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Channel Restoration Design for Meandering Rivers

Philip J. Soar and Colin R. Thorne

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Preface

The investigation reported herein was conducted by and for the U.S. Army Engineer Research and Development Center (ERDC). All research reported herein was funded by the ERDC, Vicksburg, MS, as part of the Flood Damage Reduction Research Program. The study was done under Work Unit No. 32878, "Channel Restoration Design." Dr. Ronald R. Copeland, ERDC, supervised this research.

This report was prepared by Drs. Philip J. Soar and Colin R. Thorne, University of Nottingham, United Kingdom, for the ERDC, Coastal and Hydraulics Laboratory (CHL), Vicksburg, MS. The United States sand-bed data were collected during a fieldwork program between July 1998 and January 2000. Fieldwork assistance was given by Messers Calvin Buie and David Derrick, Mses. Dinah McComas and Rebecca Soileau, Mr. Curtis Shields, CHL, and various United States Geological Survey (USGS) staff members. Bed and bank material samples collected from these sites were analysed by ERDC and USGS laboratory technicians. Mr. Jon Fripp, formerly U.S. Army Engineer District, Baltimore, MD, assisted with the fieldwork for the case study.

This study was conducted over the period from 1998 to 2001 under the general supervision of Mr. Thomas W. Richardson, Acting Director, CHL, and Dr. Yen-hsi Chu, Chief, River Sedimentation Engineering Branch, CHL.

At the time of publication of this report, Dr. James R. Houston was Director of ERDC, and COL John W. Morris III, EN, was Commander and Executive Director.

Conversion of SI to US Customary Units

The following units are used in this report and may be converted as indicated:

Length	1 m = 3.281 ft	Discharge	1 m^3s^{-1} = 35.32 ft^3s^{-1}
Area	1 km^2 = 0.39 square miles	Mass	1 tonne = 1.1 US tons

CHAPTER 1

Introduction

1.1 PROJECT BACKGROUND: PURPOSE AND PHILOSOPHY

This report presents an enhanced channel design framework for restoring the channels of meandering rivers using a geomorphic engineering approach that is based on bringing together geomorphic principles and conventional river engineering methods.

The design of stable river channels with mobile bed materials poses complex problems in the fields of hydraulic engineering, fluvial geomorphology and sedimentology. Furthermore, the contemporary viewpoint within applied fluvial studies of ecologically sound river management has established a general orthodoxy among practitioners which requires rivers to provide a range of aquatic and riparian environments capable of supporting a high biodiversity. This prerequisite, together with the provision of recreational activities and good aesthetics, presents further challenges to the channel designer. Consequently, the pre-eminence of flood-control objectives implicit in traditional engineering practises, and consequently negligible consideration of riverine ecology and catchment geomorphology, have been superseded by multidisciplinary, or ‘multifunctional’, objectives which endeavour to optimise the management interests of the river engineer, fluvial geomorphologist and riverine biologist. To complement and further this transition in management goals, a new framework for designing stable channels for river restoration is required.

1.2 CHANNELISATION AND CHANGING APPROACHES TO RIVER MANAGEMENT

The design and construction of artificial waterways has been practised since the earliest civilisations developed water management to meet the proliferation in domestic, communication and agricultural demands. The engineering community has traditionally modified the riverine landscape for many purposes:

- To expand commercial trade routes by canal design and river dredging to aid navigation. For example, the expansion of the intricate canal network in England during the late eighteenth and early nineteenth centuries exemplifies the importance of constructing and modifying waterways for improving infrastructure, industrialisation and economic gain. By 1750, Britain had some 1400 miles of navigable river and by 1830, approximately 2500 miles of canal had been built (Dodgshon and Butlin, 1990, p. 460);

- To improve irrigation by diversion and canal design (e.g. Lacey, 1930; Inglis 1949b);

- For the exploitation of floodplains for agriculture, habitation and mineral excavation (Brookes, 1988, p. 8). While natural river systems have been channelised, floodplains have been drained, and the diffuse riparian boundary, or corridor, between river and floodplain has subsequently diminished;

- To reduce the occurrence of flooding along networks of increasingly cultivated and urbanised floodplains by channelisation schemes to maximise efficiency in water conveyance, regulation and diversion. Flood control channels are described in more detailed in Section 1.3.2.

As Boon (1992, p. 11) comments, rivers ‘have been abstracted from, fished in, boated on, discharged into; their headwaters have been diverted; their middle reaches dammed; their floodplains developed’. As a consequence of these demands, the natural geometric configuration of the river channel has been extensively modified by resectioning the cross-sectional shape, realigning the planform, constructing diversion channels and/or constructing artificial flood banks (Hey, 1994a; Brookes, 1988). The term ‘channelisation’ refers to all or some of these modifications:

i) ***Resectioning***

This approach aims to increase the in-bank discharge capacity by enlarging cross-sectional area and/or elevating the slope. Engineering methods designed to increase capacity include dredging and widening. Channels are also deepened in order to lower the water table and increase the agricultural productivity of the floodplain. The result, particularly in urban areas, is a uniform, trapezoidal channel configuration that is wider and deeper than the pre-existing natural channel. Banks frequently have to be stabilised by sheet piling, riprap or concrete masonry, such as revetments, in order to produce and maintain a rectangular cross-sectional shape. In theory, a flood control channel should have a cross-sectional area that provides the maximum efficiency in discharge with the minimum of excavation.

ii) ***Realignment***

This approach aims to increase flow velocity by reducing the natural sinuosity or creating a straight alignment, thereby steepening the bed slope and increasing the energy gradient. Methods include dredging of shoals and cutting off meander bends (to produce ‘cut-offs’). During excavation, the channel is often regraded, whereby the natural pool-riffle sequence and large roughness elements are removed from the pre-existing, stepped long profile. Realignment is usually undertaken as part of a resectioning project. The term ‘canalisation’ is often used to reference projects that yield straight channel alignments.

iii) ***Diversion Channels***

This approach aims to redirect flow away from a protected area. This is often the preferred method of flood protection in urban areas where spatial restrictions inhibit resectioning.

iv) ***Flood Banks***

Levees (or embankments) are intended to increase channel conveyance capacity by reducing the frequency of over-bank flow. For large rivers in lowland floodplains, this is probably the oldest method of protecting floodplain developments from flood flows. The method conventionally involves the construction of levees as close as possible to the river to maximise the protected area. Hydraulic models are applied to define a suitable levee elevation to contain the required design discharge. In some cases, where there is an extensive floodplain, flood banks can be constructed outside of the meander belt of an actively migrating river.

Britain has a long history of channelisation extending back at least 2000 years (Brookes, 1988, p. 9). The traditional management practise was to align or divert small streams to flow parallel with field systems, which often produced shallow field depressions following changes in stream course. With urban expansion during the past 150 years, management objectives gradually focused more and more on flood protection. In England and Wales, approximately 96 percent of lowland rivers have been modified (Brookes and Long, 1990) and approximately 25 percent have been channelised since the 1930s (Brookes et al., 1983, p. 109). In the Severn-Trent basin alone, approximately 34 percent of main river is channelised (Brookes et al., 1983, p. 108). In a channelised stream, artificially repairing the bank-lines inhibits the natural migrating tendencies of the channel, creates an unnatural interface between the channel and its floodplain and, in turn, significantly changes the delicately adjusted balance between form and process of a natural stream. In many cases, this change causes deterioration of the quality of physical habitat. This type of intense management was not just confined to the U.K., with similar flood control management practises in many European countries (Brookes, 1987b, 1988, 1990; Iversen et al., 1993), in the U.S.A. (Little, 1973; Winkley, 1982) and elsewhere. For example, intensive farming and urban development in Denmark have left only 2.2 percent of rivers with sinuous courses, most of which are large rivers that are difficult to regulate and reaches unsuited for agricultural development (Brookes, 1987b, 1988). By providing legislation for new land drainage works, the Land Drainage Acts of 1861 and 1930 have played important roles in shaping the river landscape in the U.K. as it is viewed today. The number of engineering works to improve flood protection and enable more intensive agricultural production on floodplains accelerated through the period 1940 to 1980 during which there was a dramatic and insensitive deepening, widening and straightening of many of our lowland rivers (Holmes, 1993, p. 27). This period marks a *denaturalisation* of the landscape as societies came into heightened tension with their natural environments (Peet, 1989, p. 43).

In particular, the expansive wave of countryside development, agricultural mechanisation and irresponsible large-scale pollution during the ‘permissive, self-indulgent’ 1960s (Nicholson, 1993, p. 7), ‘forced a shift in resources from the still unfinished tasks of care for the natural and semi-natural environment to fire-brigade operations against powerful and pigheaded vested interests cashing in on natural resources’ (Nicholson, 1993, p. 7). Even as recent as 1976, the Land Drainage Act instructed the ten Regional Water Authorities of England and Wales to initiate a programme of river works to improve floodplain drainage for intensive agricultural production, thereby further diminishing floodplain wetlands and riparian habitats (Holmes, 1993, p. 27). The extent of channel works in England and Wales between 1930 and 1980 is depicted by Brookes (1988) and Brookes et al. (1983).

In the U.S.A. there has been an intense period of channelisation for at least 150 years during which at least 320,000 km of river have been modified (Little, 1973), primarily for the purpose of draining floodplain land for agriculture, flood control and transportation of goods. In total, approximately 53 million hectares of wetlands have been drained. This massive effort was primarily co-ordinated by the U.S. Corps of Engineers under the Flood Control Acts of 1936 and 1944 and the Soil Conservation Service which has been responsible for many smaller watershed projects under the Watershed Protection and Flood Prevention Act of 1954 (Brookes, 1988, p. 9). Leopold (1977, 430) estimated that approximately 26,550 km of river had been modified in the U.S.A. since the Flood Control Act of 1936.

In Western Europe and the United States of America, the engineered control of rivers was a post-war management approach that was deemed appropriate to meet the demands of rapid population expansion, encroaching floodplain developments and economic restructuring. Unfortunately, government policies on food production, flood defence and land drainage have been extremely damaging for wetlands and riverine wildlife.

In contrast to the engineering of flood control channels, the design of waterways which aim to mimic the physical attributes of natural alluvial systems is a science in its infancy (Osborne et al., 1993, p. 188) and engineers and fluvial geomorphologists currently strive to further their understanding of the complexities of river form and mechanics. For over a century, investigative research has been performed into the physics of fluid flow in open channels, the mechanics of sediment transport and the relationships between the flow regime and channel geometry displayed by stable canals and, more recently, rivers.

Since the mid-1970s, approaches to applied fluvial geomorphology and river engineering have undertaken a gradual change in attitude towards environmentally sensitive practices and a concern to work with nature rather than against it. In fluvial geomorphology, this is evident in the promotion of river restoration projects as alternatives to channelisation. This type of response can be interpreted in terms of the homeostatic nature of ‘Gaianism’ (Lovelock, 1979), whereby society’s detrimental impacts upon the river landscape can be rectified by learning from previous interventions and applying this knowledge to return the landscape to a condition of stability. The contemporary paradigm of ‘Person-within-environment’ (Petts, 1995, p. 17) requires a multidisciplinary approach that addresses the environmentally aware needs of all interest groups, including the public, in order to succeed. As Warren (1993, p. 15) commented, ‘naturalness is high on the agenda of ecologists, aesthetes and many others’. As many environmental questions require answers to problems regarding watersheds, hydrology and river response, fluvial geomorphologists must become part of an interdisciplinary scientific community (Smith, 1993, p. 256; Shields, 1982a, 1982b). Integrated river basin management and catchment planning in the 1990s identified the catchment as the important management unit and adopted an ecocentric, holistic appraisal of this unit by attempting to balance the requirements of all groups interested in the changing character of the river landscape (Gardiner, 1991; Newson, 1992). As the antithesis to the conventional, ‘technocentric’ flood control perspective, this new attitude requires geomorphological assessment as an integral component of contemporary management practice. Moreover, the catchment scale approach marks a revival of the hydrological dimensions of regional geomorphology explicit in the work of Davis (1899, 1902), and Schumm (1977).

Mackin (1948) considered a shift to working with nature, rather than against it, as the inevitable outcome of having to respond to the undesirable consequences of channel modifications. Half a century later, his foresight has proven true:

> ‘The engineer who alters natural equilibrium relations by ... channel improvement measures will often find that he has the bull by the tail and is unable to let go - as he continues to correct or suppress undesirable phases of the chain reaction of the stream to the ‘initial stress’ he will necessarily place increasing emphasis on study of the genetic aspects of equilibrium in order that he may work with rivers, rather than merely on them’ (Mackin, 1948).

In England and Wales, the transition to environmentally sensitive river management has been encouraged by changes in political legislation and public awareness since the early 1980s (Brookes and Gregory, 1988, pp. 56-57; Heaton, 1993; Holmes, 1993, p. 28; Ward et al, 1994, pp. 382-392). A summary of some of the recent U.K. legislation that is relative to environmentally sensitive river management is given:

- The 1973 Water Act (Section 22) stressed that Water Authorities must ‘have *regard* to the desirability of preserving the natural beauty, of conserving the flora and fauna and geological or physiographic features of special interest’.

- The 1981 Wildlife and Countryside Act extended the policies of the 1973 Water Act by requiring the Regional Water Authorities and other bodies concerned with land drainage to safeguard fisheries and further the conservation of flora, fauna and physiographic features.

- The 1982 House of Lords Select Committee on Science and Technology stressed that legislation concerning wildlife interests could not be furthered because of a lack of baseline information on rivers. This has encouraged greater data collection to further understanding of the coupling between geomorphological and ecological systems and stresses the importance of baseline information at an early stage of river management projects. In response, taxonomic River Corridor Surveys have been developed by the Nature Conservancy Council (NCC) and include typing British rivers according to their flora (Holmes, 1983) and undertaking river corridor surveys and habitat assessments (Ash and Woodcock, 1988; Holmes, 1986) to expand the knowledge of the wildlife resources to be found in riverine systems and identify potential enhancement opportunities. Numerous studies classifying invertebrate assemblages have also been undertaken (e.g. Wright et al., 1989; Wright et al., 1992, for the Institute of Freshwater Ecology). Coles et al. (1989) discussed the value of river wildlife databases for use in the preliminary stages of river management projects.

- The 1986 review of land drainage activities (Holmes, 1986) revealed that more than 2000 km of 'main' river were dredged annually and that the water authorities had minimal conservation personnel.

- The 1988 Statutory Instrument 1217 (Her Majesty's Stationary Office (HMSO), 1988) which requires land drainage projects to undertake an Environmental Assessment (EA). It is increasingly the case that good river management practice demands an EA on the basis that it provides an opportunity to incorporate issues from various disciplines, including public involvement. The EA framework is imperative for a project to be sustainable in its human setting as well as its physical and biological settings (Brookes, 1995a; Kondolf and Downs, 1996, p. 130).

- 1989 Water Act legislation brought about the splitting of the previous regional water authorities into ten privatised water supply and sewerage companies and the formation of a new regulatory body, the National Rivers Authority (NRA, now Environment Agency, EA), to promote new government policies concerning: flood defence; water quality (pollution control); water resources; fisheries; recreation (in relation to water-based activities); navigation (in some regions), and; conservation (Heaton, 1993, p. 301). Section 8(1) of the Water Act instructs the Authority to '*further* the conservation and enhancement of natural beauty and the conservation of flora and fauna and geological or physiographic features of special interest'. Notably, the main difference from the 1973 Water Act is the term '*further*' which replaces the phrase 'have *regard* for'. Furthermore, a new duty was placed upon the NRA under Section 8(4) of the 1989 Water Act, allowing the Authority to promote landscape conservation and 'the conservation of flora and fauna which are dependent on the aquatic environment', thus permitting the NRA to undertake conservation schemes independently of its other obligatory tasks (Heaton, 1993, p. 304).

- The 1991 Water Resources Act and Land Drainage Act 1991 endorsed previous Acts by instructing the NRA to *further* the conservation of flora and fauna.

- In 1992 the River Restoration Project was formed and several river restoration projects were established in Denmark and England for the purpose of demonstrating sensitive river engineering techniques (Holmes, 1998).

- The European Union Directive on the Conservation of Natural Habitats and Wild Fauna and Flora (92/43/EEC) requires members to safeguard the habitats of endangered species and to take positive conservation action throughout their country by incorporating conservation measures into decision-making procedures (Ward et al., 1994).

- Changes in agricultural policies within the European Community, for example 'set-aside', which encourages farmers to take land out of production. Designation of Environmentally Sensitive Areas (ESAs) and Countryside Stewardship have also influenced river management practises.

- Improved environmental awareness within the engineering community and the desire to integrate flood defence and land drainage Standards of Service with the aspirations of other interest groups, including improved fisheries, recreation and visual amenities.

- Increased public awareness of environmental conservation since the 1980s, public pressure for improving the physical landscape and formation of various organisations dedicated to restoring the river environment. This has been aided by handbooks outlining the conservation value of rivers which have been produced by such organisations as the Nature Conservancy Council (Newbold et al., 1983), the Royal Society for the Protection of Birds, Environment Agency and the Royal Society for Nature Conservation (Lewis and Willams, 1984; Ward et al., 1994). Setting out examples of good practice by water authorities and river managers and how managed rivers could be improved for wildlife has encouraged a fundamental change in the flood defence engineers' way of thinking such that enhancement works are now routinely incorporated into flood defence projects (Heaton, 1993, p. 308).

These developments are paralleled in the U.S.A. (Brookes, 1988; Brookes and Gregory, 1988):

- The 1958 Fish and Wildlife Co-ordination Act requires that wildlife be considered in the design and implementation of water resource development programmes and the mitigation of development impacts. The act demands that consultation should be made with the United States Fish and Wildlife Service at the planning stages of engineering projects which may compromise wildlife interests.

- The 1968 Wild and Scenic Rivers Act designated selected rivers of the nation, 'which with their immediate environments, possess outstanding remarkable scenic, recreational, geologic, historic, cultural or other similar values, shall be preserved in free-flowing condition'.

- The 1969 National Environmental Policy Act (NEPA) requires preparation and review of environmental impact statements for 'major Federal action significantly affecting the quality of the human environment'. The act requires the development of methodologies which consider wildlife resources in project planning and decision making, for example the Environmental Assessment methodology of the Soil Conservation Service (Brookes, 1988, p. 61; U.S. Department of Agriculture, 1977).

- Numerous published articles since 1970 have described the controversy surrounding river channelisation and stressed public concern over detrimental environmental consequences.

- The 1971 general policy statement of the United States Fish and Wildlife Service states that the Service would cooperate fully in the planning, formulation and implementation of proposals that are environmentally sound; minimise harmful effects on fish and wildlife and maximise enhancements. Furthermore, the policy requires that compensation should be made for unavoidable damage to fish and wildlife populations due to river engineering (Brookes, 1988).

Other relevant legislature includes the 1972 Clean Water Act, the 1973 Endangered Species Act, the 1976 Federal Land Policy and Management Act and the 1976 National Forest Management Act (Andrews and Nankervis, 1995).

In parallel with these recent directives in the U.K. and the U.S.A., the rapid development of information technology has spurred a proliferation of hydraulic modelling technology that is available to the hydraulic engineer to route flow and sediment through a river system according to known principles of fluid flow and sediment dynamics. As Smith (1993, p. 258) remarked, two and quasi-three dimensional models and analytical techniques are rapidly replacing field-based approaches in an emerging ‘push button science’ for the twenty-first century. However, it is still not possible to predict and explain the *three-dimensional* morphology and flow patterns of self-formed alluvial channels in terms of deterministic equations.

Furthermore, Mellquist (1992, pp. 6-7) identified three technical difficulties with the application of computer-based analyses for the purpose of applied fluvial geomorphology: i) there is much left to do regarding the actual development of complex numerical models; ii) it is often expensive and time-consuming to undertake the fieldwork that is necessary to collect the input and validation data required by the models, and; iii) qualified ‘universal geniuses’ are still needed to interpret and correctly apply the results. Petts (1995, p. 15) noted that 'despite numerous attempts to establish two- and three-dimensional models of channel morphology in relation to sediment and water discharge, the assumptions required to overcome mathematical indeterminacy and the lack of an adequate bed load equation remain major concerns'. Unfortunately, there has been a paucity of research that attempts to bridge the gap between observation-led research, which dominated river management during the first half of the Twentieth Century, and advanced numerical methods currently being developed within the engineering community.

Conventional flood control approaches to river management are characterised by analytical and deterministic methods (U.S. Army Corps of Engineers, 1994). Conversely, freshwater biology is a more descriptive, elaborate science whereby 'very few empirical guidelines exist for converting the engineering specifications for restoring a channel into those appropriate for sustainable aquatic habitats' (Osborne et al., 1993, p. 191). Traditionally an empirical science, fluvial geomorphology since the mid-1970s has become increasingly a science of environmental impacts, evaluation and prediction of environmental processes and environmental design (Gregory, 1984, p. 189). This is corroborated by a wealth of applied research into river channel changes resulting from channelisation (Section 1.3.2), impoundment (Gregory and Park, 1974; Petts, 1979, 1984; Williams and Wolman, 1984; Jiongxin, 1990; Church, 1995; Xu, 1997; Friedman et al., 1998; Hadley and Emmett, 1998), urbanisation (Wolman, 1967; Hammer; 1972; Fox, 1976; Booth, 1990; Gregory et al., 1992) and other land use changes such as agricultural and forestry practices and mining activities (Burns, 1972; Hill, 1976; Newson, 1980; Richards, 1979; Knighton, 1991). Park (1995) and Mackin and Lewin (1997) gave detailed reviews of river channel change resulting from various types of human impact. Disturbances that affect stream corridors in general were discussed in depth by the Federal Interagency Stream Restoration Working Group (FISRWG) (1998).

Transition to this contemporary environmentalist approach of geomorphology requires more quantitative methods of investigation. Consequently, the more analytical geomorphologist is poised to communicate effectively with both the quantitative hydraulic engineer and qualitative ecologist for consultation and specification of project objectives and appropriate management practices for channel restoration design.

1.3 RIVER RESTORATION

1.3.1 Definition and Types of Restoration Projects

The practise of river restoration is interpreted differently by different interest groups with different priorities and objectives. Brookes (1995a, 1996) and Sear (1994) listed several definitions of restoration but Cairns (1991) provided an encompassing definition as the complete structural and functional return [of the river] to a pre-disturbance state. Where measures are selectively implemented to improve specific aspects of the river, works should be termed rehabilitation or enhancement (Brookes, 1996) and the term *restoration* should be reserved for the complete restoration of habitats within a system-wide holistic framework.

Restoration is an appropriate management solution for streams which are degraded in terms of geomorphological, hydraulic and ecological diversity yet have potential to return to a more natural or quasi-natural condition, with appropriate engineering (Figure 1.1). River restoration is not a suitable practice for slightly degraded systems which can be improved via less structural interventions, such as *limitation* and *mitigation* of catchment developments. Full river restoration to *pre-disturbance* conditions is an ideal concept but not a viable solution in practise as catchment hydrology and land-use patterns change over time.

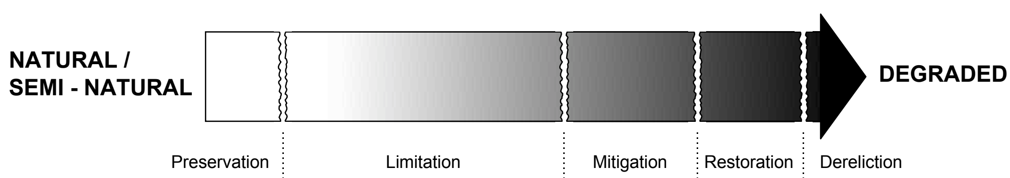


Figure 1.1 The case for conservation: A range of management options along a spectrum of decreasing conservation value (modified from Boon, 1992, p. 19).

River restoration has been used to describe different types of project (Sear, 1994):

i) Enhancement of instream habitat (Kern, 1992; Iverson et al. 1993);

ii) Reducing nutrient and sediment loads from intensively farmed agricultural land (Quinn and Hickey, 1987; Burt and Haycock, 1992; Petersen et al., 1992);

iii) Enhancing landscape quality (Pursglove, 1988);

iv) Stabilisation of eroding stream systems (Barmutra et al., 1992; Shields et al., 1993);

v) Amelioration of the detrimental effects of water pollution resulting from land use changes (Petersen et al., 1992). The Lowland Streams Restoration Workshop, Lund, Sweden, August 1991, recognised that the majority of restorative effort is aimed at water quality issues and that imitation of geomorphology and riparian vegetation of a quasi-natural or natural reference reach has received less attention (Osborne et al., 1993).

Restoration may also be required when reaches of river are relocated to further exploit the floodplain due to pressures from land use practices, such as mining and other mineral works and diverted for roads, railways and large developments. Holmes (1993, pp. 28-29) recognised four major applications for major restorative projects:

i) Urban projects. These are particularly challenging because the restored channel is constrained within a narrow corridor, with limited wildlife interest. The recreation potential of urban schemes is particularly important;

ii) Restoring straightened channels. This is a prime target for agricultural areas. Restoration of meanders within the floodplain and improvement of ecological diversity are prime objectives;

iii) River diversions. The main objective of river diversion schemes is to mimic or improve existing environmental features without compromising channel stability;

iv) Restoration in original location following temporary diversion. This is relevant for areas with temporary mineral extraction activities such as open cast mining. The target channel is the same as in iii) but with increased opportunity for floodplain restoration projects.

1.3.2 Why Restore Rivers?

Channel design approaches have developed to the point that a stable cross section can be determined for most combinations of water flow, sediment load, bed material gradation, bank material properties and valley slope. These approaches are suitable for channels with straight alignments and uniform cross sections. However, natural channels are seldom straight for a distance greater than ten times their width (Leopold and Wolman, 1957). While straight channels may perform satisfactorily as canals, geomorphologists, ecologists and engineers have recognised that the imposition of a straight alignment on a river system is not the ideal solution, and alternatives must be found.

Channelisation is the term used to embrace all processes of river channel engineering for the purposes of flood control, drainage improvement, maintenance of navigation, reduction of bank erosion or relocation for highway construction (Brookes, 1988, p. 5). In general, adjustments arising from channelisation are undesirable. There are a plethora of reasons why alternatives to channelisation must be sought, although, in general, they can be explained in terms of: i) channel instability; ii) low ecological diversity, iii) downstream flooding; iv) poor aesthetics and recreation; v) impeded recovery and; vi) unsustainable maintenance.

i) ***Channel Instability***

Straight rivers with mobile beds are prone to instability. Understanding how river channel stability has been affected by anthropogenic impacts, such as channelisation, urbanisation, impoundment and irrigation diversions, has been a major research topic since the early 1970s (Brookes, 1988, p. 4). Instability arises because the natural equilibria of process-form relationships of the pre-existing channel have been perturbed, often resulting in an imbalance between sediment supply and capacity. The most dramatic adjustments within engineered reaches occur in response to slope changes caused by channel shortening, for example as a result of bend cut-offs, or regarding, or to over-excavated bottom widths. Case studies of how channelisation schemes can induce instability have been discussed by Brookes (1987a, 1988) and detrimental impacts of engineering and management were reviewed by Hey (1997b).

Initially, increasing the energy slope elevates the stream power and may cause degradation that undermines bridges and other structures or destabilisation of the bank-lines through toe scouring. Subsequently, if the sediment supply from upstream is significantly high, there may be siltation problems in over-widened reaches which act as 'sediment traps', particularly following high-magnitude flow events (Jaeggi, 1993). Consequently, extensive bars may form at unpredictable locations, generating navigation problems and leading to a loss of flood conveyance capacity. Nixon (1959) discussed the post-project recovery of the River Tame, near Birmingham, U.K., whereby the channel had sufficient energy to revert to almost its original capacity, through bar development, in less than 30 years since resectioning. Widening a channel reduces the unit stream power, decreases the sediment discharge and stabilises alluvial deposits, encouraged by vegetation encroachment, to form permanent morphological features (Brookes, 1988, p. 101). Brookes (1992) described how the low-flow width of the low-powered River Cherwell, Oxfordshire, U.K., was reduced as a result of silt deposits and incremental vegetative encroachment over a period of 14 years after widening in 1967.

There is considerable evidence that straightened reaches can destabilise adjacent, natural reaches upstream (through head cutting by knickpoint migration) and downstream (through siltation because of the transmission of additional sediment, providing sediment supply is not limited) as the system adjusts towards an equilibrium long profile (Parker and Andres, 1976). Channelisation may result in bank failure because the natural homeostatic tendency to reduce the channel slope via knickpoint migration can lead to bed degradation and a subsequent increase in bank height. Darby and Thorne (1992) described how this equilibrium-restoring mechanism partly explained the infill of swallow holes downstream from the channelised Mimmshall Brook, Herfordshire, U.K. The processes of incision and bank widening as a response to disturbed alluvial systems, such as in channelised streams, have been developed by Schumm et al. (1984), Harvey and Watson (1986), Watson et al. (1988a, b), Simon (1989) and ASCE Task Committee (1998b). The bank-lines of straightened rivers are seldom stable because the form of a straight channel works against natural processes in turbulent flow, and in this unstable condition, rivers attempt to counteract any imposed changes and return to an equilibrium state. Therefore, controlling the tendencies for spatially organised patterns of bank retreat (through erosion) and bank advance (through accretion), leading to the re-adoption of a natural meandering course, requires considerable engineering effort. A channel that has been resectioned will only be laterally stable if the banks are artificially lined to prevent bank failure and if the bed material load is negligible, otherwise alternate bars will form.

The conventional engineering counter-solution to bed degradation is the installation of grade control structures to stabilise bed and bank toe elevations (see Biedenharn et al., 1990, for this application) and reduce the longitudinal energy gradient. These structures are both difficult to site effectively and are costly to construct. Control of aggradation may require implementation of bed load traps and maintenance dredging, both of which are costly to perform and environmentally undesirable. These measures to prevent aggradation/degradation problems by further modification of the sediment regime only tackle the symptoms of the instability, resulting in a post-project maintenance commitment, which can be very demanding on resources. Moreover, the structures may lead to further instability and may further disrupt the already damaged ecosystem.

Connectivity in the fluvial system means that morphological responses to any anthropogenic impacts at any given location can be transmitted over a wide area. This can necessitate further, expensive training and stabilisation works to prevent serious channel instability and environmental damage elsewhere in the catchment. The frequency and magnitude of flows are significantly modified following flood control projects such as resectioning and regulation, which are designed based on the conveyance of a specific design flood rather than the natural sequence of flows. Furthermore, an increased frequency of high-magnitude flow events may significantly modify the spatial and temporal distribution of basin-wide sediment transport. For example, Hey (1990, p. 337) found that 5000 to 8000 tonnes of sediment must be dredged annually from the channel of the River Usk at Brecon, South Wales, to maintain design capacity. The scale of river maintenance in England and Wales was estimated from a 1992 survey at 7850 km of channelised main rivers and 2400 km of bank protection on non-navigable rivers (Sear and Newson, 1994; Sear et al., 1995, p. 631).

In zones of siltation, vegetation encroachment of submerged, emergent and marginal plants can reduce the capacity of the channel to convey the design flow, thus compromising the flood defence function of the initial project. Consequently, maintenance cutting is an important component of river maintenance in channelised rivers.

The instability effects of channelisation can be transmitted into other stream systems by lowering of the base level for tributary streams and triggering complex response as the channel network attempts to regain equilibrium long profiles. For example, Emerson

(1971) described how tributary meanders of the Blackwater River, Missouri, became entrenched, with associated channel widening and deepening, during the 60 years following channelisation. Furthermore, rapid tributary erosion by head cutting may result in significant aggradation problems within the main channel (Brookes, 1988, p. 102).

In summary, channelisation often disrupts the equilibrium between sediment supply and available transport capacity. The effects of this imbalance are greatest in high-energy systems with an appreciable sediment load. Restoration can stabilise sediment imbalances by creating a channel shape and size suitable for transporting the supplied sediment load with negligible net erosion or sedimentation within the restored reach in the medium to long term.

ii) ***Low Ecological Diversity***

Canalised, straight channels with lined banks are frequently ecologically impoverished and aesthetically displeasing because they lack the local instream and riparian heterogeneity and complexity found in naturally meandering rivers. The result is often a homogeneous, unattractive vista in the river landscape, with low biodiversity. A wealth of literature exists which describe case studies of the biotic impacts of channelisation and a summary was provided by Brookes (1988). Between 1940 and 1985 over 80 percent of the literature concerning fisheries are from the United States of America. In contrast, since 1981 the majority of papers are from countries outside the United States of America (Brookes, 1988, pp. 121-122), reflecting a global awakening of environmentalism.

Ward et al. (1994, p. 382) presented three motives for conserving the ecological value of rivers:

> a) ***To maximise and sustain biodiversity***.
>
> This is a direct result of pressure from environmentalist and special interest groups (Micklewright, 1993) to preserve the current range and diversity of species and habitats. The central purpose of the environmental movement is ‘the scientific understanding of nature, and the promotion of activities compatible with that, while objecting to contrary activities’ (Nicholson, 1993, p. 9). As O’Riordan (1989, p. 80) commented, since the early 1980s, ‘green politics has moved from the fringe of voluntary environmental pressure groups and green parties into mainstream party

politics'. Public pressure has forced a redirection of thinking away from post-war environmental determinism, the conquest of nature and interventionist policies, towards a re-enlightenment of environmentalism and the associated rhetoric of biodiversity and sustainability. As Hey (1997b, p. 5) noted, 'public opinion is becoming disenchanted with the wholesale exploitation of river systems and is demanding that some rivers, at least, should be preserved in a natural state'. Boon (1992) argued a case for river conservation by presenting nine elements for effective river conservation, including the need for improved procedures for environmental assessment, adaptive management in river modification schemes and long-term monitoring.

b) ***Moral considerations***.

Society has a moral obligation to protect species and restore environments following the damage inflicted on the riverine environment from a history of insensitive engineering works.

c) ***Cultural considerations***.

Society values riverine wildlife as a recreational, aesthetic and educational resource. This was endorsed by Harrison (1993, p. 48): 'Why nature conservation matters is because people benefit spiritually, emotionally, intellectually, physically and socially when nature *is* accessible'.

The ecological importance of habitat diversity in natural river systems is evident in the diverse physical structure of riffles, pools, eroding river cliffs, backwaters, gravel and sand bars and channel margins which provide a variety of instream and terrestrial habitats for a diverse species assemblage. In its unconfined, natural state there is a continuum in physical habitat from terrestrial to aquatic zones. This continuum, or hydrological connectivity, can be severely disrupted by engineering works which isolate the river from its floodplain by the imposition of steep banks, set-back levees, maintenance of riparian vegetation, ground water lowering and reduced habitat suitability (Holmes, 1993, p. 28). Walker et al. (1992, p. 271) described the lateral linkages between floodplain and river as imperative for the integrity of the corridor. The transitional zone features provide several functions: i) to enhance species diversity for resident and visitor species; ii) for the exchange of matter and energy between adjacent areas; iii) as refuges and other resources for species, and; iv) as pathways for species dispersal and migration.

Channelisation reduces the dynamism of the river both in terms of the physical structure (morphology) and replenishment of fluvial habitats (dynamic ecology). The uniform spatial distribution of velocities that a straight alignment and trapezoidal cross section generate has deleterious impacts on both the structure and function of the residing ecology. This was corroborated by Hey (1994b), who used the results of a study of 18 flood alleviation schemes in England and Wales to conclude that resectioned and realigned channels considerably limit instream and bank-side flora. Alterations to the magnitude and calibre of transported sediment upset the delicate nutrient balance within the water column, thereby disrupting feeding patterns of instream species. Moreover, sedimentation of fine particles within a coarse substrate, a common side-effect of channelisation in lowland reaches, may suffocate both fish eggs and aquatic insect larvae and reduce the quantity of refuge habitats for low-tolerant fish and invertebrates which require shelter from turbulence in the water column (Jenkins et al., 1984). Furthermore, a well-graded stable substrate can support a substantially richer benthic ecosystem than a homogeneous less stable sand-silt bed. Also, rapid recovery of benthic ecology may not occur where a high sediment load in an unstable river prevents the development of a stable substrate (Hill, 1976).

Swales (1989) identified ten adverse effects of river channelisation on the habitat quality of fish and other aquatic communities, focusing on the numerous impacts from reducing channel length (hence habitat area) as a direct consequence of straightening. One of the largest current river restoration schemes is on the Kissimmee River, Florida, where a 1960s flood protection scheme transformed approximately 160 km of meandering river into a linear, trapezoidal, concrete ditch, 78 km long, which reduced fish abundance by 75 percent (Boon, 1992, p. 19; Glass, 1987). The deleterious effects are generally a function of reducing morphological diversity which, in turn, modifies the flow regime and lessens the natural sorting of bed sediments. Following a detailed study of macroinvertebrate biomass in the channelised River Welland, U.K., Smith et al. (1990) stressed the importance of reintroducing morphological variability on the channel bed by reinstating a natural pool-riffle system in future engineering operations. In the riparian zone, thinning vegetation to maximise flood conveyance may severely impair bank-side habitat and the aquatic species assemblage dependent upon the cover provided by overhanging bank-side vegetation.

Soar (1996) concluded that the steep banks of the channelised River Idle, Nottinghamshire, displace habitats of fry and juvenile fish during the rising stage of the flood hydrograph. In natural rivers, these instream habitats tend to be replaced, rather than displaced, as species have lateral mobility to procure temporary refuge habitats in shallow, often vegetated, flow zones. Luey and Adelman (1980) described how natural reaches downstream from channelised reaches could act as sheltered areas for fish displaced by channelisation. Moreover, Newall (1995) noted that aquatic macrophytes that colonise marginal deposition bars could reduce velocities by up to 80 percent, thereby increasing survival potential of low-tolerant species. Flood control channels typically require the destruction of bank-side vegetation, usually for a distance of between 10 and 15 metres either side of the channel and often up to 100 m (Hill, 1976). Furthermore, where channelised rivers have resulted in siltation, there tends to be an absence of clean gravel substrate, which is vital fish spawning and macroinvertebrate habitat. Murphy and Meeham (1991) and Garcia de Jalòn (1995) have used a ‘habitat bottleneck’ analogy to describe these types of physical conditions, which act to limit fish populations. For the case of the River Idle (Soar, 1996), fish populations are limited, or ‘bottlenecked’, at the fry stage of the species’ life cycle. The transitional zone between floodplain and channel of natural rivers provides valuable habitat for other species such as water voles, nesting in tussocks of vegetation and consuming the green shoots and rhizomes of reeds and bank-side plants, and amphibians and reptiles (Ward et al., 1994). These riparian habitats are severely limited in channelised rivers.

Bayless and Smith (1967) studied 23 channelised sites in North Carolina and revealed that the number of game fish had been reduced in number by 90 percent, with only marginal recovery 40 years after the schemes were implemented. Significantly altering the natural pool-riffle sequence of a river by modification of width and depth variables for channelisation tends to create shallow, unnatural flows which induce ecological stress during low-flow periods, in particular by impeding fish migration (Keller, 1976).

Gorman and Karr (1978) discussed the buffering effect of natural meandering streams with diverse morphologies. Sinuous alignments can alleviate flood effects and pools serve as refuges for species during drought whereas marginal zones provide refuges during the passage of flood peaks. Furthermore, the shading effects of bank and riparian trees act to regulate the water temperature.

iii) ***Downstream Flooding***

Although flood control projects implement channelisation in an attempt to alleviate the frequency of flooding, connectivity in the fluvial system means that flood peaks are often merely transmitted further downstream, beyond the modified reach, with compounding effects. Brookes (1988, pp. 10-11) documented examples of the extent of channel works in the U.S.A. and calculated that approximately 6.4 million single family homes are at risk of flooding by the 100-year flood (1970 data). A naturally meandering river within its floodplain tends to dissipate the effects of high magnitude flow events and subsequently reduces the flooding hazard further downstream. Also, floodplain wetlands are important natural regulators of stream flow by improving storage during flooding and sustaining a base flow during low flows (Hill, 1976). By increasing stream power, via resectioning and regrading, and constructing flood embankments, this buffering effect is removed from the system and the flood peak is transferred downstream, fuelled by high velocities, until over-bank flow occurs. This potential hazard is exacerbated in lowland zones that have experienced significant siltation during periods of low flow, resulting in reduced capacity. A comprehensive study by Campbell et al. (1972) for the Boyer River, Iowa, revealed how channel straightening increased peak discharge in the range 90 percent to 190 percent at 36 cross sections. Examination of single-event hydrographs showed how the gradients of both the rising and falling limbs of the hydrographs were dramatically increased with a subsequent decrease in time lag from start of the event to flood peak. Case studies of the downstream hydrological consequences of channelisation have been summarised by Brookes (1988).

iv) ***Poor Aesthetics and Recreation***

The aesthetic value of a river is more of a subjective, rather than objective, issue and generally assumed to be maximised through diversity of landscape features, for example shallow, rippling riffles contrasting with contiguous, slow-flowing deep pools, well-graded riparian vegetation from floodplain trees to marginal submerged and emergent plants and diversity of wildlife. For the purpose of environmental impact assessment, Leopold and Marchand (1968), Leopold (1969) and Melhorn et al. (1975) have produced a variety of qualitative methods for evaluating the aesthetic value of rivers based on a range of factors including: physical parameters; biology and water quality, and; human-use interest. The aesthetic value of rivers as an objective for river restoration, particularly in urban areas, was discussed by Brookes and Sear (1996).

In the U.K., the Water Act of 1989 includes legislation directed at the NRA (now, EA) to preserve access to the countryside, to make water or land available for recreational purposes, and to promote the use of inland and coastal waters and associated land for recreational purposes (Heaton, 1993, p. 315). However, although techniques for the evaluation of the aesthetic quality of rivers are well developed, little has been achieved in terms of developing and applying these techniques (Brookes, 1988, p. 240).

v) ***Impeded Natural Recovery***

The instability described in i) indicates that a river is dynamic in its attempt to respond to imposed changes via complex negative feedback mechanisms (homeostasis) to ‘recover’ a stable natural form adjusted to the flow and sediment regimes imposed by the catchment. If channelised rivers are not constrained by fixed bank-lines, they will undergo a period of recovery, or ‘relaxation time’, during which morphological change will occur. In perennial rivers, homeostasis is encouraged by riparian and floodplain vegetation which acts to limit the impacts of major floods, and vegetation re-growth promotes the processes of siltation necessary for morphological recovery (Gupta and Fox, 1974; Hack and Goodlet, 1960).

Moreover, the ecosystem recovery process tends to keep pace with the physical recovery process (Brookes, 1988, p. 112) because of the intrinsic coupling of morphological diversity and niche heterogeneity (Section 1.5). The rate of recovery is a function of the geomorphological potential to expend energy; therefore, upland systems may be able to recover, whereas lowland, low-energy streams, which experience formative events less frequently, may take centuries to recover naturally. Brookes (1992) noted that recovery of channels in North European countries have taken place over periods ranging from 1 to 150 years. Furthermore, de Vries (1975, p. 344) suggested that meander growth from channel straightening and cut-offs might take from 30 years to more than 1000 years. In cases where the ecological and/or aesthetic quality of a channelised stream is poor, river restoration can target low-energy systems that are unlikely to recover fully from the disturbance over the short- to medium-term. The relative merits of intervention by restoration or allowed natural recovery have been discussed further by Brookes and Sear (1996, p. 78).

