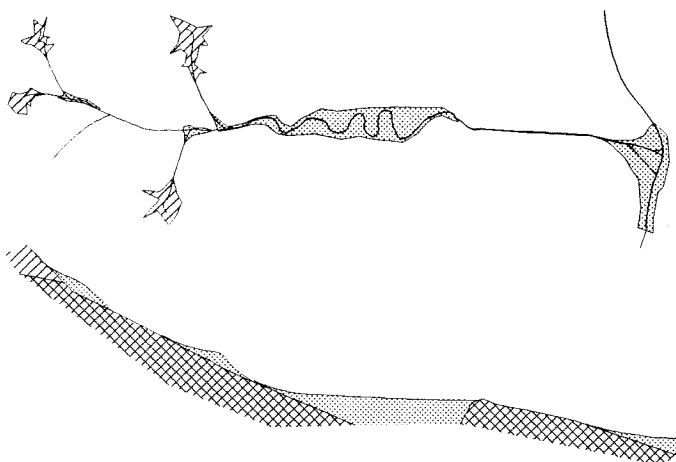# Communications on hydraulic and geotechnical engineering

Hydraulics and morphology of mountain rivers: a literature survey
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HYDRAULICS AND MORPHOLOGY OF MOUNTAIN RIVERS

A Literature Survey

by

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Abstract

Present knowledge on fluvial processes in mountain rivers should be expanded to enable the development of projects dealing with mountain rivers or mountain-river catchment areas. This study reviews research on hydraulic and morphological features of mountain rivers.

A major characteristic of mountain rivers is the variability of the hydraulic and morphological parameters. Flows can change from extremely non-uniform flow over large roughness elements at low stages to relatively uniform flow at high stages. The irregularity of geometry complicates the modelling of the turbulent, non-uniform and/or unsteady behaviour of water and sediment. It can be concluded that, due to the complexity of the conditions observed, a proper, general description of sediment movements in mountain rivers is not possible yet.

Description or prediction of morphological developments at present is limited to exceptionally isolated phenomena. Morphological responses of a river to a flood depend on (i) the size-distribution of the bed material and (ii) the distribution in time and place of hydrographs and sediment supply. The effects and relevancy of extreme hydraulic conditions have to be investigated, to enable description and prediction of long-term morphological evolution. Considering the importance of extreme, and subsequently low-frequency variables, the prospect of theoretical simulation models of morphological processes in mountain rivers seems rather remote.

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Chapter One.

Introduction.

After urbanization of low-land regions, the development of natural resources shifts to less-accessible areas. This has led to an increasing exploration of land and water in mountainous regions in the last few decades. Water resources are allocated for energy production projects, fisheries and supply of agricultural and municipal water needs. Urbanization of alluvial fans continues, as a result of occupied low lands.

Mountain streams are part of a complex system, in which hydrological, geological, ecological and morphological factors are integrated. However, in contrast to low-land rivers, little is known about the fluvial and morphological processes in mountain rivers. Consequently, an optimal planning, construction and management of river engaged projects in mountainous regions can not be accomplished. Growing fluvial hazards (Davies, 1989) and environments that are seriously affected by erosive processes, have increased the need for knowledge, and consequently scientific research on fluvial processes in mountainous regions.

This report is a literature survey of recent research on mountain rivers. The study aims to describe or refer to research on fluvial processes of mountain rivers from a hydraulic and morphological point of view. With this survey, an attempt has been made to describe the state of the art on the subject and to distinguish priorities in research.

In Chapter Two, attention is paid to the general characteristics of mountain rivers compared to low-land rivers. Chapter Three describes general hydraulic processes and includes a review of the formulation of resistance to flow. In Chapter Four, the morphological features of mountain rivers are described. This chapter includes reach-averaged characteristics and geometric features at sediment particle scale.

Movements of sediment are described in Chapter Five, which concentrates on the rate, composition and mode of sediment transport. In addition to Chapter Five, effects of a changing rheology of flow due to high rates of sediment transport are described briefly in Chapter Six. Here, also remarks can be found on discharge measurements and prediction of solute transport in mountain rivers. Conclusions are presented in Chapter Seven, where recommendations are made on topics of further research.

Chapter Two

General characteristics.

2.1. General.

From the contributing watersheds flowing down seawards, rivers encounter a wide array of physical conditions, which significantly affect the characteristics of the system. Schumm (1977) modelled the ideal fluvial system as consisting of (i) a drainage basin as sediment and run-off producer, (ii) main river channels as a transfer component and (iii) sediment deposition zones such as alluvial fans, delta's, etc.

From a topographical point of view, mountain rivers can be defined as rivers that directly drain catchment areas, and convey water and sediment from mountainous headwaters to a receiving river or storage system. Galay (1989) described a catchment area as *"a geomorphic unit, or area, that gathers water and sediment from precipitation and snowmelt and delivers to a lake, a larger river or to the ocean."* A catchment area can be considered equivalent to drainage basin or watershed.

In many ways, rivers in mountainous regions differ from lowland rivers. Mountain rivers can be characterized by relatively small watersheds, steep and often non-uniform slopes and turbulent flows. The fluvial and hydrologic processes in mountain regions are characterized by extreme variability, in both time and place. This regards (i) the input of water and sediment, (ii) the geometry of streams and (iii) the behaviour of flow.

Rain and snowfall can be very concentrated due to orographic effects. Run-off is related to complex conditions including hydrology, soil and rock characteristics, topography, vegetation and cultivation. Hydrographs can be steep and erratic. Large-scale sediment supplies exhibit non-uniform spatial and temporal frequencies. Intense, localized "bursts" of rain can produce debris flows and flash floods.

Snow avalanches, rock falls and landslides, or on a smaller scale, trees in forested areas can temporally block the stream and induce future flood waves, or drastically change the plan form of the stream (Davies, 1989).

The nature of flow can also be variable. During flash floods, the state of flow may change from turbid flow transporting gravel as bed-load to an unsteady debris flow, a dense slurry of boulders, gravel, sand, silt and clay in water.

2.2. Water and sediment sources.

2.2.1. Types of water supply.

Mountain rivers are located in or directly downstream of water and sediment delivering areas. The type, yield, frequency and sequence (in space and time) of different sources in the water and sediment supplying areas significantly affect the geometry of the river.

Water supply from catchment areas to mountain rivers concerns direct inflows from (i) run-off from precipitation or snowfall supplied by adjacent slopes, terraces or small, local streams, or (ii) indirectly by way of temporal storage; snowmelt, meltwater from glaciers, seepage from underground reservoirs or sudden inflow from lakes or reservoirs by dam failure.

The inflow can therefore be very localized and unsteady. Floods in mountain rivers can result from rapid snowmelt, excessive rainfall or from flood waves induced by the breaking of a natural or artificial dam of a lake or reservoir. Measurements are scarce due to hard accessibility of mountainous catchment areas. The complex measuring conditions of extreme events reduce the accuracy of measurements. The representability of local point-measurements for entire reaches is often questionable.

Many models have been developed to predict inflow from hydrologic processes in mountains. However, the complex, interrelated effects of climatics, topography and morphology limit the applicability of the empirically based descriptions to specified regions. Unique inflow events resulting from dam-breaks, land slides or earthquakes cannot be described or predicted accurately.

Among the empirical relations regarding the prediction of discharge, the major part aims to describe steady flow in uniform reaches. Among others, Afanas'ev and Komarov (1984) and Naef (1985) suggested the extension of peak discharge time series, which however requires a relatively long period of discharge measurement. In absence of historic streamflow data, infiltration and run-off processes should be modelled to estimate the peak discharge from rainfall measurement data (Naef, 1985). To measure rainfall of localized storms, a dense network of point rainfall measurements over the basin is required.

Time scales of base flow are determined by slope, drainage density, hydraulic conductivity and drainable porosity (e.g., Zecharias and Brutsaert, 1988). Takahashi et al. (1983) briefly reviewed some empirical recession formulae on base flow in mountainous basins. Ando and Takahashi (1983) and Ando (1987) describe a base-flow input by distinguishing run-off and infiltration terms in the catchment area.

In narrow valleys, river-flow hydrographs in short reaches with steep slopes are significantly determined by the hydrograph of inflow that is delivered to the stream (Fernández et al. 1991). To model run-off processes in first order basins, generally the kinematic wave approach is applied assuming uniform and steady flow with average values of gradient, length and roughness of a slope (e.g., Hirano, 1983; Moussavi et al. 1990; Shentzis, 1990). Shentzis (1990) additionally related precipitation to elevation using empirical parameters that represent macro-scale orographic effects.

The space frequency of landslide-induced flood waves can be predicted by analysis of the regional geology and history of slope movements, to estimate the size and localize the geographic distribution of landslides (Swanson et al. 1985). Analysis of the topography of the drainage system indicates the potential landslide dam sites. Analysis of the stability of the dam could identify the areas with flooding hazards. The temporal frequency of floods from reservoirs or ice-dammed or moraine-dammed lakes cannot be derived from analysis of historic hydrographs because the events concerned may be unique, and hydrological characteristics of the catchment change suddenly and discontinuously.

2.2.2. Types of sediment supply.

Relatively small sediment sources of mountain rivers comprise sheet and rill erosion from steep slopes, tributary degradation, gully erosion, erosion of flood plains, banks and degradation of the bed in upstream reaches. Large-scale sediment supplies to mountain streams include landslides, debris slides, semi-arid mountain or alpine mud flows, till flows in glacierized, and lahars in volcanic regions. Therefore, debris of mountain streams can be volcanic or weathered materials or moraine depositions composed of rounded boulders, cobbles and finer grain sizes such as pebbles, granules, sands, silt and clay.

Different triggering mechanisms can be distinguished. Lahars can be initiated by earthquakes, rapid melting of snow and ice, conversion of pyroclastic flows and dry avalanches to waterborne debris flows, release or ejection of crater lakes, saturation or failure of debris dams or avalanches, or excessive rainfall.

Till flows are caused by the slumping of sediment on a glacier, backwasting of slopes composed of sediment and stagnant ice, melting of debris laden ice and erosion and mobilization of sediment by a catastrophic release of melt water. Semi-arid mountain mud flows and alpine mud flows are triggered by the slumping or slipping of saturated, unconsolidated material on steep slopes.

Massive landslides can be triggered by changes in landuse (such as deforestation followed by over grazing or cultivation) or construction activities (such as roads, canals, etc.), degrading river beds or undercutting of steep slopes by shifting river

bends.

The sediment yield of surface run-off erosion in a catchment area is related to topography, the actual and antecedent hydrological conditions, the weathering rate, the type and method of cultivation of the soil cover and vegetation. Catchment-area developments such as soil conservation programs or deforestation trends, affect the sediment input and subsequently the bed geometry in mountain rivers (Renard and Laursen, 1975).

Data on temporal frequency and quantities of large-scale sediment supplying events are scarce. As erosion by run-off is concerned, the complex character obstructs the prediction of sediment yields or sediment load graphs. Many empirical relations between run-off and sediment load have been recognized for certain types of small waterheds. With regression analysis, sediment yields have been related to rainfall (Overland and Kleeberg, 1985; Qingmei and Jinze, 1985), catchment area characteristics (Kronfellner-Kraus, 1985) or discharge (e.g., Amada, 1985). However, the applicability of the relations derived are limited and the insights in the physical processes rather implicit.

Di Silvio and Peviani (1989) developed a model for prediction of long-term equilibrium profiles of mountain rivers and suggest that the total annual sediment transport of a mountain river is determined by three factors; a topographical, sedimentological and hydrological one. However, the complexity of the conditions has obstructed yet the development of models that integrate all three factors.

2.3. Channel response.

2.3.1. Erosion and deposition zones.

In relation to the geologic and morphologic features of the catchment area, the sediment supply to mountain streams is spatially varied. The morphological response of the channel to large-scale supply of sediment depends on the rate of delivery, relative to the rate of removal by fluvial processes (Swanson et al. 1985). Mass movements of sediment can reduce the width of a valley or river, and raise the bed. If the movements are slow compared to the removing fluvial transport capacity of the river, little change in geometry can be experienced.

If the rate of these movements increases, temporary changes in channel geometry and elevation may occur. Removal of sediment by fluvial processes can destabilize the volume of sediment supplied. If, in the extreme case, slides move rapidly, a large volume of sediment can bury the channel completely, forming a dam by locally raising the valley floor. Those landslide dams can eventually fail as a result of piping, incision after overtopping or mass movement, or remain stable and create a reservoir. Then a low gradient valley floor can develop upstream of the constriction.

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Along the river profile, erosion and sediment deposition zones can be distinguished (Fig.2.1).
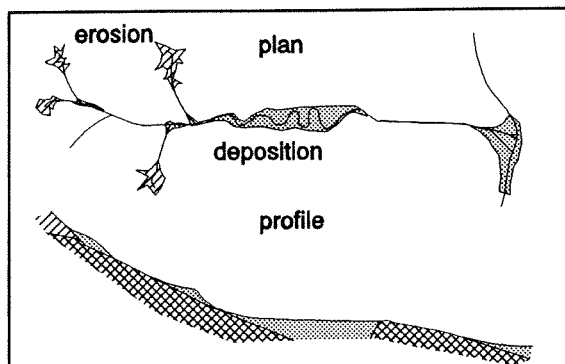
Figure 2.1 Mountain river course.

At sections with excessive supply of sediments, movements of sediment volumes can interact actively with the hydraulic behaviour of flows. Consequently, the geometry and morphological settings of mountain rivers vary from relatively stable, irregular bedrock channels at steep, degradating reaches to relatively unstable, reaches with finer sediment accumulated at flatter slopes. Plan forms include i) straight channels, ii) regular, tortuous or irregular meanders and iii) irregular channels. As a result, two types of channel geometry can be distinguished in the morphology of mountain rivers;

- reaches with a relatively *stable* geometry and a *supply-limited* sediment transport
- reaches with a relatively *unstable* geometry and a *flow-controlled* sediment transport

2.3.2. Channels in erosion zones.

At degradating bedrock channels, the river morphology is controlled by geological constraints that determine the plan form, width and elevation of the channel. The geometry of bed-rock streams is rather irregular and non-uniform with an extremely wide range of grain sizes including sands, gravel cobbles and boulders. Selective entrainment of particles coarsens the surface of the bed, which stabilizes the river morphology. The longitudinal profile in steep, narrow mountain streams often exhibit a stair-case appearance of steps and pools. These steps are formed by regularly arranged boulders and affect the depth of the flow, resistance to flow and sediment transport.

The water depth can be small, compared to the sediment particles on the bed. Large-sized sediment can even protrude through the flow. Consequently, slope, width, bed level, flow depth and velocity can be very irregular.

The transport capacity of flows often is insufficient to move particles from the coarse, armoured bed. As a result, the sediment transported is often supplied by upstream

reaches or lateral sources (wash load) and can be considered supply-limited. At small-scale sediment depositions, non-erodible rocky beds can be uncovered during high flows. Sections of non-erodible bed can act as natural check dams. Because changes in bed slope will require less accumulation of sediment between non-alluvial sections, the morphological response time will be reduced (Di Silvio and Peviani, 1991).

2.3.3. Channels in sediment deposition zones.

Sediment is deposited at alluvial fans at the base of steep slopes, delta's in lakes or reservoirs, alluvial flood plains, aggrading reaches (at wider cross-sections, Nakamura et al. 1985), channel bars or downstream of sediment supplying sources. Because the stable parent bed rock is covered partially or entirely, the river morphology exposes a more alluvial character.

With an increasing degree of valley affecting the river, Galay (1989) distinguished partially entrenched (bordered occasionally by flood plain segments), entrenched, partially confined (occasional control of channel pattern by valley) and confined channels (Fig. 2.2).
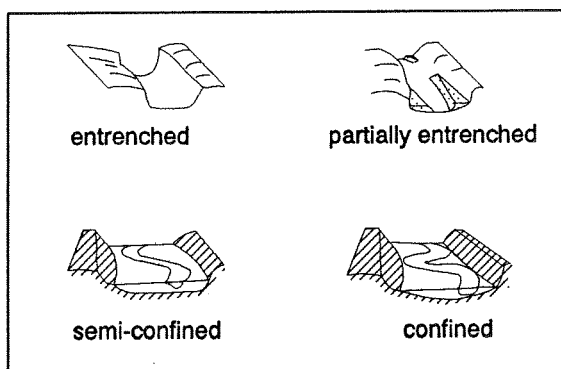
Figure 2.2. River types in valleys.

2.4. Mountain river projects.

2.4.1. Impacts of hydraulics and morphology.

The impacts of mountain rivers concern (i) hydrologic and morphological interactions with the catchment area (ii) the fluvial system of the river and (iii) the water and sediment transported. Because the relations between these elements cannot be described properly, planning, design, construction and control of mountain river projects with full account of the changes induced is not possible yet.

In Table 1.1, some general benefits and impacts from mountain rivers are briefly reviewed. Mountain streams interact with the catchment area through large-scale

sediment deposition or retrogressive erosion (MSSSY, 1982).

Among other factors, developments of mountainous regions are determined by regional gradients. The accessibility, water resources and, in general, fertile soils in flatter regions of large-scale sediment deposition provide appropriate conditions for human settlement. This is reflected in the distribution of populations and subsequent usage of land and water resources in mountainous regions, which often are concentrated at flatter slopes (e.g., terraces, flood plains, alluvial fans), whereas at steeper slopes the usage of land and water resources is limited.

As a result, effects of morphological and hydraulic phenomena in mountain rivers range from indirect impacts (erosion processes in unoccupied areas) to extremely important direct impacts (flooding at fans). The potential of a mountain river changes with the hydraulic and morphological features in different regions.

aspects	potential	characteristic
water	-supply of: irrigation projects domestic and industrial use (cooling water) -generation of hydropower -transport of products -waste disposal	-complicated diversion and storage management (erratic hydrographs) -flooding hazards -variable rates of sediment transport
environment of fluvial system	-navigation -fisheries -recreation -ecological habitat	-turbulent flows -irregular, non-uniform velocities and depths
sediment	-sediment mining -management of sediment production and transport	-large morphological responses (erosion or deposition) -significant changes of river environment -instability of structures
Table 1.1		

2.4.2. Hydraulic aspects.

In small, steep mountain streams, turbulent flows over and through the irregular, non-uniform bed material of mountain streams, have in some cases enabled the implementation of fisheries. However, management of flushing flows that maintain the geometry in the proper conditions (open, permeable bed composition) is complex (Wesche et al. 1987). The environment of a steep mountain stream represents a unique habitat for flora and fauna species. Pool and riffle sequences maintain adequate life in the ecosystem by forming alternating sections of deep and shallow water. Despite the significant ecological importance, these aspects will not be reviewed further.

Due to the extremely variable and relatively small discharges, water supply is limited to small-scale diversion for irrigation usage or power generation, implementation of fisheries or recreation (including navigation). Schälchli (1991) describes the consequences of water diversion from relatively small mountain streams in a qualitative manner.

More downstream in flatter regions, the higher, accumulated river-discharges enable a larger scale of water usage. This allows development of river projects more similar to that in low-land rivers. Due to irregular inflow and steep slopes, hydrographs in mountain rivers can be steep and erratic. At flatter sections, the variable discharges, extreme amounts of sediment transport during floods and the subsequent large-scale morphological responses of the river can cause significant complications and hazards (Table 1.1).

This complicates the management of river diversion and storage facilities. Changes in morphology can threaten both the performance and stability of structures. Whittaker et al. (1985) distinguish:
- development of a scour hole downstream of a dam
- burial of a dam at intensive sediment deposition
- lateral erosion around the dam

Therefore, diversion and storage facilities should be accompanied by degradation protection works or sediment transport management measures.

To eliminate the irregularity in discharges, reservoirs are created providing storage capacity. Apart from discharge regulation for water supply, power generation and flood control, a reservoir can be used for management of sediment transport and erosion control (Section 2.4.3.), flushing of rivers, water quality control and recreation.

Sediment deposition areas can provide a wide range of building materials, such as gravel and sands. However, mining of sediment from mountain rivers can have a significant impact on the river morphology.

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Flooding hazards have been considered largest in the relatively flat regions: at terraces, flood plains and alluvial fans where flows decelerate and land resources are allocated. Due to relatively high velocities, unpredictable changes in flow directions and large amounts of debris and sediment, the characteristics of flow can change significantly (Simons et al, 1988). Flow patterns can be unstable, meandering and braiding (Moore, 1989). This unpredictability complicates the implementation of flood control.

In the analysis of flooding hazards on fans, different approaches have been applied (Simons et al, 1988):
- statistical; equal probability of flooding everywhere on the fan (Mifflin, 1988)
- deterministic; computation of flow depths in existing channels, and flooding depths and velocities across the fan computed as sheet flow (Beer and Jirka, 1988).

Dawdy et al (1989) mention different factors that determine the future flow pattern; previous flow patterns, alluvial washes of low flows, dykes, natural ridges and existing structures, the volume of delivered sediment, as well as the scour from bed and banks should be considered.

Mathewson and Keaton (1988) considered the type of flooding in the mitigation of flood hazards. They distinguish categories based on the causative phenomena: debris flow torrents, debris flood torrents, bed-load transporting torrents and flood creeks

With the different characteristics of the flood materials deposited, Mathewson and Keaton (1988) distinguished four hazard zones in downstream direction. The first zone is near the apex, and is characterized by flooding of slow moving plastic debris flows, carrying large blocks of rock. These flows "freeze" close to the apex, and often control the path of the following more fluid flow events. Hazards can be modified by construction of debris basins that store the flow.

The second zone downstream of the first suffers from impacts of viscous debris flows, that can develop higher velocities. Runout channels and diversion walls can change the flood hazard. The third zone can be flooded by rapidly moving hyperconcentrated sediment floods. Hazard mitigation options suggested are channel modification and diversion walls (Mathewson and Keaton, 1988). The hazards of the fourth zone at the downstream end of the fan are characterized by clear water flooding. Flooding occurs if the capacity of the draining channels is exceeded, therefore flooding mitigation measures concern the preservation of sufficient channel capacity.

2.4.3. Morphological aspects.

A second aspect of activities in mountain rivers concerns the controlling of a great quantity of sediment, that is released in a short period. In the morphology of mountain streams, Whittaker et al. (1985) distinguished two situations: (i) an upper catchment

area consisting of an extensive erosion zone, a transition zone with a steep gorge or ravine followed by a deposition zone, the alluvial fan and (ii) a stable catchment area, followed by an erosion zone with unstable side slopes.

In the first situation, the passage of flows and sediment should be controlled, to prevent damage on vulnerable side slopes in the transition zone (Whittaker et al. 1985). In the second situation, deposition of sediments by construction of check dams should stabilize the slopes. After stabilization, little sediment moves through the transition zone.

In the established control policies of mountain streams, different strategies can be distinguished:

i) To reduce or eliminate the production of sediment, the first strategy regards the *prevention of mountain-stream erosion*. The factors concerning the sediment production in mountainous regions are complex. Relative to the type of erosion in a catchment area, control of sediment production can be accomplished by reforestation, changes in cultivation (e.g., contour ploughing), terracing, etc. Movements of eroded material in mountain streams and catchment areas can be controlled by consolidation check dams (Zeller, 1985), that limit the amount of sediment delivered by tributary streams. These dams act as sills in stream beds subjected to erosion.

ii) A second strategy concerns the *prevention of sediment transport* by bed load and debris flow retention dams. The object of mountain-stream control is not to stop the sediment transport entirely, but retain the part that causes damage, such as debris flow and heavy bed-load during floods. Therefore, open structures have been applied. The retaining capacity of a sediment control dam affects the reduction of the peak sediment discharge (Senoo and Mizuyama, 1983).

iii) Another strategy aims at the reduction of flooding and elimination of debris deposition on valley floors by *preserving or increasing the transport capacity* of mountain rivers. This could be achieved by channelling and squeezing rivers between high levees, enlargement of the gradient by straightening river courses (Zeller, 1985), upstream retention of sediment, diversion of additional flows or regulation of flows.

The control of sediment processes, or bed-load management concerns:
- filtering out of the coarsest fractions to prevent obstruction
- dosing and retention of the problematic quantities of excess sediment
- reduction of flow energy by breaking and dividing the current.

According to their function, two types of sediment control works can be classified: beam dams and slit dams (Armanini et al., 1989). Beam dams sort or filter the sediments. These structures enable the passage of less harmful, finer sediments, and conserve storage capacity for retention of coarser particles. Slit dams dose the

sediments during floods by backwater effects or hydraulic jumps. During minor floods, sediment deposited in sediment retention basins is eroded by undisturbed flows. To prevent the slit from blocking Ikeya (1985) suggested

$$\frac{b}{D_{max}} \geq 2.0 \qquad\qquad (2.1)$$

where b the width of the slits, and D_{max} the maximum particle diameter. Different structures have been designed (Armanini et al. 1989) to reduce the extremely high dynamic pressures and prevent obstruction of the slit and beam openings and erosion near the abutments of the structures.

Iwamoto (1985) summarized the effects of check dams on a mountain river. The check dams with sufficient holding capacity reduce the bed slope, regulate the river width and flow direction and control the sediment transport. If large volumes of sediment transport occur, the dams temporarily check and accumulate sediment from upstream reaches, and allow a gradual transport afterwards. However, a single check dam can only be expected to fix the local river bed. Therefore, Iwamoto (1985) suggested a serial execution of mountain-river works to stabilize streams.

Chapter Three.

Hydraulics.

3.1. Introduction.

Flows in mountain rivers can be unsteady and non-uniform as a result of steep hydrographs, non-uniform geometry and lateral inflow. Flow depths in mountain rivers can be small, which in combination with large-sized sediment, yields large-scale roughness. At large-scale roughness conditions, the free surface of the flow is controlled by roughness elements. Changes in depth as well as sediment can result in time and space-varying roughness scales and subsequent flow structures. As a result, the local and mainstream flow regime can vary during a flood.

3.2. Structure of flow.

3.2.1. General.

The flow in mountain rivers can in general be described as turbulent flow over a rough, rigid boundary.
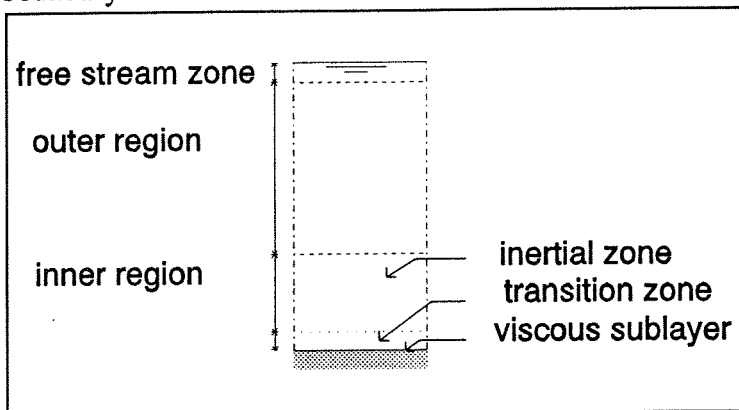
Figure 3.1. Regions in boundary layer flow.

Over the depth of flow, different regions can be distinguished (Fig.3.1) (Coleman and Alonso, 1983; Graf, 1989); an *inner region* close to the bed where the actions of shear stress and roughness are dominant, an *outer region* where direct effects of local roughness parameters are less significant and a *free-stream* zone that, in theory, is free of shear and turbulence. In nature, however, this *free-stream* zone may contain ambient turbulence generated at upstream sections, and therefore often cannot be identified.

In general, the flow is considered entirely boundary layer flow, although the boundary layer thickness occupies approximately 80 - 90% of the depth of flow (Coleman and Alonso, 1983). The inner region is limited to that part of the boundary layer where shear stresses can be assumed constant (about $z/a < 0.15$ for depths that are large relative to the particle size (Graf, 1989)). Within the inner region, again different zones can be distinguished; an extremely thin viscous sublayer, a transition zone where the flow transitions from laminar to turbulent flow, and an inertial or logarithmic zone. According to Nezu (1977), the effect of roughness on the turbulence appears up to the middle of the inner region. The inertial zone makes up the greater part of the inner region.

The boundary theory (law of the wall) for the turbulent regime yields logarithmic velocity profiles and, consequently, logarithmic hydraulic relations (e.g., Coleman and Alonso, 1983; Graf, 1989; Ayala, 1991). Integration of this profile over the depth yields

$$\sqrt{\frac{8}{f}} = \frac{U}{u_*} = \frac{1}{\kappa}\ln\frac{R_h}{k_s} + B \tag{3.1}$$

where f is the Darcy-Weisbach friction factor, U the depth averaged velocity of flow, $u_* = \sqrt{(\tau_0/\rho)}$ the shear velocity, τ_0 the bed shear stress R_h the hydraulic radius, B a numerical constant of integration, k_s a roughness parameter and κ Von Kármán's universal constant (Graf, 1989; Aquirre-Pe, 1990). Kirkgöz (1989) found a reasonable performance of Eq.3.1 for rough beds with small-scale roughness.

The constant of integration B, is related to the relative magnitude of roughness R_h/D_i (D_i is a representative particle size) and Froude number (Graf 1989; Tsujimoto 1989). The Froude number is defined as $Fr = U/\sqrt{ga}$, where U the velocity averaged over the depth a. Usually, B is taken 8.5, for flows with small-scale roughness. For gravel-bed rivers with $1 < R_h/D < 10$, B has been found to be 3.25 (Graf, 1989).

Graf (1989) found B to vary with Fr, at intermediate- and large-scale roughness. Tsujimoto (1989) proposes an empirical formula for B. In non-uniform flows with small relative roughness, B has been found to be approximately constant (Kironoto and Graf, 1990), although Tu and Graf (1992) distinguished a tendency for B to be larger at decelerating flows. Tominaga and Nezu (1992) found the value of B to increase with bed slope at supercritical flows.

In the outer region, the prevailing conditions do not warrant application of a logarithmic velocity profile; the surface velocities exceed the velocities predicted with logarithmic distributions (Jarret, 1984, Graf 1989). The turbulent energy exchange is dominated by convection, dissipation and diffusion. Consequently, unlike flows in the inner region, turbulent flows in the outer region depend on upstream, as well as local

conditions (Coleman and Alonso, 1983). This complicates the formulation of the velocity profile. As no theory is available yet, the relationships derived are empirical (Graf, 1989).

