stress are insignificant ($\tau^*_{max}/\tau^* < 1.4$). Comparison of τ^* (obtained by means of

Table 1
Variation in Manning's coefficient of roughness with discharge and site

Discharge	Riffle	Pool
< 220 $m^3 \cdot s^{-1}$	0.030	0.032
390–670 $m^3 \cdot s^{-1}$	0.028	0.035
> 1000 $m^3 \cdot s^{-1}$	0.030	0.037

Table 2
Table of bed material movement for different floods (mean per transect, in mm)

	Q 440	Q 580	Q ten year
D_{50}	21.5	23.2	24.4
D_{90}	44.5	49.0	54.3
D_{99}	69.0	74.1	86.0

u^*) and total shear stress gives on average a ratio $\tau^*/\tau \approx 0.67$. This ratio is relatively normal since τ^* is recognised as being a good descriptor of grain shear stress. Furthermore this ratio is close to that produced above when comparison of τ'/τ was made.

For discharges when movement of marked particles occurred, shear stress was calculated using the relationship put forward in Fig. 5, then weighted by the ratio τ'/τ noted above, and finally transformed to a value of θ_c from Eq. 5. A synthesis of the results is presented in Tables 2 and 3.

There is partial movement of material for a discharge of 440 $m^3 \cdot s^{-1}$ (only ten percent of the pebbles remains stable), a discharge which represents barely 30% of bankfull discharge ($0.3Q_b$) and which is reached or exceeded on average for 40 days $\cdot$ year^{-1}. The material moved on both bars studied regardless of transect, had D_{50} of 21.5 mm. In the case of the 580 $m^3 \cdot s^{-1}$ flood, a discharge occurring on average 30 days $\cdot$ year^{-1}, all the material moved ($D_{50} = 23.2$ mm). Finally on the occasion of the ten year flood, all the material moved but the shear stresses do not achieve values sufficient to set in motion fluvio-glacial material when this outcropped on the channel bottom ($D_{50} = 110$ mm, $D_{90} = 160$ mm, $D_{99} = 230$ mm). Translated into Shields' dimensionless criterion, and with all transects and floods lumped together, $\theta_c = 0.064$ (average of 83 values) is obtained. If differentiation by flood is carried out, the full set of 28 quadrats gives $\theta_c = 0.045$ in the case of the flood of 440 $m^3 \cdot s^{-1}$. The flood of 580 $m^3 \cdot s^{-1}$, produces $\theta_c = 0.052$, which compares well with the values used in the Meyer–Peter equation.

Closer analysis involving differentiation of sites reveals the following points (Table 3):

— Those quadrat-sites located upstream systematically provide lower θ_C values. This shows that shear stresses at such sites are probably slightly higher than average and that uniformity in the entrainment function cannot be assumed between micro-sites.

— An increase in θ_C occurs with discharge, whatever the site, with very high values

Table 3
Values of Shields' dimensionless criterion θ_c for movement of marked pebbles at different flood levels

	Q 440	Q 580	Q ten year
Upstream site (bar 1)	0.033	0.038	0.057
Upstream site (bar 2)	0.040	0.054	0.081
Downstream site (bar 1)	0.050	0.045	0.090
Downstream site (bar 2)	0.055	0.071	0.159

occurring in the ten year flood. However, this increase has only limited significance since, in the ten year flood, it is likely that some particles were moved by lower values of shear stress than those occurring at the peak of the flood. With regard to this if only the five quadrat-sitings comprising the coarsest material are taken into consideration, $\theta_c = 0.043$ is obtained.

Since θ_c seems to vary inversely with the size of the material, an analysis of the relationship between θ_c and the ratio (D_i/D_{50}) was undertaken, where D_i is the diameter of the material considered, in relation to the diameter of the material constituting the bed (D_{50}). This relationship in fact integrates the effects of relative protrusion, and implies that shear stresses have to be proportionally higher in order to initiate movement of material of smaller size than the material forming the bed. Conversely shear stresses have to be proportionally lower in order to initiate movement of material of larger size that the material forming the bed. Initially proposed by Andrews (1983), this relationship is expressed as

$$\theta_c = 0.083(D_i/D_{50})^{-0.87}$$

Numerous confirmations have been forthcoming both in flumes (Petit, 1994) and in natural rivers (Komar, 1987; Ashworth and Ferguson, 1989; Assani and Petit, 1995) but with appreciably different values of the coefficient a and the exponent b. However, in the case of the Miribel canal, when detailed examination is made of each of the equations established for a given flood, all sites merged together, the correlation coefficients are far from satisfactory $(r < -0.5)$. If comparison is made between coefficients and exponents of the different equations, wide variability in values is observed. The same is true when analysis of sites is undertaken, with data for all rates of discharge combined.

Three reasons can be suggested to explain the failure of this type of relationship in the Miribel canal case.