The ability of a channelised river to recover can be expressed in terms of specific stream power (stream power per unit bed area) at bankfull stage, which is an independent parameter, proportional to discharge and channel gradient. Brookes (1987b) demonstrated how reaches with stream powers greater than 100 Wm^{-2} usually fully regain their natural sinuosity. Rivers with stream powers in excess of 35 Wm^{-2}, but less than 100 Wm^{-2}, lead to aggradation and/or degradation with only partial recovery, whereas the rivers studied below this range of stream power had insufficient energy for natural morphological adjustment to occur. The majority of channelisation schemes are found in lowland zones with some degree of urbanisation and agricultural expansion within the catchment.

The greatest potential, and challenge, for river restoration is on rivers below or in close proximity to the 35 Wm^{-2} threshold. Society has significantly modified the structure of these channels, yet nature alone cannot recover effectively even when the causes of the morphological problems cease and further interventions may be required by the design engineer and geomorphologist.

vi) ***Unsustainable Maintenance***

Structural works and dredging operations to deal with the instability problems of channelisation are expensive. Sear et al. (1995) estimated that the annual bill for sediment-related river maintenance carried out by all drainage authorities in the U.K. exceeds £20 million.

Gardiner (1988) stressed how river management that encourages the natural development of the river, rather than control, is more economic because the river channel form will be stable within the present catchment landscape. If a channel design can effectively transport sediment through the restored reach with near-zero net aggradation or degradation, then a potentially expensive post-project maintenance commitment becomes a significantly less expensive and sustainable monitoring programme.

1.3.3 Channel Restoration Design

River restoration, in the context of this report, is aimed primarily at recovering dynamic channel stability in the project reach by specifying suitable instream dimensions, with the assumption that the diverse form and function of a stable river system will provide a diverse range of habitats (Section 1.5), thereby enhancing overall ecological quality. In essence, the dynamism intrinsic to geomorphology drives a healthy and sustainable ecosystem. If channelisation is the process of 'ironing' out nature's 'creases', then river restoration design is the antithesis of channelisation by emulating the morphology of natural stream channels. According to Hey (1994a, p. 355), river restoration is '...removing a legacy of uniformity and recreating the diversity that is characteristic of natural channels'.

Water quality issues are not directly addressed herein, although a stable sediment regime and managed encroachment of marginal vegetation will improve the dispersal of pollutants and improve water quality indirectly. However, 'the expectation of self-purification in a rehabilitated stream must never be a substitute for further efforts in pollution control' (Kern, 1992) and this should be addressed in river management if water quality is an issue. Methods of restoration for water quality generally focus on the creation of riparian buffer strips and wetlands which absorb pollutants and tackling pollution at the source.

A distinction must be made between the general term 'river restoration' and the more specific term 'channel restoration design'. While river restoration describes the general 'practice' of returning a river to some previous condition of equilibrium, channel restoration design is concerned with the actual 'process' of defining the physical shape and size of the restored channel and strives to mimic the form and function of a natural river system based on enhanced engineering methods and geomorphic principles as defined below (after Soar et al., 1998):

Channel restoration design is the reconstruction of a river channel to a stable geometric configuration that is self-sustaining and in balance with imposed flow and sediment regimes and the character of the catchment landscape.

As Shields (1996, p. 26) remarked:

> '…selecting dimensions (width, depth, cross-sectional shape, planform, bend radius, amplitude, slope) … is representative of perhaps the most difficult class of river restoration engineering problems'.

Intervention via realignment and cross-sectional reshaping is often termed 'full channel restoration' and is required particularly in lowland 'low-energy' environments where natural recovery rates are low. Reconnecting the river within its floodplain is not a direct aim of the report but is a beneficial side effect of reinstating a natural instream morphology and an active meander belt, or fairway, which is the first stage in stream corridor and ecoriparian restoration. A dynamic channel within its floodplain is desirable because channel migration promotes destruction of mature ecosystems, creates unique ecological niches such as river cliffs and sand bars and presents opportunities for invasive and pioneering species, which significantly improves the biodiversity.

Conventional river engineering is primarily driven by flood control objectives, whereby a channel is designed to carry a specific flood flow, rather than the range of flows which the river would naturally experience under present catchment conditions. In the past, channel design required specifying a unique elevation required for flood protection and engineering methods have previously focused on unnatural straight alignments that were designed to alleviate flooding and maximise the protected floodplain area. Today, channel restoration design involves selecting a dominant, or 'channel-forming', discharge as a function of the empirical distribution of flows, which controls channel shape and size. Ward et al. (1994, p. 168) remarked that, 'a natural river may be the most efficient in expending geomorphic energy but very inefficient in terms of space. To restore a river's form *and* function, this floodplain 'space' must be utilised where possible and existing approaches need to be modified to take into account natural planform shapes and observed width variability around the bendways of meandering rivers, as well as spatially averaged cross-sectional design.

1.4 COUPLING GEOMORPHOLOGICAL PRINCIPLES WITH ENGINEERING METHODS: GEOMORPHIC ENGINEERING

Only during the 1980s and 1990s has the importance of fluvial geomorphology been recognised as a vital component for effective river management in the U.K. (Brookes, 1995b). Recently, geomorphic principles have been developed by applied geomorphologists to complement and substantiate existing engineering approaches to management. In light of the considerable evidence of environmental problems following channelisation, Coates (1976, p. 20) suggested that the geomorphologist:

> '...must become involved in the tools of engineering because if construction causes irreparable damage to the land-water ecosystem due to lack of geomorphic input the earth scientist cannot be absolved of blame. Thus it is imperative that the geomorphic engineer be involved in the decision-making processes that plan and manage the environment.'

This coupling of geomorphology and engineering is defined by Coates (1976, p. 6) as the field of *geomorphic engineering* that:

> '...is interested in maintaining (and working towards the accomplishment of) the maximum integrity and balance of the total land-water ecosystem as it relates to landforms, surface materials and processes.'

With this necessary convergence of disciplines, Haines-Young and Petch (1986, p. 200) considered geomorphology as becoming indistinguishable from engineering: '…if anything, geomorphology is developing into a minor branch of engineering…more preoccupied with successful model building and prediction than with explanation or truth'. Arguably, this is a misinterpretation of why the two disciplines have become more closely interrelated during the 1980s and 1990s. The scientific experience of geomorphology is based on an understanding of the environment rather than engineered control and 'can usefully serve to question the assumptions of design models, to ensure a concern with the total complexity of the natural system, and to aid in evaluating system changes over time' (Baker and Twidale, 1991, p. 86). In contrast, 'the aim of engineers is to cope with the world, rather than understand it' (Baker and Twidale, 1991, p. 86).

Since the 1960s, geomorphology has been dominated by critical rationalist approaches (Haines-Young and Petch, 1986), whereby predictions generated by hypotheses based on physical laws are compared with and validated by observations. More recently, Richards (1990) has argued a case for 'real' geomorphology. The realist perspective recognises that 'complex networks of causal mechanisms operate within complex contingencies' (p. 196), rather than simple deterministic laws, whereby accepting explanations based on mechanisms is merely based on explanatory power (Baker and Twidale, 1991, p. 86). However, the fluvial system is indeterminate because there are fewer physical laws than available degrees of freedom in the system (Hey, 1978, 1988). Moreover, research by Wolman and Gerson (1978) described how channel change occurs as a response to complicated, and poorly understood, feedback mechanisms, or complex responses that cannot be modelled effectively without 'bridging principles' or assumptions. Therefore, while a realist approach might represent ideal geomorphology, the science requires further evolution before the 'ontological depth' described by Richards (1990) can be uncovered. In light of the limitations of applied geomorphology, it is necessary to develop techniques that simulate the 'real' fluvial system in the absence of advanced equations describing the complex relationships between river processes and forms. For example, by researching the natural variability of channel dimensions for a particular type of river cannel, it is possible to use simple statistical techniques to describe the variability without the need to understand fully the complex networks of mechanisms controlling that variability.

Newson (1995, p. 427) and Sear et al. (1995) considered fluvial geomorphology as providing a 'complementary medicine' for engineering in river management in terms of a catchment-scale science with an ability to identify the cause of sediment related river maintenance. Also, Sear et al. (1995, p. 629) noted that 'while the engineering community has fed from the patronage of politicians to control the river system, fluvial geomorphology has evolved as an academic discipline'. Theoretical geomorphology is concerned with the evolution of landforms while theoretical hydraulic engineering is based on a sound understanding of flow forces, in particular flow resistance. Despite these fundamental differences, geomorphology utilises many engineering methods and there are many cross-references in applied research (Figure 1.2). Sediment transport is probably the fundamental process which links theory with practice in both river engineering and fluvial geomorphology and channel restoration design may also be regarded as a central thread in Figure 1.2 because it attempts to bring together geomorphic principles and engineering methods into a framework for solving practical river management problems and improving riverine health.

The engineering approach has dominated river management in the past probably because engineering science has significantly greater experience in the application of research to real world problems over less developed approaches: '…engineering solutions are traditional, 'exact' and auditable, a powerful commendation to society against the untried, more flexible and qualitative aspects of fluvial geomorphology' (Newson, 1995, p. 415). Despite this lack of experience, environmental river engineering should consider geomorphological principles 'by virtue of their knowledge of river mechanics, morphology, erosional and sedimentation processes and landform evolution' (Hey 1990, p. 335).

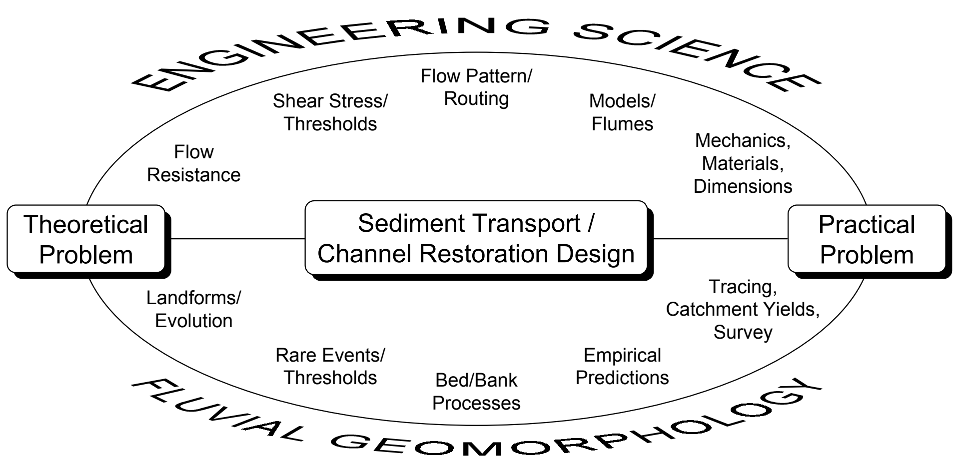


Figure 1.2 Engineering science and fluvial geomorphology - divergent research topics on routes between theory and practice (modified after Newson, 1995, p. 416 and Sear et al., 1995, p. 635). Sediment transport and channel restoration design are central threads that link theory and application.

Faced with the problems of channelisation, engineers, geomorphologists and ecologists have moved toward a new principle of 'designing with nature' (McHarg, 1969) which strives to sustain or restore natural forms and processes to the river, in harmony with the river landscape (Brookes, 1988, 1995a; Brookes and Sear, 1996; Hey, 1986, 1990; Soar et al., 1998). As Baker and Twidale (1991, p. 73) remarked: 'Hope for the reenchantment of geomorphology lies in a new connectedness to nature'. This now pervades the thinking in U.K. agencies involved in environmental management and conservation (Thorne, 1995).

Recognition of the undesirable consequences of river modification has encouraged those interested in river management to design with nature as a means of achieving a 'reverence for rivers' (Leopold, 1977) which is a basic requirement of the geomorphic engineering approach. To return a river to its natural state, an understanding of the direction and magnitude of change in channel characteristics, both intrinsic and extrinsic, are required (Simons, 1979, p. 5-61). This includes studying the river in its natural condition, acquiring detailed knowledge of sediment characteristics, including sources, understanding the flow regime and applying knowledge of catchment geology, soils, hydrology, and ecology of the alluvial river environment. Understanding fluvial processes is imperative for successful river management and channel restoration design. This caution was endorsed by Holmes (1993, p. 31):

> 'Without an understanding of fluvial geomorphology, and the impacts of catchment developments, it is likely that totally inappropriate enhancements will be proposed and executed only to be destroyed by the river's natural processes'.

Central to this approach is to consider a river within the catchment framework. According to Boon (1992, p. 25), a river is 'liquid history' and reflects the characteristics of the catchment in which it flows and shapes catchment topography. Therefore it is imperative to recognise the controls imposed by the catchment when managing rivers. This entails consideration of the impact of river management works not only at the site itself but also upstream, downstream, and the land adjacent to the channel as embodied in the concept of 'catchment management planning' (Ward et al., 1994, p. 4).

To geomorphologists concerned with principles of continuity, equilibrium and processes of channel change and sediment dynamics, treating the catchment as the fundamental management unit is axiomatic. Solving real world problems based on geomorphology raises two problems: i) successful translation of principles into policy and strategic planning (Newson, 1992) and; ii) how to incorporate principles of geomorphology into quantitative design drawings for construction purposes (Brookes and Sear, 1996; Soar et al., 1998).

The effects of channelisation are not confined solely to the channelised reach (Section 1.3.2) and often influence channel stability at the catchment scale. As the instability caused by previous engineering works must be addressed on a catchment scale, so too must effective river management to restore an equilibrium river system.

The geomorphic approach to river management involves a sequential decrease in the spatial scale of investigation from the catchment as a whole to a detailed assessment of the study reach and is, therefore, an appropriate method of investigation for multi-functional projects in which matters of flood defence and conservation value must be addressed. In contrast, conventional engineering project management is frequently parochial in its treatment of a study reach in isolation from its catchment context, dynamics and constraints. Unpredicted post-project adjustments in the fluvial system are often a result of failure to account for the intrinsic coupling between the morphology of the channel and the geomorphology of the whole river catchment (Sear, 1994, p. 170; Brookes and Sear, 1996) which are both dynamic over the medium- to long-term and usually not addressed by the hydraulic engineer involved in flood control or navigation schemes. Furthermore, it is essential that the future trajectory of catchment land-use change be considered to examine the geomorphological sensitivity of restored channel dimensions to potential modifications of the flow and sediment regimes.

Geomorphic engineering for channel restoration design aims to: i) design with nature, rather than against it, by incorporating the catchment context in the procedure and prompting the river itself to aid recovery and; ii) imitate natural systems, in particular their morphological variability, rather than engineering a rigid channel design.

1.5 COUPLING GEOMORPHOLOGICAL PRINCIPLES WITH ECOLOGICAL ASSESSMENT: THE PHYSICAL HABITAT

Fluvial geomorphology and ecology have become increasingly interrelated via their shared interest in the physical or ‘hydraulic habitat’ (Newbury and Gaboury, 1993) and ‘environmental design’ (Newson, 1995) which assume that the physical, morphological structure of the riverine environment is indicative of the ecological value of microhabitats located within this structure (Soar, 1996). Furthermore, closer integration between geomorphologists and ecologists has occurred because of shared interests in investigating the effects of vegetation and woody debris on channel dynamics (Petts, 1995, p. 16). A geomorphologically active river, that is one which is expending energy within a dynamic equilibrium framework, creates a corresponding dynamic equilibrium in the biological system. Also, Ward et al. (1994, p. 4) stated that 'the integration of hydraulic and ecological techniques is the key theme for the development of sustainable solutions to the challenges of river management'. On this basis, Ward et al. (1994) examined the hydrological link between rivers and their floodplains and the influence of processes on shaping river habitats and Harper and Smith (1992) described a study which focused upon a classification of river channel habitats that is structured according to geomorphological features. Moreover, physical habitat attributes are generally more predictable and measurable than biological ones and, therefore, preferred descriptors of streams for environmentally sensitive river management (Garcia de Jalón, 1995).

The depth, velocity and substrate of a river form the physical 'structure' within which organisms reside. Hynes (1970), Soar (1996), Ward et al. (1994), and many others, examined the interrelationships between the physical attributes of a channel and ecological quality, in particular fish habitat. Velocity variation delimits availability of oxygen, the replenishment of nutrients and is essential in the natural dispersal mechanism of species (Gore, 1978). Depth influences temperature variation, light penetration and the physical living space available for the aquatic species assemblage. The depth also affects the distribution of benthic invertebrates, important species in the food web with most preferring shallow depths (Wesche, 1985), and the available distance between predator and prey. Substrate variability is closely associated with depth and the temporally and spatially variable flow regime, influencing spawning habitats, shelter and the stability of macroinvertebrate populations. Further discussion on the importance of river ecology from the perspective of hydraulic engineering was provided by Meier (1998).

Milhous (1988), Milhous et al. (1989) and Naiman et al. (1992) stressed the importance of an initial understanding of the stream system through an assessment of channel topography and flow hydraulics prior to analysing ecological potential and showed how habitats can be delineated by physical attributes of the catchment. It is unlikely that river restoration will be successful if the project does not create a dynamically stable channel that supports a self-sustaining and functionally diverse community assemblage (Osborne et al., 1993, p. 187).

Coupling morphological diversity of natural river systems with ecological diversity, often the primary restoration objective, and recognition of the importance of geomorphic principles for channel design positions fluvial geomorphology as an invaluable catalyst for channel restoration design. The geomorphologist can effectively communicate between engineers and ecologists in a common language and emphasise common objectives for multidisciplinary projects. As Warren (1993, p. 15) remarked, 'an approach to the natural from one particular branch of science (geomorphology), can yield concepts that are useful to reserve managers, virtually irrespective of different social constructions'. This was endorsed by Coates (1982, p. 166), who concluded that:

> '...the environmental geomorphologist is in a position to not only bridge the gap with peer natural scientists but also to translate various pieces of a puzzle into a composite whole.'

1.6 NATURE OF INVESTIGATION

This report describes the framework, individual stages and testing of an enhanced channel restoration design procedure. The procedure represents 'best practice' in this regard. As there is not a scientific premise or hypothesis to test in this study, a traditional scientific approach to investigation is inappropriate. Rather an applied strategic approach is adopted. This involves:

i) Assimilation and enhancement of existing principles and methods.

ii) Investigation of how the principles and methods can be pieced together into an appropriate framework, or management tool, for application by the end-user community involved in the practice of channel restoration design.

iii) Testing the enhanced procedure against an actual restoration project. Although this investigation is concerned with the development of methods, process geomorphology is used to examine how stable channel morphology is related to the processes that have moulded the form over time.

The approaches adopted in this report include a combination of empirical and analytical techniques. Complex numerical modelling of flow and sediment patterns is not an objective of this research on the basis that two- and three-dimensional modelling have not yet advanced sufficiently to be routine techniques for channel design purposes, given the quantity and type of data required for computation and the assumptions necessary to compensate for an incomplete knowledge of fluid flow in rivers with mobile boundaries. Furthermore, it is intended that the channel restoration design procedure developed herein should be carried out immediately after a detailed reconnaissance survey of a project reach. Therefore, it is imperative that data collection in the procedure is economic and not intensive, particularly as several reaches maybe considered before choosing a suitable site for restoration. As a result of these criteria, only one-dimensional analytical methods are used.

Project engineers have directly participated in the development of the procedure by providing feedback during site testing which has helped ensure that the demands of the end-user community have been met, where possible.

1.7 U. S. CORPS OF ENGINEERS APPROACH TO CHANNEL DESIGN

The U.S. Corps of Engineers has maintained a continued interest in the relationship between channel geometry, fluid flow and sediment characteristics, since the middle of the Nineteenth Century. At this time the engineering policy of protecting the Mississippi River using levees was questioned in light of recommendations for outlets, or spillways, to lower flood water levels (Barry, 1997). This investigation required a better understanding of the processes that cause the river to carry and deposit sediment. Engineering feats such as the construction of jetties (1875-1879) at the mouth of the Mississippi near New Orleans, Louisiana, to improve navigation also demanded an understanding of the interrelationships between river form and mechanics (Barry, 1997), the task of the Mississippi River Commission, set up in 1879. Later, the devastating Mississippi flood of 1927 changed the way engineers viewed the river and led to the creation of the U.S. Army Engineer Waterways Experiment Station (UAEWES), Vicksburg, Mississippi, (currently, U.S. Army Engineer Research and Development Center (ERDC)) a research based centre with a main objective to further the engineering science of river hydraulics. With the rapid development of computer technology, analytical methods to simulate the interrelationships between river form and processes have advanced the application of river engineering to effective management of rivers.

1.7.1 SAM Hydraulic Design Package

The hydraulic design package 'SAM' (Stable channel Analytical Method) was developed over a period of eight years by Thomas et al. (1996) at the UAEWES/ERDC Coastal and Hydraulics Laboratory, through a Flood Control Channel Research Program. The package is an integrated system of computational tools with the capability to evaluate erosion, transportation and deposition in alluvial streams. Although not specifically a channel design tool, systematic application of several of the modules can assist the river engineer in evaluating the stability of an existing channel and may provide design guidance for stable channel dimensions in straight rivers. Full documentation of SAM is given in the end-users manual (Thomas et al., 1996). In summary, SAM has the capability to calculate three principal degrees of freedom in an alluvial river: width; depth, and; slope. There are six modules in SAM, although three provide the main functionality of the package, summarised below:

i) SAM.hyd (the *hydraulics* module) provides the computational tools necessary to define the normal depth of a trapezoidal cross section (depth from the top of the banks to the bed) and composite hydraulic parameters for a cross section with variable roughness defined by the user. Several flow resistance functions are available to the user for specifying how roughness varies between banks.

ii) SAM.sed (the *sediment transport* module) calculates sediment discharge from the user's choice of a range of sediment transport functions suitable for a wide range of riverine conditions. Usually, this module is executed after calculating the normal depth in SAM.hyd. With repeated execution, a site-specific sediment discharge rating curve can be developed.

iii) SAM.yld (the *sediment yield* module) usually follows SAM.sed in operation to calculate the bed material load passing the cross section per unit time given a user specified flow duration curve or hydrograph.

With knowledge of project reach and supply reach boundary conditions (aided by SAM.hyd and SAM.sed, if required), the stable channel dimensions, width, depth and slope, can be derived from a sub-module of SAM.hyd using an analytical approach. By using a channel stability assessment technique, the designer is presented with a suite of stable geometry solutions which, theoretically, will yield zero net aggradation and degradation over the medium- to long-term. An appropriate solution that satisfies project constraints, for example floodplain width restriction or channel depth restriction, can then be selected. The analytical method is discussed in detail in Chapter 6.

The analytical channel design method in SAM was chosen in the context of this report because it represents an up-to-date practical approach to examining channel stability and estimating stable channel dimensions. The package is referenced in many of the recent engineering (e.g. Copeland and Hall, 1998; Soar et al., 1998; Copeland et al., 1999), geomorphological (Shields, 1996) and multidisciplinary publications (e.g. FISRWG, 1998), but has not been widely applied. This is partialy because SAM is still in a developmental stage and several limitations have been identified but also because a procedure for applying the channel design method within the framework of a river restoration project is currently not available.

The package appeals to the geomorphologist because it can be applied to examine the systematic interaction between upstream sediment supply, project reach sediment conveyance and downstream sediment demand. In many projects, river engineers adopt a very parochial treatment of the study reach in isolation of the larger system within which it belongs. SAM provides the necessary tools to address channel stability/instability on a wider spatial scale. Also, the method is unique in its separation of bed and bank roughness that permits riparian vegetation of known roughness to be included in the overall design without significantly affecting geomorphological stability. Furthermore, because of the analytical nature by which stable dimensions are derived, the sensitivity of the design variables can be specified for each project. This is particularly valuable to the geomorphologist who aims to restore a channel configuration which is not likely to react significantly to future extrinsic influences within the catchment system. Using hypothetical scenarios, both spatial and temporal stability of the designed channel can be investigated, quantified and used to improve the design solution.

1.8 RESEARCH OBJECTIVES AND OVERVIEW OF REPORT

By bringing together geomorphic principles and engineering methods, this report presents a geomorphic-engineering framework for channel restoration design in meandering rivers. By accounting for natural systems variability, the design framework is an appropriate platform for generating restoration design solutions that mimic the natural channel morphologies and environmental attributes in undisturbed systems, while meeting multifunctional goals of channel stability and low maintenance commitments. The approach outlined in this report provides a practical solution by striking a balance between empirical-statistical and analytical (process-based) methods.

The main objective of this study is to develop a practical procedure for channel restoration design which does not require sophisticated computer models that can only be operated by an expert in hydrodynamic and morphological modelling but rather bridges the gap between simple ‘empirically based’ or ‘experience-based’ methods and those requiring complex numerical modelling. Therefore, the data that are required to support the procedure should be relatively quick and inexpensive to acquire or collect. The required flow data should be available from gauge records and other data must be retrieved during detailed stream reconnaissance. As several sites are often examined for restoration potential, execution of the procedure immediately following stream reconnaissance is essential.

The structure of this report follows the development, individual stages and testing of the channel restoration design procedure:

Chapter 2 is a detailed chronological review of existing methods for designing stable river channels and an examination of their applicability to meandering river restoration. Initially, the complexity of the river itself (as an agent of nature) is questioned as a realistic analogue for restoration. It is recognised that while the river exists within an open system, it has to be defined by geomorphologists within a black-box system to facilitate conceptualisation and development of process-form relationships. The review demonstrates that a complete understanding of the fluvial system continues to evade rational scientific explanation, with many analytical techniques only corroborating the findings from early research. The imbalance between the natural degrees of freedom posed by an undisturbed, unconstrained, meandering river and those which can be described mathematically is an important theme in this and subsequent chapters.

Distinctions are then made between the hydraulic engineer and the geomorphologist as alternative channel restoration designers. The types of engineering approaches described in this chapter include regime theory and analytical techniques such as tractive force theory and extremal (variational) hypotheses. The main geomorphological approach discussed is that of downstream hydraulic geometry analysis which precedes a short overview of recent geomorphological approaches that have been developed specifically for restoring rivers. Following from this review, the main objective of Chapter 3 is to bring together the various techniques available into a coherent design framework, thereby overcoming, to some degree, their individual limitations and providing a solution to the indeterminacy problem in stable channel design.

Chapter 3 discusses the geomorphological principles that should form the basis of a river restoration project to yield stable channel dimensions that are commensurate with the catchment context and observed natural variability. The objective of this chapter is to examine these geomorphological principles in the context of developing a best practice procedure for channel restoration design and then present the main design phases, methods and variables involved in the design framework.

Despite a plethora of deterministic morphological equations found in the engineering literature, in nature there are no unique values of stable channel dimensions for a specified design discharge. Reach-average dimensions of a natural river can vary markedly over a relatively small distance along the path of a meandering river. This uncertainty is rarely specified in engineering analysis and often expressed in terms of numerical error rather than natural variability. One of the important themes addressed in this chapter is that, by recognising this misinterpretation, natural rivers could be realistic analogues for channel restoration design. In light of this, the enhanced procedure should accommodate levels of uncertainty, as a function of natural variability, in estimates of restored channel dimensions. Therefore, one of the main objectives of this chapter is to describe the necessary statistical techniques and equations necessary to accommodate natural variability of channel dimensions in river restoration projects.

Furthermore, it is necessary to guide end-users to appropriate morphological equations by ‘typing’ the target channel through simple morphological classification. Different classification systems are examined in Chapter 3 with the objective of developing a simple typing system for use in river restoration.

Based on the discussion in Chapter 2, the geomorphic engineering approach presented in Chapter 3 recognises that the river is ultimately the best restorer of its natural morphology and should be allowed to participate in its own recovery. This objective is central to the design framework and an important theme in Chapter 3.

Re-establishing equilibrium between the sediment supply and available transport capacity in the restored reach is the primary objective of the design framework. This is discussed in Chapter 3 by adopting an approach which defines the catchment in terms of three units: an upstream supply reach (or reaches), which defines the sediment input; the restored reach, where channel dimensions and slope must be designed to transport the input sediment load with negligible erosion or sedimentation over the medium- to long-term, and; the downstream reach, which has a specific sediment load demand that must be met by output from the restored reach. It follows that a stable channel is one in which sediment supply must match transport capacity. This principle is adopted in this chapter in the development of a sediment impact assessment which should be undertaken at the end of the design procedure as a closure loop to examine the overall stability of the restored channel.

A general procedure is presented which is divided into four broad stages: i) Supply Reach Assessment; ii) Project Reach Assessment; iii) Channel Design and; iv) the Design Brief. The design methods and associated design variables for each of these stages and sub-stages are presented and form the basis for further discussion in later chapters. In summary, by adopting a geomorphic engineering approach the enhanced procedure should: i) direct the design community towards the information requirements necessary to restore a river; ii) support the user in determining the channel ‘type’ and apply different design solutions for different 'types' of channel; iii) account for local site constraints in the restored design, and; iv) use uncertainty in the functions linking form and process to produce a range of acceptable solutions that allows for local variability and non-uniformity in channel form. The various statistical techniques that are used in this report are also discussed in Chapter 3.

Chapter 4 is a discussion of the main process driver in channel restoration design: the geomorphologically important 'channel-forming' flow. The design framework presented in Chapter 3 shows that the channel-forming flow is the most important parameter in channel restoration design, from which all channel dimensions are directly or indirectly determined by. The main research objectives of this chapter are to compare the various methods of estimating the channel-forming discharge in terms of their practical utility value for river management and to present practical guidance for calculating an objective estimate of the channel-forming discharge.

By coupling discharge and sediment transport, Magnitude-Frequency Analysis (MFA) is presented as a geomorphological approach to define the effective discharge for channel restoration design. The initial objective was to identify a standard procedure for calculating the effective discharge based on existing methods. However, through a detailed investigation of the type and number of discharge class intervals used in the procedure, it became apparent that the conventional 'class-based' method should be replaced by a more 'event-based' method which yields a high resolution histogram of sediment-transporting flow events and more detailed information on channel stability. A quasi-event-based MFA method is presented at the end of the chapter which has evolved out of the conventional technique. Following the development of the various methods presented in Chapter 4, the use of a unique channel-forming discharge for river management is questioned as the recent methodological developments provide a useful starting point to identify an effective range of flows which is causally linked to channel morphology and could aid the design engineer in the restoration of instream sedimentary features and physical habitats as well as the basic stable channel dimensions.

Chapter 5 is a statistical treatment of bankfull width, based on a new downstream hydraulic geometry that recognises the presence of uncertainty in estimates. The objective of the chapter is to present the end-user of the design procedure with practical width equations for different types of alluvial river channel. The chapter is an integral part of the design framework as it uses an empirical-statistical technique, together with an estimate of channel-forming discharge based on guidance given in Chapter 4, to overcome the indeterminacy problem in analytical channel design. The estimate of bankfull width is then used as an input parameter necessary to complete the analytical derivation of slope, depth and sinuosity discussed in Chapter 6. Adopting a geomorphic engineering approach, the statistical equations presented in Chapter 3, that describe confidence intervals applied to typed hydraulic geometry equations, are applied in Chapter 5 to provide a series of engineering equations for various combinations of bed and bank characteristics and levels of statistical uncertainty.

Following an extensive fieldwork programme to investigate width-discharge relationships in sand-bed rivers, it became necessary to re-examine the procedure for calculating the channel-forming discharge outlined in Chapter 4. An effective discharge investigation is presented with an objective to identify the conditions at which bankfull and effective discharges are not equal. In particular, the results demonstrate that this deviation between discharges is related to the variability of the flow regime. Morphological relationships derived from previously unpublished data are presented. These data provide further guidance for estimating the channel-forming discharge when certain conditions prevail and identify a significant area where further research should be directed to further develop river management procedures.

Chapter 6 is a short overview of the analytical component of the channel design framework. Using estimates of discharge and width, from the preceding Chapters 4 and 5, estimates of depth and slope (and sinuosity, given valley gradient) can be determined though the simultaneous solution of flow resistance and sediment transport equations. The two objectives of this chapter are to present and critically review an existing channel design procedure for sand-bed rivers in the SAM hydraulic design package and then develop a similar method for gravel-bed rivers with mobile beds. The output estimate of sinuosity, together with bankfull width, is then input to determine meander planform geometry, as described in Chapter 7.

Chapter 7 is concerned with laying out the restored meander planform. The objectives of the chapter are to firstly provide the necessary design equations to specify the reach average meander planform and secondly to provide a series of morphological relationships for introducing local morphological variability around meander bends, as found in natural channels. The main research objective in Chapters 5 through 7 is to develop the geomorphic engineering approach through an increasing level of design detail with a balance between empirical, statistical and analytical methods. Chapter 7 is divided into the three essential stages necessary to layout the basic planimetric geometry of a restored channel according to the procedural framework given in Chapter 3: i) determination of reach-average meander wavelength from channel width with uncertainty in estimates; ii) reach-average planform layout according to the sine-generated curve and; iii) determination of local morphological variability around meander bendways with uncertainty in estimates. The main theme in this chapter is the recognition that some degree of post-project recovery is both inevitable and encouraged in river restoration and the use of confidence bands applied to simple morphological relationships is more appropriate, and provides more realistic solutions, than the specification of fixed channel dimensions.

To fully test the design framework presented in Chapters 3 through 7 would require actually applying the various methods to a real restoration project and monitoring post-project channel stability (and other success criteria) for several years. This was not feasible for this research. An alternative testing strategy is presented in Chapter 8, which describes a case study in Maryland where significant channel change has occurred in a short period since the river was restored. The geomorphic engineering approach and the various design methods developed in this report were applied and the simulated channel configuration was compared to actual channel design specifications and the resulting configuration following post-project channel adjustments. The objectives of the application are to investigate whether the channel evolution is tending toward the simulated channel configuration output from the procedure, thereby providing success criteria, and also to identify any operational difficulties in following the procedure which would highlight areas for further development.

In the conclusion, Chapter 9 identifies the salient findings in this report through a discussion of the significance and limitations of the various methods that are central to the geomorphic engineering approach and constitute the enhanced channel restoration design procedure. The various components of the approach are assessed and the most important stages in the procedure are identified. The case for specifying a range of effective discharges for channel restoration design in light of the methodological developments in Chapters 4 and 5 is also presented. Based on these discussions, recommendations for further research and the continued development and testing of the procedure are presented.

C H A P T E R 2

The Channel Restoration Designers

and their Toolkits

2.1 INTRODUCTION

The scientific and engineering literature documents a plethora of principles and methods for estimating the geometry of theoretically stable river channels. Contributions from the fields of hydraulic engineering, geomorphology and, recently, freshwater ecology describe a chronology of different approaches since the turn of the twentieth century. However, unlike the development of many procedures for solving real world problems, which evolve sequentially over time with increasing output precision, some of the greatest methodological achievements in the field of channel design were made over half a century ago, with many recent analytical techniques only corroborating the findings from early research. Technological developments have paved the way for improvements in hydraulic modelling, but a complete understanding of the fluvial system continues to evade rational scientific explanation. Subsequent 'state of the art' and 'best practice' methods for designing river channels are frequently based on established empirical techniques such as 'regime theory', which was developed by engineers researching stable canal geometry on the Indo-Gangetic plain and reached its zenith in the 1930s (Blench, 1986).

Natural channel geometry evolves within the catchment system. Its complexity exists without reference to significant anthropogenic pressures on the system. For channel restoration design, substituting the design capabilities of nature by those of an engineer or geomorphologist (or both) requires a number of assumptions to be made and the river to be represented as a simplification of its actual form and function. Several routes of enquiry naturally ensue from this scientific 'fix' and are investigated in this chapter.

This chapter is divided into three main sections. Firstly, use of the natural river as an appropriate analogue for channel restoration design is questioned. This is followed by a historical overview of hydraulic engineering methods for designing river channels, and their applicability as predictive tools for river restoration. The development of morphological relationships to predict stable channel dimensions by geomorphologists is then discussed, before a short overview of recent geomorphological approaches that have been developed specifically for restoring rivers. When full restoration is not a practical or appropriate management option, approaches from hydraulic engineering and geomorphology may be unsuitable and more ecologically based methods of assessment should be considered, such as physical habitat evaluation procedures, which can provide baseline information in less structural rehabilitation schemes. These approaches are beyond the scope of this study and are documented extensively elsewhere (e.g. FISRWG, 1998).

2.2 THE *RIVER* AS A CHANNEL RESTORATION DESIGNER

From a geomorphological perspective, three important and interrelated objectives of channel restoration design are: achieving channel stability; restoration of the form and, if possible, restoration of the function of a natural river (the 'pre-disturbance state'). This raises two questions: i) Are natural rivers stable? ii) Can the science of geomorphic engineering be used to describe the form and function of natural rivers? It follows from these questions that it is by no means certain that natural rivers should be used as realistic analogues for restoration.

2.2.1 Natural Channels: Realistic Analogues for Restoration?

The instream morphology of all natural river channels is moulded by complex interrelationships between the driving mechanisms of flow and sediment transport and the resisting forces of bed and bank stability and resistance to flow. Assuming stationarity in drainage basin controls, the undisturbed, natural geometry of a river channel is a stable geometric configuration that optimises the efficiency of flow conveyance and sediment transport so that in the medium- to long-term, sediment storage in the channel is approximately constant and inputs to the system are balanced by outputs. When a river has achieved this state of flow and sediment continuity, it is referred to as a 'graded' stream by geologists and geomorphologists and is termed 'poised' or 'in regime' by river engineers. At this condition, the river is self-regulating and strives to maintain its average condition in time and space.

An equilibrium channel is generally considered to be synonymous with a graded channel and implies both an adjustment of the channel to changes in the independent variables such as bed material load and discharge, and stability in form and profile (Leopold et al., 1964, p. 267). According to Howard (1982, p. 320), grade or equilibrium should refer to an absence of net aggradation or entrenchment and also refers to the simultaneous equilibrium of channel width, bed roughness, channel pattern, sinuosity and other channel properties. The term 'stability', as used to describe equilibrium channels, is sensitive to time scale such that over an intermediate scale, or medium- to long-term, average channel form and local gradient remain constant and in 'quasi-equilibrium' (Langbein and Leopold, 1964) with the prevailing regime of sediment load and discharge. According to Wolman and Gerson (1978, pp. 195-196):

> 'The notion of equilibrium river channels is based on the assumption that over a period of time the net effect of a variable climatic regime will produce a river channel of a given size and shape which is termed to be in adjustment with the climatically controlled runoff, sediment and vegetation within a given geological setting. All definitions recognize that both the processes and specific forms represent averages and that the characteristics which define equilibrium must be measured 'over a period of years', in Mackin's (1948) phrase, to allow for short term variations'.

Therefore, natural regime does not describe a condition of *perfect* negative feedback, or homeostasis, because within a long meandering reach there may be zones of aggradation and degradation, although at the reach scale an equilibrium channel is neither aggrading nor degrading, but *dynamically* stable. Within an open system, Strahler (1957) and Hack (1960, 1975) refer to the steady state in which channel form and profile remain unchanged as 'dynamic equilibrium'.

Deviations from the 'ideal', natural channel configuration over time are a function of the magnitude and frequency of natural and anthropogenic perturbations in the system and the reactive properties of the channel to both withstand the perturbations and recover from their detrimental effects. Therefore, rivers exhibit either 'robust' behaviour or 'responsive' behaviour (Werrity, 1997). Robust behaviour involves regulation by negative feedback, whereas responsive behaviour is a response to some change in the drainage basin controls, which causes a river to move across an extrinsic threshold to a new process regime.

Following an instance of morphological change at the reach scale, a channel's natural tendency is to recover its pre-disturbance state through re-establishment of a natural morphological configuration, or 'regime' state (Blench, 1969, p. 29). If a stream's natural form has been compromised either directly, via engineered modifications to the channel, or indirectly, for example via land-use changes in the catchment, then the morphology of the channel may represent the legacy of these influences during the recovery period. For example, when disturbance leads to channel widening, the channel may be too wide to convey the sediment load supplied from upstream. Subsequently, the lateral distribution of velocities would tend to induce deposition at the slow flowing channel margins where resistance to flow is greatest. This accumulation may then be aided by encroachment of vegetation on the juvenile berms. This process would drive progressive narrowing at a rate dictated by the energy of the system until the shear stress at the banks equals the frictional and gravitational resistance of the bank sediment.

Moreover, Wolman and Gerson (1978) recognised that channel change occurs as a response to complicated, and poorly understood, feedback mechanisms, or complex responses and that forcing events have a time-scale for effectiveness that relates the recurrence interval of an event to the time needed for a landform to recover to the form which existed prior to the event. According to Brunsden and Thornes (1977), complex response is the response to perturbing displacement *away* from equilibrium, which is spatially and temporally complex, leading to a considerable diversity in channel form (rather than average conditions). In some cases insensitive river engineering can act as a catalyst within dynamic, meta-stable equilibrium (Schumm, 1975) to carry the system over a geomorphic threshold and into a new equilibrium regime. Stevens et al. (1975) recognised that rivers frequently exhibit non-equilibrium forms because the morphology of a river channel at a given time is changing, especially in river systems with a wide range of peak flood discharges. Subsequently, process studies have begun to shift away from observing equilibria states, *per se*, to the recognition that few geomorphological systems are now 'as nature designed' and that multiple stable and unstable equilibria may coexist (Phillips, 1992a; Renwick, 1992) within non-linear dynamical systems (Phillips, 1992b; Lane and Richards, 1997) that are sensitive to rainfall-runoff processes in the catchment. Such complexity inhibits a full understanding of the fluvial system and thereby compounds the problem of engineering a restored channel.

The notion of equilibrium suggests uniformity in controlling factors and indicates that inputs are balanced by outputs either via immediate responses or change over a finite time (Howard, 1982). Based on this definition, Howard (1988) concluded that the presence of non-linearities and thresholds in geomorphological systems precludes equilibrium. Consequently, the concept is only relevant to specific parts of geomorphic systems and not to the systems themselves (Howard, 1988). However, channel restoration design can only be undertaken if it embraces the notion of equilibrium in design objectives using a catchment-based approach to ensure geomorphic stability. A natural channel is one in which is in harmony with, and delicately adjusted to dynamics and constraints within the catchment. Therefore, a restored stream can only be as stable as the drainage basin controls are constant. For example, restoring a river downstream of a rapidly urbanising area will lead to channel change in response to disequilibrium between the flow and sediment regimes and the restored channel configuration. It follows that the projected trajectory of land-use change dictates the potential life span of a restoration project.

To understand stable channel geometry, geomophologists are faced with the difficult task of describing the relationships between equilibrium channel form, the processes responsible for shaping that form and their complex interaction. The river exists within an open system, yet is traditionally defined by geomophologists within a black-box system to facilitate conceptualisation and development of process-from mathematical relationships.

Natural rivers in dynamic equilibrium possess a high degree of morphological diversity, in terms of cross-sectional, longitudinal and planform variabilities. The physical characteristics of a river channel include the shape and size of the channel cross section, the configuration of the bed along the path of the channel, sediment deposits and other instream features, the longitudinal profile and the channel pattern (Simons, 1979, p. 5-1). For straight alluvial streams, Lane (1937, p. 131) identified a list of factors that may enter into a determination of stable channel shapes: i) hydraulic factors (slope, roughness, hydraulic radius, mean velocity, velocity distribution and temperature); ii) channel shape (width, depth and chemical and physical side factors); iii) the nature of material transported (size, shape, specific gravity, dispersion, quantity and bank and sub-grade material) and; iv) miscellaneous factors (alignment, uniformity of flow and aging)

The number of morphological equations required to obtain a determinate solution of the fluvial system is controlled by the number of dependent variables that define the hydraulic geometry of the system. Hey (1978, 1988) identified nine degrees of freedom for natural channels with sinuous planforms based on: i) cross-sectional shape (wetted perimeter, hydraulic radius, maximum depth); ii) slope; iii) plan shape (sinuosity, meander arc length); iv) velocity, and; v) bed forms (bankfull dune wavelength, bankfull dune height). The controlling, or independent, variables are discharge, input sediment load, bed and bank sediment size, bank vegetation and valley slope. With only three established process equations available (continuity, flow resistance and sediment transport), the system is indeterminate unless empirical methods and other assumptions are applied. Moreover, Maddock (1970) suggested that relationships between width, depth, velocity and slope are indeterminate, because rivers do not remain in equilibrium under a wide range of drainage basin conditions. Maddock (1970, p. 2321) concluded that 'local and sometimes short-lived changes in bed configuration are the means by which dynamic equilibrium is maintained', yet the process-form mechanics of bedforms are poorly understood.