Coles (1956) presented an empirical wake function (law of the wake) to describe the velocity in the outer region. In the form of a velocity defect function, the velocity in both the inner and outer region with Coles' wake function reads

$$\frac{U_{max}-u}{u_*} = -\frac{1}{\kappa}\ln\left(\frac{y}{\delta}\right) + \frac{\Pi}{\kappa}\left[1 + \cos\left(\frac{\pi y}{\delta}\right)\right] \qquad (3.2)$$

where U_{max} is the maximum velocity, δ the boundary layer thickness and Π is Coles' wake parameter (Nezu and Rodi, 1986; Graf, 1989; Kironoto and Graf, 1990).

The wake parameter Π has been found to fluctuate between 0.0 and 0.7, and on average larger than 0.2 (Graf, 1989). The dependency of the wake strength coefficient Π on the Reynolds number is limited (Graf, 1989). The wake strength coefficient Π is affected by the non-uniformity of the flow, and the intermittency of the turbulence in the outer region. Tu and Graf (1992) found Π to be more pronounced at decelerating flows. Kironoto and Graf (1991) report Π to be affected by three-dimensional effects at small aspect ratios ($B/a < 5$).

3.2.2. Scale of roughness.

Mountain streams on steep slopes with high velocities generally have shallow depths of flow and large bed materials that affect the flow resistance. As a result, phenomena of intermediate- and large-scale roughness can be observed (Bathurst et al. 1981; Jarret, 1984). The scale of roughness in mountain streams can range from small-scale in beds composed of sands and gravel to large-scale in beds consisting of cobbles and boulders.
Bathurst et al. (1981) and Bathurst (1982a; 1982b) classify the relative submergence, or the ratio of depth a to representative bed element height D_{84} as approximately:

large-scale roughness:

$$\frac{a}{D_{84}} < 1.2 \qquad (3.3)$$

intermediate-scale roughness:

$$1.2 < \frac{a}{D_{84}} < 4 \qquad (3.4)$$

small-scale roughness:

$$\frac{a}{D_{84}} > 4 \qquad\qquad (3.5)$$

The roughness of bed material can be considered large-scale if the roughness elements affect the free surface (Bathurst, 1981; 1982b). Then, flow will essentially be of a jet and wake type and can be considered three-dimensional (Bray, 1982).

For large-scale roughness, most of the flow resistance is caused by form drag of bed elements, free surface distortion and local hydraulic jumps (Thorne, 1984). This implies that the processes of fluid mechanics are related to the Reynolds and Froude numbers, and the geometry of individual roughness elements on the bed, should be taken into account. The Reynolds number represents the effect of inertia relative to viscosity and is defined as $Re = UL/\nu$, where U is the velocity, L a characteristic length and ν the kinematic viscosity of water.

In the range of intermediate-scale roughness, both form drag and skin friction contribute to the flow resistance. For relatively small-scale roughness, variations in size-distribution and spacing of roughness elements are less significant (Bathurst, 1982) and the flow can be categorized as two-dimensional flow with similar time averaged velocity profiles across the channel (Bray, 1982).

3.2.3. Shallow flows.

In the case of small relative submergence, the turbulent structure of flow degenerates the logarithmic velocity profile near the bed surface (Bayazit, 1982). So, in streams with large relative roughness, the velocity profile is not logarithmic in the lower portions of the flow. Interpretated in terms of Fig.3.1, the transition zone is extended at the expense of the inertial zone (e.g., Kirkgöz, 1989). This is confirmed by measurements (Jarret, 1990) as shown in Fig.3.2., where point-velocity measurements are compared with a logarithmic velocity profile.

The formation of a roughness layer implies that flow velocity near the bed becomes smaller than that for flow with sufficient submergence at the same shear velocity. As a result, the hydrodynamic forces on the particles are reduced. Higher velocities occur near the free surface in shallow flows, because the major part of the discharge is forced to the surface level by roughness elements and associated low-momentum flow (Wiberg and Smith, 1991).
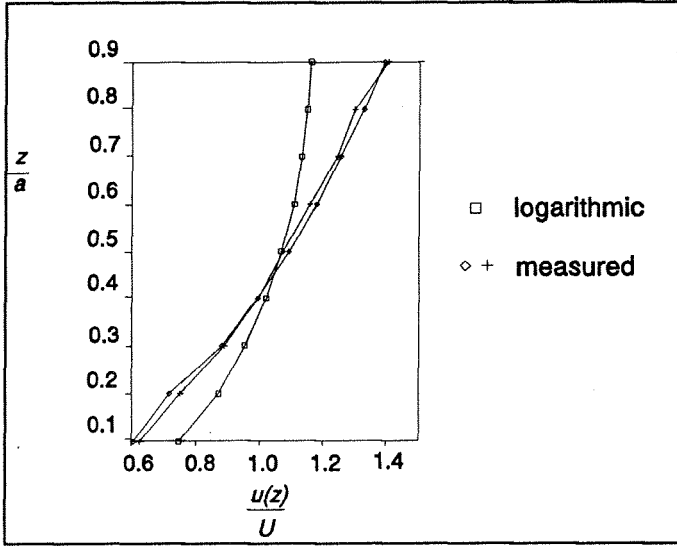
Figure 3.2. Degeneration of logarithmic velocity profile (Jarret, 1990)

The thickness of the roughness sublayer is approximately 0.3 to 1.2 times the representative bed material diameter (Tsujimoto, 1989). The flow velocity drops at a level corresponding to $z = D_{50}$, where D_{50} is the mean particle diameter of the bed surface material (Wiberg and Smith, 1991).

The degeneration of the velocity profile affects the energy coefficient α and the momentum transfer coefficient β. These coefficients are used to describe the effects of vertical velocity distributions in depth-averaged flow models. The *energy or Coriolis coefficient* (Chow, 1959) is defined as

$$\alpha = \frac{\dfrac{1}{A}\int_A u^3 \mathrm{d}A}{\left(\dfrac{1}{A}\int_A u\,\mathrm{d}A\right)^3} = \frac{\overline{u^3}}{\overline{u}^3} \tag{3.6}$$

The *momentum or Boussinesq coefficient* is defined as

$$\beta = \frac{\dfrac{1}{A}\int_A u^2 \mathrm{d}A}{(\dfrac{1}{A}\int_A u\,\mathrm{d}A)^2} = \frac{\overline{u^2}}{\overline{u}^2} \tag{3.7}$$

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In many research activities, attempts have been made to explain the degeneration of the velocity profile by correcting the reference bed level (e.g., Kirkgöz, 1989), by modifying the mixing length (Christensen, 1972) or eddy viscosity (Wiberg and Smith, 1991) or by considering the changes in turbulence structure (Nakagawa et al. 1989).

Christensen (1972) suggests the mixing length to be composed of (i) a contribution due to viscous shear stresses in the laminar sublayer, (ii) a part that is proportional to the height above the bed (Prandtl-hypothesis that is valid at approximately $z/a < 0.2$) and (iii) a part related to the formation of vortices between roughness elements. The vertical distribution of the second and third contribution have been described rather implicitly.

Kirkgöz (1989) models flow near rough beds by assuming a fictious layer with laminar flow, extended with Δz below the roughness height k. Kirkgöz (1989) mentions a range of thickness Δz from 0.3 to 0.7 times k. Kirkgöz (1989) found Δz to decrease with increasing turbulence. The "slip velocity" at $z = \Delta z$ was observed to be approximately 0.3 or 0.4 times the maximum point velocity.

Karaushev and Pozdnyakov (1987) found the turbulence intensity to increase significantly from small-scale to intermediate-scale roughness and related the turbulent intensity to a/D. In the roughness layer, however, where vortices dominate, the turbulence is suppressed (Chen and Roberson, 1974; Bayazit, 1976; Nezu, 1977), and consequently the Reynolds-stress distribution is degenerated inducing a uniform velocity distribution within this roughness sublayer (Nakagawa, 1989). Bayazit (1982) explains the decrease in turbulence intensity near the bed, by turbulence dissipating energy in the separation zones between the relatively large roughness-elements. Nakagawa (1989) noticed peaks in frequency of turbulent velocity-fluctuation to correspond to the eddy-shedding intervals from individual roughness elements.

The turbulence induced by significant bed roughness has been taken into account by identifying a wake zone, or roughness sublayer within the inner region near the bed surface (Nakagawa et al. 1989; Aguirre-Pe, 1990). Aguirre-Pe and Fuentes (1990) distinguished two zones in the velocity distribution (Fig.3.3.); in the wake zone, wakes generated by protruding roughness elements are overlapping, and the velocity is assumed constant. Above this zone Aguirre-Pe and Fuentes (1990) approximate the velocity distribution with a logarithmic profile.

The depth-averaged velocity can then be written as

$$U = \frac{1}{a}[u_w \beta D + \int_{\beta D}^{a} u\,\mathrm{d}z] \qquad (3.8)$$

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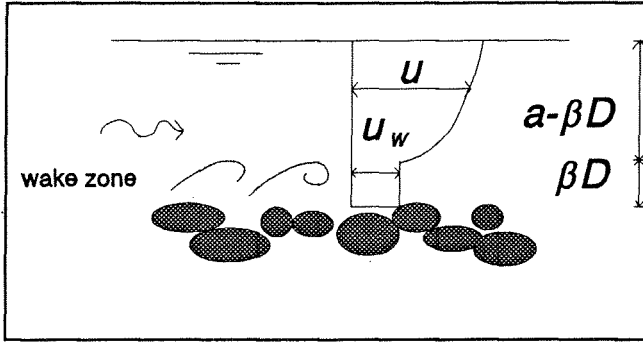
Figure 3.3. Wake zone (Aguirre-Pe, 1990).

And, after substitution of the logarithmic velocity profile of Eq.3.1 in Eq.3.8, and using that $u = u_w$ at $z = \beta D$, the friction factor f considering the wake effect can be written as

$$\sqrt{\frac{8}{f}} = \frac{1}{\kappa}\ln(\frac{a}{k_s}) + B + \frac{1}{\kappa}\frac{\beta D}{a} \qquad (3.9)$$

Wiberg and Smith (1991) predicted the degeneration of the velocity profile by adjusting the length scale for turbulent mixing to wake effects that are created by large roughness elements. They modelled the total shear stress, τ_t, as the sum of fluid-turbulence shear stress, τ_f, and drag-related shear stress, τ_d, with the latter corrected for free-surface drag τ_s

$$\tau_t(z) = \tau_f(z) + \left[\tau_d(z) - \tau_s\right] \qquad (3.10)$$

Assuming uniform flow conditions, Wiberg and Smith (1991) used a linear distribution of the shear stress τ_t. The fluid-turbulent shear stress, τ_f, was defined as

$$\tau_f = \rho u_* L \frac{\partial u}{\partial z} \qquad (3.11)$$

where L is the length scale of turbulent mixing, composed of L_1 based turbulence in undisturbed flow, and L_2 which accounts for wake effects that dominate the turbulence structure

$$L = L_1 + L_2 = (1 - \sum_{i=1}^{N} c_i)\frac{L_f}{(1-z/h)^{1/2}} + \alpha \sum_{i=1}^{N} c_i D_i \qquad (3.12)$$

where L_f is the length scale of turbulent mixing as defined for flows with relative small roughness, c_i and D_i the surface concentration and particle diameter of grain-size

fraction i respectively and α an empirical coefficient hypothetically ranging from 0.1 to 0.5.

Assuming a log-normal distribution of sediment sizes, c_i could be determined to analyse velocity profiles. The drag-related shear stress, τ_d, is defined as

$$\tau_d = \frac{3}{4} \rho \sum_{i=1}^{N} \frac{C_{di} c_i}{D_i} \int_{z_0}^{D_i} u^2 dz \qquad (3.13)$$

where C_{di} is a drag coefficient of size fraction i. Iterative solution of the shear stresses τ_d, τ_t and τ_f yields the velocity profile (Wiberg and Smith, 1991). However, it should be questioned wether the interaction between τ_f and τ_d can be neglected.

3.2.4. Regimes of flow.

In river hydraulics and morphology, the state of the flow, or flow regime is used to identify the hydraulic behaviour of the flow, or the morphologic responses of the bed. The flow regime can be classified by the Froude number Fr. In mountain rivers, different flow regimes can occur at different stages (Fernández et al. 1991) and locations. Bathurst et al. (1983) carried out flume experiments with steep slopes and coarse sediment, and concluded the Froude number, which increased with increasing discharge, to be rather sensitive to energy dissipation by transport of sediment (Fig.3.4). The flow regime of mountain rivers can be expected to change during a flood also as a result of varying resistance to flow.
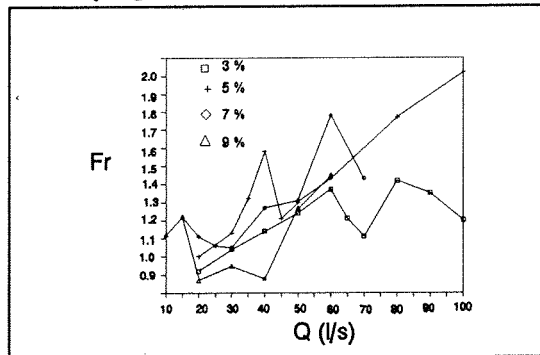
Figure 3.4 Changes in Froude number (Bathurst et al. 1983).

At small-scale roughness and a *subcritical* ($Fr < 1$) regime of the mainstream flow, gravity forces are more pronounced than inertia. At low Froude numbers ($Fr < 0.5$), the effects of large roughness elements, geometrical cross-sectional irregularities or residual bed forms will only be a slight depression of the surface (Bathurst et al. 1981; Bathurst, 1982b; Colosimo et al. 1988). Although locally non-uniform flow conditions can occur, Peterson (1960) described the regime as tranquil flow.

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If in that subcritical mainstream flow the scale of roughness or the Froude number increases, the flow can become crest-controlled. Then, the flow is *critical (Fr = 1)* on top of, or funnelling between the stable bed elements. Then the regimes up and downstream are decoupled and the local behaviour depends on upstream and downstream conditions.

If the mainstream of the flow still is subcritical, the region of critical or supercritical flow is limited and the flow depth on top or downstream of bed elements is forced above the sequent depth of the mainstream flow. As a result, a hydraulic jump is formed behind the roughness element and the free surface drag increases sharply. At higher roughness scales or Froude numbers, the supercritical flow region is extended by forcing the jump in downstream direction.

In this case, the flow locally alternates into subcritical and supercritical states (Chow, 1959; Bathurst, 1982b; Lawrence, 1984). This condition of flow, dominated by accelerating and decelerating flow is characterized by series of localized, stationary hydraulic jumps, and has been described as *tumbling flow or transcritical flow* (Peterson, 1960). The energy losses in the tumbling flow regime are caused by turbulence, water jets striking the bed elements, localized wall shear and dissipation due to oscillations around critical flow.

At high velocities, the flow can become *supercritical (Fr > 1)*. In this stage, the inertial forces are more significant than the gravity forces and the flow is usually described as rapid (Chow, 1954; Peterson, 1960). At relative large submergence depths, the effect of large bed elements is restricted to a rise in the free surface. According to Peterson (1960), flows with sufficiently large submergence depths do not exhibit hydraulic jumps in the tranquil and rapid flow regimes. If the roughness scale forces the water level above the subcritical sequent depth, hydraulic jumps can appear at the front of the element. If the relative submergence depth exceeds about 0.8, the roughness elements can be overtopped and hydraulic jumps can form above or behind the elements (Bathurst, 1982b).

However, extreme turbulence will be created by channel form and cross-section variation (for example Herbich and Walsh (1972)) inducing the formation of cross waves, large expansion and contraction coefficients, obstructions by debris and transport of large-sized bed material. Due to large bed roughness and subsequently great energy dissipation, Mussetter (1989) observed the flow resistance at steeper slopes to be much greater than for flatter slopes. This dissipation of energy results in a reduction of the velocity. Therefore Jarret (1987) indicates that supercritical flow can occur in sand and smooth bed-rock channels, but is only very localized in rough boulder bed streams. Trieste (1992) also suggests critical flow to exist at short distances only.

3.2.5. Flow discontinuities.

Apart from local phenomena at bed-element scale, cross-sectional changes in flow regime occur in non-uniform river geometries such as riffle-pool sequences. In the modelling of flows, the effects of local jumps are usually lumped in reach-averaged resistance formulae. However, if the mainstream flow regime changes, the structure of the flow is significantly affected by the geometrical discontinuities. In the modelling of (un)steady flow, generally the Saint Venant equations can be applied (Basco, 1989).

These are, however, not valid near discontinuities, where the local distribution of pressure can be non-hydrostatic due to streamline curvature effects that affect the transfer of momentum (Basco, 1989). Gharangik and Chaudhry (1991) modelled rapidly varying flow with the Saint Venant equations extended with the Boussinesq term that accounts for changes in pressure distribution. In the modelling of flow at hydraulic jumps however, the effect of the changing pressure distribution was found to be relatively small (Gharangik and Chaudhry, 1991).

In transcritical flows, local equations are used at transition points, that describe the rapidly varying non-uniform conditions of flow. At unsteady flows, the location of transition points ("shocks") can move. This has been investigated numerically in the modelling of dam-break induced waves with shock-fitting or -capturing algorithms (e.g., Botev Botev, 1991; Bhallamudi et al. 1990; Savic and Holly, 1990; Savic, 1991).

Hughes and Flack (1984) measured a decrease in sequent depth and length of hydraulic jumps, due to boundary shear-stresses of rough beds. The reduction increased for increasing values of initial Froude number and relative roughness. The stage of flow and the slope of the bed can affect the hydraulic jump significantly. At extremely steep ($i > 0.40$), uniform slopes and large depths downstream of the jump, Ohtsu and Yasuda (1991) observed in flume experiments no clearly identifiable surface roller, but noticed the supercritical flow to continue along the bed as a high-velocity jet. At steep flume slopes (with $i < 0.34$) changing to a flat section ($i = 0$) and smaller depths downstream of the jump, surface rollers appeared, partially on the steep and flat section. The formation of a surface roller significantly affected the delay in velocity, and subsequently the dissipation of energy (Ohtsu and Yasuda, 1991).

In supercritical flows, changes in geometry can induce cross waves (e.g., Hager, 1989). Jimenez and Chaudhry (1989) modelled supercritical free surface flows with shocks and rapid expansions, assuming a hydrostatic pressure distribution except near the discontinuities and neglecting interactions between different cross-waves.

3.2.6. Roll waves.

High velocity flows may exhibit instabilities, which develop from small disturbances,

and grow into a pulsating flow with series of breaking waves, or roll waves (Mayer, 1959). Many investigators derived stability criteria, by analysing the growth or decay of perturbations (Ishihara et al. 1960; Taylor and Kennedy, 1960; Escoffier and Boyd, 1962; Dracos and Glenne, 1967). If the flow becomes unstable, the dissipation of energy, and subsequently the resistance to flow increases significantly (Rouse et al. 1963; Rouse, 1965; Sarma and Syamala, 1991).

The flow can become unstable if the Froude number exceeds a critical value. This critical Froude number is a function of the velocity profile, the Reynolds number, the resistance to flow, the wave length of the initial disturbance wave and the channel width (Berlamont, 1976). In the case of uniform flow, a disturbance travelling with the current becomes unstable if $Fr > 2$ (Escoffier and Boyd, 1962; Montuori, 1963; Dracos and Glenn, 1967; Berlamont and Verstappen, 1981; Kranenburg, 1990).

The critical Froude number in rough channels increases with increasing channel roughness, and decreasing channel width (Rosso et al. 1990). The non-uniformity of flow affects the stability criterion (Kranenburg, 1990). Berlamont (1976) explained the existence of an upper critical Froude number, beyond which no roll waves develop. For shallow flows, Aziz and Prasad (1985) suggested the presence of a movable sediment layer to have a stabilizing effect on surface waves. However, sufficient experimental confirmation lacked.

Because non-linear analyses have not been carried out, the behaviour of instable flows cannot be described. In the modelling of supercritical flows, the occurrence and effects of roll waves are generally not taken into account (Beer and Jirka, 1990; Sloff, 1989).

3.3. Resistance to flow.

3.3.1. General.

Flow resistance generally refers to the processes by which the physical shape and bed roughness of a channel control the depth, width and velocity of flow in a channel. However, total flow resistance factors include free surface instabilities, secondary flows, non-uniform shear-stress distribution, cross-section irregularities, channel shape, obstructions, vegetation, channel meandering, suspended material and bed load (Jarret, 1984). The relative importance of the factors alters with depth and discharge (Bathurst, 1982b). According to Colosimo et al. (1987), the friction factor f can be described as

$$f = F(\frac{R}{D_\phi}, \Phi, \psi, Re, \xi, Fr, \tau_*)$$
(3.14)

where R is the hydraulic radius, D_ϕ particle size for which ϕ percent of the particles are finer, Φ and ψ the influence of the cross-sectional shape and of the grain-size

curve, Re the Reynolds number, ξ the flow sinuosity and Fr and τ the Froude number and dimensionless shear-stress. The dimensionless shear-stress is defined as $\tau=u^2/\Delta gD_\phi$, where u the mean shear velocity and $\Delta = (\rho_s/\rho-1)$, with ρ_s and ρ the specific density of water and sediment respectively.

Bray (1982) distinguished two approaches for the evaluation of resistance to flow. The first considers a reach to consist of a straight prismatic channel characterized by average geometry and uniform, steady flow properties. The second approach considers the detailed flow characteristics and fluid turbulence, and is therefore generally restricted to the laboratory.

Numerous equations have been developed, either based on the application of a boundary layer theory to open channel flow, or empirically based on multiple regression analysis of prototype data sets (Jarret, 1984; Mussetter, 1989). However, the general applicability of the formulae derived is limited, as the complicated physical processes that induce the resistance to the flow are difficult to describe and the data bases used for derivation and calibration of empirical relations are restricted to the regions specified.

In flows with small-scale roughness, generally two processes are distinguished: resistance associated with the roughness geometry, and resistance associated with the structure of flow. The first is resistance due to size, shape, gradation and arrangement of the roughness elements on the channel boundary (*"grain resistance"*). The second one is resistance due to flow separation and macro-scale eddies that result from Φ and ξ, slope, etc. (*"form resistance"*) (Colosimo et al. 1987). At intermediate and large-scale roughness, the distinction of the two types of resistance is arbitrary.

3.3.2. Individual effects of roughness elements.

Effects of grain resistance include form drag and surface drag of roughness elements. Bathurst et al. (1981) classified the energy dissipation due to the coarse bed elements (grain resistance) with scale of roughness. The effects of Froude number and sediment mobility (Colosimo et al. 1987; Griffiths, 1989, among others), and the effects of the non-uniformity of the flow (Kironoto and Graf, 1990; Tu and Graf, 1990) have been subject of more recent research. Due to the significant effect of the roughness geometry of the bed, the contribution of the form drag by micro-scale eddy structures to the total equivalent bed shear-stress is large, relative to viscous shear stresses caused by skin friction (Aguirre-Pe, 1975; Arisz and Davar, 1991).

According to Li and Simons (1982) the resistance to flow in steep, shallow cobble-bed channels depends on the form drag of the boulders. The form drag of a roughness element is induced by imbalanced pressure forces acting on the upstream and downstream faces of the element. The magnitude of the net pressure force is related to the type of turbulence structure induced. The eddy structures and turbulence field

change with Reynolds number defined as UD_{84}/ν. The turbulence field, and subsequently the form drag, may change from semi-smooth turbulent (isolated roughness), through hyper-turbulent (wake interference), to quasi-smooth (skimming) flow, with increasing Reynolds number, and roughness concentration (Chow, 1958; Arisz and Davar, 1991).

Within the range of Reynolds numbers of 6×10^4 up to 6×10^5 and relative roughness of 0.15 up to 0.40, the partition of the form drag in the total shear stress decreases with increasing Reynolds number (Bathurst, 1981, Arisz and Davar, 1991). This trend could possibly be explained by the transition of turbulence field from wake interference to skimming flow (Arisz and Davar, 1991). Consequently, the flow resistance can be significantly affected by the Reynolds number (Bray 1979; Bathurst, 1981; Bathurst 1982b; Arisz and Davar, 1991).

The form drag contribution increases with increasing height of the roughness elements (Arisz and Davar, 1991). Flume studies pointed out that the effects of bed-material shapes on resistance to flow are within the accuracy of flow-resistance prediction methods (Thorne, 1984). As roughness shape is generally determined by local geology, shape effects within a region of uniform geology are likely to be constant (Bathurst, 1978).

At large and intermediate roughness scales, both form and free surface drag can be observed. Then the phenomena are interrelated. The formation of an eddy structure behind an obstacle occurs after separation of decelerating flow from the bed (Huppert and Britter, 1982). Therefore, changes in depths and subsequently velocity by disturbed free surfaces affect the point of separation of flow, and therefore the form drag. This interaction decreases with increasing a/D_{84}.

At large-scale roughness, pronounced free surface distortions are exhibited (Flammer et al. 1970; Bathurst, 1982b). These surface distortions are generated by bed elements protruding into the flow and represent significant energy losses. At $a/D_{84} > 0.8$ and with increasing Fr, Flammer et al. (1970) observed an increase in free surface drag coefficient, C_d, for $Fr < 0.6$, a peak value for $Fr = 0.5$ to 0.6 and to decrease for $0.6 < Fr < 2.0$ approximately. They conclude that pronounced free surface effects exist for large-scale roughness ($a/D_{84} < 1.6$) and $Fr < 1.5$.

Moderate free surface effects were observed for intermediate-scale roughness ($1.6 < a/D_{84} < 4.0$) and $Fr < 1.5$, whereas at small-scale roughness ($a/D_{84} > 4.0$) free surface effects are negligible at all (Flammer et al. 1970; Bathurst, 1982b; Bathurst, 1985). The occurrence of hydraulic jumps significantly affects the drag coefficient (Flammer, 1970). Due to free surface distortions, the entrainment of air can be significant (Bathurst, 1982b).

The local free surface drag and its effect on the resistance to flow can be described with the Froude number, in relation to the relative roughness (Peterson, 1960; Flammer et al. 1970; Colosimo et al. 1988). However, the mainstream Froude number can be affected significantly by the resistance to flow. This implies a careful formulation of local effects such as the influence of free surface drag on the mainstream friction factor (Bathurst, 1982).

3.3.3. Empirical grain-resistance formulae.

As grain roughness generally dominates the resistance to flow in plane gravel-bed rivers, numerous equations have been developed, generally in terms of Manning's n, Chezy's coefficient C or Darcy-Weisbach's friction factor f (Bray, 1982; Thorne and Zevenbergen, 1985; Chang, 1988). The coefficients described are related as follows

$$\frac{U}{\sqrt{gRi_f}} = \sqrt{\frac{8}{f}} = \frac{R^{1/6}}{ng^{1/2}} = \frac{C}{g^{1/2}} \tag{3.15}$$

The resistance equations commonly used are the semi-logarithmic and the power form (Ferro and Giordano, 1991). Due to phenomena as described in section roughness Bathurst et al. (1981) already mentioned the semi-logarithmic form to be appropriate for small and intermediate-scale roughness, and the power form to be more appropriate for large-scale roughness. In general, the logarithmic velocity distribution has been found to develop for flows with $a/D_{84} > 5$ (Bayazit, 1976; Bayazit, 1982; Jarret, 1990; Maynord, 1991; Pitlick, 1992).

In Table 3.1, some logarithmic resistance formulae are reviewed, that have been derived for coarse-bedded flumes and uniform reaches of mountain rivers.

references (semi-log)	$\sqrt{\dfrac{8}{f}} = A + B\log(X)$			range of validity	nr.
	A	B	X		
Bray (1979)	0.701	6.68	$\dfrac{a}{D_{50}}$	$2.5 \leq \dfrac{a}{D_{65}} \leq 120$	3.16
Hey (1979)	0	15.90	$\dfrac{\alpha_m a}{3.5 D_g}$ (1)	$\dfrac{R}{D_{84}} > 1$ (2)	3.17
Thompson Campbell (1979)	5.66	$-0.566\dfrac{k_s}{R}$	$12\dfrac{R}{k_s}$	$\dfrac{a}{D_{84}} > 1.2$ (3)	3.18
Bathurst et al. (1981)	$14.7\beta_2$ (4)	$1.842\dfrac{B}{D_{84}}\beta_2$	$\dfrac{R}{1.2 D_{84}}$	$\dfrac{a}{D_{84}} > 1.2$	3.19
Griffiths (1981)	2.15	5.60	$\dfrac{R}{D_{50}}$	$1 \leq \dfrac{R}{D_{50}} \leq 200$	3.20
Bathurst (1985)	4	5.62	$\dfrac{a}{D_{84}}$	$0.3 < \dfrac{R}{D_{84}} < 1$	3.21
Bray & Davar (1987)	3.1	5.7	$\dfrac{R}{D_{84}}$	$\dfrac{a}{D_{50}} > 1$	3.22
references (power)	$\sqrt{\dfrac{8}{f}} = AX^b$			range of validity	
	A	b	X		
Bathurst (1978)	β_1 (4)	2.34	$\dfrac{R}{0.365 D_{84}}$	$\dfrac{R}{D_{84}} < 1.5$	3.23
Bray (1979)	3.85 4.19 5.03	0.281 0.276 0.268	$\dfrac{a}{D_i}$	$2.5 \leq \dfrac{a}{D_{65}} \leq 120$	3.24 3.25 3.26
Bray & Davar (1987)	5.4	0.25	$\dfrac{R}{D_{84}}$	$\dfrac{a}{D_{50}} > 1$	3.27

Table 3.1

Note: (1) $\alpha_m = 11.1 \left(\dfrac{R}{a_m}\right)^{-0.314}$ (Bathurst, 1982), (2) Thorne and Zevenbergen (1985),

(3) Bathurst (1983b), (4) $\beta_i = \left(\dfrac{B}{a}\right)^{7(\lambda_i - 0.08)}$; λ_1 = basal area of roughness elements per

m^2 bed, λ_2 = plan area of roughness elements per m^2 bed (Bathurst, 1978).

Bathurst (1978) developed Eq.3.23 based on the roughness geometry in boulder bed rivers, with steep, uniform slopes, and a tumbling flow regime. At the sites examined, the roughness shape and size distribution were approximately constant. The frontal and basal roughness concentration (λ_1 and λ_2 respectively) were related empirically to the relative roughness. Bathurst et al. (1981) suggest Eq.3.23 to be valid for $a/D_{84} < 1.2$, and Eq.3.19 for higher values of a/D_{84}.