(i) In the first place the material is very homogeneous $(D_{90}/D_{50} = 2.17)$, very well rounded [roundness index according to Cailleux and Tricart (1959) = 290], and has an open structure. Clusters, for example, occur relatively infrequently. However the effect of relative protrusion, which is the principal control of this D_i/D_{50} relationship, should persist.

(ii) The range of the D_i/D_{50} ratio which features in the equations is relatively narrow (0.47–1.61) which hinders estimation of the regression.

(iii) However, point shear stresses for each micro-site have not been used, and variations in shear stress may occur at the level of these micro-sites. The inadequacy at Miribel of the formula propounded by Andrews runs counter to observations made by Ramette and Heuzel (1962) in the Rhône, but downstream of its confluence with the Saône. The latter river, however, delivered to the Rhône a relatively coarse pebble load originating in the Azergue, the last tributary of the Saône before it joins the Rhône.

A synthesis of the experiments conducted with marked pebbles leads to the following conclusions.

(i) If one considers the grain shear stress of the discharge required for movement, and the mean diameter of the material moved, one obtains an overall $\theta'_c = 0.039$. This value is marginally higher $(\theta'_c = 0.041)$ for a discharge of 580 $m^3 \cdot s^{-1}$. These values of θ'_c

appear lower than those generally accepted ($\theta_c' \approx 0.045$ in Komar, 1987) although certain authors have found lower values, such as $\theta_c' = 0.030$ (Neill, 1968), $\theta_c' = 0.015$ (Clifford et al., 1992) or similar values (Ashmore, 1988). The Miribel material is presented in open structure and, as stressed by Richards and Clifford (1991) in their synthesis of the literature, bed particles laid out in open position can be much more easily moved than particles arranged in a structured setting as in imbrication or grouping in clusters. Moreover, in view of the high degree of homogeneity of the material, neither protuberance nor hiding are significant. On the other hand, it was grain shear stress which was used in our investigation, and not total shear stress, as is the case for most authors. On this point, if total shear stress had been used, a θ_c varying from 0.052 to 0.057 would have been obtained.

(ii) The discharge required to initiate bedload movement is relatively low ($\approx 0.3Q_b$) and therefore relatively frequent. This runs counter to what is generally accepted, the discharge for bedload transport occurring close to the bankfull discharge, the recurrence of which would be of the order of 1 to 1.5 years. Recent studies on bedload transport are relatively numerous, in terms of method of measurement (Bunte and Ergenzinger, 1989; Bunte, 1992), initial motion (Reid and Frostick, 1984; Richards, 1990) or the tendency of bedload to move in pulses/waves (Gomez et al., 1989; Hoey, 1992) and about entrainment criteria based on critical unit discharge (Bathurst et al., 1987; Ferguson, 1994). However, relatively little information is available on the occurrence of the discharge required for mobility, expressed in relation to bankfull discharge, and therefore with regard to frequency of bedload transport and to quantification of the bedload discharge, especially in large rivers. However, Thorne and Lewin (1979) have shown from experiments with marked pebbles that, in the River Severn, the bed material (D_{50} varying from 28 to 40 mm) was moved by discharges of barely one quarter that at bankfull stage, with 25 occurrences in 2 years. Similarly, in the case of the Rhône close to Lyon, the C.N.R. estimates that entrainment of small pebbles occurs on between 20 and 40 days per annum (in Tricart, 1965).

Bedload discharge was first evaluated at a discharge rate of 850 $m^3 \cdot s^{-1}$ (since the bottom sluice-gates of the Jons dam are not yet opened and there is no input from upstream). The value of τ' for this discharge is obtained by estimating first the total τ from the relationship advanced in Fig. 5, this value being then weighted by the ratio $\tau'/\tau = 0.70$ (see above). The value of τ_c is considered as the value of τ' for the discharge to initiate movement (440 $m^3 \cdot s^{-1}$). Thus, by applying the Meyer–Peter equation, a value of 4.560 $t \cdot day^{-1}$ is obtained. In the knowledge that this discharge occurs on average 10 days $\cdot$ year^{-1}, a volume of about 30000 $m^3 \cdot year^{-1}$ transported and eroded is obtained for the Miribel canal, this before any input from upstream consequent upon the opening of the bottom sluice-gates of the Jons dam. Floods of lower intensity transport less material, but their frequency is higher. For example, floods of 580 $m^3 \cdot s^{-1}$, occurring on the average 27 days $\cdot$ year^{-1} would, according to the Meyer–Peter equation, result in transport of nearly 1300 $t \cdot year^{-1}$ (about 800 $m^3 \cdot year^{-1}$).

Nevertheless, there remains a problem. The values estimated using theoretical equations are potential values which could not materialise unless material were available in sufficient quantity, which means that these equations generally overestimate the actual bedload discharge. In the case of the Miribel canal, a supply of material has remained at the river's disposal despite the lack of upstream input (resulting from the presence of the Jons dam), and despite the embankment of the river which limits input from lateral erosion. Indeed the movable pebble bed reaches a thickness of 2 to 5 m and it is only very recently, and very locally, that one finds it fluvio-glacial material which cannot be moved.