Limitations of hydraulic engineering, such as bedform mechanics, prevent the solution of all of the degrees of freedom in a natural river. Rather the geomorphic engineer must simplify reality and represent the natural forms of a channel in terms of a subset of dominant physical factors. As a result, cross-sectional shapes are usually described in terms of a simple trapezoid, with constant width along the reach, and the planform is routinely described in terms of a regular meander path rather than the complex, deformed patterns found in nature, particularly in large river systems. Few natural cross sections are uniform in shape, but exhibit a degree of asymmetry as a function of flow pattern around bends. Even cross sections at meander crossings (the point of inflexion between contiguous meander bends where secondary flow is at a minimum) usually possess some degree of asymmetry.

Secondary flows are responsible for shaping many geomorphological features in meandering rivers, for example cut banks, point bars, pools and riffles, chute channels and the complex distribution of bed sediment. However, despite the wealth of encouraging flume work investigating secondary flows and lateral momentum exchange between channel and floodplain, the lateral velocity distribution in natural streams is still poorly understood by engineers and geomorphologists. Even the most advanced models usually assume a wide channel in which secondary velocities are negligible. If the processes controlling the evolution and dynamics of secondary circulation were better understood, then the geomorphologist would be at a stronger position to understand the interplay between river form and mechanics. As Simons (1979, p. 5-52) remarked:

> 'The extent of the effect of secondary circulation on factors related to channel stability is unknown. Based on existing knowledge, its influence may be negligible, of considerable importance or somewhere in between these limits'

As so many unknown variables are involved in describing the channel configuration, 'the river is the best model of itself' (Shields, 1996, p. 26) and is ultimately the best channel restoration designer. If the necessary equations or 'physical laws' *were* available, they could be integrated and the prediction of a given phenomenon in time and space could be made mathematically (Simons, 1979, p. 5-75). Hey (1986) believes that a numerical model will eventually be developed from a more in depth understanding of natural processes, feedback mechanisms and process equations and three-dimensional simulation of scour and fill as a response to anthropogenic inputs. Until this is possible, alternative approaches must be sought.

The imbalance between the natural degrees of freedom posed by an undisturbed, unconstrained, meandering river and those which can be described mathematically is evident in the statistical uncertainty associated with morphological equations and often considerable differences between observed and predicted channel dimensions. This uncertainty is a complex function of unaccounted variables and local constraints that are difficult to quantify and impossible to account for in practical management tools. Due to hydraulic engineering being an 'exact' science, that is a study for precise, mechanistic relationships, uncertainty is rarely specified and often expressed in terms of numerical error rather than natural variability. By recognising this misinterpretation, natural rivers could be realistic analogues for channel restoration design.

2.3 THE *HYDRAULIC ENGINEER* AS A CHANNEL RESTORATION DESIGNER

2.3.1 Regime Theory: The *Empirical* Design Solution

As a natural alluvial stream attempts to sustain a dynamically stable (ideal) channel configuration, and minimise the fluctuations of morphological change over time, it is a reasonable channel restoration design goal to remove the imposed configuration and 'restore' the ideal state. However, because the river channel is a system with poorly understood complex responses, neither geomorphologists or hydraulic engineers know precisely what an ideal natural channel is or how it should function. Engineers over the past century have relied significantly on empiricism to fill the gaps in their limited understanding of the interrelationships between channel form and process.

Research as early as Davis (1899, 1902) confirmed that the gradient of a natural river diminishes with increasing distance from its source and that the quantity of flow increases as the order of the stream increases through the catchment (Strahler, 1952). Observations and experience of engineers working on major irrigation systems in the Indian subcontinent during the first half of this century revealed that there is a semi-causal link between flow and channel size which can be expressed mathematically: as the magnitude of flow increases, the width and depth of the channel tends to increase in a non-linear fashion.

The regime engineers were faced with the task of developing practical mathematical relationships to predict the three basic laws of self-adjustment of regime canals: the relationships between discharge and width, depth and slope. To derive these equations from hydraulic theory, three independent process-based equations are required, describing three different physical phenomena: hydraulic friction, sediment transport and the stability of the banks. With a complete understanding of these three processes, regime equations can be derived by their simultaneous solution. However, even if suitable equations were available, they would include very complicated coefficients as a complex function of flow and sediment properties, which are not well understood. Regime theory aimed to remove this complexity by using empirically derived formulae to represent 'in regime' conditions.

The form of a regime-type equation attempts to account for the uncertainty implicitly present in the constants and exponents of simple mathematical relationships. In this they are predictive but provide limited causal explanation. Based only on inductive reasoning, the regime method has raised a great deal of scepticism among academics, but has remained popular among practising engineers because of its strong predictive capability. More complex process-based methods of modelling the hydrodynamic and sediment transport system have only developed with the availability of computer technology, since the 1970s. However, as numerical methods have developed, they have also provided confirmation of the mathematical form of the empirical equations derived over half a century ago.

The first empirical studies which investigated the geometry of equilibrium channels were undertaken during the 1890s and early decades of the twentieth century on major irrigation canals, or *regime* channels, in the Indian subcontinent, notably in the Punjab Province and United Provinces of India and the Sind of Pakistan. By 1947, all the Indus tributaries were interconnected by canals and today Pakistan alone has in excess of sixty thousand miles of major canals and watercourses (Stevens and Nordin, 1987).

2.3.1.1 Regime Channel Physiography

The typical alluvial canal on the Indo-Gangetic Plain, as described by Blench (1952, pp. 389-391; 1957, pp. 13-15; 1969, pp. 37-40) had a mobile bed consisting of silty-sand dunes with particle sizes ranging between 0.1 to 0.6 mm but with a mean size of about 0.25 mm. While a canal bed was essentially self-formed, the banks were artificially constructed to accommodate a larger channel and encourage berm development as the channel filled. A straight alignment was maintained by the use of brush-wood spurs which stabilised the berm edges to encourage grassy banks to form. The final stage involved trimming the berm edge to form a perfectly straight canal. The method of designing set back banks allowed the channel to adjust within regime limits as the design discharge was gradually increased. A typical cross section was trapezoidal with equal side slopes of approximately 60 degrees to the horizontal (approximately 2:1) and slopes were less than 2 feet per mile (approximately 0.0004) (Inglis, 1948). In general, the canals were cut into an alluvial plain consisting of a silty-clay loam crust and carried suspended load of up to approximately 1000 to 2000 ppm during normal flow conditions (Blench, 1970; Raudkivi, 1990, p. 215; Stevens and Nordin, 1987). The early tendency in canal design was for 'economy in construction', by excavating narrow, deep channels. This type of channel aggraded over several years to yield a gradient sufficient to impose a non-silting velocity ('initial regime'). Being narrow, these channels had a further tendency to gradually widen until 'final regime' was ascertained, often taking in excess of 20 years:

> 'In general, therefore, final regime channels are more often to be expected among artificial channels of small or moderate size than in very big canals, the lateral dimensions of which are so radically fixed by man that Nature is afforded little opportunity of correcting them' (Lacey, 1930, p. 271).

The design flow carried by a regime canal was usually maintained close to the full supply discharge, therefore the *dominant* discharge, or flow expending the most geomorphic work, was the daily observed, or equilibrium, flow. This differs from a river system, where the dominant discharge is a function of the natural sequence of flow events experienced by the river (see Chapter 4).

Sediment discharge was controlled in the regime canals so that individual channels received sediment concentrations in proportion to their flows. Bed load was required to be substantially less than the off-take rivers. This was achieved by siting the off-take at the outside of a bendway so that the material accumulated at the inside of the bend as a function of secondary flow and could be subsequently dredged (Blench, 1957).

The term *regime*, synonymous with *prevailing conditions*, was coined by Lindley (1919) for canals of the Lower Chenab Canal System, India, with an average cross-sectional configuration over time and space, whereby 'the dimensions, width, depth and gradient of a channel to carry a given supply (of water) loaded with a given silt charge were all fixed by nature'.

2.3.1.2 Development of Cross Section Regime Equations

The development and demise of regime theory has been extensively documented by numerous researchers, including: Inglis (1948, 1949b); Leliavsky (1955); Chien (1957);

Henderson (1963, 1966); Graf (1971); Mahmood and Shen (1971); Shen (1971); Ackers (1972 and 1992); Kennedy (1975); Simons and Senturk (1977); ASCE Task Committee (1982); Neill (1982); Richards (1982); Blench (1986); Hey (1987, 1988, 1997c); Chang (1988); Raudkivi (1990); Stevens and Nordin (1990).

At the turn of the century, very few quantitative tools were available to the design engineer and channels constructed after 1850 suffered severe problems during self-adjustment (Richards, 1982, p. 291). The goal of the regime studies was to quantify basic observations of river behaviour in simple expressions linking form (channel geometry) and process (usually flow conveyance). Nation-wide research co-ordination was through the Central Board of Irrigation, Government of India, later to be subdivided into the Central Board of Irrigation and Power for India and the Water and Power Development Authority for Pakistan (Blench, 1986).

Two general types of regime equation were derived: i) those giving an expression for velocity (Kennedy-type) and; ii) those defining stable channel shapes (Lacey-type). The forms of the relationships were usually power function equations. The first equations expressed velocity as the dependent variable, as a surrogate for discharge, although the dependent and independent terms of the equations were used interchangeably.

Kennedy (1895) developed one of the first regime-type equations from 22 observations of the sandy-silt Bari Doab canal system, which links the Ravi and Sutlej rivers in the Punjab, near Lahore. The equation predicts a non-silting velocity, V_0, as a function of mean flow depth over the bed, D_m. Kennedy's work initiated a new engineering-science, which aimed to define a permissible velocity that would prevent siltation, as a criterion for designing a stable channel. Regime velocity was usually considered independent of all other factors. The canals studied by Kennedy had a range of discharges from approximately 26 to 1700 ft^3s^{-1} (0.7 to 48 m^3s^{-1}) (Stevens and Nordin, 1987, p. 1363) and depths ranged from approximately two to seven feet (Lindley, 1919).

$$\text{metric: } V_0 = 0 \cdot 55\, D_m^{0 \cdot 64} \quad \text{imperial: } V_0 = 0 \cdot 84\, D_m^{0 \cdot 64} \qquad (2.1a, b)$$

where 'V_0' is the non-silting velocity and 'D_m' is the mean bankfull depth. According to this relationship, it is permissible to design a narrow, deep channel or a wide, shallow one to carry the same discharge. Despite this fundamental flaw, the relationship was applied extensively until the 1930s (Blench, 1952, p. 387; Simons and Albertson, 1960, p. 35; Lane, 1937) and is still frequently quoted in the engineering literature today.

Arguably, the equation was only applicable to the Bari Doab because there was no allowance for variable sediment particle size, which is usually a significant parameter in flow-depth equations. Kennedy thought that the coefficient would be expected to vary with the quality and quantity of sediment but that the exponent would be nearly constant (Lane, 1937, p. 126). For *coarse* sand-bed channels, Kennedy suggested that the coefficient in Equation 2.1b should be raised to the range 0.92 to 1.00 and for fine sand should be reduced to the range 0.67 to 0.76 (Mahmood and Shen, 1971, p. 30-2)

The regime-based design procedure was described by Mahmood and Shen (1971, p. 30-2) and involved assuming a flow depth and calculating the flow velocity from Equation 2.1. The width could be obtained by dividing the design discharge (supply discharge) by the product of depth and velocity and the slope was calculated from the Kutter flow resistance equation (this is similar to but predates the Manning flow resistance equation). To specify the shape of the channel, Kennedy also produced guidance for appropriate width-to-depth ratio, specifying values from 3.5 for a discharge of 10 ft^3s^{-1} to a value of 7.0 for 1000 ft^3s^{-1} (Mahmood and Shen, 1971, p. 30-2).

As canals have three degrees of freedom (width, depth and slope), Kennedy was one equation short of a deterministic solution, and engineers were forced to adopt unofficial rules of thumb to compensate for an absent third equation. Despite this limitation, Kennedy recognised that the velocity-depth relationship had a limiting depth of 10 feet at which the banks began to erode and the relationship breaks down (Lane, 1937, p. 129), although no data for this limiting criteria are available.

Lindley (1919) transposed Kennedy's relation for design purposes to produce

$$\text{metric: } D_{\mathrm{m}} = 2\cdot 51 V_0^{1\cdot 56} \quad \text{imperial: } D_{\mathrm{m}} = 1\cdot 31 V_0^{1\cdot 56} \qquad (2.2a, b)$$

The canal systems which were designed from Kennedy's equation were the Lower Chenab (1900), Lower Jhelum (1901), Upper Chenab (1912), Lower Bari Doab (1913) and Upper Jhelum (1915), all located in the Punjab (Bakker et al., 1989).

The first width function was published 26 years after the Kennedy formula by Lindley (1919) and was expressed in terms of a width-to-depth ratio. Lindley discussed the development of a stable canal system and introduced the revolutionary engineering theory of *regime* channels:

> 'When an artificial channel is used to convey silty water, both bed and banks scour and fill, changing depth, gradient and width, until a state of balance is attained at which the channel is said to be in regime. These regime dimensions depend on discharge, quantity and nature of bed and berm silt, and rugosity of the silted section; rugosity is also affected by velocity, which determines the size of wavelets into which the silted bed is thrown' (Lindley, 1919, p. 63).

Lindley raised the concern that without significantly advancing from engineering judgement, channels would continue to be designed with exaggerated dimensions, resulting in excessive maintenance expenditure to dredge the unstable channels. Lindley's regime equations are based on 786 surveys made between 1915 and 1917 of the Lower Chenab Canal with a full capacity flow of approximately 11,000 ft^3s^{-1} and cross section dimensions averaged over each mile. The equations are suitable for depths less than 9 ft, widths less than 150 ft, velocities less than 3.3 ft s^{-1} and slopes between 0.000442 to 0.00127 (Lindley, 1919):

$$\text{metric: } V_0 = 0\cdot 57\, D_\mathrm{m}^{0\cdot 57} \quad \text{imperial: } V_0 = 0\cdot 95\, D_\mathrm{m}^{0\cdot 57} \tag{2.3a, b}$$

$$\text{metric: } V_0 = 0\cdot 27\, B^{0\cdot 36} \quad \text{imperial: } V_0 = 0\cdot 59\, B^{0\cdot 36} \tag{2.4a, b}$$

$$\text{metric: } B = 7\cdot 86\, D_\mathrm{m}^{1\cdot 61} \quad \text{imperial: } B = 3\cdot 83\, D_\mathrm{m}^{1\cdot 61} \tag{2.5a, b}$$

where 'B' is the bed width of a canal. The velocity-depth relationship (Equation 2.3) is similar to that of Kennedy, although the exponents differ slightly. Besides the frequently quoted Kennedy and Lindley equations linking depth to mean flow velocity, numerous others were produced and are documented by Lacey (1930, p. 260; 1958, p. 147) and Lane

(1937, p. 127) with source data from Egypt, India, Pakistan, Argentina, Thailand, Burma and the U.S.A.

Although the dependent and independent variables of regression equations should not be transposed, for design purposes Lindley presented the regime equations in the more familiar format:

$$\text{metric: } B = 38{\cdot}28 V_0^{2\cdot 82} \quad \text{imperial: } B = 4{\cdot}42 V_0^{2\cdot 82} \qquad (2.6a,\ b)$$

$$\text{metric: } D_{\mathrm{m}} = 2{\cdot}68 V_0^{1\cdot 75} \quad \text{imperial: } D_{\mathrm{m}} = 1{\cdot}09 V_0^{1\cdot 75} \qquad (2.7a,\ b)$$

Lacey (1958) remarked that:

> 'Lindley was responsible for the now classic dictum that the dimensions, depth, width, and gradient of a channel to carry a given supply loaded with a given silt charge are all fixed by nature. Nearly all subsequent research in India and Pakistan has, in effect, been devoted to an examination of this proposition' (Lacey, 1958, p. 148).

Although Lindley gave the first extensive *definition* of regime, his published equations were insufficient to adequately describe the regime *condition* because velocity was not measured but inferred from a resistance equation with an estimated roughness coefficient derived from the dimensions of the cross section. Despite this, the equations were used in India until approximately 1935 (Simons and Albertson, 1960, p. 36).

Although not accounted for in his equations, Lindley (1919, p. 67) noted that regime dimensions should be sensitive to boundary roughness, or 'rugosity'. This was developed in subsequent empirical research by Lacey (1930, 1933) who made the first comprehensive solution to the problem by expressing the channel form (wetted perimeter, P, and slope, S) as a function of bankfull discharge, Q_{b}, and rugosity for canals with design discharges up to 13500 $\mathrm{ft^3s^{-1}}$ (Mao and Flook, 1971, p. A-5). Recognising that the type of sediment was absent from previous equations, Lacey compiled 70 observations from Kennedy's and Lindley's data together with 15 observations from the Madras Godaveri western delta canals, India, to derive a 'new theory' for regime canals based on a silt factor, f_{s}, as a power function of mean bed sediment particle diameter, d_{m}. As Lacey sought to combine several data sets into a large database, his formulas were considered among the engineering community to be more reliable than previous equations and more applicable to a wide range of conditions.

As flow in the irrigation canals was maintained at approximate capacity, channel dimensions were measured at the bankfull stage. Power functions were 'fit' to the data with predefined exponents, rather than from regression analyses. The Lacey equations presented below are modified from the original relationships, as proposed in the reply to the discussion of his original paper, and are usually the ones quoted in the literature:

$$\text{metric: } V_0 = 0{\cdot}65(f_{\mathrm{s}}R)^{0\cdot5} \quad \text{imperial: } V_0 = 1{\cdot}17(f_{\mathrm{s}}R)^{0\cdot5} \tag{2.8a, b}$$

$$\text{metric: } V_0 = 10{\cdot}85\,R^{2/3}S^{1/3} \quad \text{imperial: } V_0 = 16{\cdot}12\,R^{2/3}S^{1/3} \tag{2.9a, b}$$

$$\text{metric: } P = 4{\cdot}84\,Q_{\mathrm{b}}^{0\cdot5} \quad \text{imperial: } P = 2{\cdot}67\,Q_{\mathrm{b}}^{0\cdot5} \tag{2.10a, b}$$

$$\text{metric: } \frac{P}{R} = 23{\cdot}36\,V_0 \quad \text{imperial: } \frac{P}{R} = 6{\cdot}99\,V_0 \tag{2.11a, b}$$

$$\text{metric: } S = \frac{f_{\mathrm{s}}^{5/3}}{3238{\cdot}62\,Q_{\mathrm{b}}^{1/6}} \quad \text{imperial: } S = \frac{f_{\mathrm{s}}^{5/3}}{1788\,Q_{\mathrm{b}}^{1/6}} \tag{2.12a, b}$$

where:

$$\text{metric: } f_s = (2519{\cdot}73\,d_{\mathrm{m}})^{0\cdot5} \quad \text{imperial: } f_s = (768\,d_{\mathrm{m}})^{0\cdot5} \tag{2.13a, b}$$

The equations allowed the design engineer to estimate width, depth, slope and non-silting velocity, V_0, as a function of discharge and sediment calibre, although the form of the slope equation was later demonstrated to be incorrect by Blench (1986, p. 52), who showed that the exponent of the silt factor should be 3/2 and not 5/3. The silt factor in Equation 2.13 is only a qualitative rule based on limited field data and the following dimensionless relationship:

$$f_{\mathrm{s}} = \left(\frac{V}{V_0}\right)^2 \tag{2.14}$$

where the non-silting velocity, V_0, was defined by fixing f_{s} to unity for Kennedy's data. From the velocity ratio, if the calibre of the bed material becomes finer, the critical velocity is reduced. Values of the silt factor are provided in Appendix G of the 1930 paper (Lacey,1930, p. 292) and Appendix V of the 1933 paper (Lacey, 1933, p. 453) for a range of conditions and locations for both canals and rivers ranging from 0.4 mm sand (Bari Doab Canals) to 25 inch boulders. The original d_m parameter was given in inches (rather than feet or metres), with a coefficient of 64, instead of 768 in Equation 2.13b. A summary of values for Lacey's silt factor is given in Table 2.1.

Bed Material Description	silt factor, f_s
Massive boulders (d_m 0.6 m)	39.60
Large stones	28.60
Large boulders, shingle and heavy sand	20.90
Medium boulders, shingle and heavy sand	9.75
Small boulders, shingle and heavy sand	6.12
Large pebbles and coarse gravel	4.68
Heavy sand	2.00
Coarse sand	1.44-1.56
Medium sand	1.31
Standard Kennedy silt (Upper Bari Doab)	1.00
Lower Chenab Canal silt	0.933
Lower Mississippi silt	0.357

Table 2.1 Values of Lacey's silt factor, f_s, for various types of bed material (after Lacey, 1930, p. 292).

The introduction of the silt factor gave the regime equations greater application potential, whereby the shape of a cross section depended upon the fineness of silt carried, coarse sediment corresponding to wide, shallow sections and conversely, fine silt defining narrow and deep shapes. Lacey also discovered that wetted perimeter is independent of the calibre of sediment, which tends to control the overall shape rather than the width dimension. This has since been confirmed by Hey and Thorne (1986) who indicated that bed sediment size influences the mean depth in mobile gravel-bed rivers but not significantly channel width, which is strongly a function of bank conditions, in particular riparian vegetation density.

Numerous variations of the Lacey equations can be derived by manipulating the flow continuity equation. From $Q=AV=PRV$:

$$\text{metric and imperial: } R = \left(\frac{Q_{\mathrm{b}}}{9 \cdot 74 f_{\mathrm{s}}} \right)^{1/3} \tag{2.15}$$

Hydraulic radius and wetted perimeter were preferred over depth and width because the parameters are a function of two-dimensional geometry and therefore, the overall cross-sectional shape. However, Lacey (1958) remarked that in large rivers the wetted perimeter and the hydraulic radius approximate the water surface width, W, and the mean depth, D_{m}, without appreciable error.

Although channel resistance can be indirectly determined from the above equations, Lacey developed a further flow resistance function very similar to the Manning equation, given by

$$\text{metric: } V_0 = \frac{R^{0 \cdot 75} S^{0 \cdot 5}}{N_\alpha} \quad \text{imperial: } V_0 = \frac{1 \cdot 346 R^{0 \cdot 75} S^{0 \cdot 5}}{N_\alpha} \tag{2.16a, b}$$

where:

$$N_\alpha = 0\ 0225 f_{\mathrm{s}}^{0 \cdot 25} \tag{2.17}$$

When f_{s} is unity, the rugosity coefficient, N_α, equals 0.0225 which is suitable for sand-bed channels. The hydraulic radius exponent of 0.75, rather than the conventional 0.67 in the Manning equation, is appropriate when the relative roughness (d_{m}/R) is elevated as a result of the presence of dune bed forms (Ackers, 1992).

Although the first to produce a comprehensive set of hydraulic data equations for stable channels with mobile beds, Lacey's equations do not account for the effects of bed material load and should, therefore, be applicable only to similar environments from which the original data were derived. Despite this limitation, Lacey (1930, p. 275) argued that the formulae 'cover the data of a very large range of stable channels and, if truly general, should, when extrapolated, fit the observed data of the largest available stable river discharges'.

Lacey's 1947 paper investigated how stable channel geometry may deviate from the regime dimensions estimated by the equations. He forwarded a 'shock' theory to explain these deviations, or 'abnormal channels', which refers to site-specific conditions of an individual channel which have not been adequately represented in the data set, for example very smooth or rough side slopes and head losses resulting from non-normal irregularities on the bed and resistance to flow in sinuous channels.

Lacey modified his equations in the light of new regime data (Lacey, 1947, 1958), which included substituting bankfull width and mean depth in place of wetted perimeter and hydraulic radius, respectively. The complete set of revised Lacey equations were presented by Mahmood and Shen (1971, p. 30-12). According to Lacey (1958), the equations are applicable to channels with discharges between 5 ft^3s^{-1} and 5000 ft^3s^{-1} and median bed material particle diameters between 0.15 mm and 0.4 mm (Mahmood and Shen, 1971, p. 30-12). Based on the full set of equations, Brookes (1988, pp. 41-42) described a simplified step-by-step trapezoidal channel design procedure using Lacey's equations, whereby given the design discharge (bankfull) and bed particle gradation, the wetted perimeter, hydraulic radius and slope for a stable cross section in a straight reach can be derived.

Stevens and Nordin (1987) criticised Lacey's regime theory for alluvial channels on the basis that Lacey's data were not extensive enough and the definition of the silt factor did not fully account for bed roughness. The basis of Lacey's research is the intrinsic relationship between rugosity, N_α, and silt factor, f_s, which represents Newton's second law for steady, uniform, one-dimensional flow. By combining Equations 2.13 and 2.17, a Strickler-type equation is produced:

$$\text{metric: } N_\alpha = 0 \cdot 06\, d_{\mathrm{m}}^{1/8} \quad \text{imperial: } N_\alpha = 0 \cdot 052\, d_{\mathrm{m}}^{1/8} \tag{2.18a, b}$$

This relationship only incorporates the particle roughness of a sand bed and neglects the component of roughness associated with the form of ripples and dunes. Furthermore, Chien (1957), examined the inequality of the silt factor derived from Equation 2.8 (f_{VR}) and the silt factor calculated by combining Equations 2.10 and 2.12, (f_{RS}) and the continuity equation, $Q=PRV$:

from Eqn. 2.8
$$\text{metric: } f_{VR} = 2 \cdot 40 \left(\frac{V_0^2}{R} \right) \quad \text{imperial: } f_{VR} = 0 \cdot 73 \left(\frac{V_0^2}{R} \right) \tag{2.19a, b}$$

from Eqns. 2.10, 2.12
$$\begin{gathered} \text{metric: } f_{RS} = 281 \cdot 66\, R^{1/3} S^{2/3} \\ \text{imperial: } f_{RS} = 189 \cdot 55\, R^{1/3} S^{2/3} \end{gathered} \tag{2.20a, b}$$

According to Chien (1957), only under a specific sediment concentration are f_{VR} and f_{RS} equal and therefore, sediment inflow should not be neglected from design considerations. For the general case, combining Equations 2.19 and 2.20 gives the following expression:

$$\text{metric: } \left(\frac{f_{VR}}{f_{RS}}\right) = 0 \cdot 0085 \left(\frac{V_0}{R^{2/3} S^{1/3}}\right)^2 \neq 1$$
$$\text{imperial: } \left(\frac{f_{VR}}{f_{RS}}\right) = 0 \cdot 00385 \left(\frac{V_0}{R^{2/3} S^{1/3}}\right)^2 \neq 1 \qquad \text{(2.21a, b)}$$

Using data from northern India and Pakistan, and Einstein's bed load function (Einstein, 1950), Chien (1957) investigated how Lacey's silt factor implicitly accounts for sediment load and demonstrated that f_{VR} increases greatly with the concentration of transported material (sediment discharge exponent 0.715) while f_{RS} remains relatively constant (sediment discharge exponent 0.052) but is a function of the bed material size. This explanation was later corroborated by Mao and Flook (1971) and Stevens and Nordin (1987).

Lacey had, in effect, produced two different momentum equations, creating redundancy and two silt factors. To remove this redundancy, Inglis (1948) presented an index $(f_{VR} / f_{RS})^2$ as a measure of divergence from regime and considered a value of unity represents the regime state. Using this principle, Equation 2.9 should adopt the form of Equation 2.21, such that

$$\text{metric: } V_0 = 10 \cdot 85\, R^{2/3} S^{1/3} \left(\frac{f_{VR}}{f_{RS}}\right)^{0 \cdot 5}$$
$$\text{imperial: } V_0 = 16 \cdot 12\, R^{2/3} S^{1/3} \left(\frac{f_{VR}}{f_{RS}}\right)^{0 \cdot 5} \qquad \text{(2.22a, b)}$$

Chien (1957) also reviewed this dilemma and derived relationships expressing the two forms of the Lacey silt factor in terms of bed material load concentration, C (defined as the ratio of sediment discharge to flow discharge in ppm), such that

$$\text{metric and imperial: } f_{VR} = 0 \cdot 061\, C^{0 \cdot 715} \qquad \text{(2.23)}$$

$$\text{metric: } f_{RS} = 2\cdot 2 d_m^{0\cdot 45} C^{0\cdot 05} \quad \text{imperial: } f_{RS} = 1\cdot 29 d_m^{0\cdot 45} C^{0\cdot 05} \qquad (2.24\text{a, b})$$

The above critique of Lacey's work does not detract from the geomorphic engineering value of the qualitative form of the regime-type relationships:

$$P \propto Q_{\mathrm{b}}^{0\cdot 5} \text{ or } W \propto Q_{\mathrm{b}}^{0\cdot 5} \qquad (2.25)$$

$$R \propto Q_{\mathrm{b}}^{0\cdot 33} \text{ or } D_{\mathrm{m}} \propto Q_{\mathrm{b}}^{0\cdot 33} \qquad (2.26)$$

$$S \propto Q_{\mathrm{b}}^{-0\cdot 167} \qquad (2.27)$$

Blench (1939) modified Lacey's silt factor by proposing its separation into a bed factor and side factor to account for the difference in form roughness between the dune covered bed and smooth banks. Furthermore, the hydraulic radius and wetted perimeter variables in Lacey's formulae were replaced by depth and width, respectively, for design purposes. The research by Blench covers five decades with notable publications reviewing the development and application of regime concepts in the United States in 1939, 1952, 1957, 1969, 1970 and 1986. Blench (1970) considered that the only justified modification to the Lacey equations is the resolution of the silt factor into components representing bed material effects and bank resistance, based on the critique which led to the development of Equation 2.21 and recognition that thinly vegetated channel banks usually act as smooth boundaries whereas a duned sand bed is clearly rough. He developed a bed factor, F_{b}, and as side factor, F_{s}, given by

Bed factor $$F_{\mathrm{b}} = \frac{V^2}{D} \qquad (2.28)$$

Side factor $$F_{\mathrm{s}} = \frac{V^3}{W_{\mathrm{m}}} \qquad (2.29)$$

where 'W_{m}' is the mean width, 'D' is the maximum depth in a trapezoidal channel and 'V' is the mean cross-sectional velocity. Using W_{m}, defined by A/D, ensures continuity whereby $Q_{\mathrm{b}}=VDW_{\mathrm{m}}$. According to Blench (1970), F_{b} is proportional to the square of the Froude number (units of kinematic viscosity, or force per unit mass, $\mathrm{ft\,s^{-2}}$ or $\mathrm{m\,s^{-2}}$) and is a function of median bed sediment size, bed material load concentration and fluid kinematic viscosity. Blench's side factor, F_{s}, is a measure of smooth-boundary shear stress, or tractive force intensity on the sides (units $\mathrm{ft^2s^{-3}}$ or $\mathrm{m^3s^{-3}}$), and describes a range of values with an upper limit set by the potential to erode and a lower limit set by the potential to deposit sediment from suspension at the channel margins (Blench, 1970, p. 208). Blench (1970) also proposed a momentum equation for uniform, one-dimensional flow which accounted for sediment concentration, given by

$$\frac{V^2}{\mathrm{g}DS} = 3{\cdot}63\left(1 + \frac{C}{2330}\right)\left(\frac{VW_{\mathrm{m}}}{\nu}\right)^{0\cdot 25} \tag{2.30}$$

where 'ν' is the kinematic viscosity of the water-sediment mix and 'C' is the bed material load concentration (ppm). The left side of Equation 2.30 is a friction factor expression, $f_{\mathrm{f}}/8$, where f_{f} is the Darcy-Weisbach friction factor, and the right side is a scaled width-based Reynolds number. The sediment concentration multiplier 1/2330 was recommended for non-uniform sands.

Suitable values of F_{b} and F_{s} are required for solution of Equations 2.28 and 2.29 for specific channel 'types'. The bed and side factors cannot be easily measured in the field because they are influenced by variables that cannot be readily quantified, in particular bank erodibility properties. Therefore, approximations for F_{b} and F_{s} were provided by Blench as they relate to channel type. Blench (1952, 1957, 1969, 1970) recommended a simple set of semi-qualitative rules based on observed conditions in canals and rivers by Lacey (1930).

For relatively low concentrations of sediment load and median bed material particle size less than 0.002 m, a condition is derived by combining Equations 2.8, 2.13 and 2.28 with an adjustment made for converting R to D, such that

$$\text{metric: } F_{\mathrm{b0}} \approx 18\cdot 31 d_{50}^{0\cdot 5} \quad \text{imperial: } F_{\mathrm{b0}} \approx 10\cdot 11 d_{50}^{0\cdot 5} \tag{2.31a, b}$$

where F_{b0} assumes that bed material load discharge is negligible and d_{50} is in metres or feet (modified from Blench's equation which expressed d_{50} in millimetres). Equation 2.31 is not suitable for channels with gravel beds. Blench and Qureshi (1964) presented a chart derived from gravel bed data relating F_{b0} to median sediment size and particle fall velocity, although an equation was not given. For channels with dune bed forms and appreciable bed material load, Blench (1970) suggested the following adjustment:

$$F_{\mathrm{b}} = F_{\mathrm{b}0}\left(1 + 0{\cdot}12\,C\right) \qquad (2.32)$$

Values of F_b in the Indian canals on alluvial plains ranged from 0.6 to 1.25, with a modal value approximating 1.0. Blench (1970) presented suitable values of side factor, F_s, for channels with different bank characteristics (Table 2.2).

Channel Bank 'Type'	F_s (metric)	F_s (imperial)
Friable banks	0.0093	0.10
Silty, clay loam banks	0.0186	0.20
Tough clay banks	0.0279	0.30

Table 2.2 Values of side factor, F_s, for channels with different bank characteristics (after Blench, 1970).

Using the continuity equation, $Q_b = VDW_m$, and the definitions of F_b and F_s, equations suitable for channel design can be derived and expressed in terms of the 3 degrees of freedom for trapezoidal channels:

$$W_{\mathrm{m}} = \left(\frac{F_{\mathrm{b}}Q_{\mathrm{b}}}{F_{\mathrm{s}}}\right)^{0\cdot5} \qquad (2.33)$$

$$D = \left(\frac{F_{\mathrm{s}}Q_{\mathrm{b}}}{F_{\mathrm{b}}^{2}}\right)^{1/3} \qquad (2.34)$$

$$S = \frac{F_{\mathrm{b}}^{0\cdot875}}{\left(\dfrac{3\cdot63\mathrm{g}}{\nu^{0\cdot25}}\right)W_{\mathrm{m}}^{0\cdot25}D^{0\cdot125}\left(1+\dfrac{C}{2330}\right)} \qquad (2.35)$$

where 'g' is the acceleration due to gravity (9.81 ms^{-2}). Equations 2.33 to 2.35 are suitable for straight channels with Q_b between 0.03 $\mathrm{m^3s^{-1}}$ and 2800 $\mathrm{m^3s^{-1}}$, d_{50} between 0.1 and 0.6 mm and ripple-dune bedforms.

For meandering channels, Blench (1969) indicated that the right hand side of the slope equation should be multiplied by a factor, k, where k varies from 1.25 for straight channels with alternate bars to 2 for well developed meandering channels, although Hey (1997c) questioned this adjustment on the basis that channel gradient decreases with the degree of meandering.

Blench (1952, p. 384; 1969, p. 27) declared that a river acquiring a regime is 'comparable in a general sense to stating that a territory has acquired a climate', whereby channel change is secular and fluctuations are ironed out over the medium to long-term. To Blench,

> 'A river is said to be "in regime" in a reach if its mean measurable behaviour during a certain time interval does not differ significantly from its mean measurable behavior during comparable times before or after the given interval' (Blench, 1952, p. 384).

Inglis (1942) contested the Lacey equations on the basis that the coefficients and exponents were often modified by design engineers for a particular channel without justification. He refuted Lacey's 'shock theory' on the basis that deviations from the regime condition, specified by Lacey's equations, were attributable to differences in sediment load entering different canals. From a review of the technical literature, Inglis (1942) noted that that the exponent in the velocity-depth regime equations typically varies between 0.52 and 0.64, and the coefficient may be expected to vary between 0.67 and 0.95. Using data obtained from the Lower Chenab canal system, Inglis showed divergence between observed and predicted dimensions (Raudkivi, 1990, p. 215), such that: i) P values varied from 0.82 to $1.45\overline{P}$ (standard deviation 0.178); ii) V_0 values varied from 0.89 to $1.21\overline{V_0}$ (standard deviation 0.095) and; iii) S values varied from 0.69 to $1.45\overline{S}$ (standard deviation 0.177), where, $\overline{P}$, $\overline{V_0}$ and $\overline{S}$ refer to the mean wetted perimeter, non-silting velocity and slope, respectively.

Noting the inadequacy of contemporary regime theory to incorporate variable sediment load, Inglis (1948, 1949a) proposed a new regime parameter as the product of sediment concentration, C, and mean particle fall velocity, V_s (a measure of work rate per unit plan area necessary to maintain particulate suspension). Furthermore, Inglis attempted to remove the problem of dimensional coefficients in regime equations using dimensional analysis to produce ten new regime-type equations. With only three principle degrees of freedom, W_m, D and S, to solve for a trapezoidal cross section of known side slopes, the derivation of ten new equations made the solution over-determinate. However, three equations were singled out as the most important for alluvial canals with sand boundaries (Ackers, 1992):

$$V_0 = \frac{\mathrm{a}_1 \mathrm{g}^{7/18} Q^{1/6} \left(d_\mathrm{m} C V_\mathrm{s}\right)^{1/12}}{\nu^{1/36}} \tag{2.36}$$

$$\frac{W_\mathrm{m}}{D} = \frac{\mathrm{a}_2 Q^{1/6} \left(d_\mathrm{m} C V_\mathrm{s}\right)^{7/12}}{\mathrm{g}^{5/18} \nu^{7/36}} \tag{2.37}$$

$$\frac{1}{S} = \frac{\mathrm{a}_3 \mathrm{g}^{1/18} \nu^{5/36} Q^{1/6}}{\left(d_\mathrm{m} C V_\mathrm{s}\right)^{5/12}} \tag{2.38}$$

where d_m is the mean diameter of bed material (in metres or feet) and a_1, a_2 and a_3 are constants related to channel 'type'.

Equations 2.36 to 2.38 reveal that sediment concentration exerts only a marginal effect on velocity (exponent 1/12) but has a large influence on the form ratio (exponent 7/12) and slope (exponent 5/12). This sensitivity to sediment load was the missing element in the early regime equations, although Inglis did not specify values for the coefficients. The work of Inglis marked the peak of regime theory development (Ackers, 1992).

In the United States of America, Simons and Albertson (1960) sought confirmation of the regime laws for a range of stable channel types. Their data, often referenced to as Simons and Bender data, were based on canal studies in India and the United States with a significant contribution from Colorado, Wyoming and Nebraska (Simons and Albertson 1960)). The resulting equations were reexamined by Henderson (1966) and rearranged here for simplification:

$$W_\mathrm{m} = \mathrm{K}_1 Q^{0\cdot 5} \tag{2.39}$$

$R \leq 7$ ft

$$D = \mathrm{K}_2 Q^{0\cdot 36} \tag{2.40}$$

$R > 7$ ft

$$\text{metric: } D = 0\cdot 61 + \mathrm{K}_3 Q^{0\cdot 36} \quad \text{imperial: } D = 2\cdot 0 + \mathrm{K}_3 Q^{0\cdot 36} \tag{2.41a, b}$$

$$\frac{V^2}{\mathrm{g}DS} = \mathrm{K}_4 \left(\frac{V W_\mathrm{m}}{\nu}\right)^{0\cdot 37} \tag{2.42}$$

where the constant values, K_1, K_2, K_3 and K_4 are given in Table 2.3.

Channel Type	K_1	K_2	K_3	K_4
Sand bed and banks	5.71 (3.15)	0.69 (0.63)	0.53 (0.48)	0.33
Sand bed and cohesive banks	4.24 (2.34)	0.58 (0.53)	0.45 (0.41)	0.54
Cohesive bed and banks	3.59 (1.98)	0.49 (0.45)	0.38 (0.34)	0.87
Coarse non-cohesive material	2.85 (1.58)	0.31 (0.28)	0.24 (0.21)	-
Sand bed and banks with large sediment load	2.77 (1.53)	0.45 (0.41)	0.35 (0.32)	-

Note: values in parenthesis are in imperial units for Equations 2.39 to 2.41.

Table 2.3 Coefficient values for various channel types in the Simons and Albertson (1960) regime equations (modified after Henderson, 1966).

K_4 values for channels in coarse material and those in sand with large sediment loads were not specified and it was recommended by Ackers (1972, p. 260) that they should be determined from appropriate sediment transport relationships, as otherwise the slope may be inaccurate.

A wealth of regime analyses flooded the engineering literature in the decades following the pioneering research by Lacey and others, such as the work by Bose (1936), Lapturev (1969), and many others. Other research focused more specifically on investigating why stable channels often deviate from the dimensions specified by regime equations and how to incorporate empirical deviation into design equations.

From a statistical analysis of stable regime channel data, Chitale (1976) attempted to quantify the degree to which observed values of wetted perimeter, P, hydraulic radius, R, and Slope, S, diverged from the dimensions given in the Lacey equations. Deviation was explained using residual analysis as related to the deviation in form ratio of the channel (P/R) and boundary shear stress. For a specified design discharge, Q_b, median bed particle size, d_{50}, and boundary shear stress, a wide and shallow channel section tends to exhibit a slope greater than that specified by Lacey, and conversely, the Lacey equations tend to overestimate slope for narrow and deep sections. This tendency was corroborated by

Chang (1985b). Deviations, or 'outliers', can occur as a result of natural variability or when one or more of the degrees of freedom are notably different from the range of conditions found in the Lacey channels. From a data set comprising 252 observations on stable alluvial canals in India, Pakistan, U.S.A. and Egypt, Chitale developed a series of Lacey divergence equations which account for the residual variance.