In accordance with the powers b (Eqs 3.24, 3.25 and 3concentrations by Bray (1979) from field measurements, Chen (1991) suggests a one-fourth power to be more applicable than Manning's one-sixth power for shallow flows in rough channels.

Hey (1979) used a logarithmic resistance formula for gravel beds (Eq.3.17), and reflects the influence of the cross-sectional shape by α_m that ranges from 11.1 to 13.5 (a_m is the maximum depth). For small- and to some extend intermediate-scale roughness, Thorne and Zevenbergen (1985) found a good performance of Eq.3.17. According to Marešova and Mareš (1990), energy dissipation by irregular cross-sections increases at steeper slopes, which indicates that α_m is also related to i.

Based on earlier research (Bathurst, 1978), Bathurst (1985) also suggests a logarithmic formula (Eq.3.21) for slopes ranging from 0.4 to 4 %. Equation 3.21 has been applied with reasonable performance by Codell et al. (1990) for flow over permeable, rock-covered slopes.

Thompson and Campbell (1979) propose a semi-logarithmic formula (Eq.3.18) on the basis of observations of flow on a boulder bed spillway. They used k_s, which is a characteristic roughness size of the bed material defined as

$$k_s = \mu D_{50} \tag{3.28}$$

where μ ranges from 1.5 to 8.2. According to Bathurst (1983b), this formula can not be applied for flows on large-scale roughness elements, but Thorne and Zevenbergen (1985) found an equal, reasonably good performance of Eq.3.23 and Eq.3.18 for R/D_{84} less than unity.

Bathurst et al. (1981) tried to describe the sources of energy dissipation explicitly, and

expressed f as a function of R, Fr, the effective frontal roughness concentration λ_f and the effects of the non-uniform channel geometry. However, consideration of those parameters did not yield a better result compared to other, more simple equations (Thorne and Zevenbergen, 1985).

3.3.4. Micro-scale form losses.

Form resistance at bed form-scale is present in rivers when the bed surface is mobile and deformed by sediment transport, or when the bed is rigid and covert with inert bed forms inherited from previous high stage flows (Griffiths, 1989). Consequently, concerning the effect of channel geometry on the resistance to flow, two cases can be distinguished; a static bed, and a mobile bed. According to Shen et al. (1990) form drag depends on the steepness and height of the bed form, relative to the depth of flow. If no separation of flow from the bed occurs, the contribution of changes in the cross-sectional area to energy losses is low (Kazemipour and Apelt, 1983).

Parker et al. (1982) and Pitlick (1992) observed that shear stresses τ in gravel-bed rivers can be considered small relative to the critical shear stress τ_c of the particles. In field measurements under conditions of intense bed-load of gravel, Pitlick (1992) observed the lower-stage plane bed upto approximately $\tau/\tau_c = 3$. The formation of relatively flat micro-scale bed-forms like dunes and antidunes does not significantly affect the resistance to flow (Bayazit, 1982; Bathurst et al. 1983; Smart and Jaeggi, 1983). Data presented by Pitlick (1992) and Dinehart (1992) indicate that at stages of low transport, the equivalent roughness k_s of a plane gravel bed with sediment irregularly moving over it, is proportional to a representative size of the stationary, coarser bed material.

Form resistance becomes significant for approximately $\tau > 3\tau_c$ (Griffiths, 1989). In flume experiments, Bathurst et al. (1983) observed wide variations in Froude numbers due to a varying resistance of the flow caused by sediment movement and bed-form regimes (Fig.3.4.). Due to high bed load rates and bed forms, the roughness height can increase significantly (Dinehart, 1992). In shallow flows and coarse sediments, De Jong and Ergenzinger (1992) report turmoil patterns at the water surface during passage of bed-load pulses, indicating an increase in turbulent energy dissipation.

Pitlick (1992) observed the development of aggregates of particles (clast jams, gravel sheets, bed forms and channel bars) induced by a more continuous bed-load at $\tau/\tau_c > 3$. Those very irregular, but steep ($\Delta_b/\lambda_b > 0.03$, where Δ_b is the height and λ_b the length of the bed form) gravel bed forms, with amplitudes up to 20% of the depth of flow add to the flow resistance. For equivalent roughness height, Pitlick (1992) suggests $k_s = \Delta_b$.

For a non-dimensional shear stress $\tau_{*50} = 0.056$, Griffiths (1981) assumes beds to be mobile and suggests the resistance to flow to be predicted with a mobility number

$$\frac{1}{\sqrt{f}} = 2.21 \left(\frac{U}{\sqrt{g D_{50}}} \right)^{0.340} \tag{3.29}$$

However, calibration data showed significant scatter due to effects of bed-form geometry and sediment dynamics which are not accounted for in Eq.3.29.

From experiments with slopes ranging from 3 % to 20 % (Smart and Jaeggi, 1983), Griffiths (1989) proposed for the ratio of form resistance τ' to grain resistance τ of flows with $\tau_* > \tau_c$, $1.09 < Fr < 2.94$ and $5 < a/D_{84} < 25$

$$\frac{\tau_{*f}}{\tau_{*g}} = \frac{\tau_* - \tau_{*g}}{\tau_{*g}} = \exp\left[-0.066 \tau_*^{-0.455} \left(\frac{a}{D} \right)^{1.30} \left(\frac{\tau_* - \tau_{*c}}{\tau_{*c}} \right) \right] \tag{3.30}$$

In the upper stage plane bed, at extreme high shear stresses ($\tau/\tau_c > 3$), Yalin and Karahan (1979), Wilson (1987), Griffiths (1989) and Pitlick (1992) suggest the bed surface to be protected by a layer of moving grains. Wilson (1987) proposes the dimensionless shear-stress, τ, required to set this "grain carpet" in motion, to be equal to $C_b \tan\phi'$, where C_b is the loose poured value of concentration in the sheared layer, and ϕ' is the dynamic friction angle of the solids.

This limits the grain resistance to the critical shear stress (Fernandez Luque and Van Beek, 1976), and increases the proportional contribution of the form resistance to the total resistance to flow (Griffiths, 1989). Pitlick (1992) found the equivalent roughness k_s to be represented by $k_s = 3D_{84}$, using the diameter D_{84} of the moving bed-load material.

3.3.5. Macro-scale form losses.

Macro-scale form losses can be caused by cross-sectional variations as contractions and expansions, macro-scale bed forms, bends and secondary flows. In the case of extreme roughness and shallow depths, the development of secondary flows is limited (Bathurst, 1982b). Form losses from macro-scale bed forms can be considered small for "channel forming" flows (Jaeggi, 1983; Smart and Jaeggi, 1983; Bathurst and Cao, 1986). Although some variability remains, the flow could be considered uniform at higher flows, when the bed forms are entirely submerged (Hey, 1988).

If significant sediment movement occurs, hydraulic resistance increases due to the development of large-scale bed forms as bars (Bathurst et al. 1983; Colosimo et al. 1988). Jaeggi (1983) found the grain resistance formula Eq.3.17 to perform reasonably well in flume experiments with depths that are large relative to bed deformations. Form resistance due to macro-scale bed forms becomes more substantial with steeper

slopes and larger relative roughness (Jaeggi, 1983).

For low flows, bed forms significantly affect the water surface profile; the flow is non-uniform and riffle-pool sequences can occur (Parker and Peterson, 1980; Jaeggi, 1984; Miller and Wenzel, 1985; Higginson and Johnston, 1988; Griffiths, 1989). In this case, the velocity and depth of flow are controlled by bed deformations and the flow may even experience different flow regimes, like sequences of chutes and pools (Egashira and Ashida, 1989). With a static bed, flow patterns over macro-scale bed forms control the dissipation processes of turbulence-induced eddy structures, boundary shear, contraction and expansion of the flow and hydraulic jumps (Egashira and Ashida, 1989).

As has been reviewed by Hey (1988), the contributions of friction and form losses have been distinguished and expressed in shear stress, depth or hydraulic radius, roughness height or slope. Jaeggi (1983) considered form losses by distinction of different friction slopes for grain friction i_g and form losses i_f. Assuming the total slope to be $i_{tot} = i_g + i_f$, the friction factor was defined as

$$\sqrt{\frac{8}{f}} = \frac{u}{u_*} = \frac{u}{(gai_g)^{0.5}} = \frac{u}{(gai_{tot}(1-i_f/i_{tot}))^{0.5}} \qquad (3.31)$$

For the relation i_f / i_{tot}, Jaeggi (1983) suggested

$$\frac{i_f}{i_{tot}} = \exp\left(-\xi_1 \frac{(a/D_{90})^{\xi_2}}{i^{\xi_3}}\right) \qquad (3.32)$$

In the experiments, wherein variable slopes were investigated only, Jaeggi suggested $\xi_2 = 1$ and $\xi_3 = 0.5$, and subsequently a form loss-corrected version of Eq.3.17

$$\sqrt{\frac{8}{f}} = 2.5\left(1 - \exp\left(-\xi_1 \frac{a}{D_{90}i^{0.5}}\right)\right)\ln\left(12.27\frac{a}{\beta D_{90}}\right) \qquad (3.33)$$

The flume experiments were carried out with approximately $2 < i < 12$ % and $4 < a/D_{90} < 30$. The empirical parameters ξ_1 and β were considered related to the shape, composition and size distribution of the bed sediment.

3.4. Non-uniform flows.

The flow in mountain rivers can be non-uniform due to non-uniform bed roughness or channel geometry, varying hydraulic parameters in a steep hydrograph or lateral inflow or outflow (infiltration). The non-uniformity of the flow, which can be either

accelerating or decelerating, affects the structure of the flow. The structure of the turbulent shear flow is related to the irregular, ill-defined magnitude and arrangement of the roughness elements. Consequently, the description of velocity and shear stress is complicated.

The resistance to flow varies during a flood hydrograph, and according to Tu and Graf (1990) significant deviations from values predicted by uniform flow formulae can occur. Assuming kinematic waves, Tu and Graf (1990) propose the deviation of shear velocity in unsteady flows from that in uniform flows proportional to the unsteadiness of the flow, and inversely proportional to the bottom slope, mean velocity and Froude number. For unsteady flows, Tu and Graf (1991) suggest the non-linear shear stress distribution to be composed of a contribution from bed slope (similar to uniform flows) and time variation of water depth and point velocity.

In flows with $Fr < 1.5$, Tu and Graf (1990; 1992) and Song et al. (1991) found the point velocity and time-averaged shear velocity to be larger in accelerating flow than shear velocities in decelerating flow. Measurements by Marešova and Mareš (1991) are confirmative. Song et al. (1991) and Tu and Graf (1992) found Eq.3.1 and 3.2 to be valid for the prediction of velocity distributions in unsteady flows. In decelerating flows, the turbulence develops, the wake-strength coefficient increases (Vedula and Achanta, 1985; Kironoto and Graf, 1990).

The response of flow to abrupt changes in bed roughness has been investigated under simplified conditions. Nezu and Nakagawa (1991) carried out experiments with subcritical flow and relative small roughness. In the case of a transition from rough to smooth, the bed shear-stress experiences a minimum value, which has been defined as the undershooting of the bed shear-stress.

This over- and undershooting downstream the change in roughness has also been recognized in the variations of turbulence intensities and Reynolds stresses (Nezu and Nakagawa, 1991). If bed roughness changes abruptly from smooth to rough, the velocity will be retarded within the reach of the new, developing boundary layer (Nezu and Nakagawa, 1991). Immediately downstream of the change, the bed shear stress experiences a maximum value, the so-called overshooting of the bed shear stress.

3.5. Modelling of flows.

In the case of steep, shallow flows, changes in geometry or flow conditions can affect the flow significantly. Therefore, the structure of flow should be described by considering local effects of geometry. But geometry can beHowever,tremely irregular and generally the characteristics of local geometry that affect the hydraulic behaviour of the flow are averaged over a reach, implicitly described in flow-resistance formulae or lumped into empirical coefficients.

In analogy with a stage-discharge curve, Kellerhals (1970) empirically derived a steady flow equation relating the discharge Q to the channel storage per unit length A. Kellerhals (1970) proposed

$$A = A_d (\frac{Q}{Q_d})^b \qquad (3.34)$$

where A_d the average flow area at the "formative" discharge Q_d. In Eq.3.34, the effects of channel geometry and slope are implicitly described. The empirical constant b ranged from 0.28 to 0.54 at the steeper channels.

In a tumbling flow regime, Kellerhals (1970) observed a distinct and continuous flattening of wave fronts in surge tests, indicating strong dispersive effects. Kellerhals (1970) suggested dispersive effects to result from storage in pools, between the supercritical flow at the riffles. To describe the translatory waves through the section investigated, Kellerhals defined control points at x_i, where a single stage-discharge relation can be described

$$Q = f(A)_{x_i} \qquad (3.35)$$

With the help of the mass conservation equation for liquid

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0 \qquad (3.36)$$

Kellerhals defined a stage-discharge relation at point $x = x_i - \Delta x$

$$Q(x) \approx Q(x_i) - \Delta x \frac{\partial Q}{\partial x} = f(A)_{x_i} + \Delta x \frac{\partial A}{\partial t} \qquad (3.37)$$

And, after substitution of Eq.3.37 back into Eq.3.36, Kellerhals approximates for a long channel with densely spaced control points

$$\frac{\partial A}{\partial t} + \frac{\partial f(A)_{x_i}}{\partial A} \frac{\partial A}{\partial x} + \frac{\partial^2 \Delta x A}{\partial x \partial t} = 0 \qquad (3.38)$$

However, in the kinematic approach used to derive Eq.3.38, dynamic effects which are characteristic for steep hydrograph flows are neglected. Therefore, although originally derived for description of unsteady flows, the applicability of Eq.3.38 for routing of flows can be considered limited to steady flows. Bren and Turner (1978) also used a kinematic wave to describe hydrographs but predictions were found to deviate from measurements.

Miller and Wenzel (1986) considered steady, non-uniform flow through riffle-pool

sequences. Assuming a hydrostatic distribution of pressure and neglecting dead zones, the conservation of momentum yields

$$\frac{\partial}{\partial x}\left(\alpha\frac{Q^2}{A}\right)+g\overline{A}\frac{\partial h}{\partial x}+g\overline{A}i_{tot}=0 \tag{3.39}$$

where $\overline{A}$ the reach-averaged cross-section area. The overall, energy head loss per longitudinal unit length, i_{tot}, was assumed to be composed of grain friction losses and local losses

$$i_{tot}=\frac{f_g u_r^2}{8gR_r}+c\frac{u^2}{2g} \tag{3.40}$$

where c is an empirical coefficient.

The local energy losses by expansion and contraction were analysed by considering the measured i_{tot} and a predicted grain resistance. According to Miller and Wenzel (1986), the local losses appear to contribute significantly to i_{tot} in riffle-pool sequences under low flow conditions. As the discharge increases, the significance of local losses decreases. In subcritical flows, the hydraulic parameters related to flow depth were found to increase in pool sections and decrease through riffle sections, while the velocity related parameters exhibited an opposite sequence. At higher flows, the non-uniformity in hydraulic parameters decreased.

In reality, the response of the flow to bed forms depends on the interaction of grain- and bed-form roughness. However, if the free surface profile of the flow is controlled by riffle sections in a channel exhibiting bar forms or step-pool structures, the depth and velocity of flow at the shallowest section of the riffle will depend on local grain roughness. According to Hey (1988), this enables the decoupling of grain and form resistance. He defined the friction factor f_r at the riffle section with

$$f_r=\frac{8ga_r i_r}{u_r^2} \tag{3.41}$$

From discharge continuity

$$Q=uBa=u_r B_r a_r \tag{3.42}$$

where u, B and a are the reach-averaged values of velocity, width and depth respectively and the subscript r refers to similar, local parameters at the riffle section.

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Substitution and rearrangement of Eqs 3.41 and 3.42 yield

$$\frac{f_r}{f} = \frac{a_r^3 B_r^2 i_r}{a^3 B^2 i}$$ (3.43)

which can be formulated with Eq.3.17, and solved for D_t

$$D_t = \alpha a \left(\frac{D_g}{\alpha_r a_r} \right)^{(f_r/f)^{1/2}}$$ (3.44)

Hey (1988) assumed the total, reach averaged roughness height D_t to be composed of linear contributions of grain and bed form roughness heights, respectively D_g and D_b, and formulated with Eq.3.44

$$D_b = \alpha a \left(\frac{D_g}{\alpha_r a_r} \right)^{(f_r/f)^{1/2}} - D_g$$ (3.45)

Egashira and Ashida (1989) modelled the structure of flow in step-pool sequences with step heights Δ by considering seperately the enerseparatelytion processes in a reach (eddy structures, grain friction losses and, if present, hydraulic jumps). The grain friction f_g was assumed the dominant energy dissipative factor on the riffle section between the reattachment point and the wave crest (Fig.3.5).
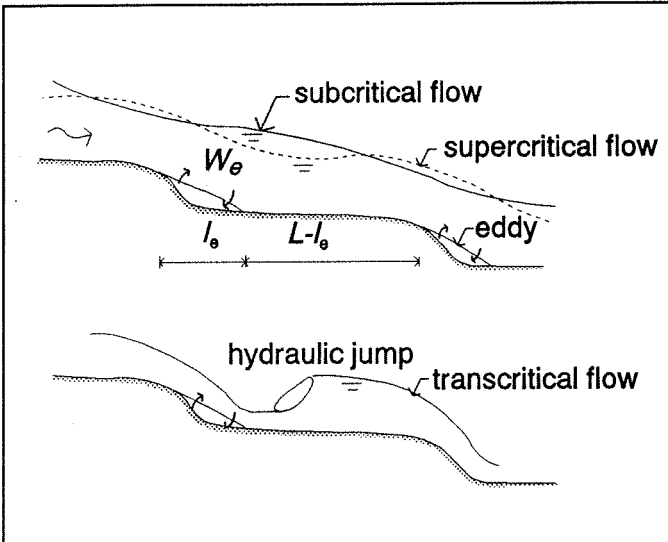
Figure 3.5. Flow over a riffle-pool form (Egashira, 1989).

In the region l_e between the crest and the reattachment point, the energy dissipation is described as

$$f_e = 4KE_n \frac{\Delta}{a} \qquad (3.46)$$

where f_e is the friction factor in the region l_e, a the reach-averaged depth and K and E_n empirical coefficients regarding eddy structure and entrainment from the order 6.0 and 0.08 respectively.

Without considering bed geometry and flow structure, Rajaratnam (1990) reviewed $c_f = 4f_e$ for scale models and prototypes of relatively uniform flows skimming over the discontinuities in step-spillways, and mentioned values of f_e indicatively ranging from 0.20 to about 0.72 at lower flows.

Averaged over the reach L, the friction factor f is

$$f = \frac{l_e}{L} f_e + (1 - \frac{l_e}{L}) f_g \qquad (3.47)$$

In case of a hydraulic jump, Egashira and Ashida (1989) suggested an additional energy loss;

$$\frac{8}{Fr^2} \frac{\delta H}{L} \qquad (3.48)$$

where δH is the energy head loss by the hydraulic jump.

Zuo et al. (1986) and Li et al. (1992) accounted for non-uniform flow in a varying geometry by using stochastically varying parameters. Furbish (1992) used a linear spectral sis for perturbations to describe flow over a complex topography and found the flow pattern to be affected by the random, long wave bed deformations. Short term variations in geometry cannot be considered, because equations used are based on gradually varying flow conditions, wherein the turbulent fluctuations are eliminated by time-averaging, and the pressure distribution is hydrostatical.

3.6. Flows over permeable beds.

3.6.1. Flow structure.

In most gravel streams, the bed surface layer is relatively permeable. In ephemeral rivers with permeable beds, infiltration losses at the rising part of the hydrograph can be considerable (Lane, 1982). Because the infiltration rate can vary with time and place, modelling of leakage is complex (Schropp and Fontijn, 1989; Boers, 1990).

Measurement data are scarce and, due to the complexity of the process, prediction formulae are rather empirical.

Zagni and Smith (1976) described two concurrent frictional mechanisms: (i) a finite slip velocity that decreases the hydraulic resistance and (ii) a lateral transport of momentum that increases the hydraulic resistance. Zippe and Graf (1983) found the velocity profile to be in accordance with the velocity-defect prediction law. Gupta and Paudyal (1985) observed the slip velocity at the bed and suggested a lowering of the hypothetical zero-velocity bed-level to preserve a logarithmic velocity-profile.

At turbulent flows and beds with increasing permeability, the exchange of mass and momentum between subsurface and surface flow dominates (Richardson and Parr, 1991), resulting in increased turbulent shear-stresses (Mendoza and Zhou, 1992). Nezu (1977), Zippe and Graf (1983) and Gupta and Paudyal (1985) observed an increase in friction factor f of flows over permeable rough beds with increasing Reynolds number and relative thickness of the permeable layer.

At small-scale roughness in uniform conditions of flow, Nezu (1977) found the contribution of shear stress by seepage disturbances relative small, but suggested the interaction between main and seepage flows to increase at larger roughness scales. In flume experiments with suction through the bed, Nezu (1977) observed a reduction in turbulence intensity in the inner region.

Consequently, at permeable beds, the structure of the flow and the bed-load transport process are affected. Primarily at low flows, the leakage of flow into a permeable bottom can significantly degenerate the structure of flow and affect the threshold conditions for particles on the bed surface (Nakagawa and Tsujimoto, 1984).

3.6.2. Subsurface flow.

Apart from leakage, a permeable bed enables a subsurface flow in the alluvial infill. Field measurement results by Castro and Hornberger (1991) suggested two modes of interactions between surface and subsurface waters in a alluviated mountain stream. The first interaction concerns short-term storage as a result of rapid mixing of water and solutes through the cobbles and gravel in the stream bed, and a slow mixing with relatively stagnant surface pools. Secondly, long-term storage can occur as a result of water and solutes moving in and out the extensive alluvium that surrounds the active stream channel. Tuzova and Filin (1990) used a natural radioactive indicator for measurements of subsurface flow.

Iwasa and Aya (1987) investigated the effects of flow over a permeable bed on the convection and dispersion of tracer clouds, and distinguished layers in the bed with immediate, delayed and no flow interactions. These layers have been defined as mixing layer, subsurface flow and impermeable bed layer respectively. Analogous to

Eq.6.10 in Section 6.3, but without considering dead zone effects, the convection and dispersion processes were described for uniform, steady flow. The concentration flux between the layers was assumed proportional to the differences in layer-concentration.

Nezu (1977) suggested the seepage flow to be driven by pressure fluctuations in the main flow. Zagni and Smith (1976) and Codell et al. (1990) assumed the subsurface component of the flow to be driven by pressure gradient and fluid shear-stress τ. Zagni and Smith (1976) suggested for steady flow

$$-\frac{\partial P}{\partial x}+\frac{\partial \tau}{\partial z}-f_d=0 \qquad (3.49)$$

where P the pressure and f_d the drag force per unit volume of bed matrix. The shear stress τ is related to the state of flow, which can change from turbulent to laminar (Nezu, 1977).

Although experimental verification is complicated, the effect of leakage on the surface flow structure has been analysed by for example Nakagawa and Tsujimoto (1984) and Nakagawa et al. (1988).

Nakagawa and Tsujimoto (1984) assumed the velocity in permeable layers of the bed to be composed of a uniform part according to the linear Darcy law and an exponential part by momentum exchange with the free surface flow via viscous and turbulent shear stresses.

In steady flow conditions, Nakagawa et al. (1988) neglected the viscous stresses and assumed the bed shear-stress equal to the Reynolds-stress, corrected for the leakage effect

$$\tau_0=-\rho \overline{uv}-\rho v_0 U \qquad (3.50)$$

where τ_0 bed shear-stress, ρ density of water, u and v respectively the longitudinal and vertical components of velocity fluctuation, v_0 leakage velocity through the permeable bottom, and U the cross-sectional averaged velocity.

Stevens (1988) and Rahuel et al. (1989) considered two cross-sectional areas: a liquid flow area A, and an alluvial bed area A_b, with bed-material porosity p. This yields a total cross-sectional area A_c composed of

$$A_c=A+pA_b \qquad (3.51)$$

The resulting conservation equations of respectively water and sediment were formulated as

$$\frac{\partial(A+pA_b)}{\partial t} + \frac{\partial Q}{\partial x} = q_l \qquad (3.52)$$

$$\frac{\partial((1-p)A_b)}{\partial t} + \frac{\partial S}{\partial x} = s_l \qquad (3.53)$$

with q_l and s_l the lateral inflow of liquid and sediment.

Stevens (1988) assumed the momentum contribution of the subsurface flow neglectable. Rahuel et al. (1989) modelled λ to be independent of time. Krishnappan (1985) and Lyn (1987) originally derived similar equations but included the change in sediment volume concentration C in the water layer in Eq.3.53

$$\frac{\partial((1-p)A_s)}{\partial t} + \frac{\partial CA}{\partial t} + \frac{\partial S}{\partial x} = s_l \qquad (3.54)$$

Chapter Four.

Morphology.

4.1. Introduction.

In contrast to low-land rivers, the geometry of a mountain river is significantly determined by features of the catchment area. External factors that determine the morphological developments of a mountain stream concern form and geologic constraints of the river reach and hydrological, geological, topographical, ecological and cultivation characteristics of the catchment area. Li and Simons (1982) indicated morphological changes of mountain rivers to result from changes in discharge, bed material, bed-load and wash-load sediment-transport, viscosity, seepage forces, vegetation and plan form.

Morphological responses of a mountain river concern changes in plan form, channel geometry and bed composition. The morphology of mountain rivers can range from irregular, relatively stable bed rock streams with large sediments and small depths to sections of relatively unstable, braided or meandering alluvial rivers with smaller sediments and larger depths. In this chapter, different geometrical features of mountain rivers at different scales will be described.

4.2. Morphological features.

4.2.1. Step-pool sequences.

In steep, narrow mountain streams, the longitudinal profile usually has a stair-case like appearance. Sequences of alternating flatter and steeper reaches can be observed, which are called steps and pools (Scheuerlein, 1973). The steps are formed by regularly arranged boulders and large-sized particles and are about the mean particle size of the armour coat (Egashira and Ashida, 1989). The formation of these structures is associated with flow conditions during low-frequency floods. Therefore, the step-pool sequences can be considered stable under "normal" flow conditions.

Schälchi (1991) distinguished six types of morphology in mountain streams ("Gebirgsbächen"), at increasing slopes and particle diameters.

1). Uniform geometry.
The first type shows a rather regular slope (1.5 to 5 %), with a homogeneously arranged particle size (D_{max} about 0.5 to 0.7 m), and relatively uniform flow conditions.

2). Riffle-pool sequences.
The second type of morphology occurs at slopes of 1.5 to 7 %, and exhibits clusters

of the larger particles (D_{max} about 0.6 to 0.9 m) arranged over the width of the flow similar to alternating bars. According to Schälchi (1991), the velocity head at the riffle sections equals about $0.7D_{max}$. The morphology induces a sequence of riffles and pools. At higher flows, the riffle-pool sequences disappear.

3). Step-pool sequences.
This type occurs at slopes of 3.5 to 12.5 %. Similar to the riffles in the second type, steps are formed by the larger particles (D_{max} about 0.9 to 1.2 m) that are regularly arranged over the cross-section. The step height is about D_{max}. Due to this relatively large step height, the step-pool sequences can remain at higher floods.

4). Boulder steps-pool sequences.
This type can be observed at slopes ranging from 9 to 30 %. The boulder steps are formed by 1 to 3 different layers of the larger particles (D_{max} about 1.1 to 2.0 m). The water flowing over the steps is usually distributed extremely non-uniform over the width. In the pools, where velocities are low, the width of the flow is larger. The cascades remain at higher flows.

5). Rounded boulder glides.
This type occurs in mountain streams at slopes of 12 to 35 % and is dominated by rounded boulders (D_{max} about 1.3 to 2.0 m), arranged homogeneously over the bed. As a result, the longitudinal profile is relatively regular. The finer particles in the bed material are sheltered by the larger ones. At low flows, the water moves as subsurface flow between the boulders. Based on the roundings of the particles, Schälchi (1991) assumed movements of the boulders at high flows.

6). Sharp-edged boulder glides.
This type can be recognized in mountain streams with slopes of 29 to 49 %. The boulders (D_{max} about 2.5 to 5.0 m) exhibit sharp edges, which indicate small distances of transport. No significant arrangement due to fluvial processes can be recognized. Similar to flows in rounded boulder glides, water flows between boulders in a turbulent state. Schälchi (1991) considered this type of creek as a transition of weathered fixed and mobile rocks.

Whittaker and Jaeggi (1982) review some theories on the origin of step-pool systems in mountain streams;

-a dispersion and sorting theory that suggests the step-pool sequences to represent an "optimum dissipation system of stream energy",
-velocity reversal in supercritical flows that predicts lower velocities at the steps and larger velocities in the pools. This assures deposition of the larger particles on the steps and thereby stabilization of the step-pool structures (Jackson and Beschta, 1982)
-an antidune theory of origin, that associates the origin of these morphological structures with the formation of antidunes in combination with armouring of the bed

surface.

Flume experiments by Whittaker and Jaeggi (1982) indicate the bed-deforming process to be basically the same as the production of antidunes. However, the formation of coarse armour layers on the surface of the bed disturbs the regularity of the process. Egashira and Ashida (1989) summarized three conditions that determine the occurrence of step-pool structures; a rapid flow regime ($Fr > 1$), non-uniform bed material and armouring or paving of the bed surface.

4.2.2. Riffle-pool sequences.

Riffle-pool sequences in channels are often associated with bar formation and meandering (Whittaker and Jaeggi, 1982). At low flows, alternating bars can affect the uniformity of the flow and induce the development of riffle-pool sequences. The riffles formed by fluvial processes are generally diagonal (Jaeggi, 1984). The pools are generally in phase with channel bends and the riffles with crossings (Chang, 1986). Erosional chutes can be formed along the bar edges at steep, downstream fronts. The spacing of riffle-pool sequences commonly corresponds to approximately 5 to 7 times the channel width (Whittaker and Jeaggi, 1982).