As far as the displacement velocity is concerned, observations made with marked pebbles show that they progress by more than 100 m during one flood. Furthermore, if the longitudinal modification of the bed is examined (Fig. 3), the total bedload can be considered to have travelled nearly 11 km in some 40 years. This is only a rough estimate, since material may have originated further upstream than the canal on the one hand, or may have been carried further downstream. For comparison, Duchesne and Pissart (1985) suggested a value of the order of 500 m per century for the River Ourthe in the Belgian Ardennes, and Tricart and Vogt (1967) produced a value of 5 km in 25 years for the Hérault, a value which can be considered to be a maximum since this is a river with a steep slope in a Mediterranean climate with pronounced peak discharges.

4. Conclusions

First, this study was an opportunity to investigate some important questions dealing with the hydraulics of a large gravel bed river. The main results are the following:

— Due to the artificial shaping of the bed, Manning's coefficients of total roughness for the Miribel canal are relatively low ($n_t \approx 0.030$). Consequently the grain shear stress (τ') represents a significant part of the total shear stress evaluated from the product of the gradient and the depth (of the order of 70%). There is a close agreement between these values of τ' and those of shear stresses calculated from friction velocities (τ_*). A very close relationship emerges between shear stress and discharge irrespective of site; slight variations occur at the level of microsites (upstream/downstream position on sampled bars).

— The frequency of bedload transport is significantly higher than generally assumed. Indeed, some movement of material occurs already at a discharge of 440 $m^3 \cdot s^{-1}$, a discharge barely one third of bankfull flow and occurring on average 40 days $\cdot$ year^{-1}. Bedload movement becomes general when the discharge reaches 580 $m^3 \cdot s^{-1}$ ($0.44 Q_b$), which occurs on about 30 days $\cdot$ year^{-1}. This high frequency of bedload transport can be explained, in the first place, by the composition of the material: fairly small size ($D_{50} \approx 25$ mm), well graded with clusters and protection playing a very small part, and absence of paving. Furthermore, since the bed-form shear stress is insignificant in view of the characteristics of the bed, a large proportion of the total shear stress remains available for transport.

— The bedload discharge was estimated using the Meyer–Peter equation, taking the grain shear stress (τ') and a value of critical shear stress (τ_c) equal to the shear stress evaluated for a discharge of 440 $m^3 \cdot s^{-1}$, i.e. the discharge required to initiate movement. Translated into Shields' dimensionless criterion ($\theta_c = 0.040$), this value is lower than the one initially employed in the Meyer–Peter equation.

Second, these results provide good arguments to revise management practices in the

Miribel canal. The studied reach was successively an embanked arm of the Rhône and a by-passed reach of this river after 1899. In the 1960s, a new equilibrium of the long profile of the canal was supposed to have resulted from hydraulic tilting over the preceding century, and from the completion of Jons dam which reduced peak floods in the by-passed reach. The signification of this equilibrium was that inputs of coarse sediment into the canal equated the outputs. The large amounts of gravel mined from the canal since the 1970s have resulted in a dramatic lowering of the river bed and in the destruction of paved riffles. Furthermore from field surveys, we made the assumption that inputs of bedload from upstream were no longer able to compensate for outputs due to bed erosion. In other words, changes at the watershed scale had probably disrupted the former equilibrium during the last 30 years.

The estimates from the hydraulic study were that, each year, nearly 45000 t of material may be transported and therefore eroded in the Miribel canal. By adding to this figure floods of lesser intensity, one would thus obtain, for floods falling between 440 and 850 $m^3 \cdot s^{-1}$, a total close to 60000 t of material per year. Floods of higher intensity can pass down the Miribel canal with higher shear stress; the quantity of material transported is consequently even greater, but it is then compensated for by input originating upstream as a result of the opening of the bottom sluice-gates of the Jons dam. The total balance is then probably negative at the reach scale.

Finally, two practical conclusions may be drawn from this study:

— The operation of the Jons dam should allow input of pebbles through the bottom sluice gates as soon as the discharge exceeds 400 $m^3 \cdot s^{-1}$, in order to provide complementary inputs of coarse sediment to the Miribel canal.

— In the long term, if the disequilibrium between inputs and outputs of bedload is confirmed, it will be necessary to control the long profile of the by-passed reach and the level of groundwaters by the construction of transverse weirs.

These conclusions illustrate the impact of the shortage of sediment supply at the watershed scale even when a by-passed reach such as this one is not isolated from upstream. The critical situation of this reach is due to the small size of pebbles derived from Pleistocene fluvio-glacial deposits in the piedmont of the Alps. The geomorphic heritage and the evaluation of sediment balance should be taken into account in the management of heavily impacted large rivers.

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