For bed material size between 0.2 and 0.6 mm:

$$\begin{aligned}\text{metric: } & P = 2\cdot 601 Q_{\mathrm{b}}^{0\cdot 488} S^{-0\cdot 109} d_{50}^{0\cdot 117}\\ \text{imperial: } & P = 1\cdot 304 Q_{\mathrm{b}}^{0\cdot 488} S^{-0\cdot 109} d_{50}^{0\cdot 117}\end{aligned} \qquad (2.43a, b)$$

and

$$\begin{aligned}\text{metric: } & R = 0\cdot 270 Q_{\mathrm{b}}^{0\cdot 354} S^{-0\cdot 061} d_{50}^{0\cdot 006}\\ \text{imperial: } & R = 0\cdot 249 Q_{\mathrm{b}}^{0\cdot 354} S^{-0\cdot 061} d_{50}^{0\cdot 006}\end{aligned} \qquad (2.44a, b)$$

For bed material size between 0.05 and 0.2 mm:

$$\begin{aligned}\text{metric: } & P = 1\cdot 949 Q_{\mathrm{b}}^{0\cdot 479} S^{-0\cdot 162} d_{50}^{0\cdot 158}\\ \text{imperial: } & P = 0\cdot 961 Q_{\mathrm{b}}^{0\cdot 479} S^{-0\cdot 162} d_{50}^{0\cdot 158}\end{aligned} \qquad (2.45a, b)$$

and

$$\begin{aligned}\text{metric: } & R = 0\cdot 067 Q_{\mathrm{b}}^{0\cdot 307} S^{-0\cdot 315} d_{50}^{0\cdot 202}\\ \text{imperial: } & R = 0\cdot 058 Q_{\mathrm{b}}^{0\cdot 307} S^{-0\cdot 315} d_{50}^{0\cdot 202}\end{aligned} \qquad (2.46a, b)$$

However, when Q_{b} and d_{50} are given, the divergence equations are not determinate but reveal that P, R and S are interdependent, with a single degree of freedom between them. Chitale (1976) suggested that the Lacey solutions are one set of many possible combinations, all of which are theoretically stable. Equations 2.43 to 2.46 were derived from channels with P between 3.0 m and 125.1 m, R between 0.27 m 4.11 m, S between 0.000046 and 0.00434, Q_{b} between 0.42 m^3s^{-1} and 4089 m^3s^{-1} and d_{50} between 0.0066 mm and 19.7 mm. The interdependence between width, depth and slope was further demonstrated by Chitale (1988, 1995) from field data derived from 36 meandering and braided rivers in India.

The initial divergence formulae were later improved to account for variant bed and bank material and sediment transport, by adding two further variables, bed material concentration, C, and bank shear stress, τ_s (Chitale, 1994). According to Chitale (1996, p.

358), the coefficient, a, in the Lacey width expression, $P=aQ^b$, is a function of both C and τ_s, while the exponent, b, is a measure of the rate of change of C and τ_s, along the channel. Sediment load was not measured but estimated from Colby's graphical relations for sediment concentration as a function of velocity, depth and d_{50}, applicable to sand-bed channels (Colby, 1964). The final Lacey divergence equations are given here as a set of dimensionless equations (rearranged from the original equations):

$$P = \frac{P_L C^{0\cdot 75} \tau^{0\cdot 625}}{C_L^{\;0\cdot 75} \tau_L^{0\cdot 625}} \tag{2.47}$$

$$R = \frac{R_L C_L \tau^{0\cdot 5}}{C \tau_L^{0\cdot 5}} \tag{2.48}$$

$$S = \frac{S_L C \tau^{0\cdot 5}}{C_L \tau_L^{0\cdot 5}} \tag{2.49}$$

where the subscript, 'L', refers to a parameter derived using Lacey's equations for the same discharge and bed material size. It is assumed that the relationship between the bed shear stress, τ, and bank shear stress, τ_s, can be approximated by the following relationship, which was considered reasonable for the type of channel banks found in the Lacey channels:

$$\tau = 0\cdot 75\,\tau_s = \gamma RS \tag{2.50}$$

The channel design method using the Lacey divergence equations involves four calculation stages, given values of Q_b, τ_s and d_{50}: i) calculation of P_L, R_L, S_L and V_L using the original Lacey equations; ii) calculate C_L using the results from stage i and the Colby relationships; iii) calculate τ_L using the results from stage i; iv) calculate τ from Equation 2.50 and; v) calculate P, R and S from Equations 2.47 to 2.49.

The proliferation of regime equations and their application to channel design prompted confirmation from both controlled flume experiments and hydraulic theory.

2.3.1.3 Laboratory Analogues of Regime Channels

In 1964, Ackers documented the results of laboratory studies undertaken at the Hydraulics Research Station, Wallingford, U.K., which attempted to validate regime channel dimensions in flume streams with mobile sand beds and sand/clay banks (Ackers, 1964, 1972). By controlling a range of discharges, the geometries of the stable streams that emerged were used to derive regime relationships for width, depth and slope. The results confirmed that the regime approach to describe stable channels was valid. Ackers (1972, pp. 265-266) summarised the conclusions from the research, and the main points relevant to this discussion are reproduced below:

i) Experiments conducted in beds of non-cohesive material with median particle diameters of 0.16 mm and 0.34 mm at constant discharges between 0.4 m^3s^{-1} and 5.4 m^3s^{-1} confirmed Lindley's dictum that 'the dimensions, width, depth and gradient of a channel to carry a given supply (discharge) loaded with a given silt charge, were all fixed by nature, ie uniquely determined'.

ii) Meandering was more likely in channels with high sediment concentrations than in those with low concentrations. This implies that any procedure for designing meandering channels with mobile beds must incorporate sediment transport to ensure channel stability in the medium- to long-term.

iii) Marked correlations were obtained in the equations relating channel geometry attributes with discharge, but these *did not* confirm that the Lacey equations were entirely applicable to streams of this type and scale.

iv) Width was closely proportional to discharge with a discharge exponent of 0.42. This indicates that the width in laboratory channels tends to adjust less with discharge than in real rivers (which have exponents approximating 0.5).

v) Streams with clayey banks were narrower than in sand-banked channels. This is in accordance with the results of Simons and Albertson (1960).

vi) Slope did not show any marked correlation with discharge, being strongly influenced by sediment concentration. This has subsequently been confirmed in the downstream hydraulic geometry analyses of Hey and Thorne (1986), and others.

The laboratory research of Ackers (1964) represents one of the earliest confirmations that ‘regime’ is not a social construct but results from the natural laws governing fluid flow and sediment transport.

2.3.1.4 Regime Theory Limitations

Regime theory was developed to overcome the problem of canal design, initially on the Indo-Gangetic plain. The channels are very simple in configuration, being originally artificially constructed before attaining a condition of regime, or dynamic stability, by aggradation. The canals were usually designed to meet one of three sets of conditions (Lane, 1937, p. 133):

i) Definition of the lowest practicable velocity, in order to reduce the slope to a minimum. For irrigation canals, this enabled the greatest irrigable area to feed from the channel per unit length. The design solution was to prevent silting by securing the lowest practicable cross-sectional form ratio.

ii) Definition of the highest practicable velocity, in order to reduce costs by making the size of channel as small as possible without inducing degradation. This was achieved by creating a cross-sectional form ratio at the threshold of motion.

iii) Definition of the greatest practicable slope, in order to reduce the costs of drop structures when the alignment is steep without inducing bed scour. This was done by making the cross-sectional form ratio as large as possible to lower the hydraulic radius, hence velocity.

From the above considerations, it is clear that, while the channel itself played a role in defining the channel shape at ’final regime’, the configuration was generally a function of the project objectives rather than natural channel evolution to some equilibrium state.

Engineers used their empirical equations with success for many decades, regarding them as almost laws of nature, despite continued scepticism on the basis of the lack of conventional hydraulics of flow resistance and sediment transport in the design methodologies. The engineering community readily adopted the equations because they were of immediate value to engineers, rather than more theoretical approaches, which had little empirical confirmation (Ackers, 1972, p. 259).

The major disadvantages and limitations of the regime approach are summarised as follows:

i) The theory was not further developed for a wide variety of physiographic and hydrological conditions found in practice, for example the regime data incorporates only a narrow range of silt sizes.

ii) The theory requires a detailed knowledge of the conditions, or channel 'type', upon which the formulae were based if they are to be applied successfully. Extreme caution must be exercised when applying regime equations to different types of channel. For example, from a comprehensive data set of regime canals in Pakistan, Mao and Flook (1971) showed that the Lacey equations markedly overestimated cross-sectional area, velocity and width. These results support the work by Chitale (1976, 1988, 1994, 1996) on divergence from regime and raises the question of whether regime formulae, developed from unlined *canals*, should be applied in the design of stable *rivers*.

iii) In general, the equations were developed for channels with relatively few modes of adjustment (straight planform, trapezoidal cross sections and homogeneous periphery sediments), rather than the complex geometries of self-formed channels found in nature.

iv) The equations were empirically derived and were therefore, not based on the mechanics of hydraulic engineering. This limitation was recognised by Bose (1936) immediately following the original work of Lacey:

> 'These relationships are frankly empirical and have not got the sanctity of the laws of nature…it must be clearly understood that these relations…are definitely empirical and as such cannot stand the usual operations of mathematics… that can be safely performed on relationships derived from strict theoretical considerations' (Bose, 1936, pp. 70, 75).

In light of this, engineers have tried to use more analytical approaches in an attempt to justify the form of the regime relationships from a hydraulic basis (Section 2.3.2).

v) Variance in the dependent variables is not fully accounted for by the independent variables, especially for the conventional bivariate equations that use discharge as the only channel forming parameter. For example, Mao and Flook (1971, p. A-5) interpreted Lacey's silt factor as representing all of the undefined influences on alluvial canal hydraulics.

vi) The theory fails to recognise the important influence of sediment transport and flow resistance on the determination of stable channel dimensions, in particular the slope (Simons and Albertson, 1960; Lane, 1937).

vii) The equations were developed for prediction rather than for investigating causal mechanisms between channel forms and processes. Consequently, regime theory has been questioned over the decades in terms of its scientific (and geomorphic) credibility.

viii) The equations are generally dimensional; that is, their coefficients have units which may be regarded as improper scientific practice.

ix) The equations are purely deterministic; that is, they represent a unique configuration, or ideal state, for a given set of independent variables rather than the range of possible stable geometries that are found in nature. The latter requires a more probabilistic route of enquiry.

Hydraulic geometry was the inevitable successor of regime theory and is discussed in Section 2.4.1.

2.3.2 Analytical Channel Design: The *Rational* Design Solution

Reviews of the numerous analytic approaches to channel design have been given by: Lane (1955); Chow (1959); Henderson (1963, 1966); Langbein (1964); Graf (1971); Shen (1971); Vanoni (1975); Simons and Senturk (1977); Neill (1982); Richards (1982); Ferguson (1986); Chang (1988); Raudkivi (1990); Chadwick and Morfett (1993); Hey (1988, 1997c); and many others.

Alluvial rivers must have the competence required to transport sediment of a particular grain size. If this threshold is not exceeded, then the river will silt. If the threshold is exceeded, then this sediment will be entrained. However, if the river is deficient in sediment entering a reach (supply limited), then the riverbed may be subject to erosion. Lane (1953) defined a stable channel as '...an unlined channel for carrying water, the banks and bed of which are not scoured by the moving water, and in which objectionable deposits of sediment do not occur'. The forces exerted by the flow arise from fluid motion and particle characteristics. The forces resisting motion include the complex frictional and cohesive forces of the mobile or static bed and banks. The rational design solution is a mechanistic attempt to balance these forces using concepts derived from fluid flow, flow resistance and sediment transport theories in order to derive stable channel dimensions (Ferguson, 1986) as an alternative to empirical, regime-type equations. The problem facing river analysts/managers is that there are fewer process equations than modes of adjustment (degrees of freedom) to solve for alluvial channels. Hence, the fluvial system is indeterminate (Maddock, 1970). The flow of water, Q, and discharge of sediment, Q_s, within any alluvial channel are governed by the Newtonian laws of conservation of mass and fluid motion. These natural laws (or models) can be subdivided and simplified as follows:

***Conservation of Mass*:**

Water: $Q = VA = VWD_m = VPR$

Sediment: $C = Q_s / Q$

***Motion*:**

Water:	A fluid momentum relationship (flow resistance expression).
Bed sediment:	A bed material load transport equation, incorporating bed load and suspended load.
Bank sediment:	Bank erodibility/mass failure models

Without considering planform or bedform characteristics of natural channels and assuming uniform, steady and essentially one-dimensional flow and conservation of mass, the simultaneous solution of three hydraulic equations would yield the mean depth (or maximum depth in a trapezoidal cross section), slope and width of a stable, straight channel for a specified discharge and known boundary roughness:

***Flow Resistance*:**

$$D_{\mathrm{m}} = \mathrm{f}\left(S,\ W,\ \text{energy losses}\right)$$

***Sediment Transport*:**

$$S = \mathrm{f}\left(D_{\mathrm{m}},\ W,\ \text{flow resistance}\right)$$

***Bank Erosion*:**

$$W = \mathrm{f}\left(D_{\mathrm{m}},\ S,\ \text{bank angle},\ \text{flow resistance},\ \text{bank sediment character},\ \text{other factors}\right)$$

where ‘other factors’ include ground water elevation, seepage, vegetation effects, near-bank flow pattern and antecedent conditions.

While flow resistance and sediment transport equations are well established, bank erosion and mass failure expressions are not very well developed because the complex interaction between bed and bank processes and materials are poorly understood in terms of quantitative mechanisms. Until further research in this area is undertaken, alternative theories, or ‘hydraulic devices’, are required to facilitate channel design. Rational theories, often termed ‘process-based’ methods, make an assumption for the last of the ‘*n*’ hydraulic relationships needed for the determinate solution of ‘*n*’ unknowns, thereby overcoming the indeterminacy problem.

There are four different hydraulic approaches available that use analytical techniques: i) maximum hydraulic efficiency in non-erodible beds; ii) tractive force theory; iii) extremal/variational hypotheses and; iv) analytical regime theory for channels with mobile beds.

2.3.2.1 Maximum Hydraulic Efficiency in Non-Erodible Beds

The concept of maximising the hydraulic efficiency or 'performance' of the channel is an appropriate criterion for the design of non-erodible (often lined) channels with a low throughput of sediment. This is the simplest of the analytical techniques as the only degree of freedom is the channel depth.

The design of non-erodible channels is often required in urban areas to maximise flow efficiency for flood protection, when floodplain development prohibits a more natural channel configuration, whilst minimising excavation. Design techniques for non-erodible channels are also appropriate for upland streams with gravel/cobble beds or where supply limitation has resulted in paving. For cases when a lined channel is conveying an appreciable quantity of sediment, it is assumed that the most efficient cross section will minimise sedimentation. Parameters to be considered include the roughness of the channel bed, the slope of both the bed and banks and the required freeboard to maintain a suitable factor of safety against flooding. Provided the bank material (or lining) is sufficiently stable, side slopes should be as steep as possible to maximise efficiency. According to Chow (1959, p. 159), freeboards between 5 and 30 percent of the depth are commonly used in practice.

The best hydraulic cross section requires an optimisation procedure. The analysis described below is modified and extended from Chadwick and Morfett (1993, pp. 444-446). Assuming uniform flow conditions in a trapezoidal channel, the Manning equation can be expressed as

$$Q_b = \frac{R^{0 \cdot 67} S^{0 \cdot 5}}{n} = \frac{A^{0 \cdot 67} S^{0 \cdot 5}}{n P^{0 \cdot 67}} \tag{2.51}$$

where 'Q_b' is the bankfull, or channel topping discharge, 'R' is the hydraulic radius, 'P' is the wetted perimeter, 'A' is the cross section area, 'S' is the bed slope and 'n' is the Manning roughness coefficient of the channel boundary (or lining material). If n and S are constant for the design discharge, then the most economic section is one that maximises the value of $R^{0.67}$ and corresponds to minimisation of P with respect to depth, D. Assuming a side slope angle of θ (acute angle with the horizontal), expressions for cross-sectional geometry can be derived for this optimised condition:

$$A = \frac{D^2}{\sin\theta}\left(2 - \cos\theta\right) \tag{2.52}$$

$$P = \frac{2D}{\sin\theta}\left(2 - \cos\theta\right) \tag{2.53}$$

$$R = \frac{A}{P} = \frac{D}{2} \tag{2.54}$$

$$W = \frac{2D}{\sin\theta} = \frac{P}{\left(2 - \cos\theta\right)} \tag{2.55}$$

$$D_m = \frac{A}{W} = \frac{D}{2}\left(2 - \cos\theta\right) = R\left(2 - \cos\theta\right) \tag{2.56}$$

To solve for depth as a function of the design variables, Q, S and n, Equations 2.51 and 2.54 can be combined to derive the expression (independent of bank angle):

$$D = \frac{2\left(nQ\right)^{1\cdot 5}}{S^{0\cdot 75}} \tag{2.57}$$

This expression is the optimum depth and can be substituted into Equations 2.51 to 2.56 to derive the other optimum cross-sectional dimensions.

Lane (1937, p. 137-141) questioned the validity of the trapezoidal shape described above. He hypothesised that the velocity near the channel banks is sufficiently low, a result of boundary resistance forces, as to induce bar formation from suspended sediments, aided by vegetation growth. According to Lane, over time the interface between bank and bed becomes less defined as the boundary deforms toward a stable 'saucer shape' or elliptical cross section. This was also suggested by Lacey (1930, p. 272) who commented:

> 'That natural silt-transporting channels have a tendency to assume a semi-elliptical section is confirmed by an inspection of a large number of channels in final regime and an examination of cross sections of discharge sites of rivers in well-defined straight reaches of known stability'.

The coarser the silt, the flatter the semi-ellipse and the greater the bankfull width (Figure 2.1). This helps to explain why straight reaches in gravel-bed rivers with composite banks appear more trapezoidal than elliptical. According to Lacey (1930, p. 277), Kennedy's cross sections (1895) also had flat beds, because the bed and banks were not composed of the same type of sediment.

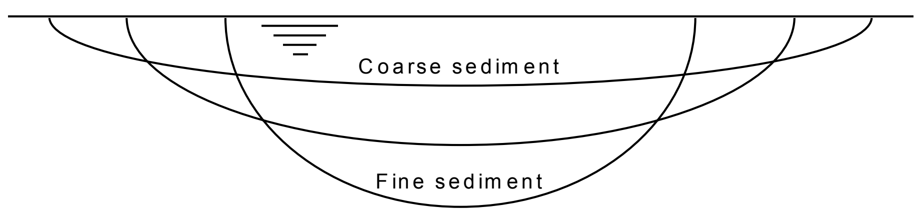


Figure 2.1 Hypothetical variation of cross-sectional shape with the size of bed material for constant discharge and wetted perimeter and homogeneous perimeter sediment (modified from Lacey, 1930, p. 273).

The theory suggests that a river assumes a major horizontal axis and a minor axis of an ellipse depending on the nature of transported sediment. According to Lane (1937, p. 140), the ratio of major to minor axes usually varies between 0.56 and 0.92 ($\pi/4$ for a perfect semi-ellipse) and the axes ratio is disproportional to the particle size of the bed material load. If there is a considerable proportion of fine material in the load, then the banks assume a steep shape tending towards to a trapezoidal section.

Lane (1937, p. 140) argued that a 'saucer shaped' cross section is stable because it keeps the thalweg in a central location and the majority of the channel at the threshold of impending sediment transport. Notably, the optimum hydraulic trapezoid described by Equations 5.22 to 5.27 (Lane 1937) is the shape that closely approximates an enclosed semi-circle with the origin at the water surface. A semi-circle has the least perimeter among all cross sections with the same area (Chow, 1959, p. 160). The best hydraulic sections for a semi-circular channel are given below (after Chow, 1959):

$$A = \frac{\pi D^2}{2} \tag{2.58}$$

$$P = \pi D \tag{2.59}$$

$$R = \frac{D}{2} \tag{2.60}$$

$$W = 2D \tag{2.61}$$

$$D_m = \frac{\pi D}{4} \tag{2.62}$$

where 'D' corresponds to the depth in the centre of the section and is given by Equation 2.57. Note that the optimum hydraulic radius for both the trapezoid and semi-circle are the same and as the side slopes approach 90 degrees, the bankfull widths approximate each other. Optimum hydraulic sections for other shapes of channel were given by Chow (1959, p. 161).

2.3.2.2 Tractive Force Theory

Lane (1937) considered the tractive force, or boundary shear stress, to be the main variable for determining the stable dimensions of channels with fixed beds. Tractive force theory is formulated on the basis that the stability of both bed and/or bank material is a function of the ability of the bed and bank to resist erosion due to the drag force exerted on them by the moving water (Simons and Albertson, 1960, p. 37). There are three types of limiting condition that can serve as criteria for channel design (Lane, 1937, p. 137):

i) Minimum width/depth ratio that will not scour the banks or result in bank failure. A complete understanding of the limiting condition of bank scour, particularly in meandering channels with complex flow patterns, requires considerable further research into the interaction of bank materials and form with fluid flow and entrainment processes.

ii) Minimum width/depth ratio that will produce the greatest mean velocity without initiating sediment motion. This criterion is appropriate for gravel-bed rivers which are sediment starved and have a critical boundary shear stress to mobilise bed sediment, significantly greater than zero. The cross section area is designed to maximise flood conveyance, while preventing particle entrainment.

iii) Maximum width/depth ratio that will yield the lowest mean velocity without leading to siltation. Non-silting velocity was the criterion adopted in the early equations of the regime engineers.

iv) Maximum slope without scouring the bed. This must be complemented by a wide and shallow cross section to lower the hydraulic radius and mean velocity to maintain flow continuity. However, there is a high potential for aggradation in the low flow marginal zones, which may increase flow depth in the centre of the channel and scour the bed via a negative feedback mechanism.

The tractive force method usually adopted for fixed-bed channel design concerns the second limiting condition and involves designing a stable cross section with boundary material at the threshold of sediment transport. This approach is often referenced as ‘threshold theory’. According to Leopold (1994, p. 5), as flow exerts an eroding force per unit area, or shear stress, on the bed and banks, the stable form of a channel is one in which the shear stress at every point on the channel perimeter is approximately balanced by the resisting stress of the bed or bank.

The criterion usually adopted for channel design based on tractive force principles is that of critical particle mobility defined by Shields (1936), which can be given in the simplified form of

$$\theta_c = \frac{R_c S}{(G_s - 1)d} \tag{2.63}$$

where ‘θ_c’ is the critical shields stress, ‘R_c’ is the critical hydraulic radius for incipient particle motion, ‘S’ is the energy gradient (equals water surface and bed slope for uniform flow conditions), ‘G_s’ is the sediment specific gravity and ‘d’ is a representative sediment particle size. Assuming θ_c equals 0.056 (for particle Reynolds numbers exceeding 400), G_s equals 2.65, sediment size can be represented by the median diameter, d_{50}, and for wide channels, R_c can be approximated by a critical depth, D_c, then

$$D_{\mathrm{c}} = \frac{0 \cdot 092\, d_{50}}{S} \qquad (2.64)$$

A critical velocity, V_c, can be derived by equating Equation 2.64 with the Manning-Strickler equation (Strickler, 1923) to give

$$V_{\mathrm{c}} = 16 \cdot 05\, R_{\mathrm{c}}^{\;0 \cdot 5} S^{0 \cdot 33} \qquad (2.65)$$

which is very similar to the Lacey equation

$$V_0 = 10 \cdot 85\, R^{2/3} S^{1/3} \qquad (2.9a)$$

The critical velocity can be given as a function of slope and sediment size to give the alternative expression

$$V_{\mathrm{c}} = 4 \cdot 903\, S^{-0 \cdot 167} d_{50}^{0 \cdot 5} \qquad (2.66)$$

This is the 'maximum permissible velocity' that will not cause erosion of the bed. Chow (1959, p. 165) noted that a deeper channel conveys water at a greater mean velocity without scouring the bed than a shallower one, assuming other conditions are constant. This is because bed erosion is caused by bottom velocities which are greater in a shallower channel for the same mean velocity. Based on irrigation channel research, Fortier and Scobey (1926) gave tables and charts expressing the maximum permissible velocity as a function of bed sediment type for straight channels with small slope. However, as a critical velocity is influenced by many other factors in alluvial channels, Chow (1959, p. 165) stressed that the 'method only serves as a guide and will not supplant experience and sound engineering judgement'. For sinuous channels Lane (1955) recommended reducing the maximum permissible velocity by as much as 22 percent for sinuous channels. Other research on permissible velocity includes work by Mavis and Laushey (1949), ASCE Task Committee (1967), Neill (1967) and Mehrota (1983).

For a trapezoidal channel, the threshold bankfull width, W_c, can be derived as a function of discharge, Q, by equating Equations 2.64, 2.66 and the continuity equation and expressed as

$$W_{\mathrm{c}} = \frac{2 \cdot 208\, Q\, S^{1 \cdot 167}}{d_{50}^{1 \cdot 5}} + \frac{d_{50}^{2} Z}{117 \cdot 127\, S^{2}} \qquad (2.67)$$

where 'Z' is the bank slope (1 vertical to Z horizontal). Assuming steep bank slopes of cohesive material, Z tends towards zero, giving

$$W_{\mathrm{c}} = 2 \cdot 208\, Q\, S^{1 \cdot 167} d_{50}^{-1 \cdot 5} \qquad (2.68)$$

Equation 2.68 is very similar to equations derived from a similar theoretical basis by Henderson (1963) and Griffiths (1981b), although Griffiths recommended estimating the coefficient (his stability index, related to the chosen value of θ_c) from a reference reach.

Equations 2.64, 2.65 and 2.67 show that a channel can be designed from the three equations of flow continuity, flow resistance and critical bed shear stress and assuming that the slope is known. In a straight channel the slope may approximate the valley slope for design purposes. To derive equations independent of slope, an alternative approach is required which uses a fourth equation for the critical shear stress for entrainment of bank sediment. Lane (1953, 1955) showed that this value may be derived by balancing forces on an individual grain and derived an expression for the ratio of critical bed shear stress to critical bank shear stress as a function of the angle of repose, ϕ, and the bank slope, θ. This equation was reduced to a depth expression and then integrated to give

$$Y = D_{\mathrm{c}} \cos\left(\frac{X \tan\theta}{D_{\mathrm{c}}} \right) \qquad (2.69)$$

where 'Y' and 'X' and the vertical and horizontal distances from the channel centreline in a curvilinear cross section with maximum depth, D_c, and the value in parentheses is expressed in radians. Equation 2.69 was derived by Glover and Florey (1951) for the United States Bureau of Reclamation for the purpose of developing a theoretically stable cross section for erodible channels, carrying clear water in non-cohesive sediments. Integration of Equation 2.69 yields expressions for stable cross-sectional area, A, and bankfull width, W, given by (Henderson, 1963, pp. 661-662; Graf, 1971, pp. 120-121)

$$A = \frac{2D_c^2}{\tan \phi} \tag{2.70}$$

$$W = \frac{\pi D_c}{\tan \phi} \tag{2.71}$$

Lane (1953, 1955) and Stevens and Simons (1971) presented charts expressing the friction angle, ϕ, as a function of sediment size and Li et al. (1976) demonstrated that ϕ exerts an important influence on channel shape with the width-to-depth ratio increasing for smaller and less angular grains. This explains why sand bed channels are often wider than gravel bed channels, other conditions being equal. If the product of mean cross-sectional velocity (Equation 2.66) and cross-sectional area (Equation 2.70) is greater than the design discharge, then a segment of constant depth must be removed from the centre of the section, and visa versa, to ensure continuity of flow (Chow, 1959; Graf, 1971).

Using a ϕ value of 35 degrees, suitable for sand bed channels, Henderson (1963, p. 663) showed that stable values of cross-sectional area (from Equation 2.70), bankfull width (from Equation 2.71), wetted perimeter and hydraulic radius for threshold channels can be estimated from the following simplified equations:

$$A = 2 \cdot 86\, D_c^2 \tag{2.72}$$

$$W = 4 \cdot 49\, D_c \tag{2.73}$$

$$P = 4 \cdot 99\, D_c \tag{2.74}$$

$$R = 0 \cdot 57\, D_c \tag{2.75}$$

By equating the continuity equation and Equations 2.64 and 2.66 a slope expression can be derived, such that

$$S = 0 \cdot 375\, d_{50}^{1 \cdot 15}\, Q^{-0 \cdot 46} \tag{2.76}$$

Combining Equations 2.64, 2.73 and 2.76 gives an expression for stable width as a function of sediment size and discharge for a ϕ value of 35 degrees, such that

$$W = 1 \cdot 105\, d_{50}^{-0 \cdot 15}\, Q^{0 \cdot 46} \tag{2.77}$$

Similarly, by combining Equations 2.64, 2.74 and 2.76:

$$P = 1 \cdot 2285\, d_{50}^{-0 \cdot 15}\, Q^{0 \cdot 46} \qquad (2.78)$$

This expression is very similar to the Lacey equation

$$P = 4 \cdot 84 Q_{\mathrm{b}}^{0 \cdot 5} \qquad (2.10a)$$

and similar theoretical equations have been derived by Henderson (1963), Li et al. (1976) and Hey (1978). A d_{50} value of 1 mm, which is suitable for the regime canals, would make Equations 2.10a and 2.78 equal. The above equations demonstrate that the rational theories can provide an explanation for the regime equations and provide a basis for understanding the processes of alluvial channel formation. However, as Henderson (1963) remarked, the similarity between regime and rational equations based on tractive force considerations is only of limited significance because regime equations generally assume a mobile bed, whereas threshold theory by definition assumes a static bed.

The threshold theory described above is only partly complete as it ignores lift forces and velocity deflection due to secondary circulation. Consequently, Li et al. (1976) and Lane et al. (1959) have developed elaborate expressions to account for these effects. Following the pioneering work of Lane (1953, 1955) significant developments in the field of threshold channel design were made by Kellerhalls (1967), Parker (1978a, 1978b, 1979) and more recently, Diplas and Vigilar (1992) and Julien and Wargadalem (1995).

Kellerhalls (1967) developed a width expression based on Canadian gravel-bed river data, canal data and laboratory data, all defined as in 'low transport equilibrium' in terms of discharge and grain size or exhibiting 'quasi-equilibrium beds' (Hey, 1988, 1997c), given by

$$W = 3 \cdot 26 Q_{\mathrm{D}}^{0 \cdot 5} \qquad (2.79)$$

where 'Q_{D}' is the dominant discharge, defined as the 'maximum sustained discharge' for canals and laboratory channels and an 'extreme flood of low frequency' with recurrence intervals between 3 and 5 years, for river data (Kellerhalls, 1967 p. 77). Analytical equations for depth and slope were based on the continuity equation, Equation 2.79, a flow resistance equation and an empirically derived threshold equation for bed material on the assumption that alluvial gravel-bed rivers with paved beds are in a state of incipient motion at Q_{D}:

$$D_{\mathrm{m}} = 0 \cdot 182\, Q_{\mathrm{D}}^{0 \cdot 4} d_{90}^{-0 \cdot 12} \tag{2.80}$$

$$S = 0 \cdot 086\, Q_{\mathrm{D}}^{-0 \cdot 4} d_{90}^{0 \cdot 92} \tag{2.81}$$

where 'D_{m}' is defined by A/W. According to Kellerhalls, the equations have a range of application for straight channels with Q_{D} between 0.03 m^3s^{-1} and 2000 m^3s^{-1}. No information was given on bank material, although Hey (1997c) suggested bank material probably consisted of gravel overlain by cohesive alluvium.

Threshold theory assumes stability of both bed and banks at any cross section. Based on the derivation of Equation 2.69, Parker (1978a, 1978b, 1979) showed that a mobile bed appears incompatible with stable non-cohesive banks. This can be demonstrated by considering a slight increase in flow depth in a threshold cross section with a flat central strip. At this condition, the threshold is exceeded on both bed and banks but the bank slope in Equation 2.69 suggests that bank material is transported towards the central strip leading to aggradation and widening before stabilisation (Ferguson, 1986). This paradox was resolved for sand-bed channels (Parker, 1978a) on the basis that the lateral diffusion of momentum resulting from to the transverse velocity gradient across the flat bed is accompanied by a backward diffusion of suspended sediment. Hence, suspended sediment preferentially settles at the channel margins and replaces any sediment transported from the banks to the channel centre, thereby maintaining sediment continuity and a stable cross section. A similar principle was used to resolve the paradox for gravel bed channels, whereby stability is maintained by a decrease in bed load transport to zero between the channel centreline and the interface between the flat bed and curved banks (Parker, 1978b).

Parker developed this theory of lateral redistribution of shear stress resulting turbulent momentum transport to derive dimensionless rational equations for straight rivers with gravel beds (1979), given by

$$W_* = 4 \cdot 4 Q_*^{0 \cdot 5} \tag{2.82}$$

$$D_* = 0 \cdot 253 Q_*^{0 \cdot 415} \tag{2.83}$$

$$S = 0 \cdot 223 Q_*^{-0 \cdot 410} \tag{2.84}$$

where 'W_*' is W/d_{50}, 'D_*' is D/d_{50} and 'Q_*' is $Q/[g(G_s\text{-}1)d_{50}^{2}]^{0.5}$. Equation 2.82 is an empirical equation used to make a determinate solution based on data from North American rivers (straight and sinuous) considered to be threshold channels and d_{50} greater than 16 mm. The discharge exponents in the Parker (1979) equations are very similar to the rational equations derived by Kellerhalls (1967). Parker's tractive force model has subsequently been revised to include sediment heterogeneity (Ikeda et al., 1988), bank vegetation (Ikeda and Izumi, 1990) and applied in the examination of the influence of suspended load on channel dimensions (Ikeda and Izumi, 1991). Further refinements were made by Diplas and Vigilar (1992) and Vigilar and Diplas (1994, 1997), whereby bank geometry is derived from a numerical model rather than being assumed. The resultant threshold channel shape is wider and deeper than the conventional cosine section, attributed to the role of momentum diffusion (ASCE Task Committee, 1998a).

Developing previous research (Julien, 1988), Julien and Wargadalem (1995) used an equation expressing the streamline deviation angle due to secondary circulation (after Rozovskii, 1961) as the rational equation in their semi-analytical derivation of stable width, depth and slope in threshold channels. The resultant equations are expressed as a function of dominant discharge, Q, particle size, d_{50}, Shields parameter, θ_c, and a flow resistance exponent, m, such that

$$W = 0 \cdot 512 Q^{(2m+1)/(3m+2)} d_{50}^{(-4m-1)/(6m+4)} \theta_c^{(-2m-1)/(6m+4)} \tag{2.85}$$

$$D_m = 0 \cdot 133 Q^{1/(3m+2)} d_{50}^{(6m-1)/(6m+4)} \theta_c^{-1/(6m+4)} \tag{2.86}$$

$$S = 12 \cdot 4 Q^{-1/(3m+2)} d_{50}^{5/(6m+4)} \theta_c^{(6m+5)/(6m+4)} \tag{2.87}$$

where 'm' is given by $1/\ln(12.2D_m/d_{50})$ and corresponds to a relative submergence exponent in the velocity expression proposed by Einstein and Chien (1954). The coefficients in Equations 2.85 to 2.87 are mean values derived from a calibration (and verification) involving 764 data sets and include several unknown parameters including the 'streamline deviation angle'. A further 115 data sets covering both sand-bed and gravel-bed channels were used to validate the equations and showed good agreement with field and laboratory conditions, with most observations within 50 percent to 200 percent of the calculations (Julien and Wargadalem, 1995, p. 320). While the equations attempted to incorporate secondary flows into the solution, the influence of the deviation angle is not explicit in the equations but accounted for indirectly by fixed coefficients. Furthermore, while the approach is based on threshold theory, the extensive data set used to derive the coefficients includes a significant number of mobile bed channels. Despite these limitations, the approach represents a significant step forward for design methods based on the governing processes of open channel flow.

When bed material transport is significant, threshold theory no longer applies and the critical shear stress equation for incipient particle motion must be replaced by a sediment transport equation to predict the stable slope given an input sediment load. However, predicting velocity, depth, width and slope, given the available equations of continuity, flow resistance and sediment transport yields an indeterminate solution, because there is one more parameter than the number of equations. Gilbert (1914) suggested that rivers with appreciable sediment loads adjust their cross section dimensions to transport the load as efficiently as possible. The approach indicated by Gilbert is that channels adjust either to convey the maximum possible bed load given the imposed grain size or to carry a specified load with the available discharge on the lowest possible slope (Ferguson, 1986, p. 18). This theory has led to the formulation of several 'variational' arguments, or 'extremal' hypotheses, which rely on the maximum or minimum of some parameter to be sought to make a determinate solution of velocity, depth, width and slope.

2.3.2.3 Extremal / Variational Hypotheses

According to Inglis (1947, p. 4), equilibrium is a condition in which the load is carried with the minimum expenditure of energy for the existing conditions. Extremal theories are based on this presupposition. There are six different types of extremal hypotheses that could be used for the purpose of channel design and have been summarised by Ferguson (1986), Hey (1988, 1997c), and others:

i) Minimum stream power (Chang, 1979a, b; 1980a, b). This hypothesis assumes that given the discharge and sediment load, the river adjusts its dimensions and velocity such that the stream power, γQS, is a minimum subject to given constraints, where 'γ' is the unit weight of water.

ii) Minimum unit stream power (Yang, 1976). The river adjusts its dimensions and velocity to minimise the unit stream power, or stream power per unit weight of water and unit cross-sectional area, VS, required to transport a given sediment and water discharge, where 'V' is the mean cross-sectional velocity.

iii) Minimum energy dissipation rate (Brebner and Wilson, 1967; Yang et al., 1981). The river at equilibrium adjusts to a stable channel configuration such that its rate of energy dissipation is a minimum.

iv) Maximum friction factor (Davies and Sutherland, 1983). This extremal hypothesis suggests that stable channel dimensions correspond to a local maximum of the friction factor.

v) Maximum sediment transport rate (Kirkby, 1977; White et al., 1981a, 1982). For a given discharge and slope, this hypothesis suggests that channel width adjusts to maximise the sediment transport rate.

vi) Minimum Froude number (Jia, 1990). Alluvial rivers will adjust their cross section form for given constraints to attain an equilibrium state which corresponds to the highest channel stability and is characterised by the minimum value of the Froude number.

Kirkby (1977) was the first to provide a quantitative solution, and in a review of the various approaches, Bettess and White (1987) demonstrated that the minimum stream power, minimum energy dissipation rate and maximum sediment transport rate were effectively the same argument. Significant developments of the extremal approach for the purpose of specifying stable channel dimensions have been made by Chang (1985a), White et al. (1981b) and Miller and Quick (1993).

Chang (1985b) developed a graphical solution of width, depth and slope for stable canal design. The method assumes a sand-bed channel in the lower flow regime with ripple bedforms and side slope in a trapezoidal section of 1.5 (1 vertical to 1.5 horizontal). Chang considered bed load to be primarily responsible for defining channel shape and used Lacey's resistance formula (Equation 2.16) and the Duboys-Straub bed load equation

(Du Boys, 1879; Straub, 1935). The equations have been rearranged here and converted to metric dimensions:

$$W = 7\cdot 55\left(S - S_{\mathrm{c}}\right)^{0\cdot 05} d_{50}^{-0\cdot 025} Q_{\mathrm{b}}^{0\cdot 5} \tag{2.88}$$

$$D = 0\cdot 049\left(S - S_{\mathrm{c}}\right)^{-0\cdot 3} d_{50}^{0\cdot 15} Q_{\mathrm{b}}^{0\cdot 3} \tag{2.89}$$

$$S = 0\cdot 359\, d_{50}^{0\cdot 5} Q_{\mathrm{s}}^{-0\cdot 789} Q_{\mathrm{b}}^{0\cdot 736} + S_{\mathrm{c}} \tag{2.90}$$

where the limiting slope at the threshold of particle motion, S_{c}, is given by

$$S_{\mathrm{c}} = 0\cdot 00039\, d_{50}^{0\cdot 5} Q_{\mathrm{b}}^{-0\cdot 51} \tag{2.91}$$

Chang (1980b, 1988) also presented equations to predict the width and depth in gravel-bed channels for imposed slope, discharge and sediment size, based on the Parker-Chang (Chang, 1980b) sediment transport equation and the minimum stream power hypothesis. The theoretical equations developed by Chang compared favourably with hydraulic geometry equations for stable channels with mobile gravel beds (Thorne et al., 1988) based on the data given by Hey and Thorne (1986), although width tended to be underestimated with increasing discharge, while depth was systematically overestimated and closer agreement required accounting for the density of bank vegetation.

The approach adopted by White et al. (1982) involves combining the White et al. (1980) flow resistance equation and the Ackers and White (1973) sediment transport function on the basis that an equilibrium cross section and slope corresponds to the condition of maximum sediment transport rate. This simultaneous solution is facilitated by a set of alluvial channel design tables (White et al., 1981b). The tables are applicable to channels covering discharges up to 1000 m^3s^{-1}, sediment concentrations between 10 ppm and 4000 ppm and sediment sizes from 0.06 mm to 100 mm.

One of the main problems in the approaches of Chang (1980b, 1988), White et al. (1980, 1982) and others is that the banks are assumed stable for slopes below S_{c}. If this is not the case, the channel is subject to widening and the approaches are invalid (Chitale, 1996). In light of this limitation, Miller and Quick (1993) devised a bank stability criterion that accounts for the increased stability of channel banks because of the consolidation of the bank sediment. The method adopted is based on the Lane (1955) bank stability analysis and an equation expressing the relative proportions of bed and bank shear stresses for trapezoidal channels proposed by Flintham and Carling (1988). Stable values of width, depth and slope are determined by maximising the rate of sediment transport estimated by the Einstein-Brown (Brown, 1950) bed load equation. Although the model includes both bank sediment size and angle of repose of bank sediment, which are not accounted for in the other methods, it is assumed that both the bed and banks are composed of the same sediment, which is an unrealistic assumption for many natural channels. Despite this limitation, the Chang method showed close agreement with various field data for gravel-bed rivers.

2.3.2.4 Analytical Regime Theory for Channels with Mobile Beds

If channel design is based on the governing equations of continuity, flow resistance and sediment transport without a fourth equation, there are an infinite number of theoretically stable combinations of width, depth and slope that satisfy the given constraints. For channels with mobile beds, a regime-type width equation can be used to make a deterministic solution. This technique, or 'analytical regime theory', represents a compromise between analytical and empirical approaches and was suggested by Chien (1956) to provide a graphical solution to the analytical method of combining flow resistance and sediment transport functions. The technique has also been used by Smith (1970) and Bakker et al. (1989) for designing straight alluvial canals but has not been widely applied for channel restoration design of rivers. The analytical method developed by Copeland (1994) for the SAM hydraulic design package (Thomas et al., 1996) is based on this approach and is discussed in detail in Chapter 6.

2.3.2.5 Limitations of the Analytical Approach

While the process-based methods of the analytical approach are based on the governing equations of open channel flow and have helped to explain the form of the earlier regime equations, there are several limitations, mainly associated with the assumptions that are required to make a determinate solution of width, depth and slope in a stable channel.