Apart from sequential scour and deposition along the channel, riffles can be formed by non-fluvial features such as exposed bed rock, rockfall or landslide debris. In steeper channels with larger particle sizes, riffle sections can develop steps by accumulated boulders. Experiments by Whittaker and Jaeggi (1986) with block ramps partially fixing the bed, showed the development of a series of sills during the stabilization of a degrading slope, due to embedding of the blocks. Often, selective scour initiates the formation of armour layers on riffles.

During small storm flows, the shear stresses at the riffles or steps are larger than in the pools (Lisle, 1979; Miller and Wenzel, 1985). In pool sections, flow decelerates and relatively fine sediments are stored. If threshold conditions for entrainment of coarser particles on the surface of the bed are not exceeded, an armour layer is formed on the riffle surface (Bhowmik and Demissie, 1982; Church and Jones, 1983; Wesche et al. (1987); Ho, 1988). Jaeggi and Smart (1983) explain the relative stability of riffles by repetitive armouring of the downstream bar fronts during floods. Church and Jones (1983) suggest the stability of bars, or the bar celerity along a channel to be affected by the stability of the anchoring riffles.

Lisle (1986) reports two mechanisms that can stabilize the form and location of gravel bars or riffle-pool sequences. The first refers to bends or large obstructions wider than one-third of the channel width, which can cause local scour that terminate upstream bars. In this case the spacing of the bends and obstructions controls the length of the bars. The second mechanism is deposition of sediment upstream of large obstruction or sharp bends, due to backwater effects.

At the rising limb of hydrographs, the bed scours and subsequently coarsens (Bhowmik and Demissie, 1982) in the pool and fills at the riffle (Chang, 1986), while during the falling stage, the riffle scours and the pool fills. At increasing flow depths, hydraulic gradients of riffle-pool sequences tend to equalize. According to for example Jackson and Beschta (1982) and Miller and Wenzel (1985), pool velocities grow larger than velocities at riffles. Coarser particles from the riffle-section that become entrained are deposited on downstream riffle-sections, preserving the riffle-pool sequences.

4.2.3. Relatively unstable river sections.

As described in Chapter Two, the river morphology is very similar to alluvial low-land rivers at low-gradient sediment deposition zones with relatively fine sediments and large depths. Instability of bed level and plan form can occur, resulting in bar formation and braiding or meandering river plans. Briefly, some major morphological aspects are reviewed.

In response to the amounts of sediment and water supplied to the river, the plan form of mountain rivers can change rapidly from straight to braiding at sediment deposition zones (Li and Simons, 1982; Newson and Leeks, 1987). Channels are unstable and vary spatially and temporally due to non-linear interactions between channel morphology, discharge and sediment transport.

Macro bed forms include bars and riffle-pool structures. Bars can be defined as relatively large bed forms at the scale of the channel width. Seminara (1989) distinguishes two classes; free bars, spontaneously developing as a result of an instability of the channel bottom or imbalance of sediment discharge, and forced bars, arising from some physical constraint imposed on the channel.

Free bars include single or alternating and multiple row bars. Alternating bars are characteristic of relatively straight channels, and multiple row bars often feature as braiding rivers. According to Church and Jones (1982), bars produced by river bed deformation constitute important flow resistance elements, but store relatively little sediment. Forced bars can be produced by confluences of rivers (tributary bars), curvature (point bars) and width variations.

Church and Jones (1982) distinguish two important functional criteria for classification of gravel bar features: hydraulic resistance and sediment storage. Sediment accumulates as macro-scale bed forms such as bars develop. The height of the bed form affects the structure of flow and subsequently the bed-load velocity. Therefore, the formation of bed forms may slow down the responses of channel morphology to varying flow conditions. However, although the local scouring or accumulation responsible for bar formation causes local variation in sediment transport, it lacks noticeable effect on the long-term average transport rate (Jaeggi, 1986).

In sections with active sediment yields, braiding can take place at low flows, whereas during floods the channels may disappear again. Bars can develop and protrude beyond the free surface and bifurcate the channel. Diversion of flow induces a shift in channel pattern. The smaller channels disappear or join into a single stream. If a channel takes a location along the front edge of a bar, it converges and tends to be stable. If a channel forms on the bar, it tends to be unstable and braided, due to local deposition of sediment. The process of channel branching repeats in time and space.

Conditions of flow are extremely non-uniform and typical confluence-scour-bar sequences are experienced. Because adaptation lengths of suspended and bed load to non-uniform bed features differ significantly, both modes of transport should be considered separately in the analysis of unstable, braided rivers (Fredsøe, 1978). If sediment transport occurs mainly as bed load, Engelund and Fredsøe (1982) considered the contribution of shear stress to the lag, and subsequently to instability of the bed to be dominant.

The time development of a bifurcation point is related to the flow and sediment distribution at the bifurcation. These distributions of water and sediment cannot be predicted accurately yet. Klaassen and Masselink (1992) distinguish symmetric and asymmetric bifurcations. The first category is characterized by two similar bifurcating channels and an upstream accretion of the bifurcation point. The latter are characterized by a larger and smaller channel, with the smaller one usually less stable.

Bifurcated channels and channels that drain the bar surface meet at confluences. The development of a confluence is related to local geometry and morphological behaviour of the channels. Maizels (1988) and Klaassen and Masselink (1992) observed only minor movements of confluences, either upstream or downstream.

In the research on the formation of bars, and subsequently the meandering or braiding of rivers, both empirical and theoretical criteria and predictors have been developed (Jaeggi, 1983; Ikeda, 1984; Engelund and Skovgaard, 1973; Parker, 1976; Fredsøe, 1978; Blondeaux and Seminara, 1985; Struiksma et al. 1985). The effect of gravity on transverse slopes can be considered stabilizing. The balance of the effects described results in a number of braids and a wave number that can be associated with a maximum growth (Colombini et al. 1987).

The linear theory formulates the conditions for incipient bar formation, the bar length, the linear growth rate of perturbations and their wave length and speed under steady conditions (Blondeaux and Seminara, 1985; Struiksma et al. 1985). The criteria that predict the braiding of rivers concern the width to depth ratio. If this ratio exceeds about 60, braiding occurs. If the width is smaller than 8 times the depth, the river will remain straight (Fredsøe, 1978).

The non-linear analysis adds the prediction of the equilibrium value and temporal

behaviour of the bar height (Tubino, 1989). Colombini et al. (1987) used a weakly non-linear stability theory to predict the equilibrium height of the bar. The destabilizing effects result from the phase lags between bed profile and sediment transport in longitudinal and transverse directions.

4.2.4. Alluvial fans.

Where mountain streams debouch from steeper slopes into a valley, the water spreads out and the sediment transport capacity of the flow decreases significantly. The resulting cone-shaped deposition of sediment is defined as an *alluvial fan*. Without any guidance of previous channels, the flow pattern generally has a braided zone with unstable channels (French, 1987).

The channel reach confined by non-alluvial rocks at the head of a fan is defined as a wash. In this wash and in the zone upstream of the fan, there is a single, active channel. Due to deposition and erosion of material, the intersection point of channel and fan moves up and down. Downstream of this point, the flow spreads out and concentrates in secondary channels. This induces the deposition of coarser material. Deposition of material may consequently divert the flow. Where series of fans intersect, an *alluvial apron* is formed, where the flow paths are reasonably parallel.

The formation of an alluvial fan is subjected to the sources and transport of sediment in the contributing basin. Therefore, the fan area and slope have often been related empirically to the contributing drainage basin area (French, 1987). In many cases, the deposition of material by debris flow can be considered a primary source of sediment to the alluvial fan. Consequently, the development of alluvial fans is determined by a number of features concerning the geology, hydrology, topography, sediment yield, etc.

Fans develop on a geologic time scale and exhibit a low, average rate of growth and slow, average changes in flow pattern. However, on an engineering time scale fluctuations can be significant. The interaction between flow pattern and deposition is non-linear. The stability of flow patterns on alluvial fans is affected by tectonic effects, rare events such as large-scale flooding, alternating sequences of depositing debris flows and eroding water flows, topographical heights of locations and climatic changes including precipitation intensity, distribution, etc.

Considering the complex interaction between varying flow pattern and flow conditions, the parameters and/or models used, can be stochastic, deterministic or a combination of both (French, 1987). The probability of flow direction is related to the previous flow direction and the slope in each direction. Jirka and Beer (1989) modelled steady, supercritical sheet flows expanding on alluvial fans with a fixed boundary, and mention the significant effects of transverse gradients.

Dawdy (1979) suggested for the probability of flooding of a point of interest, given a peak discharge q_0 occurs

$$p_{q=q_0}(flooding) = \frac{b}{B_c} \qquad (4.1)$$

where b the width of the channel, B_c the width of the fan contour at the point of interest. The probability of flooding can be described as

$$p(flooding) = \int_{q_0}^{\infty} \frac{b}{B_c} f(q) dq \qquad (4.2)$$

where the frequency curve of peak discharges $f(q)$ has been suggested log-normal (French, 1992).

4.3. Bed geometry.

4.3.1. Characteristics.

In many rivers, the distribution of particle sizes experiences a gap in the range from very coarse sand to fine gravel (Kellerhals and Bray (1971); Parker and Peterson, 1980; Klingeman and Emmet, 1982; Petts et al. 1989). In gravel-bed rivers, the bed typically features as a surface cover that is relatively coarse in comparison with the bulk mixture in the subsurface (Church et al. 1987; Diplas, 1989). This has generally been explained with armouring mechanisms (Subsection 4.3.2.). Wolcott (1988) found the deficiency in size range of river-bed material to correspond with input sources and notices that the size-distribution of input material could also be responsible for this gap.

The distributions of grain sizes in mountain river beds are the product of sediment supply events and previous hydrodynamic conditions. The configuration of the bed and the distribution of particle sizes may therefore provide information on the transporting agent, maturing of the bed, any introduction of new sediment sources, climatological changes such as changes in sea level and the direction of change (Tanner, 1989).

In mountain rivers, also temporary deposits of fine surface sediment can be observed, often non-uniformly distributed over the bed. The sediments stored in the channel are related to previous flow or sediment supply events, and form a well-mixed active layer from which particles can easily be entrained (Borah et al. 1982). At the base of the active layer, a layer of coarser particles, defined as pavement or mixing layer, protects the underlying material.

4.3.2. Armouring or paving

Due to differences in particle mobility in sediment mixtures, the composition of the bed surface changes in response to the shear stress applied. Raudkivi and Ettema (1982) distinguish four subregions in the behaviour of large particles on smaller sized bed material;

1) no particle movement
2) overpassing of larger particles by rolling or sliding modes due to exposure effects that lower the critical shear stress
3) armouring or embedding conditions by entrainment of the smaller particles
4) all grains in motion

The hydrodynamic forces required to initiate and maintain transport of sediment vary with particle size. So, once sediment transport has started, a sorting process in both vertical and horizontal directions can take place (Ribberink, 1982; Deigaard, 1982). This may result in a coarsening of the bed surface (Ribberink, 1982; Parker, 1989; Chiew, 1991) and downstream fining. Apart from selective erosion of finer sediment, particles are rearranged into groups or units (Sutherland, 1987).

Vertical sorting will occur if the shear stresses applied are between the threshold conditions of the least and the most resistant particles in a mixed size sediment. If the threshold shear-stresses of the finer particles in the river bed are exceeded, a selective-erosion or sediment-sorting process is enabled. The bed-load composition is controlled by the availability and mobility of particles at the surface of the bed. Then, the finer particles are removed whereas the larger particles remain immobile. According to Schöberl (1992) the gradation of the initial sediment material should have a minimum geometric standard deviation of about 1.4.

The coarse surface layers can be static if the sediment supply from upstream reaches is vanishing or near vanishing, or mobile in the presence of upstream sediment supply. If in the first case an armour layer has developed after a long vertical sorting process, in absence of upstream sediment supply, the bed is stable and all particles at the bed surface are static. The vanishing bed-load has been constantly finer than the bed material. The coarse, stable surface layer has been defined as *static armour* (Andrews and Parker, 1987). In the second case, the bed-load coarsens and the pavement becomes relatively finer, but frequent movement of particles at the bed surface remains; particles are exchanged continuously between the bed material and sediment discharged. This development has been defined *mobile armouring or paving*.

Parker et al. (1982) defined a pavement as *"a coarse surface layer maintained by successive periods of bed-load transport during which essentially all sizes of the bed material move."* Although at higher flows all available particles move, Parker et al. (1982) and Andrews and Erman (1986) still observed the presence of a coarse

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pavement because motion of particles at the surface of the bed is sporadic. Observations on a mobile bed by Drake et al. (1988), showed a distinction between active and inactive particle populations with small mutual exchange.

Jain (1990) distinguished the two equilibrium conditions of static and mobile armouring by shear stress; at low shear stresses, the bed-load vanishes and becomes zero (static armouring) whereas at high shear stresses, the composition of the bed-load approaches the size distribution of the bed material (mobile armouring). According to Jain (1990), the transition shear stress has been formulated by Chin (1985)

$$\tau_{*tr} = 7.9 \times 10^{-3} \left(\sqrt{\frac{1.8 D_{50}}{D_{max}}} + 1.5 \right)^2 \tag{4.3}$$

At low flows ($\tau_{*m} < 0.08$, Suzuki and Hano, 1992), fine particles are winnowed from the bed no deeper than a few median armour particle diameters (O'Brien, 1987). Without upstream supply, a static armour layer develops. According to Kulkarni (1991), the formation of an underlying filter by winnowing finer particles from underlying material induces a lowering of the coarse surface layer during the development of an armour or paving layer. Although the bed geometry at the thalweg of the stream can be stable at low flows, banks and channel margins can still act as sources and sinks for smaller sized, unstable sediment (Klingeman and Emmet, 1982).

At higher discharges ($0.08 < \tau_{*m} < 0.13$, Suzuki and Hano, 1992) or shifting flow patterns (Sawada et al. 1985), the slight movements of larger particles arranged in clasts, induce sudden and intensive releases of finer particles, yielding series of slug movements of sediment (Jaeggi and Rickenmann, 1987). The armour layer at the bed surface becomes mobile.

If available, more particles move and form small dunes, overpassing the coarser particles, as observed by Ribberink (1982) and Chiew (1991). Klaassen et al. (1986) report the occurrence of bed forms on armoured beds in flume experiments. The ripples observed consisted of fine material winnowed from the underlying sediment. As bed forms move down the channel, changes in flow structure near the bed (Klaassen et al. 1986) or the impact of finer particles (Kuhnle and Southard, 1988) tends to destabilize the larger ones. Coarser particles settled in the troughs of the bed form, and determine the level of the armour layer that reappeared after the passage of the bed forms Klaassen (1986).

At flows with sufficiently high shear stresses, the overall critical shear-stress of the entire sediment mixture in the bed can be exceeded. Then, all particle size fractions are entrained, and the sediment discharge increases rapidly (Çeçen and Bayazit, 1973; Bayazit, 1975; Suzuki and Michiue, 1988; Kuhnle, 1988). At $\tau_{*m} > 0.13$, Suzuki and

Hano (1992) observed a fining of the bed surface due to increased exposure and subsequent mobility of larger particles.

Generally, the differences between the pavement and substratum are subtle (Church et al. 1987) and affected by the sampling method used (Klingeman and Emmet, 1982). This complicates the accuracy of morphological models that incorporate river-bed compositions (Sutherland, 1987).

4.3.3. Effects of armouring.

The characteristics of coarse surface layers are important in the dynamics of mountain streams. The formation of armoured bed surfaces reduces both the amounts of sediment-material delivered to downstream reaches and the magnitude of bed degradation phenomena. Armouring of the bed surface may limit the height and steepness of bed forms and depth of bed-form troughs and the variability in bed-form size (Wilcock and Southard, 1989). Smart and Jaeggi (1983) observed in their experiments that antidunes develop more easily in relatively fine material ($D_m = 4$ mm), but almost disappear in coarser sediments at large transport rates and high relative roughness. Presumably due to the formation of a pavement layer, Chiew (1991) found the development of antidunes to be related to the gradation of the particle size.

The local composition of bed surface layers in mountain rivers responds relative quickly to changing flow conditions or large-scale events supplying large amounts of sediment by land slides or debris flows to the stream. Experiments (Suzuki and Michiue, 1988) and field measurements (Klingeman and Emmet, 1982; De Jong and Ergenzinger, 1992) showed a coarsening of the bed surface during the increasing limb of a hydrograph. The propagation of local armouring processes changes with particle size (Ribberink, 1987; Di Silvio and Peviani, 1989). With respect to changes in bed elevation or slope, the grain-sorting process has been considered quasi-steady (Parker, 1989; Pianese and Rossi, 1989).

Armouring and sorting processes increase the critical shear stress (Egiazaroff, 1965), and consequently produce a slowdown or cessation of bed degradation (Karim and Holly, 1986). Because of the increased critical shear stress, initiation of sediment transport generally occurs at much higher stresses than cessation (Klaassen, 1990). Dinehart (1992) observed that sediment released during rising stage remained in transport relatively large periods after peak river stage.

During the relatively quick disintegration of the armour layer, the armoured section of the bed area decreases and more fine-sized particles are entrained (Mosconi and Jain, 1986).

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Experiments by Suzuki and Michiue (1988) indicated that the critical conditions for the destruction of the armour layer occur if

$$\frac{\tau}{\tau_{cm}} \approx 1.1 \tag{4.4}$$

where τ is the bed shear-stress and τ_{cm} is the critical shear-stress of the mean grain-size of the armoured bed surface. Klaassen et al. (1986) observed a collapse of the armour layer downstream of ripples that deformed the bed at high discharges, due to additional turbulent shear-stress induced by bed forms.

If, at receding flows larger particles become immobile, reformation of the armour layer can be experienced, limiting the transport of smaller material. Then, at sediments with bi-modal size distributions, the bed load exhibits a drastic reduction and the median size of the transported material abruptly shifts from the coarse gravel mode to the coarse sand mode (Klingeman and Emmet, 1982; Church et al. 1991).

Experiments by Raudkivi and Ettema (1985) indicated that local scour-depth at bridge piers can be increased under armouring or paving conditions if fine particles are winnowed through the armour layer, or the armour layer is eroded.

4.3.4. Bed forms.

Bed forms affect the hydraulic conditions and morphological phenomena in rivers. As to the conditions of flow, bed roughness changes during floods due to significant bed deformations (e.g., Foley, 1975; Dinehart, 1992). Morphological research in sediment mixtures is generally limited to flat beds, but bed forms can be expected to affect armouring, paving, entrainment and deposition phenomena (Klaassen, 1990).

With increasing bed shear stress, τ, and Froude number, Fr, the flow-induced bed forms vary from a plane bed to dune bed to transition bed for subcritical flows. For supercritical flow regimes, the progression is from plane bed to antidunes (moving up- or downstream) to plane bed again, after destruction of bed forms, and subsequently to chute and pool formation. Apart from an initial plane bed without sediment movement, Bathurst et al. (1983) observed three types of bed forms in laboratory flume experiments: antidunes, alternate bars and a plane bed with sediment movement.

In rivers with coarse bed material, small-scale ripple, dune and antidune features are generally absent (Hey and Thorne, 1986). This could be explained by the fact that critical shear stresses are rarely exceeded sufficiently in gravel bed rivers (Parker et al. 1982; Pitlick, 1992).

During storm flows, Dinehart (1989) measured the production of rapidly migrating fine gravel dunes and associated changes in velocity and distortion of the free surface to the occurrence and migration of the bed forms. Gravel bed forms can result from longitudinal particle segregation: sediment accretion in coarse gravel clusters, clast jams and bed load fronts. Brayshaw et al. (1983) distinguished two types of clustering, concerning:

-particles entrapped in the train of the obstructing particle, forming a stream-lined tail of the obstructing particle
-particles arrested to form an imbricated cluster on the upstream side of the particle

Cluster processes encourage the deposition of sediment ranging from fine sediment to gravel, pebble and cobble grades. Clustering of interlocking pebbles from larger-sized fractions can induce aggradation upstream, and degradation downstream of this clast jam, resulting in a finer and coarser bed respectively upstream and downstream of the particle cluster (Kuhnle and Southard, 1988). Carling (1990) observed segregation of coarse particles from finer ones by deposition of the coarse bed-load fraction at the downstream fronts of bars where the flow separates from the bed.

Bed-load waves of fine sediments can move as clusters over the bed resulting in rapid alternations of smoother and rougher bed types (e.g., Dinehart, 1992). In a braided river, Ferguson et al. (1989) observed bed-load sheets, transverse stripes of alternating roughness and relatively long longitudinal sand ribbons. At large relative roughness, the free surface distortions can be good indications of the bed geometry and patterns of bed load as reported in field measurements by De Jong and Ergenzinger (1992).

4.3.5. Bed composition.

The composition and structure of the substrate are a function of dynamic interactions between sediment supply, hydrograph form and bed turnover or instability. This can result in quantitative and qualitative changes in bed composition. The armour layer and substrate of gravel-bed rivers provide a sink for both fine sediments such as fine sands and silt particles, and potentially toxic metals (Klingeman and Emmet, 1982).

Regulation of river discharge can reduce the frequency and depth of turnover of the bed, and the associated flushing of fines during flood events (Simons and Simons, 1989). As a result, fine sediments may accumulate progressively within the gravel bed substrate. This accumulation of fines may affect the timing of suspended load in relation to the hydrograph (Petts et al. 1989).

Fine bed-load size fractions can enter the bed by infiltration through the framework, or by deposition along with bed-load. Whether a particle is excluded, becomes trapped near the surface or passes through the base of the bed is determined by the particle size, the ratio of matrix pore size to particle size (Lisle, 1989) and by the flow

shear-stress (Diplas and Parker, 1992).

According to Petts et al. (1989) sediments can be incorporated in the structure of gravel substrates in river beds to depths of more than 30 cm. In observations by Diplas and Parker (1992), the depth of fines infiltrating in the subsurface did not exceed $5D_{90}$. Lisle (1989) found the mass of dry sediment, M, accumulated per unit bed area

$$M = 1.88 (\int_T s_B \mathrm{d}t)^{0.365} \qquad (4.5)$$

where s_B is the bed load per unit of width.

Grain particles of similar size can overlap (be imbricated), or be wedged between adjacent larger particles (vertically infilled), or be clustered around a stable keystone (Laronne and Carson, 1976). If a wide range of particle sizes is involved, the filling of interparticle voids can develop in tight structural arrangements, with difficult dislodgement of particles.

4.4. Modelling of bed geometry.

4.4.1. Transport and storage layers.

Sutherland (1987) reviews and compares models developed for predicting selective erosion, and distinguishes single-step methods, which predict the composition formed, and multi-step methods, which predict the composition after each time step. If the actual coarsening process is modelled, the exchange of sediment size fractions between bed surface and transport should be considered.

Assuming a one-diameter armour layer, Karim et al. (1983) propose the bed to be divided into armoured and mobile sections, in accordance with later observations by Drake et al. (1988). The armoured area A_{ai} occupied by the i-th fraction of stable particles with D_i after degradation of the bed with Δz_b, is described as

$$A_{ai} = c_i \frac{\alpha_2}{\alpha_3 D_i} (1-p) \Delta z_b f_i \qquad (4.6)$$

where c_i is an armouring coefficient describing the arrangement of the stable particles in A_{ai} and p is the sediment porosity.

The geometrical coefficients of the sediment α_2 and α_3 are defined as

$$\alpha_2 = \frac{A_i}{D_i^2} \tag{4.7}$$

and

$$\alpha_3 = \frac{V_i}{D_i^3} \tag{4.8}$$

where A_i and V_i the projected particle surface and particle volume. α_2 and α_3 have been assumed equal at different size fractions.

With Eq.4.6, the change in A_{ai} with degradational armouring can be described as

$$A_{ai}(t+\Delta t) = A_{ai}(t) + \int_t^{t+\Delta t} c_i(t') \frac{\alpha_2}{\alpha_3 D_i} (1-p) f_i \frac{\partial z_b}{\partial t'} dt' \tag{4.9}$$

The total armoured area can be derived by integrating Eq.4.9 over the stable fractions k to N

$$A(t+\Delta t) = \sum_{i=k}^{N} A_i(t+\Delta t) \tag{4.10}$$

However, according to Karim et al. (1983), Eq.4.10 should be corrected for complicated effects of particle arrangement in the armour layer on the process of degradational armouring. If bed forms occur, the armoured part of the bed can be covered. Karim et al. (1983) propose a coefficient α for the correction of A_a, which is empirically related to the height of bed forms, and ranges from zero in a fully active, covered bed to unity for a plane bed without bed forms.

Karim et al. (1983) add another factor to Eq.4.10 that represents the stochastical character of mobility of the armouring particles. In addition, Wörman (1991a) found the critical shear stress or relative particle stability to be affected by the spatial arrangement of armoured area A_a and to increase with an increasing rate of A_a.

Based on the stochastic bed-load transport model proposed by Nakagawa and Tsujimoto (1980) (Eq.5.45, Subsection 5.4.4.), Tsujimoto and Motohashi (1990) described the development of the bed surface composition with the help of n_i, which is defined as the number of particles of the i-th size fraction at the bed surface.

The number of particles n_i in the k-th subreach between $x = k\Delta x$ and $x = (k+1)\Delta x$, can be related to the surface size-fraction f_i as follows

$$n_{ik} = \frac{f_{ik}\Delta x}{\alpha_2 D_i^2}$$ (4.11)

where α_2 is the geometrical coefficient of the sediment defined in Eq.4.7.

In the changes of n_i, Tsujimoto and Motohashi (1990) account for the number of particles that are dislodged (ΔM_{ik}), deposited (ΔQ_{ik}) and exposed or covered by ΔM_{jk} and ΔQ_{jk} with $j \neq i$. For the k-th subreach, they describe the change in n_{ik} as

$$n_{ik}(t+\Delta t) = n_{ik}(t) - \Delta M_{ik}(t) + \Delta Q_{ik}(t) + f_{0i}\left(\sum_{j=1}^{N}[\Delta M_{jk}(t) - \Delta Q_{jk}(t)]\left(\frac{D_j}{D_i}\right)^2\right)$$ (4.12)

where f_{0i} is the particle size fraction i in the substrate. The number of particles dislodged ΔM_{ik} is described as

$$\Delta M_{iK}(t) = n_{ik}(t)p_{sik}(t)\Delta t$$ (4.13)

where $p_{sik}(t)$ is the pick-up rate of size fraction i in reach k. The number of particles deposited, ΔQ_{ik}, has been assumed to be composed of dislodged particles from upstream reaches (ΔM_{ij} with $j < k$) and from bed-load s_{Bi} supplied to the stream. Tsujimoto and Motohashi (1990) suggested for ΔQ_{ik}

$$\Delta Q_{ik}(t) = \sum_{j=1}^{k-1}[\Delta M_{ij}(t)\int_{(k-j)\Delta x}^{(k-j+1)\Delta x} f_{Xi}(\zeta)\mathrm{d}\zeta] + \left(\frac{s_{Bi}(x_o,t)\Delta t}{\alpha_3 D_i}\right)\int_{k\Delta x}^{(k+1)\Delta x} f_{Xi}(\zeta)\mathrm{d}\zeta$$ (4.14)

where $f_{Xi}(\zeta)$ is the percentage of particles dislodged with step length ζ (see Section 5.4.4).

To describe the development of the bed material size distribution and the sediment transport composition, single or multiple layer models have been developed. The layers distinguished (Fig.4.1) differentiate the mechanisms of exchange between sediment transport and storage in the bed material (Ribberink, 1987; Di Silvio and Peviani, 1989; Di Silvio and Brunelli, 1989; Parker and Sutherland, 1990).

Borah et al. (1982) developed a one-layer model and define the mixing zone as the height above the bed that is occupied with bed features. The surface layer in contact with the flow was identified as the active layer. Below the mixing zone, the bed material remains undisturbed. The mixing zone was described as composed of different layers, with each of them assumed homogeneously throughout time.
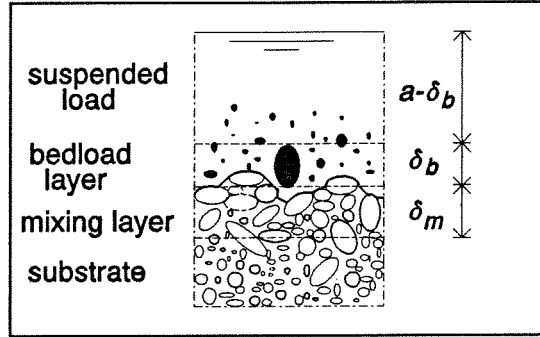
Figure 4.1 Sediment transport and storage layers (Armanini and Di Silvio, 1989).

During the scouring of fine material, the sediment exchange between bed and bed-load takes place in the active layer, which can develop asymptotically into an armour coat. To describe effects of selective entrainment, the thickness of the active layer was defined as

$$\delta_a = \frac{1}{F_L} \frac{D_L}{\sum\limits_{i=L}^{N} f_i} \tag{4.15}$$

where D_L and F_L are the size and fraction of the smallest stable size fraction L at the surface of the bed, and f_i the i-th size-fraction of the bed material.

Ribberink (1987) developed a two-layer model and distinguished a transition or exchange layer between the transport layer and the non-moving bed material as indicated in Figure 4.2. In this transition layer, no horizontal transport occurs. For each sediment size fraction, and for each layer, a sediment balance can be formulated.

Armanini and Di Silvio (1989) distinguish a layer $(a-\delta_b)$ (Fig.4.1) where water and suspended sediment is transported, a bottom layer δ_b where the bed-load transport takes place, a mixing layer or surface layer (Parker and Sutherland, 1989) δ_m where only vertical fluxes are considered, and the substratum with undisturbed sediment. Particles in the substratum are defined to be at rest, until the overlying material is disrupted and the lower boundary of the mixing layer subsides.

The mixing layer has a thickness similar to the roughness of the bed surface. If the bed is flat, δ_m is defined proportional to the coarsest particle diameter (Andrews and Parker, 1987; Petts et al. 1989; Suzuki and Kato, 1989, Di Silvio and Brunelli, 1989; Parker and Sutherland, 1989). According to Rahuel et al. (1989), this definition is valid if the time considered is nearly instantly and particles at the surface are

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exchanged only.

If bed forms develop or if the period considered is taken somewhat larger to enclose the deformation of the bed (Rahuel et al. 1989), δ_m is given by the height of the undulations (Armanini and Di Silvio, 1988). As bed-form heights are generally related to the depth of flow, Rahuel et al. (1989) defined δ_m proportional to the depth a, with δ_m/a ranging from about 0.10 to 0.20. Holly and Rahuel (1989) propose that δ_m can vary in case of persistent erosion or due to the successive lying down of depositional strata during persistent deposition. Rahuel et al. (1989) suggest that if the streambed elevation changes considerably during the period considered, the mixing layer can be thought of as the thickness of the layer eroded or deposited.

If δ_m is very small, numerical instabilities can be induced in the computation of size fractions present in the bed. Then, Armanini and Di Silvio (1988) recommended to use $\delta_m = \delta_b$. Di Silvio and Peviani (1989) define the thickness of the mixing layer δ_m as $\delta_m = 2D_{90}$.

Di Silvio (1992) suggests a four-layer model by distinction of an intrusion layer (subpavement) between the mixing layer (pavement) and underlying parent material, to explain vertical fluxes by occasional, vertical particle movements below the mixing layer.

The definition of δ_b, the thickness of the bed-load transport layer is arbitrary. It can be defined by the height of saltation jumps of bed-load particles (Van Rijn, 1984) or by the roughness height of the bed. In flows with relatively large roughness and extremely graded sediments, the thickness and width of the bed-load pattern are very irregular.

Observations in flume experiments by Smart and Jaeggi (1983) indicate that for flatter slopes, δ_b usually is the same as the maximum grain size of the sediment. For fine sediments, layers of two to three grain sizes thick were observed to be in motion. At extreme high shear stresses, the thickness of the layer occupied by moving particles, and subsequently the effective roughness of particles on the bed surface, is related to the shear stress (Wilson, 1989).

For steeper slopes, the definition of δ_b is complicated. Particles detached from their packing into a saltation motion can travel close to the surface or can even be ejected from the flow (Smart and Jaeggi, 1983). Interparticle contacts affect the fall velocities, and the distinction between suspended and bed-load transport becomes blurred.

4.4.2. Vertical sediment exchange.

In equilibrium conditions, the sediment input equals the output in one reach. Then, although a continuous exchange between the layers in the bed, and the bed-load can

exist, the net vertical flux of sediment will be zero. Under these conditions a a unique relation exists that controls the exchangement mechanisms among the layers defined and the sediment transport (Di Silvio and Brunelli, 1989). In a non-equilibrium state, however, the development of substratum and pavement needs to be described with size-specific sediment mass balances.

For the mixing zone, the sediment balance yields (see Fig. 4.2) (Armanini and Di Silvio, 1988)

$$\frac{\partial \beta_i \delta'_m}{\partial t} = -\Phi_{bi} + \Phi_{oi} \tag{4.16}$$

where β_i is the volume fraction of D_i sized particles in the mixing layer, δ'_m is the instantaneous thickness of the mixing layer and Φ_{oi} and Φ_{bi} are the net flux through the lower and upper boundary of the mixing layer respectively.
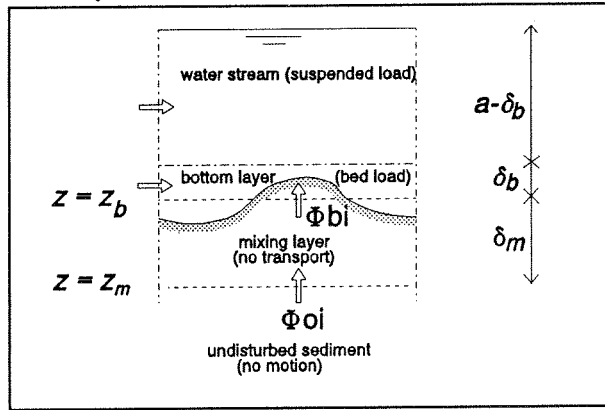
Figure 4.2 Vertical sediment flux in the bed (Armanini and Di Silvio, 1988).

The net flux through the lower boundary layer at $z = z_m$ is

$$\Phi_{oi} = -\beta_i^* \left(\frac{\partial z_m}{\partial t}\right) = -\beta_i^* \left(\frac{\partial z_b}{\partial t} - \frac{\partial \delta'_m}{\partial t}\right) \tag{4.17}$$

where β_i^* equals the percentage of the i-th fraction in the substratum when the lower mixing layer boundary moves downwards, or the percentage in the mixing layer when the boundary moves upwards ($\beta_i^* = \beta_i$).

This would suggest that if $\partial z_b/\partial t = 0$, the net flux Φ_{oi} is zero. However, since the fractions transported in Φ_{oi} through a fluctuating z_b are related to the sign of $\partial z_b/\partial t$, fluctuations in time around the average value of z_b can result in a dispersive transport of sediment.

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Ribberink (1987) defines this dispersive flux Φ_{di} as

$$\Phi_{di} \equiv v_d (\beta_i^* - \beta_i) \qquad (4.18)$$

where v_d a characteristic velocity that basically depends on the average thickness δ_m and the total bed-load transport. Consequently, Eq.4.16 can be written as

$$\frac{\partial \beta_i \delta_m}{\partial t} = -\Phi_{bi} + \beta_i^* (\frac{\partial \delta_m}{\partial t} - \frac{\partial z_b}{\partial t}) + v_d (\beta_i^* - \beta_i) \qquad (4.19)$$

Analogous to Eq.4.19, Holly and Rahuel (1990) suggest a sediment mass balance for the mixing layer extended for two dimensions in horizontal direction. They do not consider the dispersive transport of sediment and distinguish in Φ_{bi} sediment flux contributing to suspended and bed-load.

The sediment mass balance of the bottom layer (see Fig.4.3) has been formulated as (Armanini and Di Silvio, 1988)

$$\frac{\partial S_{bi}}{\partial x} + \frac{\partial C_{bi} \delta_b b_b}{\partial t} = \Phi_{bi} - \Phi_{si} \qquad (4.20)$$

where S_{bi} is the bed-load transport of fraction i integrated over the bed-load width b_b, δ_b is the depth of the bottom layer and Φ_{si} is the net flux through the upper boundary of the bottom layer.
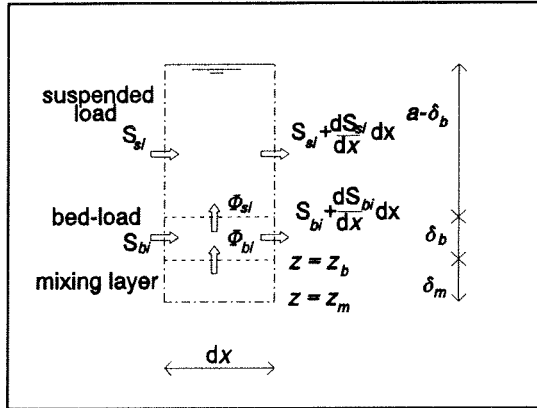
Figure 4.3 Sediment balance.

The transport-averaged bed-load concentration C_{bi} can be defined as

$$C_{bi} = \frac{S_{bi}}{U \delta_b b_b} = \frac{1}{U \delta_b b_b} \int_0^{\delta_b b_b} \int_0^{b_b} c_i(z,y) u(z,y) dy dz \qquad (4.21)$$

where U is the velocity of flow, averaged over the cross-section.

The concentration $c_i(z)$ ranges from β_i at the lower boundary, to $c_i(a)$ at the upper boundary. However, the storage term in Eq.4.20 is negligible in uniform flow and if C_{bi} immediately adapts to equilibrium sediment transport conditions (e.g. Armanini and Di Silvio, 1988; 1989).

In a similar way, the sediment balance for the layer that conveys both water and suspended sediment can be formulated

$$\frac{\partial S_{si}}{\partial x} + \frac{\partial C_{si}(a-\delta_b)b_s}{\partial t} = \Phi_{si} \tag{4.22}$$

where S_{si} is the suspended load integrated over the suspended load-width b_s, and C_{si} is the transport-averaged suspended load concentration. If the total sediment transport of fraction i is defined as

$$S_i = S_{bi} + S_{si} \tag{4.23}$$

the sediment transport S_i, integrated over the layers with bed- and suspended-load, can be written with the help of Eqs 4.20, 4.22 and 4.23

$$\frac{\partial S_i}{\partial x} + \frac{\partial \overline{C_i} A_s}{\partial t} = \Phi_{si} + (\Phi_{bi} - \Phi_{si}) \tag{4.24}$$

where $\overline{C_i}$ is the concentration of the i-th fraction averaged over the cross-section A_s, defined as

$$\overline{C_i} = \frac{1}{Q} \int_{A_s} c_i u \, dA_s \tag{4.25}$$

Elimination of Φ_{bi} in Eq.4.24 with the help of Eq.4.19 yields

$$\frac{\partial S_i}{\partial x} + \frac{\partial \overline{C_i} A_s}{\partial t} = -\frac{\partial \beta_i \delta_m}{\partial t} + \beta_i^* (\frac{\partial \delta_m}{\partial t} - \frac{\partial z_b}{\partial t}) + v_d(\beta_i^* - \beta_i) \tag{4.26}$$

In uniform sediment material $\beta_i = 1-p$, $S_i = S$, the mass balance of sediment in the transport and storage layers would be

$$\frac{\partial S}{\partial x} + \frac{\partial \overline{C} A_s}{\partial t} = -(1-p)\frac{\partial z_b}{\partial t} \tag{4.27}$$

Sediment size fractions should be routed through the morphological model by size-specific volume or mass-balance relations (Armanini and Di Silvio, 1988). For n size fractions, this would add n transport formulae, and n sediment continuity equations to

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the system of equations. If the number of equations is large, the models developed are complicated from a numerical point of view. The models can be simplified by using first, second and higher order moments of the size distribution (Armanini, 1989). However, due to armouring processes, the distribution of particle sizes can be discontinuous, which complicates a proper representation with parametric descriptions (Sutherland, 1987).

4.4.3. Prediction of armour layer or pavement.

The equilibrium surface layer composition can be predicted by inversion of non-uniform sediment transport-equations if the flow conditions and the size distribution of the sediment transport are known by measurement or surface-based sediment transport prediction models (e.g., Andrews and Parker, 1987). The bed material size-fraction F_i can be predicted using the dimensionless bed load Φ_{Bi} of size fraction i (see Subsection 5.4.2.) and p_i, which represents the proportion q_{Bi} of the ith size fraction in the total bed load q_B.

Parker and Sutherland (1990) propose

$$F_i = \frac{p_i/\Phi_i}{\sum_{i=1}^{N} p_i/\Phi_i} \qquad (4.28)$$

If surface-based sediment transport formulae are used, the predicted composition refers to an equilibrium transport situation. At low transport rates, Eq.4.28 predicts a surface distribution coarser than the bed load (Parker and Sutherland, 1989). At higher transport rates, the size distribution of the bed load approximates the composition of the bed material and no coarsening of the surface is predicted.

Assuming a homogeneous distribution of particle size fractions, Borah (1988) proposes Eq.4.15 for the thickness of the active layer, δ_a, wherein the porosity of the armour layer is not considered. The predicted scour depth Δz_b is described as

$$\Delta z_b = \delta_a - D_a \qquad (4.29)$$

4.5. Roughness geometry.

4.5.1. General.

The bed geometry affects the structure of flow and controls the sediment transport through entrainment conditions. The high rate of irregularity and the wide ranges of grain sizes of bed material in mountain rivers complicate the definition, analysis and

modelling of the geometry. The analysis of bed geometries generally concerns the size distribution of particles. In morphological computations, either the statistical characteristics of the particle size-distributions or equivalent particle-diameters are defined. Assuming geometric similarity to occur in course river-beds, Furbish (1986) suggested the use of roughness statistics.

On rough beds, the surface roughness (size, distribution and shape) can have profound effects on the structure and behaviour of the flow, (De Jong and Ergenzinger, 1992). At large-scale roughness, the roughness geometry can be considered an important resistive factor. The bed elements individually affect the structure of flow (Ashida and Bayazit, 1973; Bathurst, 1978).

4.5.2. Roughness height.

As described above, in many rivers with non-uniform bed material, flow conditions changes the bed surface. If a pavement or an armour layer is formed in response to the shear stresses applied, a wide range of different resistance levels can be experienced (Kuhnle and Southard, 1988; Schöberl, 1992). The selective sorting of sediment particles coarsens the bed surface and changes the scale of roughness. Therefore, roughness geometry is related to the history of flows. As suggested by Ayala (1991), the effect of an increased roughness could be taken into account by taking a representative pavement diameter.

If on the other hand, larger particles are moved due to exposure effects, the flow pattern changes also. Wiberg and Smith (1991) assume an instantaneous adaption of flow to the truncated grain size distribution. Sediment supply events can also change the size distribution of the bed material. Layers of fine sediment that cover the coarse bed can reduce the resistance to flow significantly (Simons et al. 1979; Li and Simons, 1982). If deposits of erodible sediments, are not uniformly distributed over the bed, strong discontinuities in resistance to flow and sediment transport can be introduced (Ghilardi and Menduni, 1989).

Bray (1982) reports of methods measuring the boundary roughness by tracing the bed surface along a transect across the reach. The roughness was determined with reference to a datum through the lowest points on the transect profile. Field measurements by Furbish (1986), De Jong and Ergenzinger (1992) and Ergenzinger (1992) included regular measurements of differences in river-bed topography at fixed intervals with rods projected downwards through a horizontally installed tube. Cross-sections of the stream were scanned and levelled with the help of a beam or measuring bridge, attached over the width of the section. Dinehart (1989 and 1992) uses sonic depth sounding records to measure mobile bed geometries in steep, coarse bedded streams.

The effect of roughness elements on the flow is often expressed by roughness height.

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As the flow is more affected by larger particles than smaller ones, the roughness height in gravel beds is generally expressed as D_{84}, multiplied by 3 to 3.5, or D_{90} multiplied by 2 to 4.5 (Bayazit, 1982; Bray, 1982; Van Rijn, 1982). Glass (1987) reported wide ranges of roughness heights to occur. According to Bathurst (1982), any diameter could be used if, at varying flows and sites, the contribution of the individual particle sizes remains proportional to that of other percentiles. Analysis by Bray (1982) indicated the equations predicting the resistance to flow to be relatively insensitive to the choice of characteristic grain size.

De Jong and Ergenzinger (1992) defined the roughness height as the maximum difference between three adjacent measuring points, located at constant intervals. The sets of three points can be chosen along a line, with an overlap of one point.

The definition of bed level or zero-velocity level is rather arbitrary. Flintham and Carling (1988) determined the zero-velocity level by comparing the known discharge with a calculated discharge. The discharge was calculated by cross-sectional integration of velocity point measurements. In the experiments carried out, the reference level was approximately equal to $0.5D_{84}$.

The bed level, with respect to which the flow depth is to be measured can be defined in different manners. The geometric bed level is the level that would arise if all the roughness elements on the bed were melted to form a uniform bed level (Bayazit, 1982). In the case of spherical roughness elements with size D, this would yield a plane levelled at $0.75D$ (Van Rijn, 1989). The separation zones behind the roughness elements could be included in the computation of the roughness volume. Another approach would be to define the theoretical bed level at the zero-velocity point of the velocity profile (Dong et al, 1992). Curve fitting with logarithmic velocity profiles yields a zero-velocity level at approximately $0.25D$ (Van Rijn, 1989).

Wiberg and Smith (1991) defined the zero bed level at the plane on which the bottoms of the grains lie. Stream depth can be measured by lowering a top-set wading rod or weight until it touches the bottom (Wiberg and Smith, 1991). The zero level is then defined as the top of the clast below the measuring rod.

4.5.3. Horizontal roughness aspects.

In experiments with intermediate- and small-scale roughness and small slope gradients, the controlling roughness has been found larger than the maximum grain size (Gessler, 1990). This implies that not only the grain size, but also the grain arrangement in the armour coat controls the friction factor. The roughness geometry should therefore comprise the relative *roughness area* of bed elements, and their spatial arrangement or *roughness concentration* over the bed. The randomness of the roughness geometry can be considered a significant source of discrepancies (Aguirre-Pe, 1991).

The relative roughness area, λ, can be defined as the proportion of the cross-sectional area occupied by significantly protruding bed elements, (Bathurst, 1981). It determines the degree of funnelling of flow between projecting elements, and therefore the average velocity of flow. The relative roughness area is significantly affected by the larger elements in the size distribution.

In general, the variability of roughness patterns along the perimeter, has been accounted for by defining an empirical relation (Bathurst, 1978), or an equivalent roughness. A general approach has been to divide the cross-section into subsections, and subsequently determine the specific resistance or discharge in the subsections (Motayed and Krishnamurthy, 1980).

To take into account the irregular plan form of bed features, the cross-section can be approximated polygonally. This "stripe method" enables the determination of transport rates with local flow and geometry conditions. However, the energy slope is assumed constant over the width, which may not be the case in nature (Jaeggi, 1987). In the case of significant differences in roughness scales between river-bed zones, the exchange of momentum will be rather strong. Consequently, cross-sectional averaging of flow and roughness conditions without considering this exchange of momentum, cannot be applied.

Pillai (1979) observed a decrease in effective depth at increasing concentrations of roughness elements. To compute the equivalent roughness, different procedures have been developed. Thompson (1979) and Bathurst (1985) empirically accounted for the effect of the bed-material size-distribution on the spatial roughness concentration, by relating the relative roughness area to the relative submergence depth. According to Bathurst (1982), the roughness spacing does not change, provided that the roughness, shape and size distribution are constant.

Experiments by Ferro and Giordano (1991) indicated a relation between the roughness concentration (the number of coarse bed elements at the surface of the bed) and D_{84} of the bed particles. If the relative roughness in the resistance prediction formula is defined as a/D_{84}, the effect of the particle concentration on the roughness, is considered included (Ferro and Giordano, 1991).

The reach-averaged energy slope can vary significantly at individual subsections (Motayed and Krishnamurthy, 1980). Assuming a uniform energy slope, Fuentes and Aguirre-Pe (1991) described flow in channels of composite roughness as formed by bands of uniform flow corresponding to bands of uniform roughness. Transverse momentum exchange was assumed proportional to the square of velocity differences in neighbouring zones.

4.6. Sampling methods.

The lateral and longitudinal variations in bed material, the varying composition of the

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bed in time, the vertical variation and a wide range in particle size complicate the application of sampling techniques (Kellerhals and Bray, 1971). Particle sizes can range from coarse sand to fine gravel. However, with particle sizes larger than about 20 cm, volume sampling and subsequent sieving and weighing is complicated (Fehr, 1987).

Three sampling methods can be distinguished; *grid or transect* sampling, *areal* sampling and *volumetric* sampling. At grid or transect sampling, grid points over the surface are established, and underlying grains are selected. Adams (1979) suggested the grain size analysis using photographs. Fehr (1986) reviewed the number-by-line analysis, at which particles are sampled along a line.

At areal sampling, all stones within a predetermined area are sampled. Grains can be collected manually, by freeze coring (Petts et al. 1989), by magnetic removal, using a spray painting with a mixture of paint and fine magnetite, or by pressing a cylinder filled with moist clay or wax, adhering the particles (Wilcock and Southard, 1989). The volumetric sampling method concerns the analysis of a standard volume that represents the surface and subsurface of the bed. For a review of sampling methods, reference is made to Church et al. (1987) or Diplas and Fripp (1992).

Kellerhals and Bray (1971) proposed a conversion method based on geometric arguments, assuming
-a volumetric fraction equals the areal fraction on a random cut sample surface.
-the area exposed by a grain on a random cut surface of the sample, is proportional to the square of its diameter.
Diplas and Sutherland (1988) refine this method by considering porosity of sediment in the sample.

Fehr (1987) also investigated different size analysis procedures, to develop a conversion and correction method to compare volume sampling results with areal or line-by-number analysis. He found a good performance of the line-by-number analysis after assuming a Fuller distribution for the finer size fractions, and applying correction procedures. Diplas (1992) recommended grid sampling for coarse particles, and clay sampling for analysis of clay particles.

Bray (1982) mentions the effects of sediment variability in a reach, when using averaged sizes to characterize the river. Access to appropriate sampling sites can be hard. The sampled material can originate from the bed, or from sediments deposited by previous floods.

Chapter Five.

Sediment transport.

5.1. Introduction.

Sediment transport in mountain rivers concerns lateral sediment input and movement of sediment in longitudinal direction. In this chapter, the sediment transport considered is induced by hydrodynamic forces of flows. A brief section on sediment gravity flows and interactions between sediment transport and flow is included in Section 6.4. and 6.5..

In mountain rivers with coarse bed material, most sediment transport occurs during the passage of floods, at high flows only. Transport rates at low or even normal flows are low compared to transport capacities (Jaeggi and Rickenmann, 1987; Musetter, 1989). Then, the transport rate is controlled by the limited sediment supply from (i) upstream reaches and (ii) bed and banks of the stream. Both the vertical and longitudinal compositions of sediment size-fractions in the river bed affect the composition and rate of the sediment transport. The supply and subsequently the transport of sediment can be episodic and spatially non uniform, with explicit seasonal patterns affected by antecedent storm history.

Jaeggi and Rickenmann (1987) characterize sediment transport in mountain rivers by large gradations in sediment sizes, a limited sediment supply from upstream reaches, bed and banks, significant in-channel storage and large quantities of fine suspended material.

If mobile sediments are sufficiently available, the steep, erratic hydrographs, irregular channel geometries and non-uniform sediment which are characteristic for mountain rivers often result in unsteady or non-equilibrium and non-uniform, selective transport.

Another factor that influences the bed-load transport, is the sediment storage pattern in upstream reaches. In forest streams, organic debris may form series of log steps and plunge pools, providing temporal sites for deposition of sediment. Sediment stored by temporal organic debris jams affect the dynamic behaviour of sediment transport. Therefore, in many cases, sediment transport rates cannot be related to water discharge alone (Griffiths, 1980; Pitlick and Thorne, 1987; Shuyou et al. 1989; Richards, 1990; De Jong and Ergenzinger, 1992).

Sediment discharges can be determined by analysing a natural or artificial sediment trap or box, sampling point values of bed-load velocity, or mass of the sediment transported. Klingemann and Emmet (1982) briefly reviewed vortex, conveyor belt or Helley-Smith bed-load samplers. Sobocinski et al. (1990) mentioned the collecting of bed load, the use of magnetic tracers (e.g., Spieker and Ergenzinger, 1990),

radioactive tracers (e.g., Walling and Bradley, 1990), acoustic measurements, metal-tagged gravel combined with metal detection devices and the tracking of painted clast. However, Laronne and Carson (1976) observed differences in stability of labelled sediment relative to *in situ* material due to disturbance of the natural, structural arrangement of the sediment. Integration of point values over the cross-section should account for the non-uniformly distributed sediment transport.

5.2. Modes of transport.

A particle can be considered *suspended load*, when completely supported by the turbulence of the flow. *Bed load* moves by rolling, sliding or saltation in a proximity to the bed. The material that is transported as bed load is displaced periodically or, at higher transport rates constitutes a layer that is about one grain size thick.

In general, clay and silts (< 0.05 mm) are transported in suspension whereas gravel, cobbles and boulders (> 2.0 mm) are transported near the bed. Sand-sized particles represent a transition between suspended and bed load (Beschta, 1987). In rivers with coarse bed material in absence of fine wash-load, bed load often is the dominant mode of sediment transport. In many rivers with coarse sediment, the ratio of bed load to suspended load can be significantly larger than in low-land rivers with sand beds.

However, the mode of transport not only depends on the particle size but is also determined by the conditions of flow; at low flows, sand can be moved as bed load, whereas at high flows, gravel can remain in suspension (Parker, 1990). At sufficiently high transport rates, the distinction between bed load and suspended load disappears, and division of modes becomes arbitrary.

5.3. Particle mobility.

5.3.1. General.

Movement of bed material is controlled by characteristics of the bed, hydraulic conditions of the flow and individual characteristics of the particle. The resistance of a particle to fluid forces is determined by the particle size, mass, shape, and the imbrication, packing and size of surrounding particles. The grain size and shape, particle weight, orientation (Carling et al. 1992), exposure to flow, and frictional forces by inter-particle stresses vary significantly. In the case of imbrication or vertical infilling, the initiation of motion of a particle requires the disturbance of neighbouring particles and the distortion of the bed structure. The instantaneous fluid forces are turbulent and the impacts of grains are irregular. As a result, forces on particles are stochastically fluctuating.

Different models have been developed (e.g., Li et al. 1976; Parker, 1978; Ikeda et al. 1988) to predict geometries in gravel-bed rivers, assuming that channel shapes are at

the condition of incipient motion and consequently controlled by the angle of repose of the particles in banks and bed. However, Wiberg and Smith (1987) mention increasing angles of repose with decreasing relative particle-size.

5.3.2. Character of particle forces.

Both the actual and critical shear stresses of a particle can fluctuate significantly. Movement of larger particles on the bed surface can be induced by instability resulting from local scour around the particle, by hydrodynamic forces or, at higher transport rates impact forces of smaller grains entrained by the flow (Kuhnle and Southard, 1988).

Drake et al. (1988) observed that the initiation of motion of larger particles predominantly occurs by rollover, displacement by rolling, and distrainment by deceleration over one or two particles. The smaller particles were observed to be entrained by liftoff mainly, displaced by saltation and distraint by head-on collisions with larger particles on the bed surface.
At higher rates of sediment transport, however, the contribution of inter-particle collision to the initiation or cessation of particle motion can be significant (De Jong and Ergenzinger, 1992).

As a result, movements of coarse size-fractions are random, infrequent and poorly related to discharge (Shuyou et al. 1989; Bunte, 1992). Prototype tests of stochastic characteristics of cobble-gravel bed-load transport by Shuyou et al. (1989) showed great variability in transport rates under constant conditions of flow. They found the probability distribution function of cobble-gravel bed load under uniform, steady conditions of flow, to be log-normal. According to the central limit theorem of probability, a variable will approximately have a normal distribution, if it is affected by the sum of a sequence of mutually independent random factors, and each factor has a relatively small effect on the variable.

Drake et al. (1988) explain the varying transport rates by shear stress fluctuations from high speed fluid-parcels impinged on the bed ("sweep" events). As a result, particles on rough beds can be entrained by accelerated, upward flow during "ejection" or "bursting" events. Kuhnle and Southard (1988) explain fluctuations in size fractions of bed-load transport by the development of bed forms, whereas Ghilardi and Menduni (1987) explain the occurrence of sand waves or clusters by the varying roughness, and subsequently varying flow conditions and transport capacity in mountain streams.

Particle segregation during transport can built coarse bed-load sheets moving in clusters (Dinehart, 1992). Kuhnle and Southard (1988) observed fine particles moving in bed-load sheets, regions with distinctively higher sediment transport. These bed-load sheets affected the entrainment of relatively larger particles by impact forces. This

implies that fluctuations of different size-fractions in the bed load are correlated. Build up and destruction of clast jams causes fluctuations in coarse fractions of bed load (Kuhnle and Southard, 1988; Dinehart, 1992).

An extensive review of research on incipient motion of bed material in gravel-bed rivers has been given by Richards (1990). Particle mobility can be expected to vary with the position of size fractions in the size distribution. Actual particle movements can hardly be predicted. The erratic characteristics can either be omitted by averaging of parameters in time, or by applying a stochastical approach (Shuyou et al. 1989; Suzka, 1989; Gessler, 1990; Richards, 1990).

5.3.3. Threshold conditions.

In the determination of incipient motion criteria, threshold conditions have been expressed in non-dimensional critical shear stresses, slopes or water discharges. In steep channels, the threshold conditions for particle movement are affected by the destabilizing component of particle weight. With neglect of the lift force, the dimensionless shear stress including the particle weight component aiding the particle movement can be described as (Bathurst et al. 1987; Graf, 1989; Suzka 1989)

$$\tau_{*i} = \frac{\tau_0}{\rho \Delta g D_i} \frac{1}{[\tan\phi\cos\theta - \sin\theta]} \qquad (5.1)$$

where τ_{*i} is the dimensionless shear stress, τ_0 the bed shear stress, D_i a particle diameter, ϕ the friction angle, θ the slope angle, and $\Delta = (\rho_s/\rho - 1)$, where ρ is the density of water and ρ_s the mass density of sediment. The destabilizing effect of gravity becomes significant at slopes steeper than approximately 1% (Graf, 1989).

In addition, velocity profiles in flows with large relative roughness are degenerated (see Subsection 3.2.3.). This results in a reduction of hydrodynamic drag forces. Relative large roughness elements dissipate energy by inducing turbulence. This results in a reduction of flow velocity near the bed surface. Consequently, the shear stress on bed particles in the wake region decreases (Ashida and Bayazit, 1973; Thompson and Campbell, 1979; Bayazit, 1982; Graf, 1989; Tsujimoto, 1989; Suzka, 1989; Wiberg and Smith, 1991).

Consequently, the critical tractive force on particles in steep channels with small relative submergence is affected by gravity and changes of velocity distribution (Bathurst et al. 1982; Graf, 1989; Tsujimoto, 1989; Aguirre-Pe and Fuentes, 1990). At increasing slopes, increasing gravity effects destabilize the particles, whereas the smaller depths resulting from higher velocities reduce the shear stress.

According to Graf (1989), the Shields stress τ_{*c} has a constant value of about 0.06 for small-scale roughness ($a/D > 25$), but depends on the roughness scale for $a/D < 25$. A simple, empirical formulation of the dimensionless critical shear stress τ_{*cr}, has been suggested by Graf and Suzka (1989)

$$\tau_{*c} = \tau_{*c0} \exp(5.06i) \tag{5.2}$$

where $\tau_{*c0} = 0.042$, which represents the shear stress without gravity effects and i is the bed slope.

Bartnik (1989) also recognized the effect of relative roughness and proposes

$$\tau_{*c} = 1.54 \frac{8}{f} \left(\frac{a}{D_i} \right)^{0.24} \tag{5.3}$$

Because the discharge can be measured more precisely than the depth, Bathurst et al. (1983; 1987) and Tsujimoto (1989) suggested the "Schoklitsch criterion", using a critical water discharge q_c

$$q_c = 0.21 g^{0.5} D_{50}^{1.5} i^{-1.12} \tag{5.4}$$

Using uniform blocks at slopes ranging from 5 to 25 %, Whittaker and Jaeggi (1986) proposed for the critical discharge at which block ramps started to deteriorate

$$q_{cr} = 0.257 \Delta^{0.5} g^{0.5} D_{65}^{1.5} i^{-1.167} \tag{5.5}$$

The critical discharge of Eq.5.5 is larger than with Eq.5.4, which according to Rickenmann (1990) might be explained by effects of different roughness scales in the original experiments.