Some of the main limitations, with implications for their use in channel restoration design in meandering rivers with mobile beds, are summarised below:

i) The maximum hydraulic efficiency and threshold methods often represent a stable cross section by a regular curvilinear shape such as the cosine relationship in Equation 2.69. The simplicity in form is partly due to the general assumption of uniformity in sediment characteristics throughout a section. However, stable geometries in natural channels are highly variable and generally exhibit a marked interface between bed and bank, because the material in the bed and banks differ. Furthermore, Pickup (1976) suggests that the condition of maximum hydraulic efficiency, which warrants a semi-circular section, and maximum sediment transporting capacity (or similar extremal hypothesis), which requires a wide flat bed of a trapezoidal section, may be mutually exclusive, especially for bed load dominated channels.

ii) The maximum hydraulic efficiency or tractive force methods assume zero bed material transport and are, therefore not applicable to mobile bed channels. These methods are best suited for paved channels or cobble-bed channels in upland catchments with 'quasi-fixed beds'. Despite this limitation, several of the equations, such as the Kellerhalls (1967) width equation is very similar to equations developed from sites with mobile gravel beds (Chapter 5).

iii) The tractive force method is based on a predefined value of the Shields parameter. Li et al. (1976) suggested that gravel-bed and cobble-bed streams must be at the threshold condition at bankfull to be in equilibrium. However, research by Andrews (1984), Hey and Thorne (1986), and others, has shown that stable gravel-bed rivers can be found with appreciable bed load. These contradictory statements are the result of uncertainty in the value of the Shields parameter, which may be as low as 0.02 in gravel-bed rivers (Andrews, 1983). Griffiths (1984) indicated that the use of sediment transport equations assumes that all channels have the same Shields stress value at bankfull, whereas in fact they vary greatly.

iv) Using a simple bank stability criterion to make a determinate solution, such as the Lane (1955) and Miller and Quick (1993) methods, may yield unrealistic geometries because river width adjustments in natural channels are poorly understood. The approach by Lane (1955), for example, does not account for the cohesive properties of bank sediments. According to Hey (1997c), a rational method for predicting the dimensions of mobile bed alluvial channels requires equations to define bank erosion and deposition, which are not available for design purposes, despite the wealth of research on river width adjustments (ASCE Task Committee, 1998a, 1998b). Hey (1997c) suggested that further research is required on the geotechnical properties of bank material in association with local flow and pore water conditions before a completely rational and reliable approach may be developed.

v) Analysts have refuted regime theory on the grounds of empiricism, however the mobile bed approaches are based on sediment transport equations which are semi-empirical and, in some cases, can yield errors of a factor of ten when compared to measured values (Chang, 1988).

vi) The sediment transport and flow resistance equations used in both extremal methods and the analytical regime technique are not truly independent of each other. This arises because mean bed shear stress determines flow resistance and, as a function of particle grain size, contributes to the sediment transporting capacity of the channel (Chadwick and Morfett, (1993, p. 450).

vii) The approaches generally assume a straight channel alignment. In light of this restriction, the use of the equations for the design of meandering channels should be applied with caution.

The following points refer to the extremal methods only and have led Griffiths (1984) to describe the variational argument as 'an illusion of progress':

i) The various extremal hypotheses lack physical justification. The approaches seek to predict an optimum channel configuration without any process-based understanding of how and under what constraints width is adjusted. Field evidence suggests that stable sites may deviate from the optimum (extremal) condition. For example, Pickup and Warner (1976) in a study of rivers in Papua New Guinea found that the width of gravel-bed rivers significantly exceeded the optimum and the width of a sand-bed site was less than the optimum. The results of a case study discussed in Chapter 8 also suggest that stable channel dimensions may not correspond exactly to extremal criteria. Bettess and White (1987) suggested that channels with erodible banks will be wider than the optimum width and channels with resistant banks will be narrower. In both cases the slope must be greater than the optimum slope.

ii) In many cases there is a wide range of width values and width-to-depth ratios that are near to the optimum condition. With the minimum slope hypothesis, the authors have found that an optimum condition is best defined in channels with appreciable sediment loads. As sediment discharge decreases toward the threshold value, the flatter the turning point of the stable slope curve as it varies with width.

iii) Some combinations of channel conditions do not yield a maximum or minimum condition.

In summary, the maximum hydraulic efficiency method assumes a non-erodible bed, while the tractive force technique assumes zero bed material transport. Both methods are inapplicable for mobile bed channels. Extremal hypotheses have attempted to provide a deterministic solution of width, depth and slope for channels with appreciable bed material load, but the approach lacks a physical basis and field data often show marked differences from the optimum geometries. Adopting a semi-analytical approach, by including an empirical regime-type relationship suitable for mobile bed rivers instead of an extremal hypothesis, may provide a more realistic solution to the design problem.

2.4 THE *GEOMORPHOLOGIST* AS A CHANNEL RESTORATION DESIGNER

The importance of accounting for channel morphology and the dynamic nature of the fluvial system when dealing with alluvial rivers is now recognised by river engineers and managers as an integral element of any management plan. The geomorphic approach is to integrate, and, in many cases, replace the 'structural', deterministic emphasis of the engineer with a desire to work with nature, rather than against it, and use more sympathetic channel modifications which require an understanding of natural process-form relationships at different spatial scales. This is best accomplished by setting aside a river corridor and allowing the river to evolve its own variable morphology within a self-formed meander belt. In practice, this ideal approach is usually seldom feasible because of floodplain constraints, and an alternative must be sought whereby the engineered channel retains as many natural channel attributes as possible. This may require, for example, a two-stage channel with an outer channel designed according to flood defence requirements and an inner channel resembling the stable channel dimensions of a natural river with the same constraints.

The geomorphologist recognises that restoration to pre-disturbance conditions is not usually attainable since catchment land-use developments impose a different set of controlling variables than may have existed prior to disturbance (Kondolf and Downs, 1996). The river catchment is an evolving system, dynamically changing its state of equilibrium/disequilibrium over the long-term scale, as hydrology, land-use patterns and river developments change, which in turn modify the natural processes and physical attributes of water, sediment and nutrient fluxes. The geomorphic approach to channel restoration design strives for equilibrium within the catchment system, therefore design procedures must accommodate catchment context dynamics, rather than some pre-disturbance condition which may have existed hundreds or even thousands of years ago. Indeed, the term 'restoration', is being gradually replaced by more geomorphologically-acceptable terms such as rehabilitation, enhancement or simply river management. The term is used in this report because it refers to the functional restoration of *natural variability* and *dynamic stability* rather than some prior *state*. The design stage of river management projects is still largely an engineering speciality but geomorphologists are becoming increasingly involved, particularly concerning mitigation of the effects of channelisation and river enhancement using instream flow-deflectors which requires a geomorphological knowledge of how the structure is likely to react with flow and sediment patterns over time and space. Unfortunately, as Brookes and Sear (1996, p. 94) remarked, 'to date, there have been few projects which have systematically followed the geomorphological guiding principles for restoring channels'.

This section discusses the utility value of regime equations for river management and reviews the geomorphic approach of downstream hydraulic geometry analysis as the geomorphologist's counterpart to regime theory. This is followed by a summary of two techniques developed specifically for river restoration by geomorphologists: i) historical reconstruction, and; ii) using a reference reach (or reaches) as a natural channel analogue.

2.4.1 Downstream Hydraulic Geometry: The *Channel Forming* Design Solution

2.4.1.1 Rivers vs Canals: Regime Equation Applicability

The design of canals on the Indo-Gangetic plain during the first half of the century required quantitative solution of only three degrees of freedom, width, depth and slope, as the irrigation network consisted of straight, trapezoidal-shaped channels. The regime channels exhibited only a small range of particle sizes, with less than 1 percent of sediment discharge comprised of suspended sediment. The side slopes were generally homogeneous and composed mainly of clay with width-to-depth ratios generally less than 30. Without the complexity of natural river morphology, the simplicity of the canal systems simplified the engineering design problem. The regime-type equations of Lacey (1930, 1933), and others, are applicable to 'in-regime' canals with the following channel conditions (modified from Blench (1952, p. 389-391; 1969, p. 52)):

i) Steady uniform discharge between 0.15 m^3s^{-1} and 285 m^3s^{-1}, with most sites between 0.55 m^3s^{-1} and 14 m^3s^{-1}. Standard practice was to operate the canals at or near full-supply discharge.

ii) Rippled or duned sand bed with bed material size between 0.1 mm and 0.6 mm (mean value approximated 0.25 mm).

iii) Suspended sediment less than 1 percent by weight of flow. In general, this was insufficient to significantly affect the form of the regime equations.

iv) Steep cohesive sides that behave as hydraulically smooth.

v) Straight planform.

vi) Uniform cross section and slope.

vii) Width-to-depth ratios between 5 and 30.

Therefore, regime canals are a very specific breed of channel and differ considerably from the complex and variable morphology of natural, meandering rivers discussed in Section 2.2.1. The practical difficulties of applying regime equations to rivers were outlined by Blench (1969) and include: i) the different behaviour of gravel-bed rivers to the duned sand-bed canals; ii) the range of flow and sediment discharge in a river may vary significantly on a day-to-day basis; iii), suspended load may be large enough as to be a significant channel forming variable, and; iv) the form of a cross section in a meandering river is highly variable and differs from the simple geometric sections found in the regime canals. However, Blench (1969) does not dismiss the use regime canal equations for rivers, specifying that they can be applied with caution because the majority of rivers of major engineering interest are either sand bed with minor bed load discharge or gravel rivers with a large D/d_m ratio (when the effect of grain size is drowned out):

> 'As the regime formulas have proved the existence of definite laws relating to the self-formation of regime-type channels, they have an application to rivers of regime type' (Blench, 1952, p. 396).

Moreover,

> 'Rivers, including models may be treated algebraically as fluctuating canals of moderate bed-load charge…that have been neglected long enough to meander' Blench (1969, p. 77).

Field studies have also revealed some similarities between river and canal morphology. For example, graphical plots of stable width, depth and slope developed by Neill (1973) for straight reaches of sand-bed rivers in Alberta showed close agreement between river form and estimated regime dimensions from Blench's (1970) equations. Furthermore, regime equations have been applied for river restoration. For example, the 'geomorphic restoration design' of a reach of the Napa River, California (Neary et al., 1998) was based on the Simons and Albertson (1960) equations for regime canals (Neary, 1998, pers. comm.).

In summary, predicting the stable form of meandering rivers presents more challenges than designing regime canals, despite the limited evidence that suggests similarities between the form of alluvial rivers and canals.

2.4.1.2 Development of Cross-Sectional Hydraulic Geometry

The term 'hydraulic geometry' was coined by Leopold and Madock (1953) to provide a quantitative description of how channel width, depth and mean velocity (and in subsequent studies, water surface slope) vary with changing discharge. Later studies of hydraulic geometry provided morphological relationships for the prediction of other physical attributes of a river such as meander planform geometry. Hydraulic geometry is a natural progression from regime theory as hydraulic engineers shifted emphasis from canal excavation problems to river management and river design applications. Despite the demise of regime canal studies, the term 'regime' has maintained its popularity among engineers and is often used instead of hydraulic geometry for fluvial studies (e.g. Ackers, 1972). The essential difference between regime and hydraulic geometry was given by Thorne et al., 1998, pp. 7-8):

> 'A self-formed alluvial channel is in regime if there are no net changes in discharge capacity or morphology over a period of years'.

Whereas, hydraulic geometry

> '…is similar to regime theory, but differs in the way that the dominant discharge is expressed. With respect to the hydraulic geometry of an alluvial river, the dominant discharge is the single flow event which is representative of the natural sequence of events which actually occur. Regime theory was developed for canals, which do not experience a range of flows. Hence, the dominant discharge for regime theory is the steady, operating discharge'.

Significant reviews of hydraulic geometry were given by Richards (1982), Knighton (1984, 1998), Ferguson (1986), Chang (1988) and Clifford (1996), who discussed the importance of Leopold and Maddock's original paper.

The fundamental assumption of hydraulic geometry theory is that a drainage area supplies a specific distribution of flow to the channel and this sequence of natural flow events moulds a specific channel geometry and shape over time, relative to boundary conditions and constraints. Therefore, cross-sectional form is inherited from the imposed natural sequence of flows and boundary sediments. As the discharge usually explains most of the variance in geometry, the relationship between discharge and dimension is usually bivariate. The concept of the dominant discharge or 'channel-forming' flow is discussed further in Chapter 4.

As Clifford (1996, p. 82) noted, Leopold and Maddock's research was essentially an investigation into water utilisation problems in south-west United States. Their work was undertaken at a pivotal period in geomorphology as the framework of qualitative investigation and deductive reasoning was being superseded by more quantitative approaches in the 1960s, whereby the identification of channel grade and equilibrium phenomena demanded more inductive methods of research to recognise the occurrence of 'stable channel geometry'. In the 1950s and 1960s hydraulic geometry was seen as the functional answer to river form regardless of region or setting (Ferguson, 1986, p. 1). To Leopold and Maddock (1953, p. 18):

> 'The channel characteristics of natural rivers are seen to constitute, then, an independent system which can be described by a series of graphs having simple geometric form. The geometric form of the graphs described these interactions suggests the term 'hydraulic geometry'.

For cross sections on 20 perennial rivers in the Great Plains and southwest United States, Leopold and Maddock developed the two complementary concepts of 'at-a-station hydraulic geometry', which describes the variation in form for a single section as discharge varies, and 'downstream hydraulic geometry', which describes how cross-sectional geometry changes in the downstream direction or between different river systems for a specific discharge frequency. For the design of simple cross- sectional dimensions, notably, width, depth and slope, it is the downstream concept which is important and will be concentrated on from this point forward.

Leopold and Maddock correlated width, mean depth and velocity with mean annual discharge (the time-averaged daily flow), Q_{m}, to produce the following relationships:

$$W = \mathrm{a}\, Q_{\mathrm{m}}^{0\cdot 5} \tag{2.92}$$

$$D_{\mathrm{m}} = \mathrm{c}\, Q_{\mathrm{m}}^{0\cdot 4} \tag{2.93}$$

$$V = \mathrm{k}\, Q_{\mathrm{m}}^{0\cdot 1} \tag{2.94}$$

Equations 2.92 to 2.94 were fitted by eye and Carlston (1969) later amended the exponents to 0.46, 0.38 and 0.16, respectively, using regression models. The exponents are very similar to those of the earlier regime equations for near-constant discharges, such as Lacey's equations. In particular, the width relationship conformed to the square-root law of both the regime and process-based equations. This has encouraged subsequent researchers to fix the exponent in the width-discharge relationship at 0.5 for practical design purposes (e.g. Hey and Thorne, 1986). However, the square root law is only an average condition. Variations of the downstream hydraulic geometry exponents for width, depth and slope were summarised by Park (1977) and Rhodes (1987) as falling most frequently in the range 0.4 to 0.5, 0.3 to 0.4 and 0.1 to 0.2 respectively, while Ming (1983) implied that these ranges may be as wide as 0.39 to 0.6, 0.29, to 0.4 and 0.09 to 0.28, respectively.

Hydraulic geometry is intrinsically linked to the mass continuity equation for uniform flow, $Q = WD_mV$, which suggests that the sum of exponents and the product of the coefficients should both equal unity. Following the work of Leopold and Maddock (1953), Leopold et al. (1964), and others, have demonstrated that downstream hydraulic geometry relationships, notably the coefficients, differ according to physiographic region and type of channel conditions.

During early research into hydraulic geometry there was no general consensus regarding the discharge frequency that should be used as the independent variable, the choice often dictated by data availability as was the case for Leopold and Maddock (1953). Their approach involved non-causal relationships, whereby channel form was related to the statistical mean annual discharge which may have little or no morphological significance and occurs considerably more frequent than the bankfull, or channel-forming, discharge. This problem was resolved in subsequent studies with equations that expressed bankfull width, W, mean depth, D_m, and water surface slope, S, as a function of bankfull discharge, Q_b. Some of the earliest hydraulic geometry equations of this type were derived by Nixon (1959) for U.K. rivers, whereby the bankfull discharge was equated with the discharge exceeded 0.6 percent of the time (2 to 3 days a year on average):

$$\text{metric: } W = 2\cdot 99\, Q_b^{0\cdot 5} \quad \text{imperial: } W = 1\cdot 65\, Q_b^{0\cdot 5} \tag{2.95}$$

$$\text{metric and imperial: } D_m = 0\cdot 55\, Q_b^{1/3} \tag{2.96}$$

$$\text{metric: } A = 1\cdot 63\, Q_b^{5/6} \quad \text{imperial: } A = 0\cdot 9\, Q_b^{5/6} \tag{2.97}$$

Because of the considerable variability of slope, Nixon suggested calculating slope from the Manning flow resistance formula. Nash (1959) revised Nixon’s equations in light of additional data from U.S. rivers and demonstrated that their application for prediction and design could lead to 33 percent error on average.

Since the 1950s there has been a plethora of downstream hydraulic geometry equations for bankfull width, mean depth and slope documented in the geomorphology and engineering literature, the majority of research relating to gravel-bed rivers in England and Wales and North America. Significant contributions have been made by Brush (1961), Leopold et al. (1964), Emmett (1972, 1975), Charlton et al. (1978), Bray (1982), Parker (1982), Hey (1982), Andrews (1984), Hey and Thorne (1986), and many others. The equations are not presented here due to space restrictions but the use of hydraulic geometry to determine width for channel restoration design is developed further in Chapter 5. Furthermore, to facilitate the design of flood control channels, Neill (1982) and Hey and Heritage (1993) have compiled a series of graphical design charts based entirely on regime and hydraulic geometry equations which could collectively be applied to a wide range of channel types.

As a surrogate for discharge, Leopold et al. (1964), Emmett (1975), Dunne and Leopold (1978), Leopold (1994) and Rosgen (1996) used the upstream drainage area to predict downstream hydraulic geometry. While these relationships often show remarkable consistency, the variability in channel dimensions is significantly greater than when discharge is used as the independent variable. While these relationships may be used to provide rough estimates of channel dimensions at ungauged sites, they should be applied with caution if used to design stable channels.

Only recently have multiple regression analyses attempted, with varied success, to account for the variance related to more complex multivariate controls and relationships that is unexplained by discharge in bivariate hydraulic geometry (Richards, 1982). Using multiple regression models applied to 72 gravel-bed streams in New Zealand, Mosely (1981) demonstrated that the variability of cross sectional area is best explained by a combination of mean annual flow ($Q_{2.33}$), mean bed material size and the percentage of silt-clay material in the banks. The width-to-depth ratio was a more elaborate expression and included stream power, a flow variability index, mean bed material size, standard deviation of bed material size, and the percentage of silt-clay material in the banks as independent variables. Independent variables used in the hydraulic geometry analysis by Hey and Thorne (1986) included bankfull sediment discharge, defined by the Parker et al. (1982), bed load equation, bed material particle sizes for which 50 percent, d_{50}, and 84 percent, d_{84}, of the sediment is finer and a semi-quantitative estimate of riparian vegetation density (see Chapter 5). More recently, based on a quantitative analysis of the influence of boundary shear stress on channel shape, Huang and Warner (1995) and Huang and

Nanson (1998) showed that a possible combination of controls on stable width and depth are slope, average bed roughness and the perimeter sediment composition.

Over the past 2 decades a number of sophisticated multivariate statistical models have been developed, such as simultaneous-equation models, continuously varying parameter models and distributed lag models. Rhoads (1991, 1992) gave a detailed review of these methods, which may be used to investigate mutual adjustment mechanisms and non-linear dynamic behaviour of rivers (Rhoads, 1992, pp. 452-3). However, these methods have not been applied widely and they require further testing to examine whether they can usefully be applied as tools for river management and channel restoration design.

Alluvial rivers with erodible boundaries tend to meander and migrate, providing stream power and sediment load are not sufficiently large as to initiate braiding. According to Leopold and Wolman (1957, p. 53), straight rivers are so rare among natural rivers as to be almost non-existent and, as a general rule, straight reaches rarely exceed ten times the channel width. Even in a straight reach, the path of the thalweg tends to adopt a sinuous alignment between alternate bar formations. General morphological relationships to predict the average size of meander attributes have been well documented since the turn of the Twentieth Century (following Jefferson, 1902) and provide useful empirical tools for restoring the planform dimensions of stable meandering rivers.

2.4.1.3 Development of Meander Hydraulic Geometry

While irregular meander planform attributes are found in nature, in general natural rivers exhibit a strong relationship between average size of channel, expressed in terms of either width or discharge, and the average magnitude of the meander loops. Almost a century ago, data compiled by Jefferson (1902) revealed linear relationships for meander belt width, A_{m} (or meander amplitude; see Figure 3.7 in Chapter 3 of this report), and meander wavelength, L_{m} (measured along the axis of the channel; see Figure 3.7), as functions of channel width. Following a re-examination of this data set by Carlston (1965), these expressions were given as

$$A_{\mathrm{m}} = 17 \cdot 6W \tag{2.98}$$

$$L_{\mathrm{m}} = 12 \cdot 6W \tag{2.99}$$

According to Jefferson (1902, pp. 377-378), just as a string can double back on itself with greater ease than a rope, the wider the river, with greater threads of current, the more difficult to make sharp meander bends. Based on a comprehensive set of data from North American rivers, meanders on glaciers and meanders in the Gulf Stream, Leopold and Wolman (1957, 1960) showed that the relationship between wavelength and width is not only independent of scale but also independent of bed and bank materials. This is the case because supraglacial streams and the Gulf stream do not carry sediment but are influenced primarily by the hydrodynamic nature of the flow (Leopold, 1994, p. 62). In their 1960 paper, Leopold and Wolman derived the following morphological equations which were the most consistent in their analysis over several orders of scale of flow:

$$\text{metric: } L_{\mathrm{m}} = 11 \cdot 0 W^{1 \cdot 01} \quad \text{imperial: } L_{\mathrm{m}} = 10 \cdot 9 W^{1 \cdot 01} \tag{2.100}$$

$$\text{metric: } A_{\mathrm{m}} = 3 \cdot 0 W^{1 \cdot 1} \quad \text{imperial: } A_{\mathrm{m}} = 2 \cdot 7 W^{1 \cdot 1} \tag{2.101}$$

$$\text{metric: } L_{\mathrm{m}} = 4 \cdot 6 \, R_{\mathrm{c}}^{0 \cdot 98} \quad \text{imperial: } L_{\mathrm{m}} = 4 \cdot 7 \, R_{\mathrm{c}}^{0 \cdot 98} \tag{2.102}$$

where ‘R_c’ is the radius of bend curvature. The geometric similarity between meander planforms have also been found on glacial ice (Dozier, 1976; Zeller, 1967) and in rill formations on Karst landforms (Zeller, 1967). Leopold and Wolman (1960, p. 772) considered the exponents in their relationships to approximate unity. Therefore, by combining Equations 2.100 and 2.102, it was shown that the radius of curvature to width ratio was 2.3. According to Bagnold (1960), values of this ratio between 2 and 3 are associated with least energy losses from boundary friction and bank erosion (Bagnold, 1960). Further discussion on the value of this ratio for channel restoration design is given in Chapter 7. Dury (1976) added further observations to the Leopold and Wolman data sets from U.S. streams and revised the wavelength relationship to

$$L_{\mathrm{m}} = 9 \cdot 76 W^{1 \cdot 02} \tag{2.103}$$

Re-analysis of the Leopold and Wolman data confirmed that a linear function, rather than a power function, fitted through the observed wavelength-width observations is acceptable. This was defined by Dury (1976) and Richards (1982), respectively, as

$$L_{\mathrm{m}} = 11W \tag{2.104}$$

$$L_{\mathrm{m}} = 12 \cdot 34W \tag{2.105}$$

Based on a data set comprising 194 sites, Williams (1986) found that cross-sectional depth and area can also be used as a scale indicator as well as the traditional channel width and derived 40 relationships between various meander attributes and dimensions of cross-section with most correlation coefficients in the range 0.95 to 0.99. However, in terms of specifying the planform geometry of a stable channel, the majority of these equations are redundant as only sinuosity and wavelength are required to specify a regular meander pattern, such as the circular curve or the sine-generated path (see Chapter 7).

In the general case, the wavelength is rarely outside the range of 10 to 14 channel widths (Leopold et al., 1964; Leopold, 1994), radius of curvature is approximately 20 percent of the wavelength (Leopold, 1994) and rarely exceeds 1.5 to 4 times the channel width (FISRWG, 1998) and belt width is usually 0.5 to 1.5 times the wavelength (FISRWG, 1998).

According to Wolman and Miller (1960, p. 66), flows responsible for shaping the path of a meander must follow the path of the waveform. As flows above the bank do not follow the waveform but assume a much straighter path, the correlation between width and wavelength is evidence that both the width of the river and its pattern must be related to a discharge close to the bankfull stage. As the exponent in the wavelength-width expression approximates unity and that in the width-discharge relationship approximates 0.5 on average, it follows that the exponent in a wavelength-discharge expression should also approximate 0.5 on average (Inglis, 1949b). A summary of existing wavelength-discharge power equations is given in Table 2.4 (converted to metric units were necessary).

Reference	Data Source	Coefficient	Exponents				
			Q_m	Q_{mm}	Q_b	Q_d	Q_{max}
Jefferson (1902) (after Carlston, 1965)	U.S.A.?	83.3	0.5				
		97.8	0.5				
Inglis (1947)	India	49.6					$0.5^{\perp}$
Inglis (1949b)	Laboratory$^{\perp}$	65.2				0.5	
	Theoretical$^{\perp}$	48.9					0.5
Dury (1965)	U.S.A.	58.8			0.47^{*}		
		48.5			0.5^{*}		
		54.3			0.5^{*}		
Carlston (1965)	U.S.A.	166.6	0.46				
		125.7		0.46			
		22.8			0.62^{**}		
Ackers and Charlton (1970a)	Laboratory	61.2			0.47		
Ferguson (1975)	U.K.	57				$0.58^{\perp\perp}$	
		36				$0.63^{\perp\perp}$	
Dury (1976)	U.S.A.	35.7			0.55^{***}		
		32.9			0.55^{***}		

Note: Q_m = mean annual (time-average) discharge (m^3s^{-1}); Q_{mm} = mean of month of maximum discharge (m^3s^{-1}); Q_b = bankfull discharge (m^3s^{-1}); Q_d = 'dominant' discharge (m^3s^{-1}); Q_{max} = 'maximum' discharge (m^3s^{-1}); * derived from drainage area relationships; ** 1.5-year recurrence interval flood, $Q_{1.5}$; *** most probable flood (1.58-year recurrence interval), $Q_{1.58}$; $^{\perp}$ assumed by Carlston (1965, p. 870); $^{\perp\perp}$ dominant discharge of one percent duration and wavelengths estimated from direction-change spectra and autocorrelograms rather than direct measurement.

Table 2.4 Existing meander wavelength-discharge equations.

In Table 2.4, the 'dominant' discharge as defined by Inglis (1941, p. 112) is 'slightly in excess of bankfull discharge' and about 60 percent of the 'maximum' discharge (1947, p. 13), although the notion of there being a maximum discharge is rather misleading as it is relative to the period of flow record.

Scattergrams of wavelength against discharge observations often exhibit considerably more variability about the best-fit line than graphs of wavelength against width. Analysis of such plots led Leopold and Wolman (1957, p. 59) to postulate that in terms of the mechanical principles controlling meander initiation and development, the wavelength is directly dependent on width and only indirectly dependent on discharge. Furthermore, Carlston's analyses indicated that there might be a range of effective flows between Q_m and Q_{mm} that is responsible for controlling wavelength. According to Carlston (1965, p. 880), these flows expend the greatest work in sediment transportation and in the processes of point bar deposition and outer bank erosion that shape the form and dimensions of unconstrained meanders in their general downstream migration. In light of these considerations, meander wavelength-width relationships are much more popular in applied fluvial geomorphology (Thorne, 1997, p. 191).

According to Ackers and Charlton (1970a) the variability of wavelength as a function of discharge in their sand-bed experiments was attributed to the differing mobilities of bed and bank sediments. To account for the influence of boundary sediments on the variation in meander wavelength, Schumm (1968) derived multiple regression equations for a sample of predominantly sand-bed channels. The expression that includes bankfull discharge, Q_b, is given by

$$L_m = 618 Q_b^{0 \cdot 43} M^{-0 \cdot 74} \tag{2.106}$$

where 'M' is a weighted silt-clay index and increases with the proportion of silt and clay found in the channel perimeter. Therefore, for a given bankfull discharge, meander wavelength increases as the composition of the channel boundary becomes less cohesive. According to Schumm (1968), M reflects the type of sediment load such that larger wavelengths are associated with friable, easily eroded banks and high proportions of bed load transport in streams.

2.4.1.4 Limitations of Hydraulic Geometry

Many of the limitations of regime theory (see Section 2.3.1.4) are applicable to the hydraulic geometry approach. In the context of river restoration, Burns (1998) discussed the appropriate use of hydraulic geometry relationships and their main drawbacks in channel design, which are summarised below.

In most cross-sectional hydraulic geometry equations, sediment transport is not explicitly considered and the relationships are generally applicable only to channels with low bed material loads. However, there are exceptions, for example Hey and Thorne (1986) showed that sediment transport was a significant parameter in depth and slope hydraulic geometry in mobile gravel-bed rivers. If sediment inputs are not accounted for, it is highly likely that a restored channel would be unstable, especially if located in the piedmont zone.

Hydraulic geometry is a black box simplification of the fluvial system and represents real phenomena in only one dimension. In empirical equations expressing width, depth, slope or a meander attribute as functions of discharge, the constants incorporate the combined influence of a variety of complex interrelating factors, including bed and bank material characteristics, riparian vegetation, geomorphic history of the catchment (Andrews, 1983, p. 371). These influences are interrelated and are difficult to identify and measure independently. On this basis, Park (1977) suggested that single values for hydraulic geometry exponents are misleading and such uniformity between physiographic regions does not exist. While multicausal models (e.g. Rhoads 1991) provide an alternative to the traditional form of hydraulic geometry equations, they have not been widely applied and are more data intensive. Furthermore, Leopold (1994, p. 178) considered hydraulic geometry to be significantly influenced by chance, such that physical laws do not dictate one and only one combination of the dependent variables. As there are fewer equations available to resolve all degrees of freedom, channels can adjust to the imposed influences in a variety of ways, which naturally leads to significant variance between observations. As hydraulic geometry is an empirical approach, the equations are appropriate for rivers with the same range of conditions as those used to derive the relationships. As with regime equations, application beyond the parent range of conditions should be treated with extreme caution. For example, Rinaldi and Johnson (1997a, b) compared the Leopold and Wolman (1960) meander equations with new equations derived from a sample of small Maryland streams and showed that the Leopold and Wolman relationships significantly overestimated measured values. In particular, the measured range of meander wavelengths were reported to be between 2.9 and 7.7 times the channel width, significantly lower than the 10 to 14 widths suggested by Leopold et al. (1964). The differences were attributed to differences in morphological channel types, control of vegetation on planform and sinuosity and channel adjustments because of intense land-use changes. In particular, urbanisation during the past few decades has resulted in significant channel widening in many of Maryland’s piedmont streams. The local wavelength-width relationship yields a significantly wider channel width for a given wavelength, when compared with the

Leopold and Wolman (1960) relationship, indicating that widening has occurred but the sizes of the meander loops have not adjusted accordingly, resulting in tighter meander bends than those of stable channels.

While hydraulic geometry recognises that discharge is the dominant channel forming variable, channel dimensions are related only to a single flow event, usually bankfull discharge, and other potentially formative flows are not accounted for. Issues relating to suitable channel forming discharge(s) for channel restoration design are discussed in more detail in Chapters 4 and 5. Similarly, bed material has influenced both depth and slope equations, yet is usually represented by only one or two particle sizes. This presents a dilemma for streams with mixed or bimodal bed material size gradations as to which sizes are shaping the channel.

For the same boundary conditions, hydraulic geometry assumes that the same process-form relationships operate throughout the fluvial system. However, Harvey (1969) showed that upstream sections adjust to more frequent flows than downstream sections and therefore, magnitude-frequency properties in the downstream direction are not fixed in nature. The influence of flow variability is examined in the context of the effective discharge in Chapter 5. Furthermore, downstream hydraulic geometry suggests a continuous relationship between channel dimensions and discharge. However, in reality tributary inputs to the fluvial system define discontinuous, or stepped, relationships that cannot be quantified easily. Also, Kellerhalls and Church (1989) used a large data set of alluvial river data to show that the rate of change of width in the downstream direction is less in smaller streams (discharge exponent of 0.4) than in larger streams (discharge exponent of 0.55).

In summary, while regime theory presented a useful tool for designing channels with simple geometric forms, regime-type equations are more problematic in their application to river problems and must be treated with caution. Variability in width, depth and slope (and planform) are significantly greater in natural rivers than in canals and subsequently, greater uncertainty is tied up in the coefficients and exponents of the equations making them less transferable to other regions or settings. This uncertainty is a result of local factors and is associated with a range of stable geometries found in nature.

2.4.2 Regular Meander Path Models

The visual similarity in curvilinear form of meander geometry in different physiographic settings and throughout a range of scales from flume studies to the Mississippi River is attributable to a strong wavelength-width relationship that is not directly a function of sediment inputs but ‘related in some manner to a more general mechanical principal’ (Leopold and Wolman, 1960, p. 774). This presupposes that meandering is predictable and sufficiently regular for a single characteristic wavelength to be identified (Ferguson, 1979, p. 229). Wavelength is a scale parameter and together with a shape parameter, such as sinuosity, a regular meander path model can be defined and used to layout a general planform configuration. Regular meander path models assume that the direction of curvature alternates between successive bends along a fixed meander belt axis and bend radius is uniform. The simplest model regards meanders as circular arcs linked end to end and has been used in many discussions of meander shape and size (e.g. Chitale, 1970; Hey, 1976). However, natural meanders rarely exhibit such uniform curvature but have tightest curvature at the apex that progressively straightens out towards the intervening inflexions (Ferguson, 1979, p. 230). Ferguson (1973a) examined several regular meander path models documented in the literature and demonstrated that properties of natural meander bends show fair agreement with those models which portray this type of variable curvature, including the sine-generated curve proposed by Langbein and Leopold (1966).

The sine-generated curve assumes that meandering is the outcome of deviations from a straight course in response to the superposition of many diverse physical causes, individually deterministic, but random in their aggregate effect (Ferguson, 1979, p. 232). As this aggregate effect is poorly understood on the basis of process-based explanations, Langbein and Leopold (1966) adopted a stochastic approach by postulating that a meander form will occupy the most probable path, defined as a ‘random-walk’ whose most frequent (and stable) form minimises the sum of the squares of the changes in direction, ϕ, in each unit length (Langbein and Leopold, 1966, p. 1). This minimum variance (or maximum entropy) hypothesis defines the channel direction with the meander belt axis, ϕ, at distance, s, along a channel centreline as

$$\phi = \omega \sin\left(\frac{2\pi s}{L_{\mathrm{m}} P}\right) \tag{2.107}$$

where 'ω' is the maximum angle a meander loop takes relative to the meander belt axis, 'L_m' is meander wavelength and 'P' is sinuosity. Julien (1985) has derived a similar meander path equation from Fourier analysis. Figure 2.2 portrays sine-generated curves for various maximum path angles, ω, at a fixed arc length. Notably, in a sine-generated curve, it is the channel direction, rather than the planform itself, which adjusts as a sinusoidal function of channel length. Furthermore, as the complex controlling variables which determine meander pattern are treated as stochastic effects, the model should be compared to individual bendways and treated as an average, or ideal, condition when addressed on a reach scale.

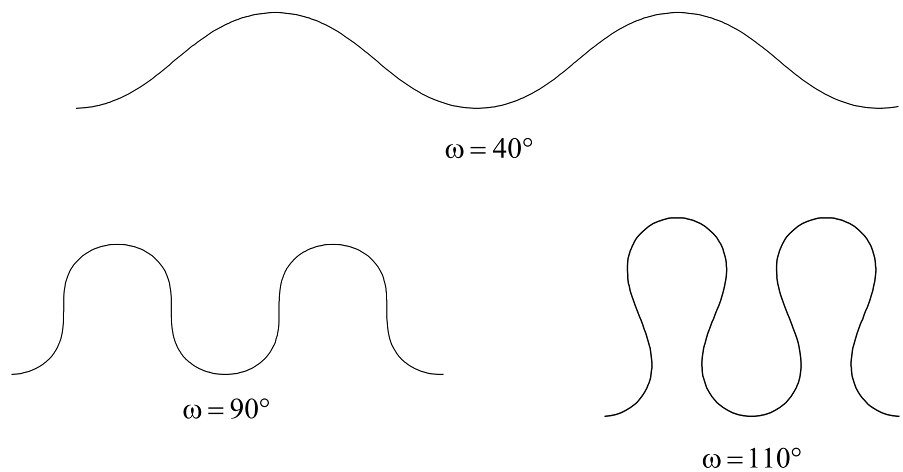


Figure 2.2 Sine-generated curves for various maximum path angles, ω, relative to the meander belt (valley) axis (modified from Langbein and Leopold, 1966).

The difference between the sine-generated meander and the circular curve is variable curvature in the sine-generated loop which is a trigonometric function of the ratio of channel distance to total length of a single meander. The presence of this positional ratio in Equation 2.107 indicates that curvature is constant at the same relative position along a meander loop for any size of meander of constant sinuosity.

Langbein and Leopold (1966) compared the planform of channels from a variety of environments in western U.S.A. and at several scales from the flume experiments conducted by Friedkin (1945) to the Mississippi River at Greenville, Mississippi, and revealed close similarity with the theoretical layout. Field data from 78 U.S. streams compiled by Williams (1986) also compared favourably with the planform of the sine-generated curve. However, as in all probability models, the most probable state is only one condition within a range of possible outcomes.

In a natural meandering channel, sinuosity, bend radius, arc length and arc angle are not constant values along a reach but portray a degree of variability such that meander patterns seldom consist of a string of identical bends. Despite advocating a regular meander path model as a theoretical stable state, Langbein and Leopold (1966, p. 15) stated that ‘nature is never so uniform’ and theorised that deviations from the sine-generated curve are a result of two causes: i) shifts from unstable to stable forms caused by random actions and varying flow, and: ii) non-homogeneities such as rock outcrops, variations in alluvium, woody debris and riparian conditions (Langbein and Leopold, 1966, p. 5).

According to Ferguson (1975, 1979), the meander planform should be characterised by an irregularity (or quasi-randomness) factor, as well as scale and shape parameters, that can be measured from the direction or curvature series for an existing stream. In a study of U.K. rivers, Ferguson (1975) showed that irregular meander geometry is the result of the distortion imposed by environmental irregularities and subsequently proposed a disturbed periodic model which recognises that events in neighbouring channel segments are dependent on one another and accounts for changes to the centreline of meander oscillation (Ferguson, 1976, 1979). Despite evidence showing the Ferguson model generates statistical meander properties similar to those found in natural streams (O’Neill, 1987), for the purpose of channel restoration, the degree of irregularity is an unknown variable.

Based on field observations in Quebec and work by other researchers, Carson and Lapointe (1983) and Lapointe and Carson (1986) proposed that natural meanders exhibit asymmetric bend geometries. They suggested that in many meandering rivers the majority of the downstream limb of a meander bend is convex facing downstream, resulting in a delayed crossover whereby the inflexion point alternates either side of the meander belt axis down the valley. This type of asymmetry is expected in an actively migrating river, as the point of maximum bank retreat is located downstream from the bend apex. Lapoint and Carson (1983) derived an index to describe the extent of delayed inflexion asymmetry from meander traces, and it is shown in Chapter 7 that a simple adjustment can be made to the sine-generated curve to account for this type of asymmetry in the design of restored planimetric geometry. More recently, kinematic models of meander migration have been proposed (e.g. Ferguson, 1984; Howard, 1984) which are based on models of flow in curved channels and assume that bank erosion is proportional to bank shear stress. These models require flow variables which are not easily measured. Further research is required to validate these models and assess their practical value for channel restoration design. They are not considered further in this study.

2.4.3 Historical Reconstruction: The *Carbon Copy* Solution

Historical reconstruction is the reinstatement of a previous channel configuration, that possessed the type of channel configuration and range of forms and features required in the target restored channel. Ideally, this ‘carbon copying’ approach involves replacing meanders exactly as found prior to disturbance (Brookes and Sear, 1996; Shields, 1996; FISRWG, 1998). Two techniques are available: i) replicate meander planform from historical sources (e.g. air photographs, maps), and; ii) excavate old river courses on the floodplain. Although case studies of successful restoration using these approaches are not well documented, Brookes and Sear (1996, p. 92) consider the carbon copy method to be one of the most widely practised techniques in northern European countries.

The fundamental problem with historical reconstruction for river restoration is in the assumption that rainfall-runoff and sediment discharge patterns in the catchment have not significantly varied over time and therefore, the restored channel will be stable within the fluvial system. However, a restored channel is only likely to be stable if the approach has accounted for watershed hydrology and supply reach sediment dynamics which are functions of *contemporary* catchment controls rather than some pre-disturbance state. If land-use patterns have changed since the date of the historical channel, then it is highly likely that the restored channel configuration would be unstable and would result in progressive aggradation or degradation. For example, if there has been a significant increase in bed load due to upstream channelisation, deforestation, urbanisation or farming practices, then the historical channel dimensions will tend to underestimate stable values of width, wavelength and slope and overestimate stable values of depth and sinuosity.

However, in some cases, channel stability could be enhanced if old channels were filled with cohesive sediments which stabilise the bank-lines (Shields, 1996, p. 32). When historical channel alignments are considered unrepresentative because the assumption of stationarity in drainage basin controls is unrealistic, then undisturbed reaches could be examined as potential design models or analogues of natural channel attributes.

2.4.4 Reference Reach Geometry: The *Natural Analogue* Solution

If there are undisturbed reaches close to the target restored channel, they may be used individually or collectively as channel restoration design blueprints or 'reference reaches'. Alternatively, if stable reaches can be identified in catchments with similar hydrological and physiographic characteristics and valley type, then channel geometry data could be extrapolated to the restored reach through the application of measured morphological relationships. Rosgen (1998) discussed the geomorphological value of the reference reach in channel design applications and considered that, although 'pristine' reaches are very rare to find in catchments targeted for river restoration, stable segments of river can usually be found.

The Rosgen method is based on classifying reference reaches to group measured variables by morphological similarity and to reduce statistical variance between groups. For a specific type of channel, dimensionless ratios of measured attributes, for example width-to-depth ratio, riffle elevation-to-bankfull elevation, etc. can then be extrapolated using regression models for channel design. This approach is preferred over more analytical methods based on the application of sediment transport equations which often yield significant errors in estimates of the design discharge and supply load that could affect the deign specification. However, one of the main limitations of the reference reach approach is the subjectivity in locating 'stable' reaches and sites with similar boundary conditions as the target restored channel and identifying bankfull dimensions from field indicators. While reference reaches may not provide the perfect blueprint for channel design in all cases, they should be examined, where possible, to determine the general target 'type' of restored channel, including the type of meander bend geometry, bed and bank material properties, riparian vegetation on stable banks, riffle-pool spacing and dimensions and other natural attributes.

2.5 SUMMARY: PRACTICAL APPROACHES FOR CHANNEL RESTORATION DESIGN

In this chapter, nature was acknowledged as the best channel restoration designer on the basis that the complicated process-form relationships in rivers that continue to evade a complete understanding from hydraulic theory are intuitive to the river with its inherent complex response mechanisms. However, natural recovery rates are dictated by the available energy to remould the channel boundary and in many disturbed lowland streams, natural recovery is unlikely to reset the imbalance between form and process over a period of years or, in some cases, decades. In such cases, the river is an ineffectual channel restoration designer and approaches from hydraulic engineering and geomorphology can provide effective solutions to direct the river toward a stable configuration in an attempt to restore the form and function of a natural fluvial system. However, not all of the methods discussed in this chapter are appropriate for, or indeed capable of, meeting this management objective.

Restoring river channels with mobile beds precludes the use of regime equations which are correctly applied to canal systems only and the numerous analytical methods developed for static-bed or threshold channels. Hydraulic geometry equations and analytical regime theory based on sediment transport continuity are more appropriate to channel restoration design. Also, despite their simplicity, regular meander path models also provide a suitable starting point for laying out meander planform geometry.