However, the critical discharge can be obtained by conversion of the Shields equation with the help of the Manning-Strickler flow resistance equation (Graf, 1971; Whittaker and Jaeggi, 1986; Jaeggi and Whittaker, 1987). If definable, the use of explicit physical parameters as flow depth, velocity or shear stress enable a better insight in the processes.

5.3.4. Hiding and exposure effects.

In sediment mixtures, the particle mobility can be affected by effects of vertical and horizontal hiding and exposure. In case of vertical hiding, finer particles in the substratum are protected by coarser particles in the pavement layer. This effect is quantified by the size distribution of the bed surface (see Section 4.4.) which represents the particle availability. At horizontal hiding, finer particles hide in the

shade of larger particles where eddy structures reduce the flow velocity near the bed surface.

Horizontal hiding is related to the reduction of velocity downstream of large particles and the relative sizes of particles and their spatial arrangement at the bed. This effect is often described by empirical hiding functions (see Section 5.3.5.). Wörman (1991 and 1992) observed an increase in critical shear stress of stable fractions at increasing portions of armoured areas on the bed, and mentioned that spacing of the armoured spots affect the particle mobility.

Due to coarsening of the bed surface, coarse particles experience a relatively greater exposure to fluid forces. This reduces the critical shear stresses of coarser particles (e.g., Fenton and Abbott, 1979; Parker et al. 1982). Michalik and Bartnik (1986) measured the incipient motion of non-uniform sediment in mountain rivers with large submergence depths and relative flat slopes. The Shields criterion shear stress was found to overpredict the actual value.

The re-distribution of actual and critical shear stress over the particles by size-selective hiding and exposure effects results in a reduction of differences in mobility of bed size-fractions (Parker and Klingeman, 1982).

5.3.5. Shear-stress related mobility.

The level of shear stress re-distribution is related to the magnitude of the shear stress applied. If the critical shear stress of all particles in the sediment mixture is exceeded, the bed-load size-distribution approaches the composition of the substrate rather than that of the bed surface (Andrews and Parker 1987). At high shear-stresses, Parker et al. (1982) observed particles with different sizes to approach an equal mobility and introduced the *equal-mobility* concept. Equal mobility implies that the bed-load size-distribution in equilibrium conditions approximates that of the pavement, and that, in accordance with the classical approach of Meyer-Peter and Müller (1948), the total bed-load could be represented by one grain size of the pavement, e.g. D_{50}.

This has been formulated in the similarity hypothesis (Parker and Klingeman, 1982) which suggests one singular relation for the dimensionless, standardized bed-load transport Φ_{Bi} and the dimensionless, standardized particle shear-stress τ_i for each size-fraction i

$$\frac{\Phi_{Bi}}{\Phi_{Br}} = \left(\frac{\tau_{*i}}{\tau_{*r}}\right)^{m_i} \tag{5.6}$$

Both parameters are defined in Section 5.4.2.

Church et al. (1991) examined the mobility of finer particles, upto 10 mm in size, in

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a cobble-gravel bed channel. Although close analysis of bed-load size composition yielded selective transport, a single threshold condition, or equal mobility seemed sustainable for finer particles.

5.3.6. Hiding functions.

In the classical procedure of calculating transport of graded sediment it is assumed that each size fraction is not affected by the presence of other fractions and that each fraction is distributed uniformly in the bed (Ribberink, 1987). Then, the transport rate of a size fraction s_i can be assumed equal to the proportion f_i of the size fraction i in the bed surface layer, times the transport capacity s_c in the case of steady flow and uniform sediment) (Einstein, 1950)

$$s_i = f_i s_c \qquad (5.7)$$

The total sediment load S summed over all n size fractions is

$$S = \sum_{i=1}^{n} s_i \qquad (5.8)$$

However, in a graded sediment, effects of hydrodynamic forces are related to particle size and arrangement of particles on the bed surface. To enable the use of transport formulae for uniform sediment, Einstein (1950) introduced the hiding factor or sheltering coefficient that corrects the bed shear stress by adjusting the particle mobility relative to a value of a uniform bed material with D_i. Hiding functions correct either the particle shear stresses, τ_{*i}, or the critical shear stresses, τ_{*ci}, for hiding and exposure effects and correspond to different sediment transport formulas (Einstein, 1950; Egiazaroff, 1965; Day, 1980; Parker et al. 1982; Proffit and Sutherland (1983); Ranga Raju 1985; Andrews and Parker, 1987).

A general definition is (Ribberink, 1987)

$$h_i = \frac{\tau_{*i}(corrected)}{\tau_{*i}} \qquad (5.9)$$

where h_i is the hiding factor for the fraction with particle size D_i. The correction factor should enable the prediction of a transported size fraction q_{Bi} (Proffit and Sutherland, 1983), that is defined as

$$q_{Bi} = p_i q_B \qquad (5.10)$$

where q_{Bi} is the i-th size fraction of the bed-load q_B, predicted by a transport formula with the corrected τ_{*i}.

Egiazaroff (1965) suggested

$$h_i = \left[\frac{\log 19}{\log(19 D_i/D_m)} \right]^2 \qquad (5.11)$$

where D_m the mean size of the sediment mixture.

Ashida and Michiue (1973) suggested Eq.5.11 for $D_i/D_m \geq 0.4$, and present for $D_i/D_m < 0.4$

$$h_i = 0.85 \frac{D_m}{D_i} \qquad (5.12)$$

Andrews (1983) considered the critical shear stress and proposes

$$h_i = \frac{\tau_{*c_i}}{\tau_{*c_{50}}} = p\left(\frac{D_i}{D_{50}}\right)^{-r} \qquad (5.13)$$

where τ_{*c_i} is the critical dimensionless shear stress for D_i, $\tau_{*c_{50}}$ the critical dimensionless shear stress for D_{50} and p an empirical constant of about 0.083. Here, the particle mobility is adjusted relative to a particle with size D_{50}. The empirical constant r would be zero without hiding effects (Shields conditions), and unity in case of equal mobility of the size fractions. The value of r has been found to vary; 0.74 (Ashworth and Ferguson, 1989), 0.87 (Andrews, 1983) and 0.88 (Ferguson et al. 1989), 0.94 (Diplas, 1986), 0.98 (Parker et al. 1982) and unity according to Tsujimoto (1989).

Because transport of coarse particles is rather random (Section 5.3.1.) Bunte (1992) considers the hiding and exposure effects as described empirically to be more relevant for smaller particle sizes. Ribberink (1987) reviewed several empirical shear-stress correction formulae and noticed rather large differences in absolute values of predicted exposure effects.

Hiding functions can only be applied in combination with the transport formula that has been used for derivation. The conditions (threshold or transport) under which the function was derived should be considered (Sutherland, 1992). The present stage, the accuracy of predicting transport rates is still low (e.g., Range Raju et al. 1992).

5.4. Formulation of sediment transport.

5.4.1. Introduction.

Most of the formulae developed concern uniform, steady flow-conditions with movable

sediment particles sufficiently available on the bed surface. Therefore, transport formulae generally predict bulk sediment movements at uniform flow that can occur at a heavy input of sediment upstream by landslides or other mass movements only. Consequently, the application of the formulae requires modification for non-uniform flow conditions, non-uniform sediment, armouring or paving of the bed surface, etc.

5.4.2. Transport of uniform sediment.

The transport of sediment has often been the subject of research. In this section, formulae are described that predict transport of relative coarse sediments. The development of sediment transport formulae is based on flume experiments, field measurements or a combination of both. In general, the formulae designed can be applied to predict the bed-material sediment transport capacity, without considering effects of unsteady, non-uniform flow, restricted sediment supply, bed armouring or wash-load.

In uniform, steady flow, the dimensionless shear stress τ_* is defined as

$$\tau_* = \frac{\tau}{\rho \Delta g D_x} = \frac{\rho g R_h i}{\rho \Delta g D_x} \qquad (5.14)$$

In relatively wide rivers, $R_h \approx a$ and the dimensionless bed-load Φ_B is defined as

$$\Phi_B = \sum_{i=1}^{N} F_i \Phi_{Bi} = \sum_{i=1}^{N} F_i \frac{\Delta q_{Bi}}{F_i \sqrt{g(ai)^{1.5}}} = \frac{\Delta q_B}{\sqrt{g(ai)^{1.5}}} \qquad (5.15)$$

with q_B the volumetric total bed-load per unit width and F_i the size fraction i at the surface of the bed.

The formulae developed are generally based on effective shear stress (actual minus critical shear stress). However, in shallow, turbulent flows, the wild flow pattern complicates the definition of shear stress. Therefore, Bathurst et al. (1985) recommend the use of a formula developed by Shocklitsch (1962)

$$\Phi_B = \frac{2.5}{\Delta + 1} i^{3/2} \frac{(q - q_c)}{\sqrt{gRD^3}} \qquad (5.16)$$

where q_c as defined in Eq.5.4. Takahashi (1987) found a good performance of Eq.5.16 in the range $0.05 < i < 0.09$ and $0.1 < \tau_* < 0.4$, but noticed an underestimation of bed-load predictions for $i < 0.01$ and $i > 0.1$.

From field measurements on a small forest stream with riffle-pool sequences, Sidle

(1988) evaluated bed-load transport rates. The three predictors examined were stream energy, antecedent storm history and seasonal effects. The variables that were found significant are discharge Q, instantaneous peak discharge of the previous flood Q_{prev}, and cumulative flow $\sum Q$ for all discharges exceeding the "threshold" discharge for bed-load entrainment. The formula predicting the bed load Q_B reads (Sidle, 1988);

$$Q_B = a Q^b Q^c_{prev} (\sum Q - Q_{cr})^d \qquad (5.17)$$

where a, b, c and d regression coefficients. These coefficients were found to vary annually, due to effects of armouring and organic debris that were not considered in the formula.

In Eq.5.16 and 5.17, shear stresses are described implicitly by using a uniform flow resistance formula or reach specific stage-discharge curve. It should be considered that explicit physical parameters improve the applicability of the formula and enable a better insight in physical processes.

Bagnold (1980) developed a bed-load equation based on "stream power" per unit width $\omega = \rho g R_h i U$

$$\frac{s_b}{s_{br}} = \Delta \left[\frac{\omega - \omega_0}{(\omega - \omega_0)_r} \right]^{3/2} \left(\frac{a}{a_r} \right)^{-2/3} \left(\frac{D}{D_r} \right)^{-1/2} \qquad (5.18)$$

where the reference values s_{br}, $(\omega-\omega_0)_r$, a_r and D_r are empirical coefficients. Because Eq.5.18 is not sensitive to bed state, grain size distribution or bed-flow interactions, Gomez and Church (1989) suggested Eq.5.18 not suited for the prediction of local transport, but considered it to have an indicating potential mainly.

At the following pages, attention will be paid to shear-stress-based transport formulae. Some formulae for prediction of coarser sediments at steep slopes are listed in Table 5.1.

references	dimensionless bed load $$\Phi_B = \frac{\Delta g q_B}{(gai)^{1.5}}$$	remarks	nr.
Meyer-Peter and Müller (1948)	$8\tau_*^{1.5}\left[\left(\frac{K_s}{K_r}\right)^{1.5}-\frac{\tau_{*c}}{\tau_*}\right]^{1.5}$	K_s/K_r correction for bed form roughness	**5.19**
Parker et al. (1982)	$\Phi_B \phi_{50}^{14.2}$ $\Phi_B \exp[14.2(\phi_{50}-1)-9.28(\phi_{50}-1)^2]$ $5474\Phi_{Br}\left(1-\frac{0.853}{\phi_{50}}\right)^{4.5}$ with $\phi_{50}=\frac{\tau_{*50}}{0.0876}$; $\Phi_{Br}=0.0025$	$\phi_{50}<1$ $1<\phi_{50}<1.59$ $1.59<\phi_{50}$	**5.20** **5.21** **5.22**
Daido (1983)	$20\delta_t \tau_*^{1.5}(1-\frac{\tau_{*c}}{\tau_*})^{1.5}$		**5.23**
Misri et al. (1984)	$4.6\text{x}10^7\tau_*^8$ $8.5\tau_*^8\left(1+\frac{5.95\text{x}10^{-6}}{4.7\tau_*}\right)^{-1.43}$	$\tau_* \leq 0.065$ $\tau_* > 0.065$	**5.24** **5.25**
Smart and Jaeggi (1983)	$4\left(\frac{D_{90}}{D_{30}}\right)^{0.2} i^{0.6} \sqrt{\frac{8}{f}} \tau_{*m}^{1.5}\left(1-\frac{\tau_{*c}}{\tau_{*m}}\right)$	$0.03 \leq i \leq 0.20$ (gravel/sand)	**5.26**
Ashida et al. (1978)	$\frac{12-24\sqrt{i}}{\cos\theta}\tau_*^{(1.5-\sqrt{i})}\left(1-\alpha^2\frac{\tau_{*c}}{\tau_*}\right)\left(1-\alpha\sqrt{\frac{\tau_{*c}}{\tau_*}}\right)$	$\alpha^2 = 2\frac{0.425\Delta-\tan\theta}{\Delta-\tan\theta}$	**5.27**
Graf and Suzka (1987)	$10.4\left(1-\frac{0.045}{\tau_*}\right)^{2.5}$ $10.4\tau_*$	$\tau_* \leq 0.068$ $\tau_* > 0.068$	**5.28** **5.29**
Table 5.1			

Meyer-Peter and Müller (1948) suggested Eq.5.19 for sediment with $D_m > 0.4$ mm, and $D_{max} \leq 29$ mm. However, Smart (1984) showed an underestimation of the bed load predicted by Eq.5.19 at slopes steeper than 3 %.

Parker et al. (1982) approximated the transport of sediment mixtures by assuming equal mobility of the particles. Parker (1991) rewrote the formulations proposed by Parker et al. (1982) for the conditions assumed, and proposed Eqs 5.20, 5.21 and 5.22. The parameter ϕ_{50} is defined as the ratio of τ_{*50}, the Shields stress for the median diameter of the subpavement, and $\tau_{*r50} = 0.0876$, a reference value of τ_{*50}.

As substrate sediment parameters require complicated sampling procedures, Thorne and Hey (1983) suggested either to assume the surface layer sediment parameters to approach the substrate parameters in sediment deposition areas (bars), or to use an empirically derived formula

$$D_{50substrate} \approx 0.58 D_{50surface} \tag{5.30}$$

Daido (1983) measured bed-load transport on flume slopes upto 10 % and suggested Eq.5.23. Here, δ_t is the thickness of the transport layer defined as $\delta_t = \mu D$. Daido (1987) suggested μ to be a function of slope, τ_{*c}/τ_*, a/D_{84} and Fr. If $\tau_* >> \tau_{*c}$, which generally occurs at lower slopes, $\mu = 12$. At the range $0.005 < \tau_* - \tau_{*c} < 0.1$, $\mu = 3.7$ (Daido, 1983).

Misri et al. (1984) investigated the transport of relatively finer sediment with particle sizes ranging from 0.5 upto 5 mm, and present Eq.5.24 and 5.25. Because critical shear stresses are small, Φ_B is suggested related to the actual shear stress.

Partially based on Meyer-Peter and Müller (1948) data sets, Smart and Jaeggi (1983) developed Eq.5.26 that predict the transport of gravel-sand mixtures for slopes ranging from 3 to 20 %.

$$\Phi_B = 4 \left(\frac{D_{90}}{D_{30}} \right)^{0.2} i^{0.6} \sqrt{\frac{8}{f}} \tau_{*m}^{1.5} \left(1 - \frac{\tau_{*c}}{\tau_{*m}} \right) \tag{5.31}$$

Presumably due to the non-negligible presence of sediment within the wetted cross-sectional area, Smart and Jaeggi (1983) found the mixture depth a_m to increase with an increasing sediment concentration. The mixture depth a_m is defined as

$$a_m = \frac{q}{u} + \frac{q_b}{v_b} \tag{5.32}$$

where q the water discharge per unit width, u the depth-averaged velocity, Φ_b the bed-load discharge per unit width and v_b the sediment velocity.

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Empirically, Smart and Jaeggi (1983) derived

$$a_m = \frac{a_w}{(1-1.41i^{1.14}\Phi_B^{0.18})} \quad (5.33)$$

where a_w the clear water depth. To account for the change in Φ_B with increasing clay suspension concentration, Rickenmann (1990) suggested to replace the constant 1.41 in Eq.5.33 with $2.37/\Delta$.

Rickenmann (1989; 1990) found a deviation of the transport rates predicted by Eq.5.26 from measured values for slopes steeper than 10 to 15 %, and modified Eq.5.26 considering the effect of suspended particles on the transport capacity

$$\Phi_B = \frac{3.1}{\Delta^{0.5}} \left(\frac{D_{90}}{D_{30}}\right)^{0.2} \tau_*^{0.5}(\tau_* - \tau_{*c})Fr^{1.1} \quad (5.34)$$

with τ_m and τ_f to be calculated with the mixture depth a_m and the clear water depth a_w.

According to Takahashi (1987), Eq.5.26 underestimates the bed load in the range of large τ_* values in steep channels (10 to 20 %), and in all ranges of τ_* at flatter slopes of about
1 %. Ward (1986) added sediment transport data to the sets of Smart and Jaeggi (1983), and modified Eq.5.26 by including grain characteristics

$$\Phi_B = 7.16\Delta\sqrt{\frac{8}{f}}\tau_*^2(D_{50})^{-0.25}\left(\frac{D_{90}}{D_{30}}\right)^{0.2}i \quad (5.35)$$

Takahashi (1987) reviewed Eq.5.27, a bed-load formula for flat channels proposed by Ashida et al. (1978). Here, θ is the slope angle. At flume slopes ranging from 2 to 15 %, with sediment sized 0.2, 0.4 and 1.0 mm, Mizuyama and Suzuki (1987) used Eq.5.27 to predict suspended and bed load with reasonably good fit.

To accomplish a better fit with data from Smart and Jaeggi (1983), Takahashi (1987) modified Eq.5.27

$$\Phi_B = \frac{1+5\tan\theta}{\cos\theta}\sqrt{\frac{8}{f}}\tau_*^{3/2}\left(1-\alpha^2\frac{\tau_{*c}}{\tau_*}\right)\left(1-\alpha\sqrt{\frac{\tau_{*c}}{\tau_*}}\right) \quad (5.36)$$

Tsujimoto (1989) slightly modified Eq.5.36, by using τ_* corrected for gravity and large-scale roughness effects, instead of α.

Based on flume experiments with coarse sediments ($D_{50} = 12.2$ and 23.5 mm), Graf and Suzka (1987) proposed Eq.5.28 and 5.29. In the table, the formulae are rewritten for $R_h \approx a$. In the experiments, flume slopes varied from 0.5 upto 2.5 % and relative roughness a/D_{50} ranged from 4.00 to 20.9. Suzka (1989) modified Eq.5.28 by including the effect of the relative depth on the critical shear stress, and proposed for Eq.5.28

$$\Phi_B = 10.4\tau_*^{1.5}(1 - \frac{\tau_{*c}}{\tau_*})^{2.5} \tag{5.37}$$

Tsujimoto (1989) used the general form of the bed-load formula suggested by Meyer-Peter and Müller (1948), and considered the effect of gravity by $\Psi_1(i)$ and, implicitly in τ_{*c}, the effect of relative roughness

$$\Phi_B = A_0 \left(\frac{\tau_*}{\Psi_1(i)}\right)^{3/2}\left(1 - \frac{\tau_{*c}}{\tau_*}\right)^m\left(1 - \sqrt{\frac{\tau_{*c}}{\tau_*}}\right)^n \tag{5.38}$$

where A_0, m and n empirical constants.

In flume experiments with supercritical, uniform flow on sand beds, Khaleel and Sarginson (1986) correlated sediment transport rates to Froude and Reynolds number, relative roughness and the ratio of flow velocity to sediment fall velocity.

Regarding river-bed sections with fine-sized sediments, Voogt et al. (1991) compared predictions of sediment transport at high velocities and found good performance of the Ackers and White formula (Ackers and White, 1973) for $u < 1.35$ m/s, the Engelund-Hansen formula (Engelund and Hansen, 1967) for u ranging from 1 to 3 m/s and the Van Rijn total load formula (Van Rijn, 1989) for $u > 2$ m/s.

Bathurst et al. (1987) concluded that most bed-load formulae perform reasonably well in the test range for which they have been developed. As verification data sets are relative scarce (Gomez and Church, 1989), this indicates the limitations of applicability of the formulae described.

5.4.3. Transport of non-uniform sediment.

Experiments by Kuhnle (1988) in flumes with non-uniform gravel beds indicate sediment transport rates to be related to effective bed shear-stresses as generally considered in most bed-load formulae.

Parker et al. (1982a and 1982b) found that the relative mobility of sediment size fractions varied with shear stress, deviating from the equal mobility concept. This implies different relations of bed-load size fractions to particle shear stress, for

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different particle size fractions (Andrews and Parker, 1987). Therefore, Eqs 5.20, 5.21 and 5.22 serve as a first approximation of the transport of sediment mixtures. At $\phi_{50} > 1.4$, the ratio r_i of bed-load content to subpavement content asymptotically reaches an empirical value r_{io}, specified for different size ranges i. To account for selective transport, Parker et al. (1982) suggested the use of the original formulations of Eqs 5.20, 5.21 and 5.22, multiplied by r_{io}.

With the help of sediment-transport data from Parker et al. (1982), Diplas (1986) attempted to include the effect of selective mobility by suggesting for Eq.5.6

$$\frac{\Phi_{Bi}}{\Phi_{Br}} = (G_s(\phi_i))^{m_i'} = \left((\phi_i)^{m_i}\right)^{m_i'} \tag{5.39}$$

Diplas (1986) proposes for $\phi_{50} < 1.4$

$$\frac{\Phi_{Bi_i}}{\Phi_{Br_i}} = \left(\phi_i^{(D_i/D_{50})^{0.3214}}\right)^{13.71} \tag{5.40}$$

And, for wider ranges of ϕ_i

$$\frac{\Phi_{Bi}}{\Phi_{Br}} = 4 \times 17^{(1-1.205\phi_i^M)} \qquad \text{with} \qquad M = -1.843\left(\frac{D_i}{D_{50}}\right)^{0.3214} \tag{5.41}$$

At lower shear stresses without upstream supply, the surface layer is the source of bed load, and therefore determined by the surface layer rather than subsurface content (Parker et al. 1982). Parker (1990) transformed Eqs 5.20, 5.21 and 5.22 from a substrate- to a surface-based relation and included the standard deviation of the size distribution.

For prediction of bed-load transport of coarse, graded sediment, Misri et al. (1984) propose the use of Eqs 5.24 and 5.25 and suggest the shear stress τ_i effective for transport of size fraction i to be corrected with

$$\xi = \frac{\tau_i}{\tau} = \frac{0.038K\left(\frac{\tau}{\tau_c}\right)^{0.75}}{\tau_{*i}^{1.2}\left[1 + 0.0031\tau_{*i}^{-2.1}\right]^{0.33}} \tag{5.42}$$

with K related to the gradation of the sediment, ranging from 1 to about 1.6. Samaga et al. (1986) extend the use of Eqs 5.24, 5.25 and 5.42 to predict transport of finer-sized sediment.

Hsu and Holly (1992) decouple the prediction of bed-load composition and total bed-load rate to enable better calibration. The bed-load size-distribution is predicted with classical uniform-sediment transport formulae, corrected for hiding and exposure effects and particle availability at the bed surface. As already mentioned by Ribberink (1987) the number and distribution of size classes in the model affect the performance of the model.

5.4.4. Stochastical modelling approach.

As already described in Section 5.3.2., sediment transport can exhibit "random" fluctuations. Under similar flow conditions, Hassan and Church (1992) observed different travel distances for one particle size, which emphasizes the stochastic nature of bed load in gravel-bed rivers. Therefore, starting with a model suggested by Einstein (1937), stochastic approaches have been developed. Griffiths (1980) developed a simple stochastical bed-load yield- prediction model by assumption of flood- and corresponding bed-load yield frequency distributions, formulated with the help of reference catchment areas.

Van Niekerk et al. (1992) assume a Gaussian-distributed turbulent shear stress to predict bed-load transport. In general, the effect of the stochastic character of shear stress on a sediment transport formulae implicitly is accounted for by calibration coefficients in the empirical formula.

Nakagawa and Tsujimoto (1980) propose a model for prediction of uniform sediment bed-load transport constituted by an empirically formulated pick-up rate and a stochastic step length. In this model, the pick-up rate p_s has been defined as the probability per unit time for a bed particle to be dislodged by fluid action from a bed area occupied by the particle. For p_s in uniform sand, per time interval and particle occupation area, Nakagawa and Tsujimoto (1980) suggest

$$p_{*s} = \frac{p_s}{\sqrt{\Delta g/D}} = F_0 \tau_* (1 - \frac{\tau_{*c}}{\tau_*})^3 \qquad (5.43)$$

where the empirical coefficient F_0 is chosen 0.03, and τ_{*c} is 0.035.

After division of Eq.5.43 by particle occupation area $\alpha_2 D^2$ and multiplication of p_s by particle volume $\alpha_3 D^3$, Eq.5.43 yields the volume of sediment dislodged per unit bed area and time, where α_2 and α_3 are geometrical coefficients of the sediment as defined in Eqs 4.7 and 4.8.

The step length can be defined as the distance for a particle to travel as bed load. The probability density function f_X is interpreted as the relative number of particles dislodged with step length ζ. Nakagawa and Tsujimoto (1980) proposed an exponential distribution of f_X with ζ

$$f_X(\zeta) = \exp\left(-\frac{\zeta}{\Lambda}\right) \tag{5.44}$$

Nakagawa and Tsujimoto (1980) suggest the mean step-length Λ to be about 10 to 30 times D. With the average travel distance of a particle equal to Λ, the bed load at x_m can be written as

$$s_B(x_m) = \int_0^{x_m} \frac{\alpha_3 D}{\alpha_2} \frac{p_s(x)}{\Lambda} \left(\int_{(x_m-x)}^{\infty} f_X(\zeta) d\zeta \right) dx \tag{5.45}$$

Nakagawa et al. (1988) modified Eq.5.45 for non-uniform sediments and suggest

$$s_{Bi}(x_m) = \int_0^{x_m} \frac{\alpha_3 D_i}{\alpha_2} \frac{f_i(x) p_{si}(x)}{\Lambda} \left(\int_{(x_m-x)}^{\infty} f_{Xi}(\zeta) d\zeta \right) dx \tag{5.46}$$

Based on Eq.5.43, Tsujimoto and Motohashi (1990) present for size-specific pick-up rates

$$p_{*si} = 0.03 \tau_{*i} \left(1 - \frac{0.7 \tau_{*ci}}{\tau_{*i}} \right)^3 \tag{5.47}$$

The size-specific step length Λ_i has been suggested 10 to 30 times D_i.

Laronne and Carson (1976) suggest the distance of travel to be affected by the particle size relative to the bed roughness, the location of emplacement, specifically the local gradient of the bed, and the steepness of the bed in the downstream reach. Drake et al. (1988) observed variations with particle size in the distribution of travel distance. This is resulting from the different transport processes involved. The fine particles tend to concentrate at shorter distances, while larger particles exhibit a wider range of longer distances.

Hassan et al. (1991) tested stochastic distributions of the distance of movement of particles in a stream. At low flows, the number of steps or movements was found to be relatively small and local. In those small events, the distribution of the distances moved by the particles was rather monotone, and corresponded reasonably with an exponential distribution as assumed by Nakagawa and Tsujimoto (1980). However, at larger flows, the number of particles and the distance they move increased, and the distribution of the distance of movement became a skewed one.

The movement of the particles was found to be affected by bed forms such as bars. This interaction and the effects of complex hydrographs disturbed the stochastic

modelling of local dispersion of particle movements (Hassan et al. 1991). The effects of flood stage and bed forms on the particle travel distance as described limit the applicability of Eq.5.45 to small flows with relative flat beds.

5.5. Morphological interactions.

5.5.1. General.

The uniformity of flow and sediment transport can be affected by channel morphology. Both unstable bed forms and stable bed configurations can control the sediment transport conditions. Jackson and Beschta (1982) distinguish;

-sediment transport of finer particles over a relatively stable gravel surfaced bed
-sediment transport on a mobile bed

Likewise, Klingeman and Emmet (1982) suggest the use of two bed-load transport modes;

-one mode at low flows, when selective entrainment and transport occurs
-one mode applying to high flows, where almost all the bed material is moving and the effects of sediment heterogeneity are minimal

Apart from the sediment transport in relation to stability of the geometry, Sawada et al. (1985) recognize a relation between sediment transport, rate and pattern of flow and stability of banks.

5.5.2. Sediment transport over stable beds.

At stable or rigid beds, the larger particles at the paved bed surface cannot be moved by the flow. Then, the usual sediment transport during small storm flows concerns wash-load through stable structures of the stream. Sediment particles are transported as over-passing loads, periodically filling and eroding pools and pores in the paved bed surface. The sediment is supplied by a finite number of inputs and comprises material that is usually much finer than the coarse bed material. Consequently, the configuration of the bed remains unchanged but the concentration of movable grain size fractions at the bed surface can change temporally and spatially.

Wesche et al. (1987) measured the largest and most variable volumes of deposited material in lower gradient pool sections. Flume tests (Egashira and Ashida, 1989) showed the propagation of over-passing loads to be retarded by step-pool topography. Whittaker (1987) suggests that long wave transport of sediment moving through stable step-pool structures propagates as a set of smaller waves.

In modelling the transport of sediment on a stable bed with step-pool morphology,

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Egashira and Ashida (1989) distinguish two sediment transport processes. In pool regions, the transport rate is related to the volume of mobile particles that are temporally stored in the pool, as already suggested by Sawada et al. (1985). In riffle sections, flows accelerate and little material is stored. The transport rate is affected by the concentration of mobile particles at the paved bed surface, the so-called coverage ratio. Daido (1987) defined the ratio of area affected by stable particles to the bed area as the shade area. If, however, sediments move over the surface of the bed, the measurement of such a coefficient is complex.

Because the quantity of sediment stored in rivers depends on sediment production and discharge, the relation between sediment discharge and flow rate can show some hysteresis (Sawada et al. 1985). The hysteresis in sediment transport at different stages of flow can be explained by the storage and release of sediment stored behind large organic debris in upstream reaches, by depletion of available sediments from the channel (Sidle, 1988) or by armouring mechanisms (see Section 5.5.3.). Nanson (1974) measured a lag of bed-load transport with flood hydrograph, and suggested three explanations;

- a rise in temperature and subsequently viscosity of the flow during the flood
- dislodgement of armoured particles
- the corresponding pattern of controlling sediment-supply processes

Nanson (1974) observed a decrease in sediment concentration during the run-off season, which indicated the dominant influence of sediment supply.

Sidle (1988) observed significant differences in bed-load transport on the rising and falling limb of a flood hydrograph. Sidle (1988) found the movement of both total and fine bed-load transport to be influenced by antecedent storms, while no such correlation for movement of coarse material was found. The transport of fines also showed significant variability at the rising and falling limbs of the storm.

As bed-load transport is affected by local conditions of bed composition, the hysteresis experienced in relations between bed load and discharge can vary at different sites (Klingeman and Emmet, 1982). If the relation is determined downstream of local sediment storage areas, the bed load will increase steeply at the increasing limb of the flow, and decrease again as the stored sediment becomes depleted. If bed load would be sampled upstream of such a storage area, an opposite sequential loop could be experienced. Therefore, lags in sediment transport should not be interpreted as the difference in arrival between sediment wave and the carrying flood wave (Borah et al. 1982), since this time interval is affected by the location of sediment available to transport.

5.5.3. River-bed controlled transport.

Without sufficient supply of sediment from upstream reaches, the surface layer of the river bed provides the sediment for bed load. Then, the composition and rate of the transported sediment can be regulated by the distribution of particle sizes at the bed surface (e.g. Parker and Sutherland, 1989). In the case of a paved bed surface, the sediment entrainment from the bed is controlled by preexisting pavement conditions. This implies that t,he history of flows should be considered in predicting sediment transport rates. Due to changing bed geometry, Klingeman and Emmet (1982) reported variation in composition and rate of bed-load transport in flows with different duration periods.

The bed load composition changes with shear stress. At low shear stresses, more fine-sized particles are transported in the bed load. According to Shih and Komar (1990), the grain-size distribution of the bed load exhibits a rather symmetrical, Gaussian distribution, presumably due to the random, size-selective processes. At increasing shear stresses, Ashworth and Ferguson (1989) measured an increase in both maximum and mean particle sizes of the bed load. At lower rates of shear stress, a decreasing number of coarse particles of tracer pebbles was observed to travel smaller distances. The composition of bed-load material measured by Schöberl (1989) was finer than the sediments on the bed, except during extreme floods.

As shear stresses increase, the amount of coarser particles transported increases (Komar and Carling, 1991). Then, at bi-modal size distributions, the gravel mode tends to dominate over the sand mode (Ashworth and Ferguson, 1989). The bed-load size distribution shifts from a symmetrical one with relatively fine-sized particles, to a skewed or Rosin distribution, approaching the composition of the bed material (Shih and Komar, 1990). In gravel beds, the grain-size distributions can be typically bi-modal, exhibiting a grain and a sand mode. This often results in a gap in mobility, which can imply discontinuous changes in sediment transport. In streams with mono-modal bed-material, the trends described are smooth rather than abrupt (Klingeman and Emmet, 1982).

In the case of moving bed forms in non-uniform sediment, a macro mixing process occurs: coarser particles are temporarily exposed and transported before re-deposition and covering with finer sediments at local decelerating flow (Chiew, 1991). Covering of armoured surfaces result in local reductions of the number of stable particles at the bed surface. This affects the resistance to flow and the sediment transport rate (Karim and Holly, 1986; Wilcock and Southard, 1989).

Hystereses in discharge-sediment transport relation can be explained by in-channel storage as decribed in Section 5.5.2., but also by the unsteady storage and release of sediment during sporadic armour formation and breakup (Klingeman and Emmet, 1982; Klaassen et al. 19s86; Sidle, 1988). The formation of a pavement layer at the

increasing limb of a flood reduces the number of mobile particles on the bed surface and the shear stresses by hiding effects. This can limit the bed-load transport significantly (Klingeman and Emmet, 1982; Kuhnle, 1988).

Without a pavement layer, the critical shear stress of particles is lower. Therefore, the destruction of the pavement at the increasing limb of a hydrograph can result in higher sediment transport rates at the receding limb (Klingeman and Emmet, 1982; Suzuki and Michiue, 1988).

Fukushima et al. (1985) suggest the suspended sediment concentration to be related to the discharge. However, field measurements by Ohta and Fukushima (1985) show that suspended sediment concentrations during floods exhibit large fluctuations that do not correspond to changes in discharge. They suggest the rate of suspended load to be controlled by movements of large particles and explained this correlation by turbulence induced by moving sediment.

Schöberl (1989) attempted to relate the suspended load to run-off wave but suggests it to be affected by various factors;

- seasonal influences affecting the flow evolution and subsequently the sediment delivering erosion processes
- the run-off wave exhibits a lag with wave of suspended sediment
- climatological trends and meteorological phenomena affect the erosion processes
- non-uniform response of the different sediment supplying areas

5.5.4. Sediment transport at mobile beds.

The second stage of sediment transport, transport at mobile beds, occurs at higher shear stresses when more particles at the bed surface become unstable. In this case, the bed-load transport is characterized by complex interactions between material in transport and bed material. However, because this stage of sediment transport is also engaged to alluvial, low-land rivers, some aspects will be highlighted briefly.

If bed forms develop in mobile sediments, the thickness of the bed-load transport layer is extended from several grain diameters up to the bed-form height. The direction and magnitude of the bed-load velocity are affected significantly by the features of the bed. Consequently a strong interaction between bed-load transport and the formation and migration of bed forms exists.

The level of complexity can be extended by considering effects of erodible banks (e.g. meandering or braided rivers). In braided rivers, interactions between conditions of flow and morphology are significant. Due to this non-linearity, diversion of flows in se,perate channels can be unstable and subsequently unpredictable. Sediment transport

exhibits significant temporal and spatial variations (Thompson, 1985). Continuity of sediment demands that there be either storage or erosion of material in the reach, which implies changes in geometry.

In unstable gravel-bed rivers Thompson (1985) and Pitlick and Thorne (1987) identify reaches with predominantly transport of sediment ("sediment conduits"), and reaches where sediment is stored or eroded ("sediment stores"). Therefore the prediction of sediment transport should be based on local values of flow parameters, integrated across the width (Griffiths, 1986). Bed-load calculations have been based on point measurements, sediment budgets and input sets of stochastic data (Davies, 1987).

Ferguson et al. (1989) investigated sediment transport and hydraulics in a braided river section and report small and large roughness heights in zones of sand and zones of gravel respectively. The measured gravel and sand transport rates and gravel particle sizes were highest and largest on the smooth sand and transitional sand-gravel zones of the bed.
Satofuka et al. (1991) present a relation between the variation of the sediment discharge and the channel variation, composed of widening, bifurcating and merging regimes.

Today, the behaviour and interaction of the reaches described cannot be predicted. The limited insights in the morphological processes in the complex and unstable geometry of braided rivers prevent the development of deterministic models (Klaassen and Masselink, 1992). Predictions are often made with scale-model studies (Davies, 1987).

5.6. Transients in sediment transport.

5.6.1. Non-equilibrium sediment transport.

Transient processes in sediment transport can be the result of changes in flow (unsteady or non-uniform) or boundary conditions (non-uniform geometry) (e.g. Tsujimoto et al. 1990). The response of sediment transport to a bed with a non-uniform composition and topography is related to the mode of transport (suspended or bed-load ,transport).

The non-equilibrium state of bed-load transport can be caused by (i) a lag of pick up rate with bed shear stress or a phase shift between the flow velocity profile and bed shear stress (Nakagawa and Tsujimoto, 1980) and (ii) the step length of particles. Parker (1975) mentions lags in bed-load transport due to inertia of particles.

Non-equilibrium bed-load transport affects small-scale phenomena as the stability of bed levels (Kennedy, 1963; Hayashi, 1970; Parker, 1975; Fredsoe, 1976; Nakagawa and Tsujimoto, 1980) and, with graded sediments, the bed-surface composition. Unstable sorting due to unstable non-equilibrium bed-load can result in fluctuating

sediment transport by formation of gravel sheets.

In general, the lag resulting from a changing pick-up rate is small compared to the particle step-length and, presumably, the relaxation of the bed shear stress. The mean step length is in the order of the sediment diameter (Nakagawa et al. 1989). Therefore, in many models, bed load transport is assumed to adjust instantaneously to local conditions of flow and boundary.

In the case of suspended load, the concentration profile, transport rate and their interaction affect the non-equilibrium conditions of sediment transport. Sloff (1990) found numerically an unstable response of bed level to large concentrations of non-equilibrium suspended sediment-transport.

With the help of experiments, Bell and Sutherland (1983) related local, non-equilibrium sediment-transport to local sediment transport capacity in a non-uniform geometry (scour hole). For uniform sediment and steady flow conditions they suggest

$$s_b = \left[1 + \left(\frac{s_b}{s_b^*} - 1 \right) \exp(-K_l(x - x_1)) \right] s_b^* \tag{5.48}$$

where s_b^* is the equilibrium value of the unit-width bed-load s_b and K_l a loading-law coefficient. Equation 5.48 yields after differentiation

$$\frac{\partial s_b}{\partial x} = K_l(s_b^* - s_b) + \frac{s_b}{s_b^*} \frac{\partial s_b^*}{\partial x} \tag{5.49}$$

with the second term on the right side additional to the generally used bed-load lag-law (Di Silvio and Armanini, 1991).

Armanini and Di Silvio (1988) used for the exchange Φ_s of sediment between bed-load layer and flow in case of non-equilibrium suspended load;

$$\Phi_s = \frac{1}{L_* b}(S_{sc} - S_s) \tag{5.50}$$

where S_{sc} represents the suspended load transport capacity of the flow and S_s the actual rate of suspended load.

The characteristic length L_* has been formulated to transform two-dimensional suspended-sediment concentration-profiles into depth-integrated suspended sediment-transport formulae. In the derivation of L_*, a semi-empirical or an analytical, asymptotic formulation (Galapatti and Vreugdenhil, 1985) of the concentration profile

can be used, that approximates the depth-integrated suspended transport.

Armanini and Di Silvio (1988) propose for L^*

$$\frac{L_* w_f}{aU} = \frac{\delta_b}{a} + \left(1 - \frac{\delta_b}{a}\right) \exp\left[-1.5\left(\frac{\delta_b}{a}\right)^{-1/6} \frac{w_f}{u_*}\right] \qquad (5.51)$$

Di Silvio and Armanini (1991) consider Eq.5.51 valid for suspended load as well as bed load. Jain (1992) derived empirical values for time and space lag coefficients based on differences in particle mobility.

5.6.2. Sediment transport at unsteady flow.

At highly unsteady flows in mountain rivers, the sediment transport can be affected significantly by unsteady, non-uniform conditions of flow, as has been observed in flume experiments and field measurements (Jaeggi, 1987). If no morphological constraints affect the particle entrainment and transport (non-uniform in-channel storage, armouring), the highest transport rates are generally measured at the rising limb of flood hydrographs.

In accordance with measurements described in Section 3.4., Nouh (1989) reports the velocity and shear velocity, and subsequently the transport of sediments, to be higher at the rising branch of a hydrograph. However, the opposite has been observed also if sediment transport exhibit lags with the hydrograph due to morphological constraints such as the behaviour of armoured beds during floods (see Subsection 4.3.3) and erratic sediment supplies to mountain rivers.

The classical procedure of modelling effects of unsteady flow conditions is to determine an equivalent steady flow that represents the actual unsteady flow by a finite serie of steady state intervals. Then, sediment transport formulae derived for uniform flow conditions are used to predict the transport at an "equivalent steady flow" rate. At unsteady flow, corrections to the actual sediment rate transported are often related to the steepness of hydrograph and the instantaneous shear stress (Paul and Dhillon, 1987; Suzka and Graf, 1987; Tingsanchali and Rana, 1987).

Schöberl (1989) measured bed-load transport under unsteady conditions. The transport processes were found to be regulated by erosion processes in the supplying areas, and therefore pavement conditions of the bed were not considered. Changes in bed load at the increasing and descending limbs were related empirically to the shape of the hydrograph.

Among others, Nakagawa et al. (1989), Philips and Sutherland (1990) attempted to model temporal lags in sediment transport with the help of an impulse response

function. Philips and Sutherland (1990) expressed the deviation of the equivalent steady discharge $q_e(t)$ "experienced" by the bed, from the base flow q_b with the help of

$$q_e(t) - q_b = \int_0^t [q(t') - q_b] g_T(t - t') dt' \qquad (5.52)$$

where $q(t')$ the real discharge per unit width and $g_T(t-t')$ an impulse response function, empirically based on the time scale of micro-scale bed-form development.

Philips and Sutherland (1990) use an equivalent discharge $q_e(t)$ to determine the equivalent steady bed roughness, and subsequently the predicted unsteady bed roughness and sediment transport capacity.

To include all the relevant mechanisms, the formulation of non-equilibrium sediment transport should be based on the conservation laws of mass and momentum.

5.7. Hydrodynamics and sediment transport

Bed-material transport occurs when critical values of the particle shear stress are exceeded. Except for debris flow, the heavier sediment particles that are transported do not move with the same velocity as the fluid. Movements of particles can induce turbulence, and affect both magnitude and pattern of the flow velocity. The transport of bed material affects the resistance to flow through development of bed forms and through transfer of momentum from flow to particles (Bathurst et al. 1982; Bathurst et al. 1983; Smart and Jaeggi, 1983).

Kobayashi and Seo (1985) assumed a relatively simple fluid-sediment interaction and found the velocity profile and sediment concentration to be affected by momentum exchange with moving sediment.

At shallow flows, moving sediments can occupy a considerable part of the wetted cross-sectional area. At significant transport rates, the fluid velocity therefore can not be determined by the cross-sectional area and discharge rate alone (Rickenmann, 1990). Smart and Jaeggi (1983) developed an equation for resistance to flow in steep slopes, considering large concentrations of sediment. They present

$$\frac{U}{u_*} = 2.5 \left[1 - \exp\left(\frac{-\alpha a}{i^{0.5} D_{90}} \right) \right]^{0.5} \ln\left(\frac{12.27}{\beta} \frac{a}{D_{90}} \right) \qquad (5.53)$$

with the friction velocity $u_* = \sqrt{(ga_m i)}$, and the water-sediment mixture depth a_m defined as $a_m = q/v_w + s_B/v_B$. The changes in rheological behaviour of flow at higher

rates of sediment transport are reviewed briefly in Section 6.4..

Chapter six.

Miscellaneous.

6.1. Discharge measurement.

6.1.1. Introduction.

Direct measurement of discharge can be accomplished by time recorded discharge collection. However, given the volumes concerned, discharges are generally determined indirectly with closure equations with parameters easier to obtain. These equations can be mathematical equations of mass or momentum conservation or empirical relations.

6.1.2. Velocity-area methods.

An indirect determination of discharge can be obtained by using the cross-sectional area and velocity of flow. In steady conditions with uniform flow, the mass balance relates velocity and cross-sectional area (or width and depth) to discharge. Irregular geometries should, however, be simplified or characterized to enable description (e.g., Bathurst, 1978). Another significant source of error is the field measurement of water surface levels. At very rough flows, surface waves disturb the free surface level, which also is not constant over the section. Consequently, accurate measurements of mean water depth, surface level and slope are complicated (Bathurst, 1986).

In his surge tests, Kellerhals (1970) used stable pools formed by bedrock outcrops or large boulders to avoid the turbulent fluctuations disturbing the measurements of water levels. Bathurst et al. (1985) suggested the construction of a measuring well by arranging boulders that shelter a site from the main flow, but which is in full hydraulic connection. Whether the water level in the well represents the real, free-surface level without any local back-water effects depends on the position of the well relative to the direction of mainstream flow velocity.

At steep sections, boulder arrangements or bed-rock outcrops can act as natural weirs. If the state of flow is critical, the velocity is related to water depth $(q \sim a^{1.5})$ which eliminates either the depth or velocity from measurements. The peak discharge can be calculated with the maximum observed depth of flow (Bathurst, 1990). Naturally, critical conditions should remain over the entire cross-section and at different stages of flow.

If the vertical distribution of velocity in a hydraulic section is known, one point velocity would enable the integration over the depth. Unlike in small-scale roughness flows, the depth-averaged velocity cannot be approximated by the point velocity at a $0.4*a$ height above the bed, due to degeneration of velocity profiles (Jarret, 1984;

Jarret, 1990; Wiberg and Smith, 1991). According to Tu and Graf (1992), this measuring height varies in unsteady flows. Therefore, Bathurst (1990) recommends a more detailed measurement of the velocity profile for flows with $a/D_{84} < 4$.

Because of spatial differences in roughness scale, water surface levels can vary over a section. The distribution of velocity profiles over the cross-section can be very irregular, which complicates the use of current meters. To reduce errors that are introduced if point measurements values are intergrated over the cross-section, cross-sectionally averaged parameters, such as water depth or velocity, could be determined by integration over representative subsections.

Due to the irregular features and inaccessibility of mountain rivers, a non-contact method of measuring hydraulic parameters is favourable. Klein and Yufit (1984) suggest the use of a Doppler meter for measuring surface velocities. This measuring device uses Doppler shifts in acoustic signals that are reflected by the water surface to determine the surface velocity. However, random errors due to turbulent fluctuations of the free surface reduce the accuracy of this method.

If bed-load transport is negligible, bed geometry and water surface topography can be measured with the help of a point gauge. At increasing bed-load transport, point gauge measuring technique becomes inaccurate due to movements of individual gravel particles and the development and passage of bed forms (Bathurst et al, 1987). If a current meter is used, an adequate gauging cross-section should be selected. To avoid risks of damage from moving cobbles and organic debris, Bathurst et al. (1985) recommend a "Flovane", which employs the angular departure of a metal flap in the flow, and enables a velocity reading on a circular scale.

6.1.3. Tracer methods.

In turbulent, rough flows, the velocity is usually measured with the help of tracers. In mountain river flows, relatively short mixing-lengths of lateral and vertical dispersion of injected solutes are exhibited due to high rates of turbulence. This adds to the applicability of the method for description of rough, irregular flows in either mountain rivers (Kellerhals, 1970; Meijer, 1992), or flume experiments (Davies and Jaeggi, 1981; Beltaos, 1982; Smart and Jaeggi, 1983; Rickenmann, 1990).

With the help of mathematical formulations on dispersive transport processes in flows (see Section 6.3.), discharge rates are derived from the transport of a flow-injected substance. In steady flows, the discharge can be calculated using the tracer mass conservation equation

$$Q = \frac{M}{\int [c(x_m, t) - c_0(x_m, t)] \mathrm{d}t} \qquad (6.1)$$

where Q the calculated discharge, M the injected tracer mass, c the measured tracer-concentration at x_m and c_0 the background concentration. A simplified method is to use an NaCl-solution and a conductivity meter. The concentration curves can be measured with either the relative salt dilution method, based on electrical detection of an NaCl-solution, or with the dye dilution method, based on fluorometric detection of a fluorescent dye (Kellerhals, 1970).

According to Davies and Jaeggi (1981), the accuracy achieved in predicted flow depth and friction factor is considerable, compared to measurements based on a fluctuating free surface and irregular bed level. With $c_0 = 0$ g/l Kellerhals (1970) defined the mean travel time of a tracer cloud as

$$T_t = \frac{\int c(x_m,t)\mathrm{d}t}{\int \frac{c(x_m,t)}{t}\mathrm{d}t} \tag{6.2}$$

Bencala et al. (1990) mentioned two aspects of the usage of injected, in-stream tracers:

- sorptic characteristics may affect the conservative nature of the tracer as a result of extensive contacts between sorbing solutes and sediment
- dye tracers can be found inappropriate for use in acidic streams due to tracer mass losses by chemical instability

6.2. Identification of peak-discharges.

6.2.1. Slope-area methods.

Bathurst (1986) has investigated prediction of peak discharges in mountain rivers by slope-area methods. In the slope-area method, discharge is calculated from channel conveyance, friction slope and peak water-level profiles (obtained from records or high water debris marks) assuming uniform and steady conditions of flow. This implies a continuous profile of water levels without transitions of flow regime (Bathurst, 1990).

The basic equation can be formulated as

$$Q = K\sqrt{i_f} \tag{6.3}$$

where Q is the discharge, K the channel conveyance and i_f the friction slope.

In a uniform reach, and steady flow, K can be defined as

$$K = A \sqrt{\frac{8gR_h}{f}}$$

(6.4)

where A is the cross-sectional area, R_h the hydraulic radius and f the Darcy-Weisbach resistance coefficient. The field parameters to be measured for application of the method include slopes and cross-sectional areas. Consequently, with the help of stage-discharge curves, a quick discharge prediction can be made.

However, the determination of f requires usage of a resistance equation, applicable for mountain rivers. Due to the complicated processes involved, accurate description of resistance to flow cannot be accomplished yet (see Section 3.3). Secondly, the cross-sectional area can be overestimated if the wetted cross-sectional area of large boulders is not considered. In unsteady flows, discharge rating curves that have not been corrected for dynamic effects cannot be used. Consequently, in mountain rivers, peak discharge computations with the slope area method can easily result in large errors (Jarret, 1987).

6.2.2. Paleohydraulics.

Attempts have been made to develop a paleohydraulic technique, to compute velocity, depth and peak discharge from remaining channel flood deposits and cross-sections (e.g., Bradley and Mears, 1980; Wohl, 1992). The approaches developed use flow competence, which often is defined as *"the ability of flow to entrain a certain maximum size of bed material, for which flow conditions are said to be competent"*.

The distribution of grain sizes over the bed surface depends on the history of flow rates (Bouvard, 1989; Tanner, 1989). Kuhnle (1988) found the size of the coarsest particle in the mixing layer to be related to maximum flow shear stresses available for sediment transport. Based on measurements of bed-load size fractions responding to the stage of flow, Komar and Carling (1991) suggested that the size of the largest particle transported can serve as the basis for flow-competence assessments yielding mean bed stresses.

Shih and Komar (1990) found the entire distribution and range of the transported particle sizes related to the stage of flow. However, during floods, when significant quantities of nearly all size fractions in the bed material are transported, Andrews and Erman (1986) observed a relatively constant size-distribution of particles in pavement layers at the bed, which indicates that bed-material size-distributions do not represent historic conditions of flow in a singular way.

However, the evaluation of transport of large-sized sediments in floods is complex and the collection of data is difficult (Graf, 1979). In the analysis of deposits, the nature

of the cause which can be a large flood, land slide or debris flow, should be considered to avoid inaccurate estimates of peak discharges (Jarret, 1990). Material deposited can be delivered by banks nearby with minimal subsequent transport. The flood magnitude that is implied by grain sizes deposited can be underpredicted by absence of representative, larger particles (Komar and Carling, 1991) or entrainment-controlling structural arrangements of the bed surface (Laronne and Carson, 1976).

6.3. Transport of solutes.

6.3.1. Effects of irregular geometry.

Due to increasing interests in environmental impacts, much research has been carried out on the physical transport of solutes in low-land rivers. However, in complex flow patterns with rapid changes in direction and unsteady velocity distributions that cannot be described properly yet, exact modelling of solute transport by diffusive, dispersive and convective terms is not possible. In irregular, shallow mountain streams, dispersive effects of non-uniform velocity profiles in lateral and longitudinal directions can be significant, which explains the rather large differences in longitudinal dispersion coefficients that have been observed (e.g., Fischer, 1967; McQuivey et al. 1974; Nordin and Troutman, 1980; Singh et al. 1992).

Dispersion of solute in flows can be caused by turbulent mass exchange between streamlines of different velocity or streamline patterns and diffusion in and out dead zones (Kellerhals, 1970). The pronounced tail of a solute pulse observed in mountain rivers is often accounted for by modelling *storage zones or dead zones* (Bencala and Walters, 1983).

At the rising phase of the pulse, dead zones behave like sinks removing the solute. At the decreasing limb of the concentration cloud, dead zones behave like internal sources with a delayed supply (Bencala and Walters, 1983). As a result, the diffusion induced by storage zones has a skewing effect on the time concentration curve of passing concentration clouds, and the equilibrium or Taylor period wherein variance increases proportionally with time is postponed (e.g., Valentine and Wood, 1977; Nordin and Troutman, 1980; Meijer, 1992).

Beltaos (1982) assumed the dead zones to represent stagnant fluid pockets attached at the bottom boundary of the flow. According to Bencala and Walters (1983), the area of storage zones in a cross-section can be affected by:

- turbulent eddies generated by large-scale bottom irregularities
- large circulation zones in expansion areas along the side of the pools
- small, rapidly mixing recirculation zones behind flow obstructions such as significant protruding cobbles, small boulders and vegetation in riffle sections
- side pockets of water, acting as dead ends for solute transport

- secondary flow into, out of and through coarse gravel and cobble bed (Davies and Jaeggi, 1981)

In the modelling of storage zones, the following assumptions are often made (Bencala and Walters, 1983):

- storage zones are not moving
- uniform and instant distribution of solute
- mass exchange proportional to the concentration difference
- the cross-sectional area, solute concentration and exchange coefficient of the storage zones are measurable

Valentine and Wood (1979a and 1979b) found the ratio of the mean tracer cloud velocity to the streamflow velocity to decrease and the variance of the concentration to increase with an increasing relative volume of the dead zones. Castro and Hornberger (1991) mentioned tracer losses due to transient storage of the tracer in the stream alluvium.

6.3.2. Solute transport models.

In the tumbling flow regime, Kellerhals (1970) observed strong dispersive effects. He explained this dispersion by temporal storage in pools between short segments of supercritical flow. Kellerhals (1970) and Beltaos (1982) modelled tumbling flow regimes by sequences of weirs and reservoirs or pools. Kellerhals (1970) assumed that a solute entering a pool is mixed instantaneously, so that concentrations in pools are distributed uniformly. Thus, the solute mass conservation equation for the i-th pool can be formulated as

$$\frac{\partial c_i}{\partial t} = (c_{i-1} - c_i)\frac{Q}{V} = \frac{c_{i-1} - c_i}{T_r} \tag{6.5}$$

where V is the pool volume, Q is the discharge into and outof the pool and T_r is a characteristic time interval defined as $T_r = V/Q$.

For a sequence of n pools, the concentration in the n-th pool can be written as

$$c_n = \frac{M}{n!V}\left(\frac{t}{T_r}\right)^n \exp\left(-\frac{t}{T_r}\right) \tag{6.6}$$

Beltaos (1982) compared the results with the exponential "Taylor" concentration-time curve

$$c(x,t) = \left(\frac{M}{\sqrt{2\pi\xi Ax}}\right)\left(\left(\frac{Ut}{x}\right)\exp\left[1-\left(\frac{Ut}{x}\right)\right]\right)^{1/\beta} \qquad (6.7)$$

where ξ a dimensionless coefficient. Flume experiments, however, did not justify a physical reality of the storage-dispersion model described with Eq.6.6 (Beltaos, 1982).

In the modelling of solute transport in flows with dead zones, Valentine and Wood (1977) formulated the exchange of solute between storage zones and stream flow as

$$\frac{\partial c_s}{\partial t} = \frac{K\bar{U}}{d}(c_\delta - c_s) \qquad (6.8)$$

where c_s the solute concentration in the storage zone, K an eddy entrainment coefficient from about 0.02 for the geometry investigated, d the dead zone depth and c_δ the solute concentration near the interface between eddy structure and mainstream. The use of U for representing flow conditions near the bed can only be applied in uniform flows with a well-defined velocity distribution.

Less consistent, Thackston and Schnelle (1970) used the cross-sectionally averaged concentration instead of c_δ. Bencala and Walters (1983) used

$$\frac{\partial c_s}{\partial t} = -\alpha_s \frac{A}{A_s}(c_s - c) \qquad (6.9)$$

where A_s the cross-sectional area of the storage zone and α_s the stream-storage zone exchange-coefficient. A_s and α_s are not defined well physically and have been found to vary significantly in mountain streams (Bencala et al. 1990). Thackston and Schnelle (1970) and Bencala and Walters (1983) noticed a clear trend of decreasing importance of storage area A_s with decreasing f, or diminishing friction.

To reduce the number of variables, moments have been considered to describe statistical parameters of the dispersant concentration distribution (e.g. Valentine and Wood, 1977; Tsai and Holley, 1978; Purnama, 1988; Denton, 1990). With this method of moments, Denton (1990) derived asymptotic solutions for longitudinal dispersion with dead zones.

In accordance with Sabol and Nordin (1978), Purnama (1989) defined an equivalent retentive layer of uniform thickness that represents the dead zones in a river, on the bed and along the banks. The rate of solute flux across the retentive layer has been assumed proportional to concentration rates exhibited at earlier times.

At increasing values of a/d, Valentine and Wood (1977) found the distance required to reach equilibrium to increase. Valentine and Wood (1979b) found the mean velocity of the tracer cloud to decrease at increasing roughness.

Valentine and Wood (1979b) defined the depth of the storage zone d as twice the standard deviation σ_z of the bed profile from the average $\overline{z_b}$. The proportion of the bed area in storage zones was taken as the number of level readings below $\overline{z_b} + \sigma_z$. Purnama (1988) modelled the dead zone geometry as a random distribution of stagnant pockets with variable depth. Cross-sectional averaging eliminated the explicit presence of the geometry in the mathematical equations.