Empirical methods are directly related to actual observations of stable channels and through data plots, provide an insight into natural morphological variability. However, they do not adequately account for sediment inputs which control channel stability and are not based on hydrodynamic or morphological process-based equations. Analytical approaches are based on physical equations which account for sediment inputs and can be used to examine the sensitivity of design variables in response to changes in flow and sediment patterns and whether a project will meet physical habitat criteria. However, they yield unique deterministic solutions with no information on natural systems variability and the river system remains indeterminate using only process-based equations. It is therefore inevitable that empirical and analytical methods should be combined in applications of sediment-related river management and stable channel design.

The length of this chapter reflects the wealth of research into stable channel design methods during the Twentieth Century. Within the engineering community there has been a shift in methodological approach from the empiricism of regime theory to more analytical (process-based) methods which have sought to explain the shape and size of natural river channels from an understanding of flow dynamics, particle physics and process-based relationships. However, despite advances in hydrodynamic and morphological modelling, a complete understanding of the fluvial system continues to evade scientific explanation. The geomorphological response to this dilemma was to further develop channel design equations through the concept of downstream hydraulic geometry and more recently, to use natural reference reaches and/or historical channel configurations as suitable analogues for channel restoration design.

Throughout this chapter the limitations of the different approaches have been identified. By exploiting the strengths of the available methods, it is the objective of the next chapter to bring together the most suitable approaches and techniques into a coherent design framework, thereby overcoming, to some degree, their individual limitations and providing a solution to the indeterminacy problem in stable channel design. Chapter 3 is divided into two main sections: i) a discussion of the geomorphological principles that form the basis of the geomorphic engineering approach to ensure the restoration of stable channel dimensions and dynamic stability within the catchment system and to ensure realistic solutions that mimic the morphological variability found in natural channels, and; ii) an overview of the design procedure, methods and design parameters that builds from the review in this chapter.

CHAPTER 3

Channel Design Framework

Principles, Techniques and Procedure

3.1 FRAMEWORK FOR GEOMORPHOLOGICAL ASSESSMENT AND ENGINEERING DESIGN

Channel restoration design is a nested phase within a larger procedural framework, or blueprint, for morphological studies which forms the methodological basis for geomorphological and environmental assessment, project planning and river management. Prior to the design work and project implementation a geomorphological assessment is imperative which requires a reasonable knowledge of the river system, its parent catchment hydrology and project level assessment of river mechanics and geomorphology to account for potentially destabilising phenomena within the system. In the U.K., the Environment Agency (1998) has set out a coherent approach to address these requirements whereby geomorphological studies are related to specific management tasks and activities.

The Environment Agency Framework involves a sequential decrease in spatial scale through the initial stages of a river study whereby a 'catchment baseline survey' and project level 'fluvial audit' provide the necessary information to: i) classify the river system in terms of spatial influences and temporal changes; ii) prioritise reaches for further investigation; iii) identify possible reference reaches in the system which could be used to specify stable channel dimensions and examine the magnitude and frequency of sediment-transporting flow events, and; iv) identify differences in the geomorphological conservation value within the prioritised reaches. In light of the project objectives, these studies aim to address the nature of instability within the system and provide the baseline information to reassess the target restoration criteria prior to undertaking a more detailed geomorphological assessment of the flow and sediment regime supplying the project reach and identifying site constraints which would influence the final design. Kondolf and Downs (1996) discussed catchment level and historical analyses of geomorphological influences and character in more detail. Following implementation of the project design, the final phase in the framework is a monitoring programme which examines both the compliance and performance of the design through geomorphological appraisal, which also provides a basis for specifying a maintenance commitment and design revisions where necessary (Skinner, 1999).

The channel restoration design procedure developed here should be carried out following initial catchment baseline surveys. The following sections examine the geomorphological principles intrinsic to the procedure and the main design phases, methods and variables involved.

3.2 RESTORING DYNAMIC STABILITY WITHIN THE CATCHMENT SYSTEM

Engineers involved in river management are primarily concerned with flow mechanics, sediment properties and yields, whilst geomorphologists are concerned more with sediment sources, fluxes and sinks (Sear and Newson, 1994; Environment Agency, 1998). A stable channel configuration may be defined when the prevailing flow and sediment regimes do not lead to progressive changes in aggradation or degradation over the medium- to long-term. Short-term changes in sediment storage and natural planform migration are inevitable in all meandering channels with unprotected banklines and are permitted in this definition. Therefore, the restoration design of a stable channel centres on ensuring sediment continuity and therefore, equilibrium sediment transfer through the fluvial system during the life-span of the design. This requires bringing together the interests and practical skills of both the engineer and the geomorphologist.

A stable design solution and effective river management post-implementation require knowledge of both flow and sediment routing in the upstream supply reach, through the restored channel and in the existing channel downstream. This necessitates a holistic geomorphic appraisal that extends beyond the project reach scale (Figure 3.1). Using a simplistic one-dimensional approach, the sediment load entering the restored reach, together with the design discharge, define the dimensions of the restored channel to ensure that there is no net aggradation or degradation. These dimensions then determine the sediment load entering the downstream reach which has a specific sediment load demand to maintain channel stability. Therefore, design variables pertaining to the character of the flow and sediment regime should be defined from the upstream supply reach(es). In the restored reach, the nature of the floodplain influences the design (in terms of site constraints, for example existing structures, constrictions and proposed floodplain developments) together with the nature of target riparian vegetation, as it influences roughness.

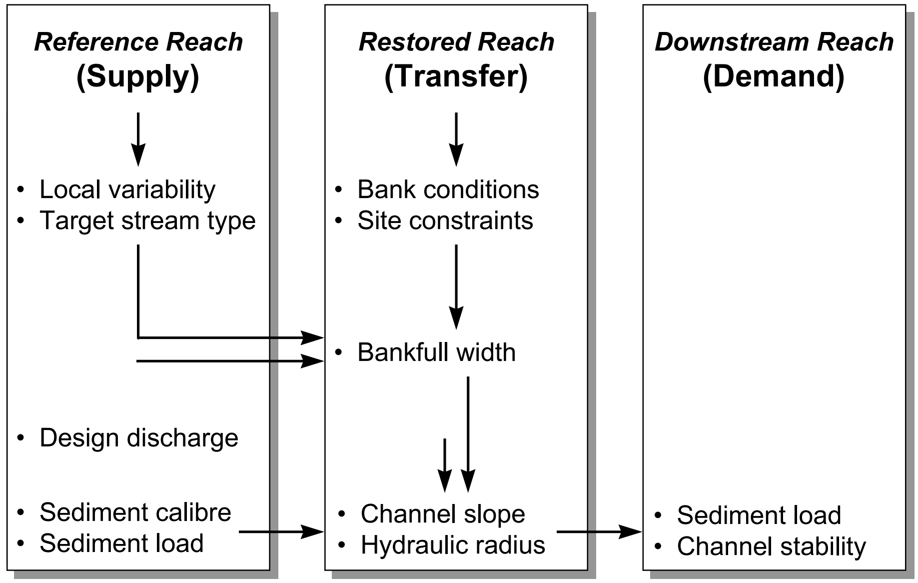


Figure 3.1 Systematic nature of the fluvial system in terms of sediment supply, transfer and demand.

The potential 'success' of a river project is often defined in terms of performance based on a single flow event and the sediment load transported by this event. This approach does not account for the potential for instability driven by other flow events in the long-term record. The potential for restoring sediment continuity through the restored reach requires an assessment of the sediment budget, which is determined by the magnitude and frequency of all sediment-transporting flows. To attain geomorphic stability through sediment continuity in the medium- to long-term, the mean annual sediment load for the restored channel (capacity) must match the mean annual sediment load in the supply reach (supply).

The concept of 'total restoration potential' was developed by Downs et al. (1999) to assess the impact of in-stream structures for river rehabilitation and is based on the ability of the natural sequence of flows to transport the quantity of sediment necessary to modify the channel morphology significantly. In channel restoration design, this concept is redefined as a Capacity-Supply Ratio (CSR), which is calculated as the bed material load transported *through* the restored reach by the natural sequence of flow events over an extended time period divided by the bed-material load transported *into* the restored reach by the same flow events over the same time period. Therefore, by definition a successful project design has a CSR close to unity. Values greater than 1.0 indicate potential degradation and values below 1.0 indicate potential aggradation. Disparities between sediment supply and capacity pertaining to individual flows will inevitably indicate potential for short-term channel changes, which is not unexpected in a dynamically stable channel. However, matching the medium-term sediment input and output should result in a low maintenance channel, with environmental and economic benefits that are sustainable in the medium- to long-term. Therefore, it is recommended that this sediment impact assessment should be performed at the end of the design procedure as a closure loop to examine the stability of the restored channel.

Evaluating the restoration potential based on this kind of sediment impact assessment assumes stationarity in drainage basin controls, whereby the recorded flow distribution prior to restoration is a reasonable template for the post-restoration distribution. However, from a review of British rivers, Lewin et al. (1988) showed that many channels are sensitive to relatively small changes in sediment supply and runoff and adjust their shape and dimensions frequently in response to anthropogenic influences to the catchment system. As a result of this sensitivity, Lewin et al. (1988) considered regime theory and environmental change as irreconcilable concepts. This is particularly relevant in urbanising catchments where spatial and temporal changes in rainfall-runoff and sediment supply are reflected in channel change rather than static morphologies (Wolman, 1967).

Sensitivity is ‘the propensity of a system to respond to a minor external change. If the system is sensitive and near a threshold, it will respond to an external influence; but if it is not sensitive, it may not respond’ (Schumm, 1991, p. 78). A restoration project should consider possible future trends of land-use change, which could influence channel stability. This necessitates an examination of whether the design configuration is robust or sensitive to changes in the flow and sediment regimes. Undertaking a sensitivity analysis prior to implementation may also indicate appropriate levels of bank protection and post-project maintenance and monitoring to be budgeted into the costs for the design proposal. Qualitative directional predictions of post-project channel change can be derived using the conceptual treatment given by Schumm (1977) based on aggradational and degradational tendencies (Table 3.1). However, this type of assessment has limited practical value. To test quantitatively the sensitivity of channel dimensions to variations in flow and sediment variables, the design method must include an analytical component, since the empirical hydraulic geometry approach does not adequately account for sediment transport.

Change	River Bed Morphology
Q_s + and Q +	process increased in intensity
Q_s = and Q +	incision; channel instability; wider and deeper channel
Q_s − and Q +	incision; channel instability; deeper, wider? channel
Q_s + and Q =	aggradation; channel instability; wider and shallower channel
Q_s = and Q =	no change; stable channel
Q_s − and Q =	incision; channel instability; narrower an deeper channel
Q_s + and Q −	Aggradation
Q_s = and Q −	aggradation; channel instability; narrower and shallower channel
Q_s − and Q −	process decreased in intensity

Note: Q_s = sediment discharge; Q = water discharge; '-', '+', '=' are increase, decrease and unchanged respectively, '?' = uncertain response.

Table 3.1 Qualitative impacts on river bed morphology because of changes in water and sediment rates (modified after Werrity (1997, p. 55) and based on Schumm's (1977) river metamorphosis concept).

3.3 NATURAL SYSTEMS VARIABILITY

3.3.1 Background: Deviations from Regime

Natural rivers which are in regime have stable morphologies that broadly conform to regime or hydraulic geometry relationships, linking the dependent parameters of channel form to independent controls of flow regime, boundary materials and riparian vegetation between controlling and controlled variables. However, rivers do not follow regime laws precisely. In fact, every river displays local departures from the expected channel form described by morphological equations and possesses inherent variability in space and time. While it is true that natural channel forms are in general predictable, it is also true that each river is unique in detail. Therefore, regime dimensions in the natural domain should only be interpreted as representative average, ideal or 'target' conditions about which channel morphology fluctuates in time and space.

An engineering project requires the specification in design drawings of channel dimensions, notably for width, depth and slope in order that machine operators can work effectively. Often the inherent variability of natural systems has been neglected in conventional approaches to engineered channel design. When designing a restored channel configuration, it is important to recognise that there is a range of possible solutions rather than a single 'correct', or regime, design and that some uncertainty concerning the exact form of the channel is not only inevitable but also quite acceptable.

Given that uncertainty in the prediction of the regime dimensions of a channel is an advantage in channel restoration design, the question which then arises concerns the best method of using uncertainty to create latitude for the designer to fine tune the specification for the channel and allow for natural variability. To identify the degree of uncertainty in the predicted channel dimensions, a statistical analysis of the regression variables is required.

Downstream hydraulic geometry relationships (such as the width equations developed in Chapter 5) are generally expressed in the form of a power function:

$$Y = \mathrm{a}\, X^{\mathrm{b}} \tag{3.1}$$

where Y is either a bankfull cross-sectional parameter (width, mean depth or area), reach average channel slope, average velocity or a planform parameter (usually meander wavelength), X is usually bankfull discharge or bankfull width if predicting meander wavelength and 'a' and 'b' are constants. In the general case, 'a' and 'b' are determined from the regression model.

These types of bivariate equations are derived from log-transformed data for both the independent and dependent variables. The transformation is required to linearize the data such that a regression model of the form

$$\ln(Y) = \ln(\mathrm{a}) + \mathrm{b}\ln(X) + \varepsilon \tag{3.2}$$

exhibits a normal (Gaussian) distribution of error, ε, with zero mean and homogeneous variance. In Equation 3.2, ln(a) and 'b' are the regression coefficients. The most common technique used to calculate the coefficients is the least squares formulation.

The appropriate method to quantify the uncertainty in Equation 3.2, because of the error variance or 'scatter', is the application of confidence intervals (also termed bands or limits). The advantage of using confidence intervals is that they account for variability in both regression constants and may be used to describe the degree of variability at a user-defined probability level. Furthermore, applying a measure of uncertainty to hydraulic geometry equations is necessary because of the statistical precept that the level of confidence associated with the predicted regression value tends towards zero with increasing precision. In other words, the probability of a value falling *exactly* on a regression line is near zero. An alternative interpretation is that there is a near 100 percent probability that the required channel width is actually either somewhat greater or less than that estimated by a regime-type equation. Also, in the context of river morphology, Equation 3.1 suggests a fixed, deterministic relationship that fails to account for the inherent spatial and temporal variability in channel geometry found in nature. Heterogeneity in channel form is aesthetically pleasing, ecologically valuable and essential for channel stability in a meandering river. Confidence bands can be used effectively to introduce non-uniformity into the design geometry, as required in channel restoration design, without the need for complex numeric modelling to simulate this uncertainty.

Burns (1971) was one of the first researchers to use statistical confidence bands to explain (albeit graphically) the observed spread of data around downstream hydraulic geometry equations. He attributed the data scatter to the presence of different types of cross section including riffles and pools. Burns developed 'state-wide' regional relationships of bankfull width, depth and average velocity as functions of fixed-frequency discharges from observations in the Smoky Hill, Republican and Kansas Rivers, Kansas, and confidence limits of 60 percent and 90 percent were used to 'indicate that individual predictions are subject to considerable uncertainty' (p. 19).

The two most common methods for applying statistical confidence limits are developed here: i) the confidence interval on the mean response and; ii) the confidence interval on a single response. These are described below in terms of a general template for hydraulic geometry equations with variable and fixed exponents, based on techniques in advanced statistical texts (e.g. Myers, 1990; Graybill and Iyer, 1994).

3.3.2 Variable Exponent Model

Confidence Interval on the Mean Response

Equation 3.2 is interpreted as the *mean* of the distribution of $\ln(Y_i)$ at a given value of $\ln(X)$, where 'i' refers to measured values of the dependent variable. Since a regression model can only be used to make inferences about a population, this 'mean response' is likely to vary between different samples. If repeated regressions are conducted, based on the same levels of the independent variable, the degree of variation in the dependent variable will be found within confidence intervals of the mean response with given probability, p, and 100(1-p) percentage confidence.

For the case of bivariate linear regression in the form of Equation 3.2, the shape of the mean response limits tends to be hyperbolic. Within 100(1-p) percent confidence limits, the mean response, Y_p, is given for the case when the independent variable X equals X_0 as

$$Y_p = e^k \tag{3.3}$$

$$k = \ln\left(aX_0^b\right) \pm t_{p/2,n-2}\, s \sqrt{\frac{1}{n} + \frac{\left[\ln\left(X_0\right) - \alpha\right]^2}{\sum_{i=1}^{n}\left[\ln\left(X_i\right) - \alpha\right]^2}} \tag{3.4}$$

where 'n' is the sample size, '$t_{p/2,\,n-2}$' is the upper $p/2$ percent point of the t-distribution with n-2 degrees of freedom, 'X_I' are the measured X values, 'α' is the mean of the logarithm of measured X and 's' is the regression error standard deviation, given by

$$s = \sqrt{\frac{\sum_{i=1}^{n}\left[\ln\left(Y_i\right) - \ln\left(aX_i^b\right)\right]^2}{n-2}} \tag{3.5}$$

where 'Y_i' are the measured Y values.

The confidence interval widens when: i) the required confidence level increases (p increases and the t-value decreases); ii) the standard deviation of Y at a given X_0 value increases (s increases); iii) the sample size, n, decreases, and/or; iv) X_0 deviates farther away from α. Equation 3.4 can be presented in a simplified form that identifies the different constants which must be identified for any given data set, such that

$$k = \ln\left(aX_0^b\right) \pm c_{1,p}\sqrt{\frac{1}{c_2} + \frac{\left[\ln\left(X_0\right) - c_3\right]^2}{c_4}} \tag{3.6}$$

For a specified probability, p, $c_{1,\alpha}$ is the product of the upper $p/2$ percent point of the t-distribution with n-2 degrees of freedom and the regression error standard deviation, c_2 is the sample size, c_3 is the mean of the logarithm of measured X and c_4 is given by

$$c_4 = \sum_{i=1}^{c_2}\left[\ln\left(X_i\right) - c_3\right]^2 \tag{3.7}$$

Confidence Interval on a Single Response

When examining regression variability, it is often necessary to consider the confidence interval applied to a single response, or observation, rather than the mean response. In bivariate equations, limits of this nature describe the probable range in the dependent variable of adding a further data point to the scattergraph, with given probability, p, and 100(1-p) percentage confidence.

For the case of bivariate linear regression in the form of Equation 3.2, the form of the single response limits is very similar to that of the mean response but differs in the value of k, given by

$$k = \ln\left(\mathrm{a}X_0^{\mathrm{b}}\right) \pm t_{p/2,n-2}\, s \sqrt{1 + \frac{1}{\mathrm{n}} + \frac{\left[\ln\left(X_0\right) - \alpha\right]^2}{\sum_{\mathrm{i}=1}^{n}\left[\ln\left(X_{\mathrm{i}}\right) - \alpha\right]^2}} \tag{3.8}$$

Equation 3.8 can be presented in the simplified form of Equation 3.6 when the constant c_2 is assigned a new value of $n/(n+1)$.

The confidence limits on a single response are significantly wider than the mean response limits, reflecting the additional error variance in Equation 3.8. As a result, these confidence limits are useful to describe upper and lower bounds of the actual spread of data rather than the regression line, *per se*, although these bounds are strongly influenced by outlying data points.

It is highly probable that several of the ‘outliers’ in a regime, or hydraulic geometry data set may be unreliable for three reasons: i) inaccurate measurement of channel geometry and difficulties estimating the channel-forming discharge; ii) the channel width may be partially influenced by parameters or constraints that are not common to the majority of observations for a particular type of river. Furthermore, the distinction between different river types is not distinct but transitional. As hydraulic geometry equations usually relate to a particular type of river, it is plausible that ‘outliers’ may fall within this transitional category, and; iii) it is unlikely that all observations are truly ‘in regime’. Outliers may not be true regime channels but either exhibit discontinuity in sediment transport or are approaching a state of quasi-equilibrium following disturbance. In light of these influences, it is good practice to select the more stable confidence interval of the mean response for channel design purposes. Observations may fall outside the mean limits and be stable, but confidence cannot be assigned accurately. Therefore, caution must be exercised when using single response limits to explain channel stability/instability.

The shape of single response limits tends to be much more linear than the curved mean response limits. Single response limits are also referenced as prediction limits.

3.3.3 Fixed Exponent Model

For the purpose of developing equations for channel restoration design, it is often appropriate to simplify the form of a relationship by making an assumption about the coefficient, a, or exponent, b, in Equation 3.1. For example, in width equations based on downstream hydraulic geometry, the exponent may be fixed at 0.5 with negligible error in the mean response (see section 2.4.1.2 and Chapter 5), such that

$$\ln\left(Y\right)=\ln\left(\mathrm{a}\right)+0\cdot5\ln\left(X\right) \tag{3.9}$$

Logarithmic transformation is required to normalise the error variance. By adopting this principle, a regression analysis is no longer required, because there is only one unknown value to predict. The mean response is then given by

$$Y=\mathrm{a}^{*}X^{0\cdot5}=\mathrm{e}^{\alpha}X^{0\cdot5} \tag{3.10}$$

Where a^* is the coefficient in the fixed exponent model and 'α' is the mean of the natural logarithm of $Y/(X^{0.5})$. Confidence limits can then be applied to the coefficient only in Equation 3.9. At a given probability, p, 100(1-p) percent confidence limits applied to a hydraulic geometry relationship with fixed exponent are given below.

Confidence Interval on the Mean Response (Fixed Exponent)

Confidence intervals on the mean response with independent variable exponent fixed at 0.5 are given by

$$Y_p=\mathrm{e}^{k}X^{0\cdot5} \tag{3.11}$$

$$k=\alpha\pm t_{p/2,\ n-1}\frac{\beta}{\sqrt{n}} \tag{3.12}$$

where 'α' is the mean of the natural logarithm of $Y/(X^{0.5})$, 'n' is the sample size, '$t_{p/2,\, n-1}$' is the upper p/2 percent point of the t-distribution with n-1 degrees of freedom and 'β' is the sample standard deviation of the natural logarithm of $Y/(X^{0.5})$.

Equation 3.12 can be presented in a simplified form that identifies the different constants which must be identified for any given data set, such that

$$k = c_5 \pm c_{6,p}\, c_7 \qquad (3.13)$$

where c_5 has replaced 'α', c_7 has a value of 1 for mean response limits and $c_{6,p}$ is given by

$$c_{6,p} = t_{p/2,\, n-1} \frac{\beta}{\sqrt{n}} \qquad (3.14)$$

Confidence Interval on a Single Response (Fixed Exponent)

Equation 3.12 is replaced by

$$k = \alpha \pm t_{p/2,\, n-1} \frac{\beta}{\sqrt{n}}\left(1+\sqrt{n}\right) \qquad (3.15)$$

Similar to Equation 3.13, Equation 3.15 can be presented in the simplified form when the constant c_7 is assigned a new value of $n^{0.5}$+1.

3.3.4 Bias in Hydraulic Geometry Equations

Logarithmic transformations are used routinely in fluvial geomorphology as a method of generating prediction equations. Two of the most common examples are hydraulic geometry equations and sediment-rating curves that predict sediment load as a power function of discharge. While the transformation is necessary as to not compromise the assumptions of parametric regression, it also introduces a systematic bias into calculations. This is because the residuals about a regression line do no cancel arithmetically. In a log-transformed regression model, the error, ε, in Equation 3.2 is 'additive'. However, the error becomes 'multiplicative' when Equation 3.2 is transformed onto arithmetic scales, such that the error in the power function of Equation 3.1, ε^*, is given by

$$\varepsilon^{*} = aX^{b}\left(e^{\varepsilon} - 1\right) \tag{3.16}$$

Hence, ε^* increases with aX^b and is greater for positive log residuals, ε, than for negative log residuals. Therefore, the absolute regression error is larger above the regression line than below it and the mean response calculated by Equation 3.2 underestimates the true mean.

This bias is widely documented in the statistical and scientific literature (e.g. Sprugel, 1983; Miller, 1984; Ferguson, 1987; McCuen et al., 1990; Rhoads, 1992; Hey, 1997a). A correction factor, F, is necessary to eliminate this bias and is given by Sprugel (1983, p. 209) as the multiplier

$$F = e^{\left(s^{2}/2\right)} \tag{3.17}$$

where 's' is the error standard deviation (Equation 3.5). Because of the nature of its derivation, Sprugel (1983, p. 210) notes that this factor cannot be given in base-10 logarithms. The correction factor is used to correct for underestimated sediment loads calculated from sediment rating curves (Ferguson, 1987; Hey, 1997a) and Rhoads (1992) recommended correcting for the downwards bias in hydraulic geometry equations.

Applying the correction factor, an unbiased hydraulic geometry equation is given as

$$Y = FaX^{b} = e^{\left(s^{2}/2\right)}aX^{b} \tag{3.18}$$

where 'a' and 'b' are defined from ordinary least squares regression on the natural logarithmic transformed variables. The degree of bias increases exponentially with the error variance and is, therefore, likely to be greatest in relationships derived from small data sets with high variability.

With this correction, it is also necessary to correct for bias in estimates of confidence. Confidence limits on the mean response represent the likely range of responses with repeated sampling and are derived from the variance of the mean response only. Confidence limits on a single response are derived from the variance of the mean response added to the variance of residuals (the error variance) (Myers, 1990, pp. 42-45). Assuming that both these variances are constant for each regression calculated from repeated samples, both types of confidence limits can also be adjusted by multiplying by the correction factor. As this mechanism does not give symmetrical confidence limits about the corrected mean response, it is not truly correcting for bias in the confidence levels but adjusting for the change in mean response. By modifying Equation 3.3, the corrected confidence limits for bivariate linear regression are given as

$$Y_p = F\,\mathrm{e}^k = \mathrm{e}^{\left(k+s^2/2\right)} \tag{3.19}$$

where k is calculated from Equations 3.4 and 3.8 for mean response limits and single response limits, respectively.

For the fixed exponent case, a correction factor, F^*, similar to Equation 3.17 may be used, such that

$$F^* = \mathrm{e}^{\left(\beta^2/2\right)} \tag{3.20}$$

where 'β' is the sample standard deviation of the natural logarithm of $Y/(X^{0.5})$. An unbiased hydraulic geometry equation with exponent fixed at 0.5 is therefore given by

$$Y = F^* a^* X^{0\cdot 5} = \mathrm{e}^{\left(\alpha+\beta^2/2\right)} X^{0\cdot 5} \tag{3.21}$$

where a^* is the coefficient in the fixed exponent model and 'α' is the mean of the natural logarithm of $Y/(X^{0.5})$. At a given probability, p, 100(1-p) percent confidence limits are given by modifying Equation 3.11 to give

$$Y_p = F^* \mathrm{e}^k X^{0\cdot 5} = \mathrm{e}^{\left(k+\beta^2/2\right)} X^{0\cdot 5} \tag{3.22}$$

where 'k' remains unchanged and is given by Equations 3.12 and 3.15 for mean response limits and single response limits respectively.

3.3.5 Which Confidence Level?

Without appropriate guidance, there could be a tendency for project engineers to resort to using the predicted best-fit value from hydraulic geometry relationships in all cases and ignore natural variability altogether. In the statistical literature (e.g. Myers, 1990) the 95 percent confidence level is usually taken to be a suitable measure of observed variability and is recommended here to describe uncertainty of the mean response. The wider single response limits are sensitive to outlying data points in a regression analysis which often exhibits characteristics uncommon to the majority of the data set. In many cases, the 90 percent level delineates a band which best fringes the upper and lower limits of the data scatter and excludes obvious outlying data points.

It is recommended that restored channels should be designed within 95 percent confidence limits of the mean response (adjusted to account for hydraulic geometry bias) where possible. In cases where this is not feasible because of floodplain 'right-of-way' constraints, confidence limits on a single response should be considered as tentative design guidance.

3.3.6 Prompting Natural Channel Morphology

Modifications to a river system that are inconsistent with natural processes can lead to a series of complex responses throughout the system. If the imposed channel dimensions or environmental features are not commensurate with the position of the restored reach in the fluvial system, the channel may adjust to a more stable form, dictated by the interaction of water and sediment transfer through the system and the energy available for this 'recovery'. Natural rivers are never in a state of continuous equilibrium but a variable, shifting equilibrium, or quasi-equilibrium (Langbein and Leopold, 1964). In some cases, insensitive river engineering can act as a catalyst within dynamic, meta-stable equilibrium (Schumm, 1975) to carry the system over a geomorphic threshold into a new equilibrium regime. Alternatively, the attribute of *prompted recovery* can be an integral part of providing a dynamically stable geomorphic design (Downs and Thorne, 1998). In damaged ecosystems, if the rate of natural recovery is very low, morphological recovery can be accelerated by artificial prompting using, for instance, flow deflectors to reduce the width of an over-wide channelised reach and promote alternate bar growth, scour and fill and the development of a more diverse and natural bed topography. The ideology of prompted recovery can be incorporated into the channel restoration process by designing an approximate configuration, or channel mould and allowing the river itself to design the cross-sectional detail and intra-reach morphological features into the mold. Using this principle, the river is *directed* toward a stable channel configuration, suitable for the target 'type' of channel but is 'prompted' to participate in its own recovery. However, if the designed channel mould deviates strongly from a sustainable morphology for that channel type, then the river may become unstable and eventually adjust to a different dynamically-stable form, for example, where a single-thread channel begins to braid due to over-widening or a high energy gradient. Theoretically, by designing a channel configuration within confidence limits, the final recovery period during which the river makes post-project adjustments in response to the imposed dimensions is minimised and the amplitudes of any temporal oscillations of channel change are subdued (Figure 3.2).

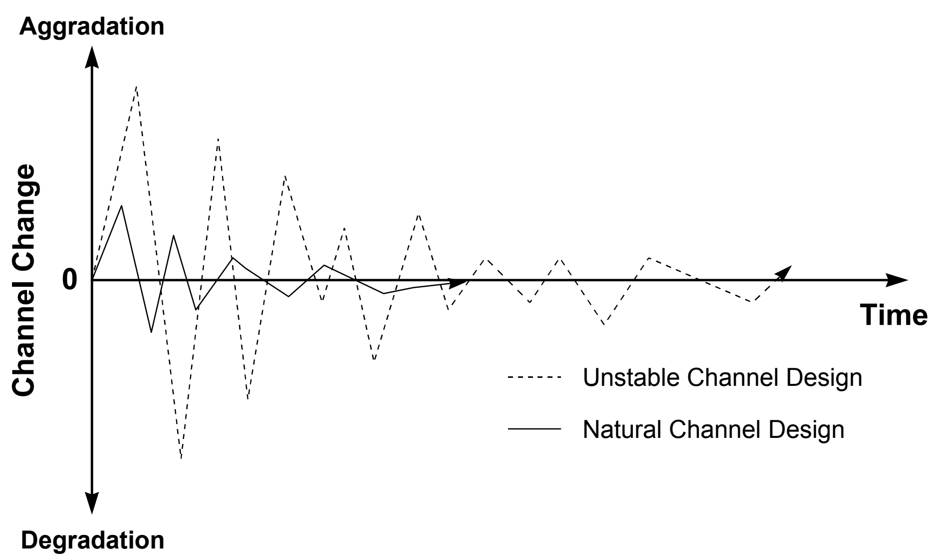


Figure 3.2 Post-project channel change in response to unstable and natural (stable) channel design.

As it is inevitable that the river morphology will respond to a degree in response to imposed channel dimensions, it is recommended that achieving an optimum CSR, within 10 percent of unity, should ensure dynamic stability while allowing the river itself to recover some of the morphological detail that cannot be designed a priori.

Complex two- and three-dimensional design models do not provide adequate alternatives because, although intellectually satisfying, they fail to replicate the true complexity of natural phenomenon. Until engineers and geomorphologists can better understand this complexity, one-dimensional approaches can design approximate geometries which are sufficient to prompt complete recovery.

3.4 TARGET CHANNEL TYPING

3.4.1 Realistic Typing Schemes for Restored Channels

Regime-type relationships which express width as a function of dominant discharge commonly have R^2 values of 0.8 or greater, suggesting that such relationships might be used to predict the stable width of a restored channel. However, the equations are only valid for the 'type' of river and range of parameters from which they were derived. Hence, it is essential to apply only the morphological equations appropriate to the 'type' of target restored channel. A geomorphic appraisal should be used to type the *existing* channel and recommend an appropriate *target* channel type based on reference reach characteristics. Classification of rivers might be used as a basis for 'typing' the channel. There are five different methods of classifying a meandering, alluvial river at the reach scale ranging from simple descriptions to more comprehensive systems:

i) Mobile or Fixed bed
ii) Bed and/or bank material/vegetation type
iii) Semi-quantitative meander planform type (e.g. Schumm, 1963, Brice 1975, 1984)
iv) Comprehensive typology based on longitudinal, cross-sectional, planform and channel material types (e.g. Rosgen, 1994, 1996)
v) Trends and styles of morphological change (e.g. Downs, 1995)

The types of target river channels considered in this study are stable channels with mobile beds, which leaves the classification schemes ii to iv (above) to distinguish between different channel types. Rivers with negligible sediment transport at stages below bankfull (usually upland streams with paved cobble or boulder beds) should be restored using a different method based on threshold theory (see Chapter 2). The simplest typing scheme is based on the type of bed and bank material. According to Thorne (1997, p. 179), ‘the action of the driving variables of water and sediment inputs on the boundary conditions presented by the floodplain topography, bed sediments bank materials and riparian vegetation produces the characteristic channel morphology of an unconfined alluvial stream’. More comprehensive typologies such as the Rosgen (1994, 1996) method are limited in practice because they require strong geomorphological insight and understanding to apply consistently and usefully (Thorne, 1997, p. 213) and there are insufficient morphological equations to match the number of subcategories (94 in the full Rosgen classification). On this basis, it is recommended that hydraulic geometry equations should be typed according to the nature of bed sediments and bank characteristics. Enhanced width equations with typed bed and bank characteristics are developed in Chapter 5.

Very little design guidance currently exists for laying out the planform geometry of meandering channels. Existing methods often rely on the user locating a reference or control reach on either the study stream or another suitable stream from which to develop a template for the meander planform. This may often be problematical because of the non-availability of a reference reach, subtle but important fluvial, sedimentary or morphological differences between it and the study reach, or restrictions on the right-of-way which preclude the import of meanders with the amplitudes observed in the reference reach. Empirical relationships between channel width and meander geometry in dynamically stable alluvial channels have been developed through numerous and widespread observations of river planforms over the last four decades (see Chapters 2 and 7), but such relationships are rarely categorised in terms of meander shape (an exception being Annable, 1996). Theoretically, relationships between bankfull width and meander wavelength are independent of bed and bank characteristics because the effects of the boundary sediments/vegetation have already been accounted for implicitly in the width-discharge relationship used to estimate width (Thorne, 1997). The ubiquitous nature of the width-meander wavelength relationship has been confirmed from observations in various environments, including ice-cut meanders, channels incised in bedrock and flume channels (Leopold, 1994; Chapter 2).

Three of the most referenced classification schemes are those devised by Schumm (1963, 1977), Figure 3.3, Rosgen (1994, 1996), Figure 3.4 and Table 3.2 and Brice (1975, 1984); Figure 3.5. Close examination of the broad categories in these schemes reveals marked similarities between three of the most common types of meander bend found in stable single-thread channels. This tripartite system is given below:

i) ***Equiwidth Meandering***

(Schumm Type 3a; Brice Type A/B; Rosgen Type E)

'Equiwidth' indicates that there is only minor variability in channel width around meander bends. These channels are generally characterised by: low width/depth ratios; erosion resistant banks; fine-grain bed material (sand or silt); low bed material load; low velocities, and; low stream power. Channel migration rates are relatively low because the banks are naturally stable.

ii) ***Meandering with Point Bars***

(Schumm Type 3b; Brice Type C; Rosgen Type C)

Meandering with Point Bars refers to channels that are significantly wider at bendways than crossings, with well-developed point bars but few chute channels. These channels are generally characterised by: intermediate width/depth ratios; moderately erosion resistant banks; medium grained bed material (sand or gravel); medium bed material load; medium velocities, and; medium stream power. Channel migration rates are likely to be moderate unless banks are stabilized.

iii) ***Meandering with Point Bars and Chute Channels***

(Schumm Type 4; Brice Type D; Rosgen Type C/D)

Meandering with Point Bars *and* Chute Channels refers to channels that are very much wider at bendways than crossings, with well-developed point bars *and* frequent chute channels. These channels are generally characterised by: moderate-to-high width/depth ratios; highly erodible banks; medium-to-coarse grained bed material (sand, gravel and/or cobbles); heavy bed material load; moderate-to-high velocities, and; moderate-to-high stream power. Channel migration rates are likely to be moderate to high unless banks are stabilized.

These types differ in terms of local width variability around meander bends. The extent of this variability is examined further in Chapter 7. The remaining stream types in the Rosgen classification would not present realistic or attractive targets for a restoration scheme for various reasons. Stream types Aa+, A and B are defined by slopes in excess of 0.02, width-to-depth ratios less than 12 and sinuosities less than 1.2. These steep, entrenched step-pool streams are very stable, characterised by colluvial deposits and bedrock forms (rather than alluvial deposits) and are highly resistant to channel change from upstream degradational effects (Annable, 1996). Types D and DA streams are braided and anastomosing channels, respectively, and lie outside the scope of this study. Stream types F and G are considered to be transitional channels that are not stable but recovering a different configuration. In the Brice classification of channel patterns, the irregular width in a type E stream is a likely result of hard points and heterogeneous bank sediments, while types F and G represent channels adjusting to altered flow and sediment regimes where recovered configurations underfit previous alignments.

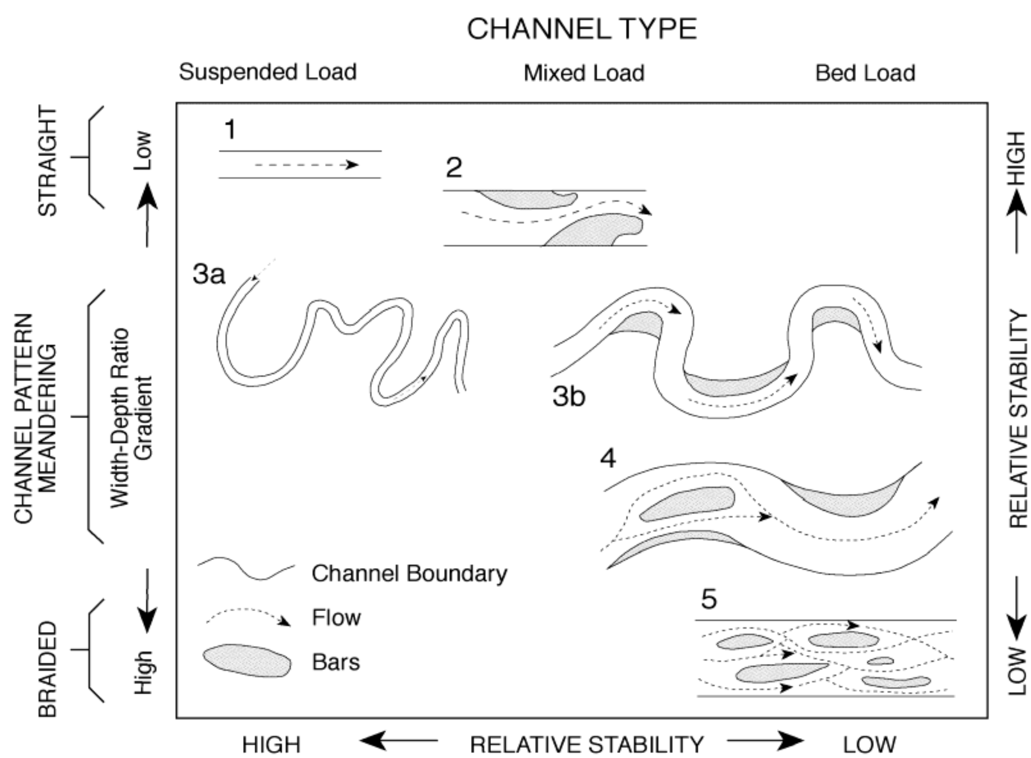


Figure 3.3 Classification of channel pattern based on sediment load and system stability (after Schumm, 1963).

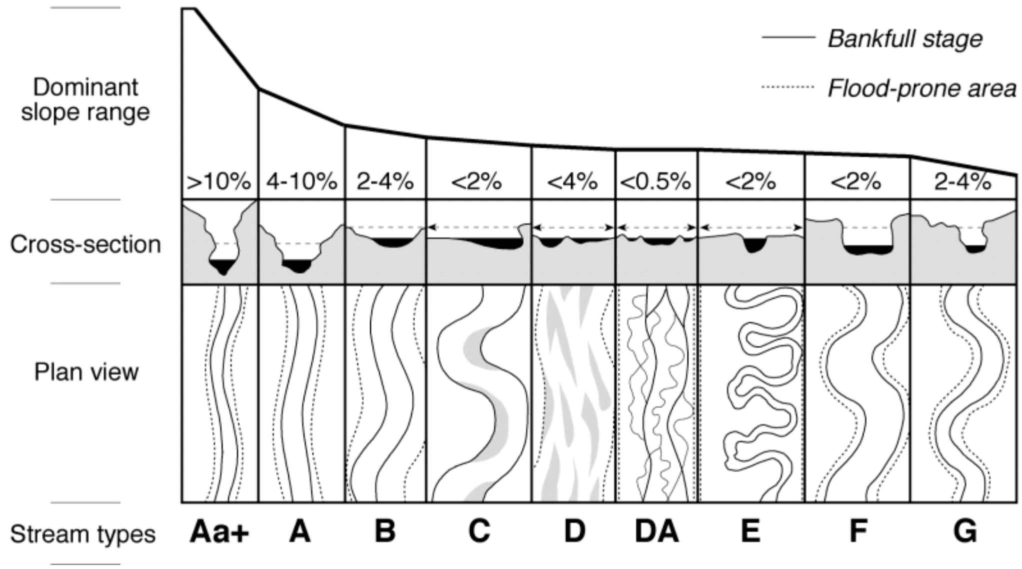


Figure 3.4 Longitudinal, cross-sectional and planform views of major stream types in the stream classification method devised by Rosgen (1994).

To determine whether a morphological relationship is unique to a particular type of river (equiwidth, point bar or point bar and chute channel), it is necessary to investigate the significance level of equality between different data sets. Also, when applying the fixed exponent model, it is important to examine the significance level of rejecting or accepting the assumed value. Two statistical methods are available to describe the similarity between different morphological relationships: i) the General Linear Hypothesis (GLH), and; ii) joint confidence regions on the regression constants. Summaries of these methods as they are applied in this report are given in the next two sections. Their respective applications are dealt with in relevant chapters.