Bencala and Walters (1983) formulated for the solute concentration c in a uniform stream channel with steady flow

$$\frac{\partial c}{\partial t} = -\frac{Q}{A}\frac{\partial c}{\partial x} + \frac{1}{A}\frac{\partial}{\partial x}\left(AD\frac{\partial c}{\partial x}\right) + \frac{q_L}{A}(c_L - c) + \alpha_s(c_s - c) \qquad (6.10)$$

where D is the dispersion coefficient and q_L and c_L are the lateral inflow rate and solute concentration respectively. The terms on the right-hand side of Eq.6.10 represent the physical processes of convection, dispersion, lateral inflow and solute exchange between the surface stream channel and areas of relatively immobile water along the channel and in the streambed gravel. In relation to the chemical stability of the type of solute considered, chemical processes should be included into Eq.6.10 (Bencala et al. 1990).

Bencala et al. (1990) analysed test observations of a lithium tracer to determine the physical parameters to be used in Eqs 6.9. and 6.10. Parameters were identified by curve fitting of simulated concentration-time curves to measured curves. Analogously to the combined simulation and curve fitting approach applied by Bencala et al. (1990), Meijer (1992) developed a method for the determination of discharge in unsteady flows, using a simulation model and curve-fitting procedures. Model simulation parameters were identified from curve fitting to water level and concentration measurements.

Although, according to Bencala and Walters (1983), a transient storage model as described above cannot describe the observed transient storage processes exactly, identical equations can simulate empirically the processes. As a result, the best-fit parameters used in the mathematical equations only represent hypothetical features of a mountain river geometry.

Iwasa and Aya (1987) considered permeable beds and distinguish a layer of surface flow and a layer of subsurface flow, defined as mixing layer. In equilibrium conditions, the amounts of solute mass in the surface flow and mixing layer were found to be constant and the average convective velocity of tracer clouds in both

layers approaches the weighted average of the velocities in surface and subsurface flows. Similarly, the average dispersion coefficient is composed of the weighted average of dispersion coefficients in both flows and a term that represents the mutual mass exchange (Iwasa and Aya, 1987).

6.3.3. Solute absorption onto streambed sediments.

At low flows in large-scale roughness areas, solutes are expected to have an intensive contact with sediment (Bencala, 1983). Reversible exchange reactions between water and sediment retard downstream transport of solutes (Cerling et al. 1990). Irreversible adsorption reactions can result in high contamination rates on gravel streambed. Sequential transport takes place by physical rather than chemical processes such as bed-load transport or abrasion followed by suspension.

Kellerhals (1970) reported tracer losses by comparison of measured tracer masses in long and short reaches. To identify tracer reactivity, Bencala et al. (1990) used different tracers for field experiments. Cerling et al. (1990) investigated sediment-water interactions in riffle-pool streams and reported the significance of underflow for vertical dispersion processes of solutes. In low flow conditions. Cerling et al. (1990) found a considerable part of contaminant absorbed by the bed-load fraction.

Bencala et al. (1983) suggested a kinetic, first order mass-exchange model for the interaction between streambed sediments and stream flow. This model relates the change in c_*, the solute concentration of the sorbing sediment, to the difference between c_* and the potential equilibrium concentration in the stream $K_d c$

$$\frac{\partial c_*}{\partial t} = \mu (K_d c - c_*) \qquad (6.11)$$

where μ is an empirical rate coefficient expressing the degree of solute transport to the sediment and K_d a distribution coefficient at a specified concentration (Bencala et al. 1983). Both parameters have been found to vary with particle size.

However, different stages can be distinguished in the sorption process;

(1) macro-scale transport of solutes to the bed
(2) micro-scale transport of solutes to the chemically active sites of the sediment
(3) chemical reaction at a receptive site

The transport processes (1) and (2) are lumped into μ. This implies that Eq.6.11 cannot describe the detailed processes, but simulates the absorption of solute on the sediment on a macro-scale.

Bencala (1983) coupled Eq.6.11. to Eq.6.10 and 6.9 to include the sorption losses in the simulation of solute transport. The following equations are presented:
Solute in the stream flow

$$\frac{\partial c}{\partial t} = -\frac{Q}{A}\frac{\partial c}{\partial x} + \frac{1}{A}\frac{\partial}{\partial x}\left(AD\frac{\partial c}{\partial x}\right) + \frac{q_L}{A}(c_L - c) + \alpha_s(c_s - c) + \bar{\rho}\mu(c_* - K_d c)$$ (6.12)

where $\bar{\rho}$ is the mass of "accessible" sediment, in effective contact with a given volume of water.
Solute in the storage zone

$$\frac{\partial c_s}{\partial t} = -\alpha_s\frac{A}{A_s}(c_s - c) + \bar{\mu}_s(\bar{c}_s - c_s)$$ (6.13)

where $\bar{\mu}_s$ is an empirical rate coefficient for the storage zone analogous to μ, and $\bar{c}_s$ the equilibrium solute concentration in the storage zone.
Sorbate on the streambed sediment

$$\frac{\partial c_*}{\partial t} = -\mu(c_* - K_d c)$$ (6.14)

However, in his case study, Bencala (1983) concluded the performance of the model to be equally affected by transfer and reaction processes and the spatial variability of model-parameters.

The bed-load of tracer-tagged sediment in steady flow conditions has been described by Sobocinski et al. (1990)

$$\frac{\partial C_g}{\partial t} = D_M\frac{\partial^2 C_g}{\partial x^2} - U_g\frac{\partial C_g}{\partial x} + \sum R$$ (6.15)

where C_g is the average concentration of tracer tagged on the sediment, D_M the longitudinal mechanical dispersion coefficient for the labeled sediment, U_g the average bed-load velocity and R represents processes affecting the concentration C_g on the sediment (adsorption, desorption, radioactive decay, etc.).

6.4. Rheology of flows.

6.4.1. General.

In a mountain stream, sediments can be transported as a dynamic transport due to dynamic forces of water or as a sediment gravity flow. The mode of transport is related to the morphology, quality and quantity of the sediment, the mode of supply and temporary logging of the stream. In many mountainous basins, different modes

of sediment transport such as bed load, suspended load, wash-load and debris flow can even be experienced under various conditions of rainfall and discharge (Sawada et al. 1985).

At large concentrations of suspended sediment, the physical and dynamic characteristics of the flow are affected. The fluid viscosity and density are increased, turbulence intensity, distribution of velocity and sediment concentration, flow resistance and sediment transport capacities are changed (Bradley and McCutcheon, 1987).

According to Bradley and McCutcheon (1987), sediment-laden flows are classified by triggering mechanisms, sediment concentration or rheological and kinematic behaviour. A qualitative classification by triggering mechanisms has some overlap, and does not cover lower concentration flows on lower slopes. Although disagreement exists on the terms and definitions used, a quantitative classification by sediment concentration has been used by many investigators.

However, at higher concentrations, a greater opportunity for particle collision exists and water content, particle size, gradation and size distribution, shape, cohesion and composition gain relative importance. Then, classification by concentration alone will not be appropriate. Models have been developed to describe and classify flows at high sediment concentrations, with affected rheological behaviour. These models describe the rheologic behaviour of a fluid only partially (Rickenmann, 1990).

6.4.2. Types of rheology.

If flows are classified by rheologic and dynamic behaviour, some of the problems of using concentration as the only variable can be avoided. Characteristics of highly sediment-laden flows such as debris and mud flow, the mechanics of movement and fluid properties can be determined from rheological models (Takahashi, 1987).

The rheology of fluids and water mixtures can be analysed from measurements of shear stresses at various rates of angular deformation. In the transition region between normal streamflow with fluvial sediment transport and a debris flow, sediment-laden flow can be distinguished. At increasing amounts of fine material (sediment concentration by volume between about 30 and 80 per cent (Bradley et al. 1989; Rickenmann, 1991)) a hyperconcentrated slurry is formed. Hydraulic characteristics such as velocity distribution, resistance to flow and particle fall-velocity change. This can be described as a *non-newtonian* fluid where the sediment-water mixture can resist shear stress without deformation due to inter-particle yield-stresses caused by cohesion or shear strength.

If the sediment concentration becomes dense enough, the sediment mixture including the very coarse particles, is able to disperse throughout the entire flow (Takahashi,

1987). This mass movement of rock fragments, soil and mud (Bradley and McCutcheon, 1987) is designated *granular or debris flow*.

6.4.3. Transport capacity of sediment-laden flow.

According to Bradley and McCutcheon (1985), in unstratified flows with sediment concentrations below 20 percent by volume, the turbulent, hydrodynamic stresses are the controlling factor in flow behaviour and sediment transport, although the magnitude and frequency of turbulence intensity decrease with increasing concentration (e.g., Xinghui and Ning, 1989). In a literature review, Rickenmann (1990) reports of lower concentration limits in case of highly viscous clay-suspensions. In this range, the fundamental structure of turbulence does not essentially change in sediment laden flow (e.g., Lam Lau, 1983; Xinghui and Ning, 1989), the logarithmic velocity profile is valid and the "common" Manning and Chezy formulae can be applied for uniform flow.

Fukushima and Fukuda (1986) investigated effects of suspended sediment on the structure of turbulent flows with a k-ϵ turbulence model, which describes changes in turbulence, velocity and sediment concentration.

The changes in rheological behaviour of flows due to high concentrations of sediment are rather complex and are related to the state of flow; laminar or turbulent. In laminar, viscous flow conditions, the deformation shear stress and therefore the resistance to flow increases with increasing viscosity. Due to a decrease in settling velocity, more suspended load remains into suspension (e.g., Rickenmann; 1991). If the viscous, laminar sublayer covers the sediment particles at the bed, rates of bed load were found to decrease (e.g., Rickenmann, 1989) due to energy dissipation by higher viscous stresses.

In the turbulent flow regime, viscous effects are relatively small. Winterwerp et al. (1989) found that turbulence damping effects of suspended sediment tend to reduce the suspended load for concentrations upto 20 percent per volume, while for increasing concentration rates, hindered settling effects dominate the turbulence dampening and uniformalize the concentration profile. If the flow around the particles at the bed surface is turbulent or transient, bed-load transport rates increase considerably at higher clay concentrations due to an increasing fluid density and viscosity (Bradley and McCutcheon, 1985; Wan and Song, 1987; Rickenmann, 1987; Rickenmann, 1991).

Bradley et al. (1989) reviewed some procedures that modify traditional bed-load equations and correct the sediment discharge for increased transport capacity due to high concentration values. However, the impacts of changing rheology cannot be predicted accurately across the entire range of high concentration flows.

6.5. Debris flows.

6.5.1. General.

If particle motion on the bed surface is induced by hydrodynamic forces, the transport of sediment can be identified as "normal" bed-load transport. If shear stresses applied by gravity induce instability of a saturated bed or slope surface layer, sediment gravity flow occurs (Takahashi, 1987).

According to Takahashi (1980), debris flow can be defined as *a fluidized mixture of all sediment sizes, with boulders accumulating and tumbling at the front of the debris wave.* Behind the front, the finer grained, more fluidic debris follows. If the particles cannot disperse throughout the entire depth of flow, a fairly clear separated upper and lower layer can be distinguished, which is specified as immature debris flow (Takahashi, 1987). Rickenmann (1991) described debris flow as an unsteady, pulsating flow of a mixture of water and both coarse and fine sediments. Debris flows can increase in volume by entraining additional sediments, water and organic debris.

According to Takahashi (1981) three mechanisms for the initiation of debris flow can be distinguished; a landslide from a hill slope transforming into a debris flow, mobilization of bed material at sufficiently large bed shear stresses and steep slopes, and motion of temporary barriers of material deposited in the channel.

The first mechanism refers to debris flows following burst of intense rain during long storms (Zimmerman, 1990), or initiated by snowmelt or moraine-burst. The area of moving particles is a function of the slope; the steeper the slope the larger the depth of the sediment gravity flow. With increasing gradients, the thickness of the unstable layer and, if the bed slope exceeds a critical value, the sediment concentration increase.

The instability of the underground has also been observed by Smart and Jaeggi (1983), when, at slopes steeper than 20 %, the bed material generated a bulk movement of material. Okunishi and Suwa (1985) recognized the attribution of rainfall and (sub)surface run-off, and statistically determined the critical discharges required for initiation of debris flow.

6.5.2. Rheology of debris flows.

Debris flow can be stony if composed of gravel, cobbles and rocks, or muddy. Debris flows can be considered mud flow, if more than half of the solid material is smaller than gravel sizes (Bradley and McCutcheon, 1987). In the classification of the debris flow, the Bagnold number N can be used, which represents the ratio of inertial to viscous stresses

$$N = \frac{\rho_s \lambda^2 D^2 \frac{\partial u}{\partial z}}{\mu_T} \qquad (6.16)$$

where ρ_s is the particle density, D the particle diameter, μ_T the viscosity of the bulk of the flow, and u velocity of the fluid. λ represents a "linear concentration of the solid in the mixture" (e.g. Takahashi, 1991) and is defined as

$$\lambda = \left[\left(\frac{c_*}{c} \right)^{1/3} - 1 \right]^{-1} \qquad (6.17)$$

where c is the volume concentration of the solid in the flow and c_* is the maximum possible concentration of the solid when packed. Takahashi (1991) rewrites the Bagnold number by considering viscosity effects of dense populations of particles.

At high Bagnold numbers, Takahashi (1991) distinguishes inertial flow including stony debris flows, immature debris flow, turbulent mudflow and hybrid debris flow (partially dispersed).
At lower Bagnold numbers, the flow is in the macro-viscous range (Takahashi, 1991).

Different theories describing the rheologic behaviour of debris flows have been developed. In the past, two principally different theories have been applied (e.g., Chen, 1988; Rickenmann, 1990). The visco-plastic model applied to flows containing fine material in a viscous slurry is based on the "Bingham-approach". The second model refers to Bagnold's (1954) concept of dispersive pressure, and considers grain collisions in debris flow (Takahashi, 1978; Takahashi, 1980; Takahashi 1991). As a result, the relation between velocity gradient and shear rate is non-linear.

To describe both fluid effects and grain-grain interactions, the combination of both approaches has resulted in more complex models (O'Brien and Julien, 1983; Chen, 1988a; 1988b; Julien and Lan, 1991). Because of the complex examination of grain and fluid processes, testing of theories is difficult (Rickenmann, 1990)

The transport capacity of the mixture, and subsequently the development of the debris flow, will increase if fine material such as clay and silt is suspended (Bradley and McCutcheon, 1985; French, 1987).

Since mass movements of debris constitute a damaging form of erosion, methods for predicting initiation should be complemented with methods predicting deposition. With the different rheologic models used, threshold slope conditions predicting the occurrence of a steady, uniform debris flow have been derived (Takahashi, 1978; Chen (1988). With conservation equations of momentum and mass, Takahashi and

Nakagawa (1991) suggested a method, which assesses the hazards under a volume and intensity of rainfall.

The deposition of debris flow is determined by its rheologic behaviour, and the geometry of the system of mountain streams, and has been described mainly empirically (Ikeya, 1981: Hungr, 1984). According to Takahashi and Nakagawa (1991), debris flow behaves very similar to a continuous fluid, until just before it stops. Debris flow deposition usually occurs where slope gradients are declining, or where the flow abruptly entered a low gradient channel at tributary junctions. Benda and Cunda (1990) developed a simple model for the prediction of the debris flow deposition, using channel gradient and tributary junctions, without considering rheologic properties.

Chapter seven.

Conclusions and recommendations.

7.1. General.

Research on hydraulics and morphology of mountain rivers is required to enable an optimum land and water usage in mountainous regions. In this chapter, lacks in knowledge on hydraulic and morphological processes in mountain rivers are reviewed.

In current research, conceptual theories or empirical relations in deterministic, statistical or combined models have been formulated. Although descriptive, statistical models incorporate the complexity of the phenomena described, physical processes and parameters often remain unknown or unquantified when black-box parameters are manipulated statistically. This endangers the interpretation of results and limits the validity of predictions.

In deterministic models, the movements of water and sediment are modelled with rigorous conservation equations and semi-empirical equations, either differential or algebraic (e.g., Rahuel et al. 1989). Considering the complex boundary conditions, the complicated flow behaviour (Davies, 1989) and the importance of extreme, and subsequently low-frequency variables (Ashmore and Day, 1988), the prospect of theoretical simulation models of morphological processes in mountain rivers seems rather remote.

7.2. Recommendations for further research.

7.2.1. Hydraulics.

Flows in mountain rivers can be unsteady and non-uniform as a result of steep hydrographs, non-uniform geometries and lateral inflow. The prediction of quantities and qualities of water supplied to mountain rivers is complex and cannot be described accurately. Due to differences in meteorologic or hydrologic processes in catchment areas, flood hydrographs in mountain rivers can exhibit different populations. Distinction of streamflow and data by causative processes can improve the analysis of flows and morphology in mountain rivers (Jarret, 1990).

Recommended research topics include

- *effect of different inflow types on river hydrograph and morphology*
- *development of discharge measuring/runoff routing methods*
- *estimation and/or prediction of peak discharges*

In mountain rivers, slopes, width, depths, roughness and flow patterns can vary

significantly along the river. Flow in non-uniform geometries has been modelled by calibration of irregular geometries in empirical, reach-averaged constants, by distinction of control-sections with local stage-discharge curves or by linear summation of energy dissipation processes (e.g., Miller and Wenzel, 1985; Hey, 1988; Egashira and Ashida, 1989).

Recommended research topics include

- *effects of non-uniform geometry (at various stages)*
- *effects of transitions in flow regime on sediment transport and morphology*
- *propagation of flood waves through non-uniform reaches*
- *flooding hazards at alluvial fans; flood plains and terraces*

In mountain rivers, clusters of roughness elements with varying scales can be observed (debris, rock outcrops or larger-sized sediment particles on the river bed). Therefore, the structure of flow can vary significantly at different locations. Flows can change from extremely non-uniform over large roughness elements at low stages to relatively uniform at higher stages.

At intermediate- and large-scale roughness, turbulent flows are controlled by individual heights and erratic arrangement of roughness elements and are locally accelerating and decelerating. Free surfaces can be turmoil as a result of drag waves, cross waves, hydraulic jumps, etc. At high Froude numbers and small roughness scales, roll waves can develop.

Recommended research topics include

- *effects of scale and distribution of roughness elements*
- *occurrence, behaviour and effects of roll waves*

On permeable river-beds, a subsurface flow-component can be observed that affect the structure of flow and the transport of sediment. Although verification measurements are complex, subsurface flow through the permeable bed has been suggested considerable.

Recommended research topics include

- *structure of free surface flow*
- *structure of subsurface flow*
- *effects on initiation and behaviour of sediment transport*

7.2.2. Morphology.

The morphological stability of mountain river reaches changes significantly in place

and time due to varying, non-uniform conditions of flow and localized, erratic input of sediment. In an ideal mountain river course, stable sections can be found in erosion zones, where sediments are large and discharges are small. Unstable morphology can be located in flatter sediment deposition areas, where flows and sediments accumulate.

Recommended research topics include

- *sediment movements in mountainous regions*
- *effects of changes in composition and rate of sediment input on river morphology*
- *interactions between river morphology and sediment input*
- *response of morphology to sediment control structures*

In long-term evolution models, seasonal and other short-term variations are often considered compensated, assuming constant, equilibrium morphological stream characteristics (e.g., Parker, 1989; Di Silvio and Peviani (1989). However, most processes in mountain rivers are initiated by localized, extremely intense events (e.g., Naef et al. 1988), that individually affect the river morphology (e.g., Whittaker, 1985). Single discharge values such as "dominant discharge" cannot account for effects of sequence, duration and magnitude of stream flow variability (Novak, 1991). The effects and relevancy of the extremely variable hydraulic conditions should be investigated, to enable description and prediction of long-term evolution. Due to the low-frequency character of the relevant, large-scale processes, measurement data are hard to obtain.

Recommended research topics include

- *effects of extreme variability of flow patterns on river*
- *effects of irregular sequences of water and sediment input*

At large-scale deposition areas, sediments usually are finer with smaller size ranges, and threshold conditions can be exceeded more frequently. Changes in flow conditions have often been assumed instantaneous relative to morphological changes (e.g., Ghilardi and Menduni, 1989; Di Silvio and Peviani, 1989), in unstable reaches, the non-linear interactions between morphological changes and non-uniform movements of sediment and flow are relevant (De Vries, 1965). Recently, non-linear solutions have been developed to describe the non-linear morphological behaviour.

Recommended research topics include

- *morphological changes during floods*
- *extension of (weakly) non-linear morphological analyses to non-uniform sediments*

Beds in mountain rivers can exhibit a wide range of particle sizes that can be arranged in clusters or patches of stable and unstable sections (e.g., stable steps, armoured

sections at riffles, etc.). Sediment can be stored and transported relative to conditions of flow and stability of neighbouring elements (debris or wood logs or imbricating, larger-sized sediment). Instability of those stable sections induces the release of finer sediments.

As a result of non-uniform flows, in-channel storage of fine sediments at or in the stable matrix of bed material can occur. In absence of upstream sediment supply, the bed composition affects the rate and composition of bed load and suspended load. In return, the level and size distribution of surface layer and substrate are affected by the history of flood events that have eroded or deposited sediment. Knowledge on the behaviour of partially and entirely armoured beds during floods is rather scarce.

Recommended research topics include

- *development of horizontal bed surface composition*
- *changes in bed composition and level during floods*
- *infiltration and flushing of sediments in coarse matrices of bed material*
- *effects of a non-uniform morphological stability on flow pattern and sediment transport*
- *effects of armouring on bed deformation, hydraulics and sediment transport*
- *effects of bed forms on particle entrainment, stability of armour layers*

7.2.3. Sediment transport.

In mountain rivers, sediment transport often cannot be determined by the discharge alone. Sediment transport and subsequent morphological responses of a river to a flood depend on (i) the bed material size-distribution and (ii) the distribution in time and place of hydrographs and sediment supply (Mizuyama, 1989; Di Silvio and Peviani, 1989).

Empirical transport-formulae only predict flow-controlled sediment movements in uniform conditions of flow. However, in mountain rivers hydrographs are steep, beds can be non-alluvial, irregular and non-uniform with graded sediments. The non-uniformity of sediment transport can be expected to change with the structure of flow at different stages. Considering the variable streamflow pattern, the non-uniformity of flow and sediment transport should be accounted for at different stages.

Recommended research topics include

- *effects of non-uniform flow conditions*
- *sediment transport in unsteady flows*

The rate and composition of sediment transport can be controlled by (i) conditions of flow or (ii) entrainment and supply from bed, banks and upstream reaches. The size

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distribution of the sediment transported varies at different reaches and stages of flow; in general, selective transport occurs at low flows, while the bed-load size-distribution approaches the size distribution of the bed material at higher flows. Sediment transport has been observed to fluctuate stochastically due to non-uniform conditions of supply, entrainment and flow.

Recommended research topics include

- *effects of (horizontally and vertically) non-uniform bed composition*
- *transport of sediment mixtures*

At higher rates of sediment transport, moving particles occupy a significant part of the cross-section and the momentum exchange between flow and transported sediment increases. At extremely high rates of sediment transport, the rheology of the flow changes. The behaviour of high concentration flows ranging from sediment-laden to granular or debris flow has been subject of recent research. Due to the complex rheology and the scarcity of field data, current knowledge is limited.

Recommended research topics include

- *effects of changing cross-section*
- *contribution to mass and momentum transfer*
- *changes in rheology of flow*
- *occurrence and behaviour of granular flows*

List of main symbols.

Symbol	Definition	Dimension
a	depth of flow	[L]
A	liquid-flow area	[L^2]
A_c	cross-sectional area	[L^2]
A_d	flow area at formative discharge	[L^2]
A_b	permeable, alluvial bed-area	[L^2]
A_i	area occupied by particle with D_i	[L^2]
A_s	area of storage zones	[L^2]
A_{ai}	armoured-area of the bed, occupied by the i-th fraction	[L^2]
A_d	size of the catchment-area	[L^2]
a/D_{84}	relative roughness	[-]
a_m	depth of water-sediment mixture	[L]
B	channel width	[L]
B_c	width of fan-contour	[L]
b_b	width of bed-load transport	[L]
b_s	width of suspended-load transport	[L]
C	Chezy's roughness-coefficient	[L$^{1/2}$T^{-1}]
c	tracer-concentration	[ML^{-3}]
c_0	background tracer-concentration	[ML^{-3}]
c_s	tracer-concentration in storage zones	[ML^{-3}]
c_L	tracer-concentration of lateral inflow	[ML^{-3}]
c_δ	tracer-concentration near interface of mainstream and storage zone	[ML^{-3}]
c_g	tracer-concentration on sediment	[ML^{-3}]
c_i	armouring arrangement coefficient of fraction i	[-]
$c_i(a)$	volumetric sediment-concentration at $z = a$	[-]
C_b	loosely-packed sediment concentration	[-]
C_{bi}	volumetric bed-load concentration of fraction i	[-]
C_{si}	volumetric suspended-load concentration of fraction i	[-]
D	dispersion coefficient	[L^{2}T^{-1}]
D_x	particle-size coarser than x % by weight of the sediment	[L]
D_L	smallest stable partice-size	[L]
D_a	maximum sediment particle-size in the active layer	[L]
D_{max}	maximum sediment particle-size	[L]
D_m	median sediment particle-size	[L]
f	Darcy-Weisbach's friction factor	[-]
f_D	drag force per unit volume of bed sediment	[ML^{-2}T^{-2}]
f_i	volumetric fraction of sediment with D_i	[-]
F_i	volumetric fraction of sediment with D_i in the surface layer	[-]
F_L	volumetric fraction of smallest stable fraction L	[-]
F_{oi}	volumetric fraction of sediment with D_i in the substrate	[-]

Symbol	Description	Dimension
Fr	Froude number, defined as $Fr = u/\sqrt{ga}$	[-]
g	gravitation constant	$[LT^{-2}]$
g_T	impulse response-function	[-]
h_i	hiding function for fraction i	[-]
i	bed slope	[-]
i_f	friction slope	[-]
k_s	equivalent roughness	[L]
L_*	characteristic length-scale of sediment transport-response	[L]
M	mass of injected tracer	[M]
M_s	mass of dry sediment per unit area	$[ML^{-2}]$
ΔM_i	number of particles dislodged of fraction i	[-]
n	Manning's roughness coefficient	$[TL^{-1/3}]$
P	pressure	$[ML^{-1}L^{-2}]$
p	porosity of sediment	[-]
p_i	portion in bedload of fraction i	[-]
Q	discharge	$[L^3T^{-1}]$
Q_d	formative discharge	$[L^2T^{-1}]$
q	discharge per unit width	$[L^2T^{-1}]$
q_b	base-flow discharge per unit width	$[L^2T^{-1}]$
q_c	critical discharge per unit width	$[L^2T^{-1}]$
q_e	equivalent steady-discharge per unit width	$[L^2T^{-1}]$
q_L	lateral inflow of water	$[L^2T^{-1}]$
ΔQ_i	number of particles deposited of fraction i	[-]
R	hydraulic radius	[L]
Re	Reynolds number, defined as $Re = UD_{84}/\nu$	[-]
S	total, volumetric sediment-transport	$[L^3T^{-1}]$
s	total, volumetric sediment-transport per unit width	$[L^2T^{-1}]$
s_i	transport rate of size-fraction i	$[L^2T^{-1}]$
s_L	lateral inflow of sediment	$[L^2T^{-1}]$
s_B	bed load per unit width	$[L^2T^{-1}]$
S_{Bi}	bed load of fraction i per unit width	$[L^2T^{-1}]$
s_S	suspended load per unit width	$[L^2T^{-1}]$
S_{Si}	suspended load of fraction i per unit width	$[L^2T^{-1}]$
U	cross-sectional-averaged velocity of flow	$[LT^{-1}]$
u	longitudinal component of velocity	$[LT^{-1}]$
u_*	friction velocity	$[LT^{-1}]$
v_d	characteristic dispersive sediment-flux velocity	$[L^3T^{-1}]$
V_i	particle volume with size D_i	$[L^3]$
w_f	particle fall-velocity	$[LT^{-1}]$
z_b	bed level	[L]
α	energy or Coriolis coefficient	[-]
α_2	two-dimensional geometrical coefficient	[-]
α_3	three-dimensional geometrical coefficient	[-]
α_s	mainstream-storage zone exchange coefficient	$[T^{-1}]$

β	momentum or Boussinesq coefficient	[-]
β_i	volumetric fraction of D_i-sized particles in the mixing layer	[-]
β^*_i	volumetric fraction of D_i-sized particles in the substrate	[-]
Δ	relative sediment density, defined as $\Delta = \rho/\rho_s\text{-}1$	[-]
Δ_b	bed-form height	[L]
δ	thickness of boundary layer	[L]
δ_a	thickness of active layer	[L]
δ_b	thickness of bed-load layer	[L]
δ_m	thickness of mixing layer	[L]
δ'_m	instantaneous thickness of mixing layer	[L]
η	fluid viscosity	$[ML^{-1}T^{-1}]$
θ	bed slope angle	[-]
Λ	mean particle step-length	[L]
λ_1	wetted frontal-area of roughness elements per unit bed-area	[-]
λ_2	plan area of roughness elements per unit bed-area	[-]
λb	bed-form length	[L]
ν	kinematic vicosity	$[L^2T^{-1}]$
Π	Coles' wake parameter	[-]
ρ	density of water	$[ML^{-3}]$
ρ_s	density of sediment	$[ML^{-3}]$
τ	uncorrected bed shear-stress	$[ML^{-1}T^{-2}]$
τ_{ci}	critical shear-stress of D_i	$[ML^{-1}T^{-2}]$
τ_*	dimensionless grain shear-stress	[-]
τ_{*c}	critical, dimensionless grain shear-stress	[-]
τ_*'	dimensionless form shear-stress	[-]
τ_{*tr}	dimensionless transition bed shear-stress	[-]
Φ_B	total dimensionless bed-load	[-]
Φ_{Br}	bed-load reference value	[-]
Φ_{Bi}	dimensionless bed-load of fraction i	[-]
Φ_{oi}	sediment flux of fraction i through lower boundary of mixing layer	$[L^3T^{-1}]$
Φ_{bi}	sediment flux of fraction i through upper boundary of mixing layer	$[L^3T^{-1}]$
Φ_{si}	sediment flux of fraction i through upper boundary of bed-load layer	$[L^3T^{-1}]$
Φ_{di}	dispersive flux of fraction i through lower boundary of mixing layer	$[L^3T^{-1}]$
ϕ_{50}	ratio of τ_{*50} to a reference value τ_{*r50}	[-]
ϕ'	dynamic friction angle	[-]

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