3.4.2 The General Linear Hypothesis

In many statistical applications that require linear regression, it is often useful to gain an insight into whether or not the parameters of separate regression analyses derived from different data sets, differ significantly between data sets. The GLH can be used to determine the level of statistical significance at which:

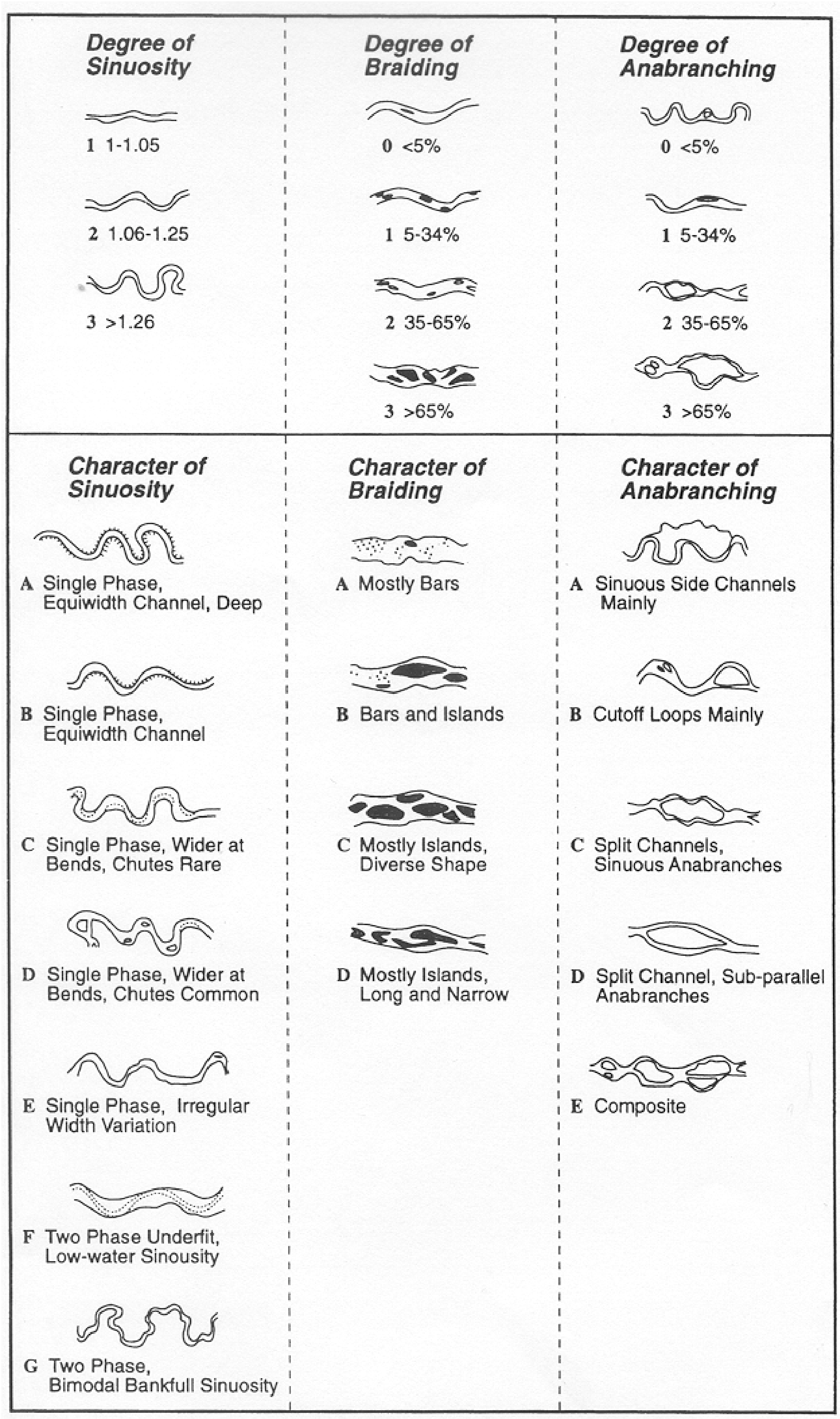


Figure 3.5 Channel pattern classification devised by Brice (after Brice, 1975).

Stream type	General description	*E*	*W/D*	*P*	*S*	Landform / soils / features
Aa+	Very steep, deeply entrenched, debris transport streams	<1.4	<12	1.0 to 1.1	>0.10	Very high relief. Erosional, bedrock or depositional features. Debris flow potential. Deeply entrenched streams. Vertical steps with deep scour pools. Waterfalls
A	Steep, entrenched, cascading, step-pool streams. High energy/debris transport associated with depositional soils. Very stable if bedrock- or boulder-dominated channel.	<1.4	<12	1.0 to 1.2	0.04 to 0.10	High relief. Erosional or depositional and bedrock forms. Entrenched and confined streams with cascading reaches. Frequently spaced, deep pools, associated step-pool bed morphology.
B	Moderately entrenched, moderate gradient, riffle-dominated channel, with infrequently spaced pools. Very stable plan and profile. Stable banks.	1.4 to 2.2	>12	>1.2	0.02 to 0.039	Moderate relief, colluvial deposition and/or residual soils. Moderate entrenchment and *W/D* ratio. Narrow, gently sloping valleys. Rapids predominate with occasional pools.
C	Low gradient, meandering, point-bar, riffle/pool, alluvial channels with broad, well defined floodplains.	>2.2	>12	>1.4	<0.02	Broad valleys with terraces, in association with floodplains, alluvial soils. Slightly entrenched with well-defined meandering channel. Riffle-pool bed morphology.
D	Braided channel with longitudinal and transverse bars. Very wide channel with eroding banks.	n/a	>40	n/a	<0.04	Broad valleys with alluvial and colluvial fans. Glacial debris and depositional features. Active lateral adjustment, with abundance of sediment supply.
DA	Anastomosing, narrow and deep with expansive well vegetated floodplain and associated wetlands. Very gentle relief with highly variable sinuosities. Stable streambanks.	>4.0	<40	range	<0.005	Broad, low-gradient valleys with fine alluvium and/or lacustrine soils. Anastomed. Geological control creating fine deposition with well vegetated bars that are laterally stable with broad wetland floodplains.
E	Low gradient, meandering riffle/pool stream with low width/depth ratio and little deposition. Very efficient and stable. High meander width ratio.	>2.2	<12	>1.5	<0.02	Broad valley/meadows. Alluvial materials with floodplain. Highly sinuous with stable, well vegetated banks. Riffle-pool morphology with very low *W/D* ratio.
F	Entrenched meandering riffle/pool channel on low gradients with high width/depth ratio.	<1.4	>12	>1.4	<0.02	Entrenched in highly weathered material. Gentle gradients, with a high *W/D* ratio. Meandering, laterally unstable with high bank erosion rates. Riffle-pool morphology.
G	Entrenched 'gully' step/pool and low width/depth ratio on moderate gradients.	<1.4	<12	>1.2	0.02 to 0.039	Gully, step-pool morphology with moderate slope and low *W/D* ratio. Narrow valleys, or deeply incised in alluvial or colluvial materials, i.e. fans, deltas. Unstable, with grade control problems and high bank erosion rates.

Note: *E* = entrenchment ratio, *W/D* = width-to-depth ratio; *P* = sinuosity; *S* = slope.

Table 3.2 Criteria used in the Rosgen Classification method (after Rosgen, 1994).

i) Two or more equations, each with the same number of variables, are different ('equivalence' test);

ii) The values of one or more of the coefficients in a regression equation are different between two or more data sets ('parallelism' test);

iii) The values of one or more of the coefficients in a regression equation do not equal user-defined values between two or more data sets ('user-defined parallelism' test).

A detailed description of the GLH, together with examples for simple and multiple regression models, is given by Myers (1990) and further details of the assumptions and derivation of the 'linear model' are described by Graybill (1976). The test procedure for the GLH uses a modified analysis of variance F-test in the form (given in matrix format):

$$F = \frac{\left[\mathbf{Cb} - \mathbf{d}\right]' \left[\mathbf{C}\left(\mathbf{X}'\mathbf{X}\right)^{-1}\mathbf{C}'\right]^{-1}\left[\mathbf{Cb} - \mathbf{d}\right]}{r\, s^2} \tag{3.23}$$

The number of regression equations to be compared may be written in terms of a combined model, such that

$$\mathbf{Y} = \mathbf{Xb} + \varepsilon \tag{3.24}$$

where $\mathbf{Y}$ is the dependent variable vector, $\mathbf{b}$ is the regression coefficient vector, $\mathbf{X}$ is a matrix containing the independent variables and ε is the regression error vector. In the case of hydraulic geometry equations with a single independent variable of the form given in Equation 3.24 with log-transformed data, the combined model is given by

$$\begin{bmatrix} \ln(y_{1,1}) \\ \ln(y_{1,2}) \\ \vdots \\ \ln(y_{1,n_1}) \\ \cdots \\ \ln(y_{2,1}) \\ \ln(y_{2,2}) \\ \vdots \\ \ln(y_{2,n_2}) \end{bmatrix} = \begin{bmatrix} 1 & 0 & \ln(x_{1,1}) & 0 \\ 1 & 0 & \ln(x_{1,2}) & 0 \\ \vdots & \vdots & \vdots & \vdots \\ 1 & 0 & \ln(x_{1,n_1}) & 0 \\ \cdots & \cdots & \cdots & \cdots \\ 0 & 1 & 0 & \ln(x_{2,1}) \\ 0 & 1 & 0 & \ln(x_{2,2}) \\ \vdots & \vdots & \vdots & \vdots \\ 0 & 1 & 0 & \ln(x_{2,n_2}) \end{bmatrix} \begin{bmatrix} \ln(\mathrm{a}_1) \\ \ln(\mathrm{a}_2) \\ \mathrm{b}_1 \\ \mathrm{b}_2 \end{bmatrix} + \varepsilon \qquad (3.25)$$

where n_{j} is the sample size of the jth data set. The matrices **C** and **d** are defined according to the test undertaken, such that the null hypothesis, H_0, (equations *are* equivalent or parallel, depending on the test) and alternative hypothesis, H_1, (equations are *not* equivalent or parallel, depending on the test) are given by:

$$H_0 \qquad \mathbf{Cb} = \mathbf{d} \qquad (3.26a)$$

$$H_1 \qquad \mathbf{Cb} \neq \mathbf{d} \qquad (3.26b)$$

In Equation 3.23, '*r*' is the rank (number of rows) of the matrix **C**. It represents the number of linear combinations under test (or test degrees of freedom). The matrix $\mathbf{C}'$ refers to the transverse matrix of **C**. The error standard deviation, *s*, is of the form of Equation 3.5 for simple linear regression.

The form of the matrices **C** and **d** for the various tests on Equation 3.26 using the GLH is given in Table 3.3.

Test	**C**	**d**
i) Equivalence of entire regression	$\begin{bmatrix} 1 & -1 & 0 & 0 \\ 0 & 0 & 1 & -1 \end{bmatrix}$	$\begin{bmatrix} 0 \\ 0 \end{bmatrix}$
ii) Parallelism of regression slopes	$\begin{bmatrix} 0 & 0 & 1 & -1 \end{bmatrix}$	$\begin{bmatrix} 0 \end{bmatrix}$
iii) User-defined parallelism of regression slopes at 2.0	$\begin{bmatrix} 0 & 0 & 1 & 0 \\ 0 & 0 & 0 & 1 \end{bmatrix}$	$\begin{bmatrix} 2 \\ 2 \end{bmatrix}$

Table 3.3 User-defined matrices for bivariate linear regression used in the General Linear Hypothesis.

Rejection of H_0 is made on the basis of an upper-tailed *F*-test. The degrees of freedom to test the significance of the calculated *F* value are given by '*r*' and the cumulative sample size in the combined regression model of Equation 3.25 (number of rows in **X**). Experience in using the GLH has shown that the significance level of rejecting the null hypothesis is very sensitive to the goodness of fit in the equations (described by the coefficient of determination, R^2) and sample sizes of the data sets compared.

3.4.3 Confidence Regions for Regression Coefficients

All of the confidence techniques described so far refer to single predictions. A useful simultaneous test is to define a confidence region, or ellipse, for the coefficients in Equation 3.2 such that with repeated sampling and regression computation, the coefficients from 100(1-*p*) percent of the regression equations will fall within the region. This technique can be applied to bivariate morphological equations derived from hydraulic geometry analysis, to examine the degree of uncertainty in both regression constants (usually the coefficient and exponent of the independent variable) and the similarity between equations derived from different data sets, relating to different stream types. From the theory of linear models, the following inequality expression can be used to develop the confidence region (modified from the matrix format given by Myers, 1990, p. 48):

$$n\mathrm{a}_*^2 + 2\sum_{\mathrm{i}=1}^{n} \ln\left(X_\mathrm{i}\right)\mathrm{a}_*\mathrm{b}_* + \sum_{\mathrm{i}=1}^{n} \left[\ln\left(X_\mathrm{i}\right)\right]^2 \mathrm{b}_* - 2s^2 F_{p,2,n-2} \leq 0 \tag{3.27}$$

where X_i are the measured *X* values, *n* is the sample size of the data set, '*s*' is the error standard deviation given in Equation 3.5, $F_{p,2,n-2}$ is the upper tail *p*th percentage point of the *F*-distribution with 2 and *n*-2 degrees of freedom and a_* and b_* are given by

$$\mathrm{a}_* = \ln\left(\mathrm{a}_p\right) - \ln\left(\mathrm{a}\right) \tag{3.28}$$

$$\mathrm{b}_* = \mathrm{b}_p - \mathrm{b} \tag{3.29}$$

where $\ln(a_p)$ and b_p are the regression coefficients on the boundary of the region at the 100(1-p) percent confidence limit and ln(a) and b are the coefficients in Equation 3.2 that are always centred in the middle of the region. Equation 3.27 is a quadratic, which facilitates constructing the ellipse. For equations without logarithmic transformed variables, Equations 3.27 and 3.28 are modified accordingly (required in Chapter 7). When the ellipses from two data sets significantly overlap at a given level of confidence, the two data sets are very similar and are likely to be derived from the same population. Conversely, when the ellipses are isolated from each other, it can be deduced that the data sets are derived from different populations.

In the fixed exponent case or with univariate data (a sample of a single parameter), data sets can be compared using the conventional Analysis of Variance, F-test. This is used in Chapter 7 for comparing different types of meander bend in terms of their width variability.

3.5 THE DESIGN LANDSCAPE

The channel restoration design procedure and methods developed in this report are suitable for a specific type of river system conforming to the following criteria:

i) Self-formed alluvial rivers which have been modified by an extrinsic influence, such as channelisation. Bedrock channels are not considered as their forms are governed by lithological constraints and structural influences, rather than instream processes;

ii) Rivers which have previously possessed a sinuous or meandering planform. The channels of interest in this study have sinuosities greater than 1.2, above which there is a discernible difference between straight and sinuous channels. 1.1 is usually considered to be straight (e.g. Chitale, 1973). Braided channels are not considered as process-form relationships in braided systems and are very poorly understood;

iii) Rivers which exhibit instability as a direct consequence of previous channel modification(s).

iv) Lowland rivers. The reaction to disequilibrium between sediment input and sediment transport capacity in lowland streams is generally a slow process (Jaeggi, 1993, p. 10) and full restoration may be an appropriate management solution. Conversely, upland rivers are generally self-adjusting (Hey, 1990, p. 339) and less disturbed by anthropogenic influences. Where upland river management is necessary, alternative techniques such as limitation, mitigation or small-scale enhancements should be considered rather than full restoration.

Where the river has previously incised and has not significantly recovered a stable configuration, a restored channel can be designed within the lowered valley (Hey, 1994a, pp. 346-347). If the channel is allowed to recover naturally then a new floodplain will develop, after a considerable time period, below the former floodplain which would be left as a terrace feature. When designing a stable channel within an incised valley, the engineered floodplain width must be able to accommodate the meander belt. This may require considerable excavation and may not be an economic solution.

3.6 THE DESIGN PROCEDURE

As demonstrated in Chapter 2, a design solution can be found by deriving channel dimensions from a reference channel (or channels), from hydraulic geometry analyses, using analytical approaches, or through a combination of methods. In light of the geomorphological principles discussed in this chapter and a comprehensive review of existing techniques (Chapter 2), an enhanced design procedure for restoring stable channel dimensions has been identified (Figure 3.6) which is intended to represent ‘best practice’ guidance. The procedure requires a range of different techniques including: field reconnaissance, detailed site survey, magnitude-frequency analysis, analytical solution of non-linear equations and hydraulic geometry analysis. Until three-dimensional numerical modelling can accurately replicate the intricate form of natural river channels, the procedure provides an appropriate solution to bridge the divide between reconnaissance level geomorphological designs at one extreme and numerical modelling of hydrodynamics, sediment transport and morphological change at the other.

The procedure assumes that a stable reference reach (or preferably reaches) can be identified and used to supply essential baseline information, such as the magnitude and frequency of sediment-transporting flow events and a channel-forming discharge suitable to base the restored design. If a catchment baseline survey reveals that the majority of the catchment is highly unstable, then restoration of an individual reach to a stable (natural) configuration will usually not be sustainable within the system without a considerable maintenance commitment. In most cases, it is unlikely that such a commitment will be feasible or economically viable. However, in many disturbed catchments, especially lowland streams that have been enlarged, sediment continuity recovers faster than the channel dimensions, so aggradation and degradation are insignificant over periods of decades. In these cases, restoration of a more natural channel configuration using a design procedure based on sediment continuity may still present a viable management solution with a low maintenance commitment.

It is important to note that restoration can never proceed in a vacuum. The design procedure here must be supported by the results of earlier strategic studies that set the catchment context for the restoration scheme. The design procedure is structured as four sequential stages: Supply Reach Assessment; Project Reach Assessment; Channel Design, and Final Design Brief:

i) ***Supply Reach Assessment***

This assessment should follow from a strategic baseline survey of the catchment system, which has already identified reference reaches and classified the entire system in terms of potential destabilising phenomena and geomorphic conservation value (Section 3.1). Stream reconnaissance methods should have been used to provide classificatory data on boundary materials and riparian vegetation and semi-quantitative information on the characteristic channel morphologies found throughout the catchment. Further details of these strategic studies are not given here as they are well documented elsewhere (e.g. Simon and Downs, 1995; Thorne, 1998; FISRWG, 1998). Bed material samples should be collected at reference reaches to provide particle size gradations of the supply sediment.

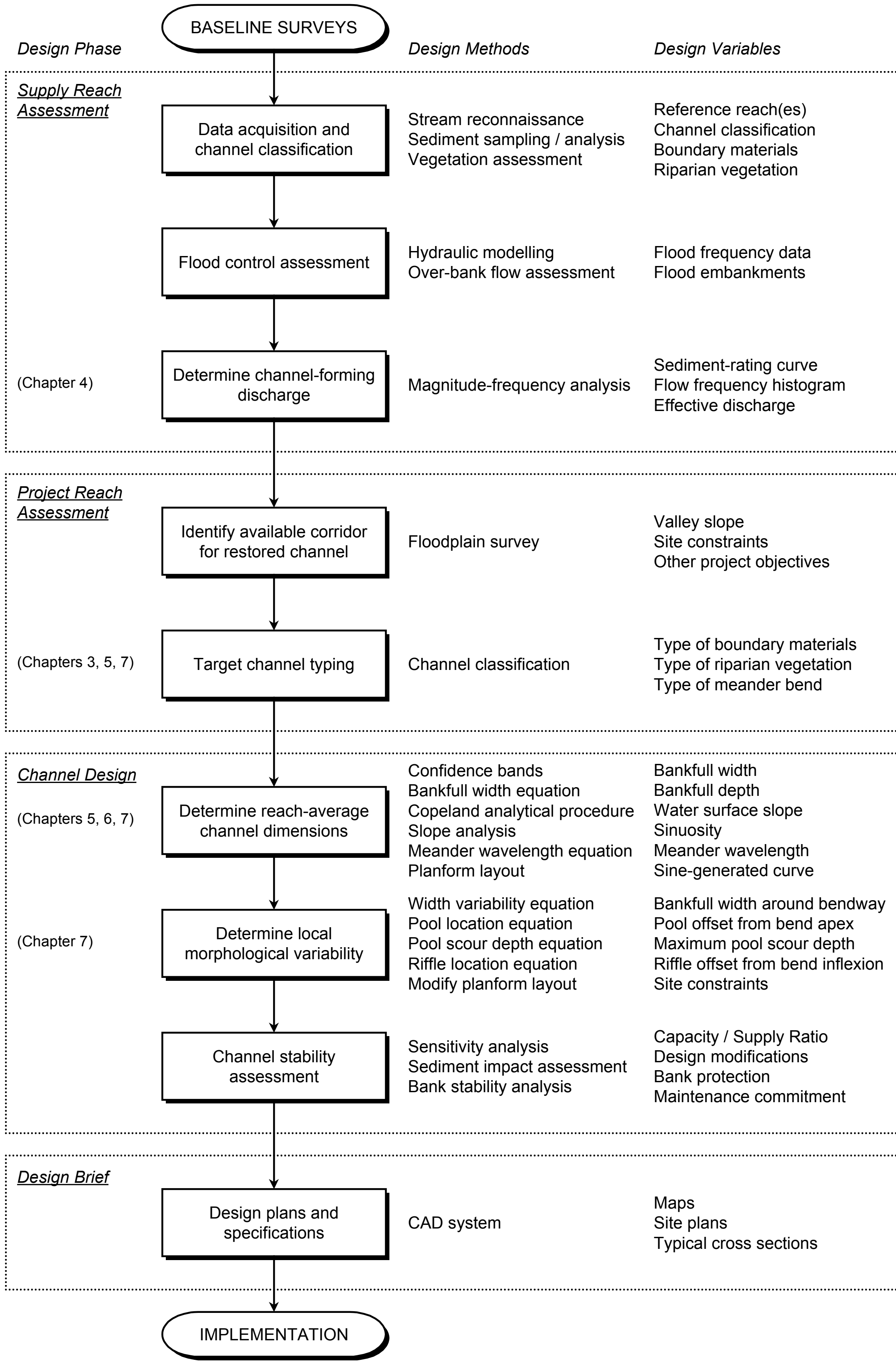


Figure 3.6 Best practice channel restoration design procedure.

In many cases, the preferred channel design solution will involve a two-stage channel (or a channel with set-back flood embankments) with a primary channel designed to convey the natural bankfull discharge and an over-bank zone designed to convey additional water during floods up to a specified recurrence interval (based on partial duration or annual maximum series). The specification of flood channels usually relies on hydrodynamic modelling which is beyond the scope of this study.

The channel forming discharge is the main driving variable for channel restoration design, however identification of bankfull stage from field indicators has been shown to be problematic and subjective (Williams, 1978a). On this basis, the effective discharge, determined from magnitude-frequency analysis, provides a more objective measure of the channel-forming flow. The effective discharge is the flow which transports the most sediment over a period of years (Andrews, 1980) based on flow frequency data and a sediment-rating curve(s). Bankfull, effective and channel-forming discharges are often assumed to be equivalent. The channel-forming flow is discussed in Chapter 4, together with best practice calculation procedures, and is revisited in Chapter 5 where this equivalence is examined further in light of new findings. From a geomorphological perspective, magnitude-frequency analysis provides important information on the distribution of sediment-transporting events which collectively influence channel stability. In a sediment impact assessment, this distribution is used at the end of the procedure to calculate the CSR and examine the efficiency of the restored channel configuration and resultant net stability.

Project Reach Assessment

This assessment requires a survey of the available right-of-way (land take) for laying out the restored meander planform. The available right-of-way is influenced by site constraints, such as floodplain constrictions and existing structures, and other project objectives which require utilising floodplain area. The valley slope should be measured from topographic maps.

The reference data obtained during the supply reach assessment should be used to determine the target channel type, in terms of boundary materials, riparian vegetation and meander pattern. In many cases, the type and density of bank vegetation will be different from that present in the reference reaches due to ecological, aesthetic and recreational objectives. It is imperative that target vegetation is identified prior to channel design as it influences flow resistance otherwise the stability status of the channel could be affected. The type of target meandering should be identified using the tripartite system discussed in Section 3.4.1, consisting of: i) equiwidth meandering; ii) meandering with point bars, and; iii) meandering with point bars and chute channels.

Channel Design

There are generally three stages in designing the geometry of a stable meandering channel:

i) Reach average dimensions and layout (bankfull width, bankfull depth, bed slope, sinuosity, wavelength and regular meander path);

ii) Local morphological variability around meander bendways (including variable width, location of pools and riffles, maximum scour depth in pools and adjustments to the layout to account for natural variability and site constraints);

iii) Fine-tuning of the initial design, based on channel stability assessment using the CSR.

Bankfull width is the diagnostic parameter with least uncertainty in estimation (Dunne and Leopold, 1978) and is the most consistent parameter in that it is highly correlated with discharge in hydraulic geometry analysis. Depth and slope estimates based on hydraulic geometry are less accurate as sediment inputs are very significant. Therefore, width-discharge equations usually yield the greatest R^2 values. On this basis, it is recommended that a range of widths within confidence limits should be calculated from a hydraulic geometry equation appropriate to the type of target channel (Chapter 5), while depth and average bed slope (equals water surface slope in one-dimensional analysis) should be determined from process-based equations which account for sediment transport. Analytical solution of flow resistance and sediment transport equations provides a possible design solution to fulfil this requirement. The preferred technique is the Copeland analytical regime method (Copeland, 1994) which involves the simultaneous solution of flow resistance and sediment transport equations for the range of stable width, to yield a range of stable depth and slope. This method is a component of the hydraulic design package, SAM, (Thomas et al., 1996) and is discussed further in Chapter 6, where the method is developed further for greater applicability.

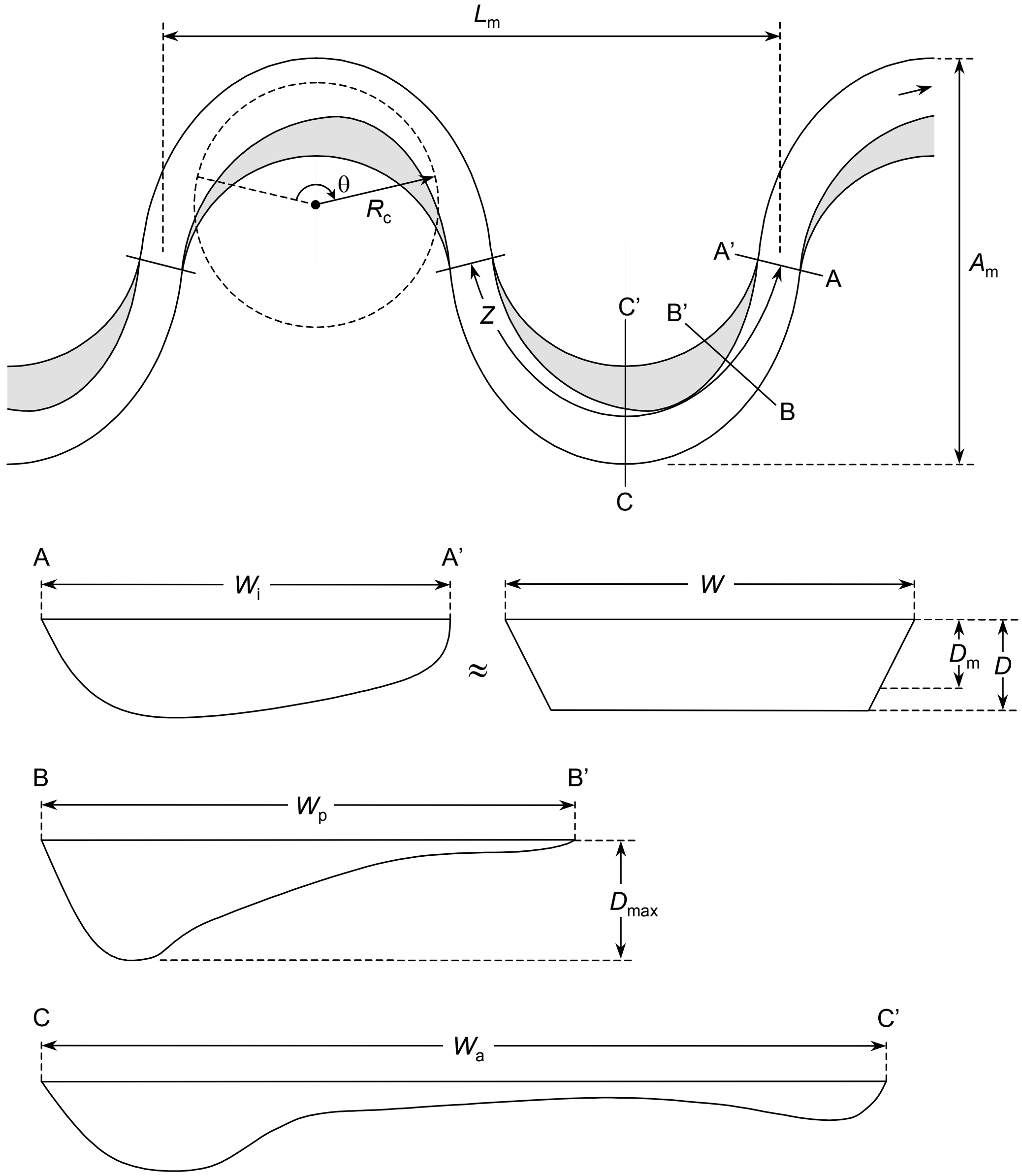
Once a stable bed slope has been determined, sinuosity is defined as the ratio of valley gradient to channel bed slope. Hey (1976) showed that for a given sinuosity an infinite number of meander patterns are possible and a determinate solution requires estimation of the meander wavelength. As wavelength is closely associated with width and only indirectly associated with discharge (Leopold and Wolman, 1957, see Chapter 2), it is recommended that a morphological relationship expressing wavelength as a function of width should be used in restoration design (Chapter 7). A range of stable wavelengths should be determined within confidence limits to account for natural variability. Although there are a wealth of equations in the literature (e.g., Williams 1986) which predict other planform variables (arc length, radius of curvature, arc angle and meander belt width), they are redundant once wavelength and sinuosity have been determined since only these two parameters are necessary to layout a regular meander pattern.

There is currently little design guidance for laying out the planform geometry of meandering channels. Detailed studies of meander bend patterns have revealed that regular meander paths are very rare in nature because most rivers exhibit considerable variability in meander form and orientation. In particular, simple geometric alignments fail to account for the downstream asymmetry in meander bends that is an essential feature in natural migrating rivers (Ferguson, 1973a; Carson and Lapoint, 1983). However, on a reach-scale level, a regular meander shape is a reasonable average condition from which to design a proto-channel alignment. The preferred geometric shape is the sine-generated curve proposed by Langbein and Leopold (1966) as the model is based on energy principles and accounts for the fact that meandering rivers often exhibit straight reaches in between bendways that are not provided by circular or parabolic curves. From the sine-generated curve, sinuosity and wavelength, the radius of curvature and required meander belt width can be determined. The sine-generated curve should be interpreted as a template or reach-average configuration and not a fixed solution for successive meander bends. In extreme cases, where the floodplain is very constricted, the desired meander belt width will exceed the available right of way and grade control structures should be implemented to reduce the energy slope, thereby allowing a reduction in the design sinuosity. However, grade control structures are difficult to design, implement and maintain (Watson et al., 1999) and so this solution should only be adopted when floodplain constraints dictate it.

Following reach-average channel design, modifications should be made to the cross section template and regular meander path to account for local morphological variability. Meander planform parameters are defined in Figure 3.7, together with cross sections at three significant locations around a meander bendway: i) the meander inflexion point with width, W_i; ii) the maximum scour location with width, W_p, and maximum scour depth, D_{max}, and; iii) the meander bend apex with width, W_a. These cross sections are shaped by the relative magnitudes of downstream and transverse velocities.

The bend apex (C-C') is very wide and characterised by a thalweg channel adjacent to the outer bank and often a minor secondary channel(s) or chute channel(s) across the inner bank which become significant at high stage flows when the alignment of the maximum velocity filament straightens. An emerging point bar covers a significant portion of the cross section where the contribution to downstream flow is negligible. The cross section at the maximum scour location (B-B') is essentially triangular in shape with a pool adjacent to a steep outer bank. Secondary circulation may also include a small cell of reverse circulation next to the outer bank (Bathurst et al., 1979) and flow along the inner bank above the pointbar crest is in an outward direction due to a progressive downstream decrease in depth along the point bar (Dietrich and Smith, 1983). While the bankfull width at this location is usually wider than at the inflexion point, the width between outer bank and point bar crest, where flow is in the downstream direction and associated with flow convergence, is less than the width at the inflexion point, where flow lines diverge.

Downstream from the pool, the width continues to decrease and velocities increase for a short distance, as the point bar is skewed in the downstream direction (Figure 3.7), before widening as the next bend is approached. The inflexion point (A-A') is characterised by decayed secondary circulation and relatively uniform depth, although it may exhibit a degree of asymmetry as the core of maximum velocity tends to cross the channel (the



Note: point bars are defined by shaded regions; L_m = meander wavelength, Z = meander arc length (riffle spacing); A_m = meander belt width, R_c = radius of curvature; θ = meander arc angle; W = reach average bankfull width; D = depth of trapezoidal cross section; D_m = mean depth (cross-sectional area / W); D_{max} = maximum scour depth in bendway pool; W_i = width at meander inflexion point; W_p = width at maximum scour location; W_a = width at meander bend apex.

Figure 3.7 Meander planform and cross section dimensions for restoration design.

crossing) at a variable distance between the inflexion and the entrance to the next bend. This is in accordance with flume and field observations (Leopold and Wolman, 1960, p. 779). If coarse material is found in the bed, a riffle feature often defines this crossover.

For the purpose of channel design, the cross section at the inflexion approximates a trapezoidal shape (Figure 3.7). Assuming this equivalence, the channel at the inflexion resembles the geometry of a straight channel.

Therefore, to restore the dynamic balance between meander form and process, the variability in width between these cross sections should be specified in the design drawings of a restoration project. In an active meandering channel, the location of the bendway pools and riffles in relation to the bend apex and meander inflexion points should be determined. Local variability around meander bendways is examined empirically in Chapter 7.

The likely maximum scour depth in the bendway pools should also be specified to check the mass stability of the outer bank and recommend bank protection where necessary. Localised bank retreat may not be permissible because of right-of-way constraints or other factors. The maximum scour depth also facilitates the design of asymmetric cross sections at pool locations. If this is not achieved, significant channel change should be expected as the river attempts to recover a balance between downstream and transverse sediment transport. Furthermore, the shallow zone above the point bar has important ecological value, for example as refugia habitat for fry during high stage flows, and conversely the pool is a refuge during low flows. Existing procedures are available for estimating the near-bank scour depth as a function of approach depth, width-to-depth ratio and radius of curvature-to-width ratio. Design guidance on scour depth is given in Chapter 7.

The final stage of the channel design is a channel stability assessment. Bank stability charts expressing critical bank height as a function of bank angle and sediment properties (e.g. Chen, 1975, based on work by Carson and Kirkby, 1972) should be consulted to investigate the stability status of the bank-lines. If required, there are numerous bank protection methods available to the engineer. These methods are beyond the scope of this study and are well documented elsewhere (e.g. FISRWG, 1998; Environment Agency, 1999). Following guidelines given in Section 3.2, a sediment impact assessment is required at the end of the design procedure to: i) validate the efficacy of the restored channel geometry; ii) identify flows which may cause aggradation or degradation over the short term, and; iii) recommend minor adjustments to the channel design to ensure that dynamic stability will be ensured over the medium- to long-term. The assessment involves calculation of the CSR and where necessary, should be used to refine the initial design configuration, thereby bringing the CSR closer to unity and improving potential stability. Achieving an optimum CSR, within 10 percent of unity, should ensure dynamic stability while allowing the river itself to recover some of the fluvial detail that cannot be engineered. If this cannot be achieved by adjusting the design parameters within confidence limits, it may be possible to delicately adjust the slope until the CSR is within the optimum range.

Finally, a sensitivity analysis on discharge and sediment load should be undertaken to examine the potential sensitivity of the restored channel to changes in flow regime and/or catchment sediment inputs. In catchments with predicted or projected changes in land use, results from the sensitivity testing could indicate appropriate levels of post-project monitoring and maintenance.

Design Brief

To complete the design procedure, engineering drawings of the planform and typical cross sections should be produced using the appropriate Computer Aided Design (CAD) system, for use by site engineer and construction contractor.

3.7 SUMMARY: GEOMORPHIC ENGINEERING FRAMEWORK

This chapter has demonstrated the importance of geomorphological principles in channel restoration design and how they should be addressed in the design procedure outlined in the previous section. It is imperative that river restoration is not undertaken in isolation of the larger system, the catchment, within which the river belongs. This necessitates a catchment based approach to the problem that requires knowledge of both flow and sediment routing in the upstream supply reach (or reaches), through the restored channel and into the existing channel downstream. Therefore, stability of the restored channel can only be ensured by using an approach that is based on sediment continuity principles. Therefore, the Supply Reach Assessment, during which the input parameters are quantified, and estimation of the CSR at the end of the procedure are recognised as the most important stages in the design procedure.

Confidence bands applied to hydraulic geometry equations have been shown to provide a mechanism through which natural rivers can be used as realistic analogues for channel restoration design. Imitating natural systems variability is a central component of the geomorphic engineering approach as each river is in detail unique and there is a range of stable design solutions, other than the regime condition. A significant portion of this chapter is concerned with the development of standard equations that allow uncertainty to be accommodated into design specifications. Ultimately the geomorphic engineering approach recognises that the river itself is the best restorer of its natural morphology (as suggested in Chapter 2), and should be allowed to participate in its own recovery. This can be accomplished by designing an approximate channel mould based on the broad cross-sectional and planform dimensions and then allowing the river itself develop the intricate morphological detail, provided that the CSR is close to unity.

Real world projects require practical solutions that must be based on sound engineering methods and geomorphological principles. However, river restoration and channel restoration design are site-specific issues. All projects are different and should not attempt to follow a prescribed series of definitive stages in any mindless manner. There are no ‘cookbook’ solutions and to prevent inappropriate and poorly designed river management, it is essential that the designer is attentive to detail, seeking expert guidance whenever a problem becomes apparent. The procedure is not a panacea and in many cases an alternative approach will be more appropriate. The procedure is not designed to be a substitute for professional experience but is aimed at providing a design framework within which the sound judgement of practitioners with experience in river management may be applied.

Through the development of the design procedure, it has been shown that the determination of the channel-forming discharge during the Supply Reach Assessment is a critical stage in the procedure, from which all estimates of stable channel dimensions follow. Incorrectly estimating the channel-forming discharge may therefore lead to significant instability in the restored reach. In the following chapter, the different methods of defining the channel-forming discharge are compared before presenting a standardised procedure for its calculation based on the conventional method of Magnitude-Frequency Analysis. However, through a detailed discussion of the various components in the analysis, it becomes apparent that the method should be revised to better represent the full distribution of sediment transporting flow events. In light of this and evolving out of the conventional method, a quasi-event-based approach to Magnitude-Frequency Analysis is presented at the end of the chapter as an approach with the potential for wide-ranging applications in fluvial geomorphology and river engineering.

CHAPTER 4

The Channel-Forming Flow

4.1 INTRODUCTION: CONCEPT AND THEORY

The role of the geomorphologist in river management is to recognise the delicately adjusted tripartite balance between: i) the driving, or ‘formative’, variables that shape a naturally variable channel morphology; ii) the mechanistic processes that fuel these variables, and; iii) the dynamism and variability of the river course within the catchment system. The natural spectrum and sequence of flow events are largely responsible for regulating this balance over time and moulding dynamically stable channel forms through sediment erosion, transportation and deposition.

Discharge varies between seasons in the water year and the annual distribution of flows varies on a year-to-year basis according to the frequency and duration of flood events. Alluvial rivers have the potential to adjust their shape and dimensions to all flows in this distribution which transport sediment (Lane, 1955). While regime theory is appropriate for channels with negligible flow variability, whereby a single, *steady* discharge (usually the full supply discharge) can be used to describe the size and shape of canal morphology, an alternative theory is required for river channels with *variable* flow regimes, dictated by the hydrology of the river catchment rather than regulation. In the 1940s and 1950s, engineers were faced with the problem of how to translate one-dimensional, process-form relationships into a variable discharge system for investigating the geomorphology and mechanics of alluvial rivers. ‘Channel-forming’ flow theory argues that there is a unique flow which, over a prolonged period, would theoretically yield the same bankfull morphology that is shaped by the natural sequence of flows (early references include Inglis, 1941, 1947, 1949b; Blench 1952, 1957; Ackers and Charlton, 1970a, 1970b; Hey 1975; Bray, 1975). This unique flow is often termed the ‘dominant discharge’ (after Inglis, 1941, 1947):

> ‘At this discharge, equilibrium is most closely approached and the tendency to change is least. This condition may be regarded as the integrated effect of all varying conditions over a long period of time.’ (Inglis, 1947, p. 6).

If the underlying principle of time-event compression can be accepted in applied geomorphology, then the channel-forming flow is an attractive simplification and has wide application potential (Thorne et al., 1998; Biedenharn et al., in preparation). Its uses include channel stability assessment, river management using hydraulic geometry relationships and stable channel design.

In humid and temperate environments, perennial rivers often recover their equilibrium following a major event over a period of about 10-20 years, partly because rapid vegetation growth encourages sedimentation (Hack and Goodlett, 1960; Gupta and Fox, 1974). The recovery process in the ephemeral channels of semi-arid regions tends to be longer, reflecting the influence of relatively wet and dry periods on vegetation growth (Schumm and Lichty, 1963; Burkham, 1972). In very arid areas infrequent floods impart lasting imprints on the channel as lesser events do not have the work potential to restore a regime condition (Schick, 1974). The concept of a channel-forming discharge is closely related to the concept of dynamic-equilibrium, which is characterised by fluctuations of channel form around an average condition through time. For ephemeral rivers that exhibit highly variable flow regimes, the notion that there may be a single discharge that can explain channel form is less applicable (Stevens et al., 1975; Baker, 1977). This is the case because the channel morphology is likely to be in *disequilibrium* with the prevailing flows rather than fluctuating around an average state. Therefore, in arid environments an alternative approach is required to describe the shape and size of channels where high magnitude, low-frequency flows may control the overall capacity, while frequent, minor flow events, which underfit this capacity, are largely responsible for defining instream sedimentary features.

The channel-forming flow or ‘dominant discharge’ is a geomorphological *concept* and not a measurable parameter *per se*. However, there are three definable flows which have been taken to represent the ‘dominant’ flow based on the application of repeatable geomorphological and hydrological techniques: i) bankfull discharge; ii) flow of a specified recurrence interval, and; iii) effective discharge.

4.2 DOMINANT DISCHARGE APPROACHES AND THEIR EQUIVALENCE

Based on field observations, Inglis (1947) considered that flows at or near the bankfull stage might approximate the dominant discharge. Further investigations in the 1950s and 1960s documented the consistency in the frequency of bankfull discharge for rivers with active floodplains (Wolman and Leopold, 1957; Dury, 1959, 1961; Dury et al., 1961; Leopold et al., 1964; Woodyer, 1968). The equivalence of bankfull and dominant discharges has since been supported by research into the process-form relationships of hydraulic geometry (Nixon, 1959; Simons and Albertson, 1960; Kellerhalls, 1967; Charlton et al., 1978; Hey 1975, 1982; Hey and Thorne, 1986; and others) and laboratory studies of shallow overbank flows (Ackers, 1992; James and Brown, 1977; Ackers and Charlton, 1970a, b; and others). For example, Ackers and Charlton (1970b) studied the influence of variable flows on meandering planforms for scaled down annual hydrographs and concluded that bankfull flow was responsible for generating the observed dynamic planform geometry, providing that the channel is not entrenched to the extent that bankfull flow does not occur at least annually. However, research into hydraulic geometry has not elucidated *why* 'bankfull' discharge should control the average channel morphology.

Williams (1978a) presented a detailed review of the bankfull discharge condition, including ten definitions based on sedimentary features, cross-sectional morphology and changes in bank vegetation. In a natural river the most appropriate definition is the discharge conveyed at the elevation of the *active* floodplain (after Wolman and Leopold, 1957; Dury, 1961; Emmett, 1972, 1975; Williams, 1978a; Andrews, 1980, 1984; Nolan et al., 1987; Hey and Thorne, 1986; and others). In practice, accurate location of bankfull indicators is not a routine procedure and is therefore problematic (Williams, 1978a) and most methods are highly subjective. A frequently applied method is to identify the width at the minimum width-to-depth ratio (Wolman, 1955). Once a bankfull elevation has been identified, its associated discharge can be determined by several techniques: direct gauging; using a stage-discharge rating curve from a nearby gauge; applying at-a-station hydraulic geometry equations; applying downstream hydraulic geometry equations, or; using appropriate flow resistance equations which can synthesise a stage-discharge curve (see Williams, 1978a, pp. 1152-1153). Hey (1978) stressed the use of bankfull discharge for design purposes because of its 'morphogenetic' significance.

Numerous studies have attempted to determine the frequency of occurrence of the dominant discharge, usually expressed in terms of recurrence interval in the annual maximum series. Nixon (1959, p. 159) remarked:

> 'From observations it is clear that rivers will remain dormant for long periods of time during low flow conditions and only become active when there is a higher than normal flow. As the process of channel adjustment is purely mechanical the determining factors must persist for sufficient time for this adjustment to take place and stability to be reached. The 'dominant' discharge, or whatever discharge determines the river channel shape, must occur often enough to permit the channel to reach regime'.

Based on data from twenty four American rivers, Wolman and Leopold (1957) found that the recurrence interval for bankfull flow in natural rivers with well developed floodplains ranged between 1.0 and 5.0 years, using the annual maximum flood series. A recurrence interval of 1.5 years was considered as a suitable average frequency. The 1.5-year flood, $Q_{1.5}$, was later confirmed as a suitable dominant discharge frequency from further U.S. data (Leopold et al., 1964; Leopold, 1994) and as the bankfull flow for gravel-bed rivers in the U.K. (Hey, 1975) using the annual maximum flood series. This flood frequency closely approximates the most-probable (modal) annual flood with a recurrence interval of 1.58 years which was advocated by Dury (1973, 1976) and Riley (1976) as the dominant flow.

Dury (1973) demonstrated from American river data that the bankfull discharge is 97 percent of the modal discharge in the annual maximum series. However, there is considerable scatter around this modal value and Woodyer (1968) found that bankfull discharge may have a frequency between 1.02 and 2.69 years in the annual maximum series, depending upon the degree of incision. This range was later expanded considerably by Williams (1978a) who found that only one third of thirty-six cases examined had recurrence intervals near the 1.5 year peak, the range being between 1.01 and 32 years, and concluded that a *range* of frequencies rather than a *single* value is more appropriate for rivers with an active floodplain. Further evidence supporting a range of bankfull discharge frequencies was presented by Pickup and Warner (1976), who demonstrated that bankfull recurrence intervals may range from four to ten years in the annual maximum series and Andrews (1980) documented 50 percent of study sites in the Yampa River basin of Colorado and Wyoming as having bankfull discharge recurrence intervals greater than 1.75 years and less than 1.25 years, the range being from 1.18 to 3.26 years. This variability was attributed to differences in climatic, geological and physiographic factors.

The Corps of Engineers manual on channel stability assessment (USACE, 1994) recommends that on average the channel-forming discharge has a recurrence interval of approximately 2 years, Q_2, but may be found between the 1- and 10-year flood flows. The 2-year flow event was also found to be a suitable surrogate for bankfull discharge in downstream hydraulic geometry relationships developed by Bray (1973, 1982) based on gravel-bed data from 71 Alberta streams presented by Kellerhalls et al. (1972). Bray (1975, p. 143) considered the dominant discharge to be the discharge which, when flowing continuously, would result in the water-surface width and the cross-sectional area of a relatively stable natural channel. This discharge was estimated by Bray from the flow providing the best correlation with water-surface width and cross-sectional area. Arguably, this statistical technique is based on the theory of minimum variance (after Langbein and Leopold, 1966), whereby the most probable, hence stable, channel configuration is the one with least variance. As variance in hydraulic geometry is measured by the scatter of observations either side of the best-fit line, stable values of width and cross-sectional area may be derived from the flow which results in the lowest variability of data points. However, although the correlation was strongest for the 2-year flow, the coefficient of determination, R^2, was not significantly different for the other discharge frequencies used in the analysis, hence the 2-year flow was not clearly defined as the 'dominant' event. Despite this degree of uncertainty, the 2-year flow was considered as representative of the dominant discharge because the bed was in motion at or near this flow, the median discharge in a log-normal analysis may be approximated by the 2-year flow, and the number of reaches with stages greater than bankfull would have substantially increased if a less frequent flood flow was used in the analysis.

From an analysis of partial duration series for 14 gravel-bed rivers in England and Wales, Hey and Heritage (1988) revealed that the modal value of bankfull discharge recurrence intervals was 0.9 years with a range of 0.56 to 3.44 years, although Brush (1961), while working in Pennsylvania, showed that the frequency of bankfull discharge in the partial duration series may be as great as 10 years. A different approach was adopted by Nixon (1959) who used flow-duration curves, rather than instantaneous flow data, from 29 rivers in England and Wales, and demonstrated that the bankfull discharge was equalled or exceeded on average 0.6 percent of the time (slightly greater than 2 days per year duration).

In summary, there is not a general consensus regarding the modal recurrence interval for the bankfull, or dominant, discharge, with considerable scatter in the data. It is generally considered among practitioners that the modal value of the bankfull discharge frequency for perennial rivers in humid environments is likely to fall in the range of one to two years, using an annual maximum flood series (Kilpatrick and Barnes, 1994; Leopold et al., 1994; Andrews, 1980; Carlston, 1965; Leopold, 1994). This variability is partly attributable to the various definitions of bankfull discharge and different characteristics in flow regime. For example, Harvey (1969) suggested that flashy streams flood more frequently in the headwater zone, while base flow dominated streams flood more frequently in the lowland zone where floodplains are well developed.

Consideration of a river's sediment budget is required to explain why bankfull discharge controls channel morphology. Any local imbalance in the sediment regime must generate channel change via erosion or deposition in an attempt to attain a new state of equilibrium, or recover the pre-disturbance condition. Therefore, the dimensions of a dynamically stable river must be delicately adjusted to the sediment balance (Mackin, 1948), so that over the medium- to long-term, sediment inputs and outputs are balanced. Wolman and Miller (1960) showed that rivers adjust their bankfull capacity during this time scale to the flow that transports the greatest quantity of sediment load over a period of years (usually the period of flow record), or the flow which expends the greatest geomorphic work per unit time. Andrews (1980) termed this flow the *effective discharge*, although *effectiveness* and the notion that the dominant discharge is an effective flow were used previously (e.g. Wolman and Miller, 1960, pp. 65-66; Pickup, 1976; Pickup and Warner, 1976; Wolman and Gerson, 1978). The effective discharge is derived from consideration of the magnitude and frequency of flows at a study site over a designated period of time.

4.3 MAGNITUDE-FREQUENCY ANALYSIS (MFA) AND THE EFFECTIVE DISCHARGE

Wolman and Miller (1960) found that both the magnitude of a sediment-transporting flow event and the frequency with which an event occurs are both significant factors in shaping and sizing a river channel. The effective discharge corresponded to an intermediate flood flow. This is because very frequent flows below the effective discharge only transport a minor fraction of the total sediment load and can be neglected from explanations of reach average morphology. Conversely, flow events above the effective discharge, while having the potential to transport a very high sediment load per event, occur too infrequently to be effective in shaping the channel boundary. Wolman (1959) demonstrated how intermediate events were responsible for controlling riverbank erosion and channel form of a small Maryland stream. Even though peak flows occur following summer storms, the greatest rate of measured bank retreat occurred during moderate, below bankfull, flows in winter which attack previously wetted banks and occur on average eight to ten times per year.

Although high magnitude-low frequency events can impart a marked change to the morphology in the short-term, in the medium- to long-term the long duration of lesser events may allow the river to recover its average morphological condition (Wolman and Gerson, 1978). This was demonstrated by Wolman and Miller (1960, p. 57), for rivers in western U.S.A., who found that approximately 90 percent of the total suspended load in alluvial rivers is transported by flows recurring more frequently than once every five years. This does not mean that catastrophic events do not have an influence on shaping fluvial landforms since they are responsible for gully development, rapid incision, avulsions and floodplain features such since alluvial fans (Wolman and Miller, 1960, p. 71). Furthermore, Harvey (1969) discussed how channel size appears to be related to much rarer events in base flow-dominated streams than in flood flow-dominated, or ‘normal’, streams because the bank fabric is often resistant enough to withstand flow stages up to and including bankfull.

Wolman and Miller (1960) found that *intermediate* flood flows are those responsible for transporting most sediment over a period of years in both perennial and ephemeral channels. The geomorphic effectiveness of the intermediate flow range was recently demonstrated by Costa and O’Connor (1995) based on stream power. However, the effective discharge is a function of the shape of the flow-frequency distribution and the form of the sediment rating relationship, so that an intermediate flood event is a highly likely effective discharge but not a generic condition in all cases. An effective discharge with a relatively low or high exceedance probability may be the result of a combination of factors. More extreme floods are likely to be the most effective in transporting sediment in rivers with varied flow regimes characterised by platykurtic distributions with high variance, often in impermeable catchments. The converse applies in base flow dominated streams with infrequent high magnitude events. Differences in sediment transport characteristics also influence the frequency of the effective discharge. In streams with high threshold discharges for incipient sediment motion (gravel- and cobble-bed streams) and/or if sediment throughput is capacity limited, the high magnitude events are likely to be more effective in shaping the channel boundary. Conversely, where sediment is mobilised at very low discharges (silt and fine sand-bed streams) and/or if there is an abundant sediment supply, the low magnitude, high frequency flows tend to be the effective flows (Hey, 1975).

Consequently it is not surprising that the frequency of reported effective discharges is highly variable. For example, Kircher (1981) calculated that the effective discharges of the North Platte, South Platte and Platte Rivers in Nebraska are exceeded on average between 1 and 30 percent of the time and Ashmore and Day (1988) found the range to be between 0.1 and 15 percent of the time for 12 sites in the Saskatchewan River basin, western Canada. In summary, the recurrence interval of the effective discharge differs between sites as different sites exhibit variations in the shape of their flow distributions and sediment rating relationships. Hence, the effective discharge is influenced by drainage area, drainage basin topography and geology, the temporal pattern of precipitation inputs and the nature of the sediment load (Ashmore and Day, 1988, p. 864). The frequency will decrease with increasing flow variability and/or increasing rate of change in sediment discharge with flow, such that more extreme flow events are effective in rivers with flashy flow regimes and supply limited transport and more frequent events are effective in base flow dominated rivers which are hydraulically controlled. In the latter case, the effect of increasing the critical discharge for incipient bed load transport in a gravel bed stream, tends to offset the influence of a low sediment rating curve exponent (Wolman and Miller, 1960; Andrews, 1980).

By coupling discharge and sediment transport, the flow which transports the greatest sediment load over a prolonged period is of particular interest to the applied geomorphologist and hydraulic engineer involved in understanding process-form relationships, such as downstream hydraulic geometry, and assessing stability and has been applied widely by: Schaffernak, 1922; Wolman and Miller, 1960; Benson and Thomas, 1966; Marlette and Walker, 1968; Prins and de Vries, 1971; Hey, 1975, 1997a; Pickup and Warner, 1976; Fisk, 1977; Andrews, 1980, 1984; Walling and Webb, 1982; Biedenharn et al., 1987; Nolan, Lisle and Kelsey, 1987; Ashmore and Day, 1988; Carling, 1988; Leopold, 1992; Lyons et al., 1992; Biedenharn and Thorne, 1994; Nash, 1994; Andrews and Nankervis, 1995; Watson et al., 1997; Goodwin et al., 1998; Doyle et al., 1999; Sichingabula, 1999; Soar et al., 1999; Tilleard, 1999; and others.

In theory, the effective discharge should equate to the bankfull discharge in a dynamically stable river. This can be explained in terms of energy considerations. By definition, for zero net aggradation or degradation over the medium- to long-term, the channel must change its shape and size to that just sufficient to accommodate the effective discharge. In a channel with a well-developed floodplain, to maximise energy efficiency the channel will convey the effective discharge at the greatest depth (minimising in-channel boundary shear stresses) and without overtopping (preventing loss of energy to floodplain vegetation resistance and lateral momentum). The effective discharge refers to a delicately adjusted state involving repeated erosion and deposition of sediment as the channel morphology is shaped and reshaped by destructive high magnitude events followed by periods of low flow which facilitate recovery. Therefore, the term *channel-forming discharge* would be better replaced by *channel forming-deforming discharge*, *morphology-regulating discharge* or *quasi-equilibrium discharge* (constantly fluctuating around an apparently stable state in the medium- to long-term).

The equivalence between effective and bankfull flows has been demonstrated for a range of river types and for a range of physiographic regions (Wolman and Miller, 1960; Leopold et al., 1964; Andrews, 1980; Knighton, 1984; Carling, 1988; Andrews and Nankervis, 1995; Hey, 1997a). Hey (1975) suggested that following a period of instability, the river would, over the long-term, adjust its bankfull shape and dimensions to the flow transporting the most sediment. From this statement it can be interpreted that dominant discharge, bankfull and effective discharge are interchangeable terms. Furthermore, Biedenharn et al. (1987) demonstrated close agreement between Q_2 and Q_e for the Mississippi River, Red River and Pearl River and Watson et al. (1997) confirmed this equivalence for ten Demonstration Erosion Control (DEC) study streams in northern Mississippi, with the 5-year recurrence interval flood providing an upper bound to the distribution of computed Q_e for most sites. More recently, Orndorff and Whiting (1999) calculated the recurrence interval of the Red River in Idaho as 1.46 years, which is very close to the average bankfull frequency suggested by Hey (1975).

Magnitude-frequency analysis is a useful geomorphic tool to facilitate predictions of the direction and magnitude of channel response to hydrologic change (Tilleard, 1999) and has been used as a mechanism to assess the restorative potential of rehabilitation schemes by comparing observed channel response (a function of flow events since project implementation) with the potential for morphological change, inferred from the full spectrum and range of flows in the long-term record (Downs et al., 1999).

4.4 CHANNEL-FORMING DISCHARGE FOR CHANNEL RESTORATION DESIGN

Although channel restoration design is associated with stable, bankfull dimensions, the difficulties in identifying the bankfull reference level inhibits the use of the bankfull discharge as the primary design variable in many cases, particularly in incised river systems. Furthermore, the target riparian corridor in the restored reach may be characteristically different to that in a reference reach. As width is strongly influenced by density of bank vegetation, measured bankfull width at a reference reach may not be stable in the restored reach. However, this should not rule out designing according to measured bankfull parameters if a stable reference reach can be found with similar bank conditions and the bankfull reference level clearly identified.

The dominant controls on channel form adjustment are discharge and sediment load. In a natural river, these independent variables integrate the effects of local climate, vegetation, soils, geology and overall basin physiography (Knighton, 1984, p. 87). Both parameters vary significantly over time and space. Thus, the application of an appropriate design discharge is critical for long-term channel stability. As the effective discharge is closely associated with the shape and dimensions of a dynamically stable river, it is paramount that the concept is employed as a central pillar in channel restoration design. This is also stressed in the conclusion by Doyle et al (1999) who demonstrated from three rivers in the U.S.A. that the effective discharge should be used in preference to other methods and concluded that the effective discharge is the most critical geomorphic and hydraulic parameter in channel design. As Tilleard (1999, p. 629) noted, the effective discharge concept provides a relationship between the hydrologic characteristics of the catchment, the hydraulic characteristics of the channel and the geomorphic characteristics of the project reach. Furthermore, with the increasing availability of flow records and computational capabilities, the effective discharge can be readily calculated following field reconnaissance during the early stages of project design.

A number of the morphological equations to determine bankfull width documented in the literature are based on the mean daily, or time-averaged, discharge (e.g. Leopold and Maddock, 1953; Leopold and Wolman, 1957; Leopold et al., 1964; Cherry et al. 1996). This discharge is frequently used because it is readily available and does not require the level of computation involved in determining the effective discharge, or the fieldwork required to identify bankfull discharge. However, these equations are purely statistical predictors and do not link form and process by causation as required in downstream hydraulic geometry relationships. The mean annual discharge of a perennial river is considerably less than bankfull discharge and usually fills the channel to approximately one third of the channel depth (Leopold, 1994, p. 129) with flow exceeded about 25 to 40 percent of the time (Dunne and Leopold, 1978, p. 620; Leopold, 1994, p. 44). Based on extensive fieldwork in eastern U.S.A., Leopold (1994, pp. 146-147) estimated the mean daily discharge as a percentage of bankfull discharge to be 3.4 percent for streams in California, 14 percent for streams in Colorado and 12 percent for all streams studied. However, although this flow has ‘no morphological significance’ (Leopold, 1994, p. 170) and should not be used to determine bankfull dimensions, it is particularly important from a hydrological and ecological perspective in terms of minimum flow requirements for habitat and species welfare.

In summary, there are three possible approaches to determining the channel-forming discharge: bankfull discharge; flow of a given recurrence interval, and; effective discharge. Ideally, the method used should have general applicability, the capability to be applied consistently, and integrate the physical processes responsible for determining the channel dimensions. Of the three possible approaches listed above, only the effective discharge has the potential to meet these requirements.

4.5 EFFECTIVE DISCHARGE CALCULATION

A standardised procedure is required for the purpose of supplying practitioners with practical guidance that has not previously been documented in the literature, despite the wide range of reported applications. The method based on MFA (after Wolman and Miller's 1960 model) is generally known throughout the engineering community as the 'integration of sediment transport with flow-duration'. Previous descriptions of the method have been poorly described. For example, the following extract describes the procedure used by Pickup and Warner (1976, p. 52) but does not provide detailed practical guidance for others to follow:

> 'The most effective discharge was determined by dividing the flow into small classes, finding the duration of flow within each class, calculating the mean bed-load discharge within the class and multiplying it by its duration. A histogram showing bed-load transport regime, i.e. the amount of load transported by each class, may then be constructed. The most effective discharge is taken as the mid-point of the class that transports the most bed-load.'

Furthermore, a standardised procedure is required so that effective discharges between sites may be comparable. The practical guidance must require only available data or limited additional information if it is to be used within the engineering and geomorphology communities and readily identified following site reconnaissance.

Calculation of the effective discharge requires a combination of empirical, statistical and mathematical methods (noted by Orndorff and Whiting, 1999, p. 559, as a 'hybridization of solution techniques'). Benson and Thomas (1966) were the first to show that the Wolman and Miller (1960) model could be specified from discrete data, based on increments of the discharge range, to yield a histogram of total load transported, as continuous flow data are not available. Following this methodology, two variations of the procedure are available based on different flow data formats: i) flow-frequency histogram method, which calculates frequency directly from unprocessed gauge data, and; ii) flow-duration curve method, which synthesises a flow-frequency distribution by first constructing a flow-duration curve. The procedures for effective discharge calculation presented here are intended to represent current 'best practice'. The methods are designed to have general applicability, have the capability to be applied consistently and to integrate the effects of physical processes responsible for determining the channel dimensions.

4.5.1 Compilation of Hydrological Data

4.5.1.1 Basic Principles

The range of flows experienced by the river during the period of record is divided into a number of classes, and then the total amount of sediment transported by each class is calculated. This is achieved by multiplying the frequency of occurrence of each flow class by the median sediment load for that flow class (Figure 4.1). The primary input data consist of: i) flow data, and; ii) a sediment transport rating relationship. The calculated value of the effective discharge depends to some extent on the steps used to manipulate the input data to define the flow regime and sediment transport function. At gauged sites, the first step is to group the discharge data into flow classes and determine the number of events occurring in each class during the period of record. This is accomplished by constructing a flow-frequency histogram, which is the empirical frequency distribution function of discharges measured at the gauging station. Four critical components must be considered when developing the flow-frequency histogram: i) the type of discharge interval scale; ii) the number of discharge class intervals, iii); the time base for discharge averaging, and; iv) the length of the period of record. These are examined in turn before presenting methods of synthesising a flow distribution for ungauged sites.

4.5.1.2 Type of Discharge Interval Scale

Previous investigations that have used MFA to calculate the effective discharge have used either an arithmetic discharge scale or logarithmic discharge scale to develop histograms of discharge and sediment load frequencies from measured data (Table 4.1). Class intervals of discharge are required because instantaneous flow measurements over time are unavailable. Instead, gauge records contain a sample of the real flow distribution

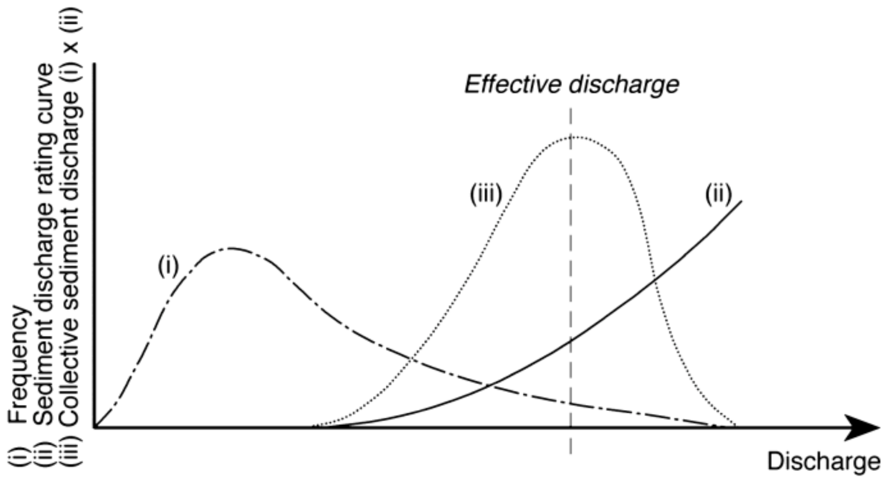


Figure 4.1 Derivation of bed material load-discharge histogram (iii) from flow-frequency (i) and bed material load rating curves (ii).

Reference	Class Interval	Number of Classes
Benson and Thomas (1966)	arithmetic	various
Pickup and Warner (1976)	arithmetic	not given
Andrews (1980)	arithmetic	20
Webb and Walling (1982)	arithmetic	23
Ashmore and Day (1988)	arithmetic	15-24**
Carling (1988)	arithmetic	7-8
Lyons et al. (1992)	arithmetic	35
Biedenharn and Thorne (1994)	arithmetic	50-54
Nash (1994)	logarithmic*	n/a
Hey (1997a)	arithmetic	25
Watson et al. (1997)	logarithmic	35
Goodwin et al. (1998)	arithmetic*	n/a
Thorne at al. (1998)	arithmetic or logarithmic***	25
Tilleard (1999)	arithmetic	not given
Soar et al. (1999)	logarithmic	23
Thorne at al. (1999)	arithmetic or logarithmic***	25
Sichingabula (1999)	arithmetic	20
Biedenharn et al. (in prep.)	arithmetic	25
Biedenharn et al. (submitted)	arithmetic	25

Note: *used theoretical probability distribution; **from example histograms presented only; ***recommended using logarithmic intervals if there are zero frequencies in the flow-frequency histogram or if the effective discharge falls within first class.

Table 4.1 Type of class interval and number of classes used to calculate effective discharge in a selection of studies.

(population) relative to a specific time interval (or time base), such as 15-minute data. Subsequently, the frequency of any discharge (say, to the nearest cumec) cannot directly be determined from a sample distribution but must be inferred from the frequency of a specific *range* of measured discharge. This section examines whether the effective discharge differs according to the method used to subdivide the range of recorded flows into classes and quantifies any likely error of estimation that may result.

Table 4.1 summarises the type of discharge scale and number of class intervals used in previous investigations and demonstrates that the majority of calculations have used an arithmetic discharge scale. Nash (1994) was the first to use a logarithmic discharge scale in an analytical definition of effective discharge. Logarithmic discharge classes were later used in an investigation by Watson et al. (1997), which used empirical data from northern Mississippi streams.

Wolman and Miller (1960, p. 56) stated that:

> 'if the stress [on the landscape (discharge)] is log-normally distributed and continuous and if the quantity or rate of movement [sediment load] is related to some power of this stress, then the relation between stress and the product of frequency times rate of movement must attain a maximum'.

This is the original concept of MFA and its application to determine the effective discharge. However, Wolman and Miller described a theoretical case of *continuous* discharge (stress) data and, therefore, did not discuss discharge class intervals. If discharge is log-normally distributed then it conforms to the log-normal probability density function (PDF) (as given by Chow, 1964, and Shahin et al., 1993) and the frequency of the logarithm of discharge, $f(\ln Q)$ is defined as

$$f(\ln Q) = \frac{1}{\beta\sqrt{2\pi}} e^{\frac{-(\ln Q - \alpha)}{2\beta^2}} \qquad (4.1)$$

where 'α' and 'β' are the mean and standard deviation of the natural logarithm of discharge, respectively. The log-normal distribution has been used to describe the distribution of many hydrological events, including stream flow (Chow, 1954; Krumbein, 1955; Kuczera, 1982). The use of this distribution function to describe flow-frequency was further supported by Leopold (1994, p. 112). If the sediment transport rate, Q_s, is expressed as a rating relationship in the form of a single, power function of discharge, then

$$Q_s = a\,Q^b \tag{4.2}$$

where ‘a’ and ‘b’ are empirically determined, the latter being sensitive to the availability of, and capacity to transport sediment (as discussed in previous section). If the ‘product of frequency times rate of movement’ in Wolman and Miller’s (1960) model is expressed as effectiveness, E, then

$$E = \frac{a\,Q^b}{\beta\sqrt{2\pi}}\,e^{\frac{-(\ln Q - \alpha)}{2\beta^2}} \tag{4.3}$$

which corresponds to curve ‘iii’ in Figure 4.1. As the effective discharge, Q_e, is a maximum condition, it can be expressed mathematically as the discharge at which the derivative of the product of magnitude, Q_s, and frequency, F, with respect to discharge, is zero, such that

$$\frac{dE}{dQ} = E\left[\frac{b}{Q} - \left(\frac{\ln Q - \alpha}{\beta^2 Q}\right)\right] = 0 \tag{4.4}$$

Rearranging this equation gives the effective discharge as

$$Q_e = e^{b\beta^2 + \alpha} \tag{4.5}$$

This analytical definition of effective discharge was also given by Nash (1994, p. 81) and reproduced by Watson et al. (1997, p. 37), who used logarithmic class intervals in their determination of the effective discharge in northern Mississippi streams. The fundamental assumption of Equation 4.5 is that for any discharge, the frequency of occurrence of that discharge, $f(Q)$, is the same as the frequency of occurrence of the logarithm of that discharge, $f(\ln Q)$.

As expected, Equation 4.5 indicates that the recurrence interval of the effective discharge increases with variability in the flow regime and the magnitude of the exponent ‘b’. The recurrence interval of the effective discharge must also increase with the critical discharge for sediment movement, yet this effect is not incorporated in Equation 4.5. Nash (1994) and Baker (1977) recommended that an apparent threshold can be determined by using a low value of ‘a’ and high value of ‘b’ in Equation 4.2. Thereby, increasing ‘b’ in Equation 4.5 can indirectly simulate the effect of a threshold.

It is hypothesised that the difference between the actual effective discharge and that calculated using MFA may be a result of two possible types of error (ignoring miscalculation error), both a function of the type of discharge class interval, arithmetic or logarithmic, adopted to transfer measured flow data into frequency data, such that

$$\text{total error} = f(\text{approximation error and/or misrepresentation error}) \qquad (4.6)$$

The definition of the effective discharge as given by Andrews (1980) suggests the existence of a *unique* flow event that is dominant in forming the channel. Approximation error is an inevitable result of statistical inference on some idealised discharge record with *continuous*, instantaneous sampling, based on *discontinuous*, measured flow data which are limited both in terms of the period of flow record and the discharge time base available (15-minute, hourly, or mean daily). Subsequently discrete frequency data corresponding to *ranges* (or intervals) of discharge are used to estimate a *unique* discharge, the effective discharge. Approximation error assumes that there is no misrepresentation, i.e., $f(Q)$ equals $f(\ln Q)$, as in the case of Nash (1994) and Watson et al. (1997). Misrepresentation occurs if the type of discharge class interval leads to a consistent bias (overestimation or underestimation) that cannot be explained in terms of approximation error alone.

Approximation Error

If it is assumed that given continuous flow data, arithmetic and logarithmic discharge scales both yield the same effective discharge, then arithmetic class intervals may give better definition in the intermediate to high discharge range but inadequate resolution at low discharges, which may be important in water quality and habitat investigations. A logarithmic scale overcomes this potential problem by significantly reducing the actual class size at low discharges. However, the range of discharge values in the larger class sizes can be enormous with a logarithmic scale and therefore, impossible to identify a specific effective discharge. This reservation was stressed by Hey (1997a, p. 12) who noted that as the flow-duration curves are generally log-normally distributed, it would be expected that equal logarithmic class intervals should be used. Therefore, Hey assumes $f(Q)$ equals $f(\ln Q)$, but preferred arithmetic intervals on the basis of lesser approximation error.

In a flow-frequency histogram, all discharges within any class have a constant frequency. For the purpose of MFA, this frequency is usually associated with either the arithmetic mean, if arithmetic class intervals are used, or the geometric mean, if logarithmic class intervals are used. Approximation error measures the possible inaccuracy in choosing the relevant mean value over some other discharge within the same class. Intuitively, approximation error increases as the size of the class interval increases. This tends to suggest that the likely inaccuracy will be greatest at high discharges for the logarithmic case, as stressed by Hey (1997a).

The error involved in approximating can be evaluated using two different definitions, termed here the 'effective error', based on percentage deviation from the actual effective discharge (therefore, dependent on sediment load), and the 'relative error', based on percentage deviation from a representative mean flow (independent of sediment load and a surrogate for channel capacity).

The absolute 'effective error' produced using arithmetic, $E_{\mathrm{e,a}}$, and logarithmic, $E_{\mathrm{e,l}}$, discharge classes maybe defined as

arithmetic classes
$$E_{\mathrm{e,a}} = 100\left|\frac{Q_{\mathrm{e,a}} - Q_{\mathrm{e}}}{Q_{\mathrm{e}}}\right|\% \qquad (4.7)$$

logarithmic classes
$$E_{\mathrm{e,l}} = 100\left|\frac{Q_{\mathrm{e,l}} - Q_{\mathrm{e}}}{Q_{\mathrm{e}}}\right|\% \qquad (4.8)$$

where 'Q_e' is the actual effective discharge, '$Q_{e,a}$' is the calculated effective discharge using arithmetic class intervals, and '$Q_{e,l}$' is the calculated effective discharge using logarithmic class intervals.

The absolute 'relative error' produced using arithmetic, $E_{r,a}$, and logarithmic, $E_{r,l}$, discharge classes are defined as

arithmetic classes $$E_{r,a} = 100\left|\frac{Q_{e,a} - Q_e}{Q_m}\right|\% \tag{4.9}$$

logarithmic classes $$E_{r,l} = 100\left|\frac{Q_{e,l} - Q_e}{Q_m}\right|\% \tag{4.10}$$

where 'Q_m' is a representative mean discharge in the flow record (arithmetic mean flow or geometric mean flow).

If the actual effective discharge, Q_e, has a magnitude at some fraction, p, of the arithmetic interval that contains Q_e, then the error in estimating the effective discharge using arithmetic-based MFA is given by

effective error $$E_{e,a} = 100\frac{i_a}{2}\left|\frac{1-2p}{pi_a + Q_{L,a}}\right|\% \tag{4.11}$$

relative error $$E_{r,a} = 100\frac{i_a}{Q_m}\left|\frac{1-2p}{2}\right|\% \tag{4.12}$$

where 'i_a' is the constant arithmetic interval, and '$Q_{L,a}$' is the lower bound discharge in the arithmetic interval that includes Q_e, such that

class interval $$i_a = \frac{1}{n}\left(Q_{max} - Q_{min}\right) \tag{4.13}$$

lower bound discharge $$Q_{L,a} = Q_{min} + \mathrm{int}\left(\frac{Q_e}{i_a}\right)i_a \tag{4.14}$$

where 'Q_{min}' is the minimum recorded discharge, 'Q_{max}' is the maximum recorded discharge and 'n' is the number of classes. The function int(q) refers to the integer part of the quotient q. Equation 4.12 is constant for any class interval.

For the arithmetic case, the maximum absolute error is produced when p has a value of 0. Solving Equations 4.11 and 4.12 at this condition gives

max effective error $$\max\ E_{\text{e,a}} = 100\left(\frac{i_{\text{a}}}{2Q_{\text{L,a}}}\right)\% \tag{4.15}$$

max relative error $$\max E_{\text{r,a}} = 100\left(\frac{i_{\text{a}}}{2Q_{\text{m}}}\right)\% \tag{4.16}$$

Alternatively, if the actual effective discharge, Q_{e}, has a magnitude at some fraction, e^p (where 'e' is exponential), of the logarithmic discharge interval that includes Q_{e}, then the error in estimating the effective discharge using log-based MFA is given by

effective error $$E_{\text{e,l}} = 100\left|\,\text{e}^{i_{\text{l}}(0\cdot 5-p)} - 1\right|\% \tag{4.17}$$

relative error $$E_{\text{r,l}} = 100\frac{Q_{\text{L,l}}}{Q_{\text{m}}}\left|\,\text{e}^{0\cdot 5 i_{\text{l}}} - \text{e}^{p i_{\text{l}}}\right|\% \tag{4.18}$$

where 'i_{l}' is the constant logarithmic class interval and '$Q_{\text{L,l}}$' is the lower bound discharge in the logarithmic interval that includes Q_{e}, such that

class interval $$i_{\text{l}} = \ln\left(\frac{Q_{\text{max}}}{Q_{\text{min}}}\right)^{1/n} \tag{4.19}$$

lower bound discharge $$Q_{\text{L,l}} = Q_{\text{min}} + \text{e}^{\left[\text{int}\left(\frac{\ln Q_{\text{e}}}{i_{\text{l}}}\right) i_{\text{l}}\right]} \tag{4.20}$$

Equation 4.17 is constant for any class interval. For the logarithmic case, the maximum absolute effective error is produced when p has a value of 0 and maximum absolute relative error when p has a value of 1. Solving Equations 4.17 and 4.18 at this condition gives max effective error $$\max\ E_{\mathrm{e,l}} = 100\left(\mathrm{e}^{0\cdot 5 i_l} - 1\right)\% \tag{4.21}$$

max relative error $$\max E_{\mathrm{r,l}} = 100\frac{Q_{\mathrm{L,l}}}{Q_{\mathrm{m}}}\mathrm{e}^{0\cdot 5 i_l}\left(\mathrm{e}^{0\cdot 5 i_l} - 1\right)\% \tag{4.22}$$

The absolute *effective* error ratio, $E_{\mathrm{e,l}}/E_{\mathrm{e,a}}$, is derived by combining Equations 4.13, 4.14, 4.15 and 4.21:

$$\frac{E_{\mathrm{e,l}}}{E_{\mathrm{e,a}}} = \frac{2Q_{\mathrm{L,a}}n}{\left(Q_{\max} - Q_{\min}\right)}\left[\left(\frac{Q_{\max}}{Q_{\min}}\right)^{0\cdot 5/n} - 1\right] \tag{4.23}$$

The absolute *relative* error ratio, $E_{\mathrm{r,l}}/E_{\mathrm{r,a}}$, is derived by combining Equations 4.16, 4.19, 4.20 and 4.22:

$$\frac{E_{\mathrm{r,l}}}{E_{\mathrm{r,a}}} = \frac{2Q_{\mathrm{L,l}}n}{\left(Q_{\max} - Q_{\min}\right)}\left(\frac{Q_{\max}}{Q_{\min}}\right)^{0\cdot 5/n}\left[\left(\frac{Q_{\max}}{Q_{\min}}\right)^{0\cdot 5/n} - 1\right] \tag{4.24}$$

As the coefficient $(Q_{\max}/Q_{\min})^{0.5/n}$ is always greater than unity, both types of error ratio are greater than unity when

$$Q_{\mathrm{L,a}} > \frac{1}{2n}\left(Q_{\max} - Q_{\min}\right) \tag{4.25}$$

and

$$Q_{\mathrm{L,l}} > \frac{1}{2n}\left(Q_{\max} - Q_{\min}\right) \tag{4.26}$$

The conditions given in Equations 4.25 and 4.26 assume $f(Q)$ equals $f(\ln Q)$, hence $Q_{\mathrm{e,a}}$ equals $Q_{\mathrm{e,l}}$. Assuming a log-normal flow distribution, Equations 4.5 and 4.14 can be combined to give expressions for the lower boundary discharge:

$$Q_{\mathrm{L,a}} = Q_{\min} + \mathrm{int}\left(\frac{n\mathrm{e}^{b\beta^2+\alpha}}{Q_{\max} - Q_{\min}}\right)\left(\frac{Q_{\max} - Q_{\min}}{n}\right) \tag{4.27}$$

Similarly by combining Equations 4.5 and 4.20:

$$Q_{L,l} = Q_{min} + e^{\left[\mathrm{int}\left(\frac{n\left(b\beta^2+\alpha\right)}{\ln Q_{max} - \ln Q_{min}}\right)\ln\left(\frac{Q_{max}}{Q_{min}}\right)^{1/n}\right]} \tag{4.28}$$

When substituted into error ratio equations, 4.23 and 4.24, the error involved in using logarithmic classes in favour of arithmetic classes can be determined for different values of Q_{max}, Q_{min}, n, b, and β. The maximum error is independent of α if the minimum and maximum discharges are at equal log-deviates from α.

To investigate approximation error, a log-normal PDF was applied with a range of 'β' from 0.5 to 1.75 and 'b' from 0.5 to 3. The minimum and maximum discharges, Q_{min} and Q_{max}, were defined as the 99.9 percent and 0.1 percent exceedance probabilities and 25 class intervals, *n*, were used. The arithmetic-based effective error, $E_{e,a}$, was calculated by substituting Equations 4.5 and 4.27 into Equation 4.7 and the logarithmic-based effective error, $E_{e,l}$, was calculated by substituting Equations 4.5 and 4.28 into Equation 4.8. Similarly, the arithmetic-based relative error, $E_{r,a}$, was calculated by substituting Equations 4.5 and 4.27 into Equation 4.17, and the logarithmic-based relative error, $E_{r,l}$, was calculated by substituting Equations 4.5 and 4.28 into Equation 4.18. The results for each scenario are plotted in Figures 4.2 and 4.3, together with the maximum possible error calculated by Equations 15, 16, 21 and 22. The graphs show that logarithmic classes yield the greatest error at a threshold of 'b' between about 1.0 and 1.5. However, the absolute difference in error increases markedly as 'β' increases, which confirms that using discrete data in a logarithmic-based flow-frequency histogram leads to greatest approximation in supply limited streams with highly variable flow regimes (assuming that the flow distribution is similar to a log-normal PDF).

Misrepresentation Error

To reiterate, Equations 4.23 to 4.28, which describe approximation error, assume no misrepresentation, that is $f(Q)$ equals $f(\ln Q)$. The appeal of using logarithmic classes is twofold. Firstly, flow-frequency often conforms to a log-normal distribution, so it may be expected that logarithmic classes are appropriate to represent this distribution (Hey, 1997a). Indeed, log-probability paper is usually used to portray a flow-duration relationship. Secondly, examination of approximation error suggests more accurate effective discharge at low flows if logarithmic class intervals are used.

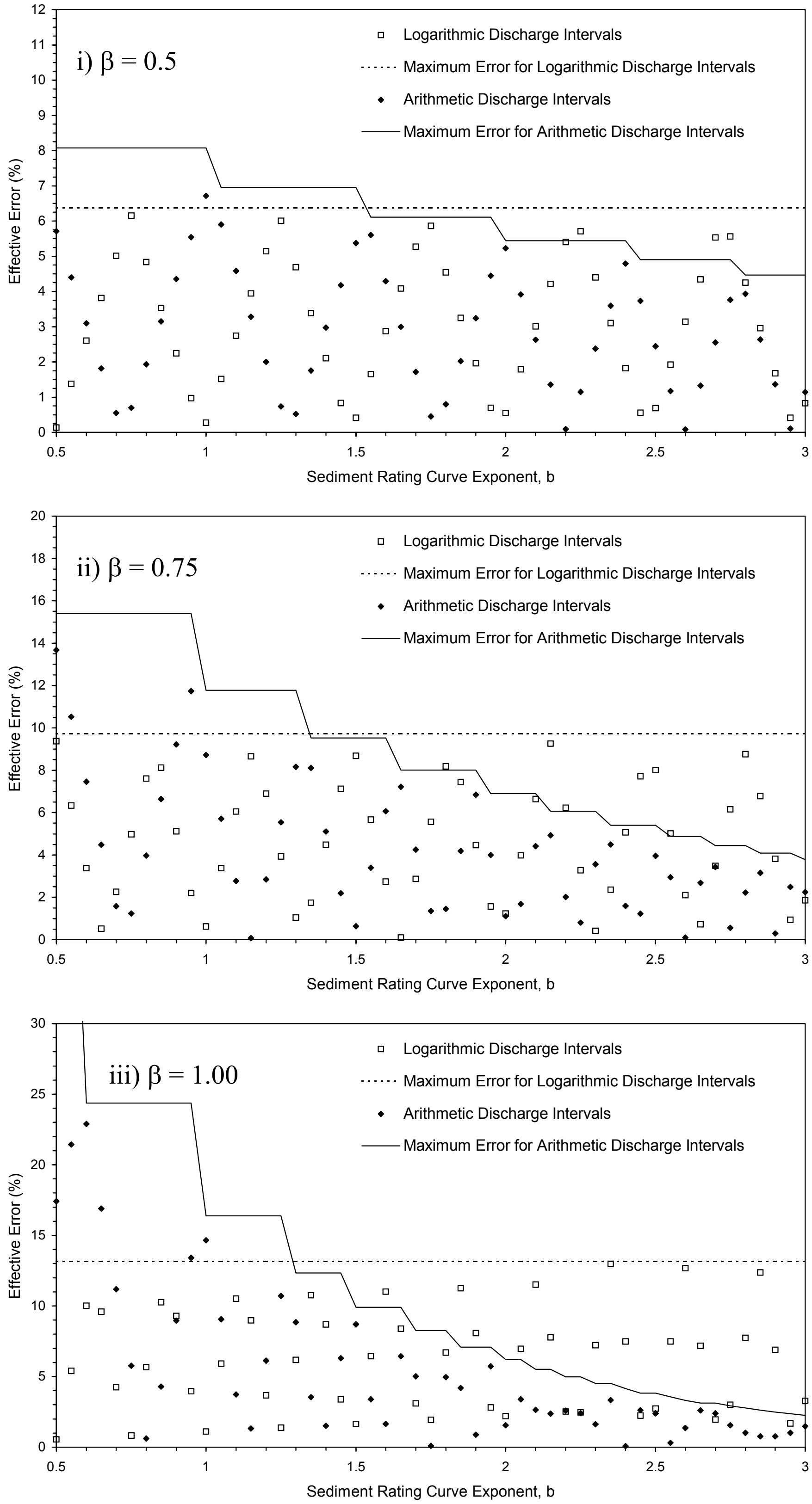


Figure 4.2 Effective error, E_e as a function of sediment rating exponent, b, and log-standard deviation, β, for a log-normal flow distribution. Only values are shown if the effective discharge is less than the maximum discharge (corresponding to 0.1 percent exceedance). (Continued)

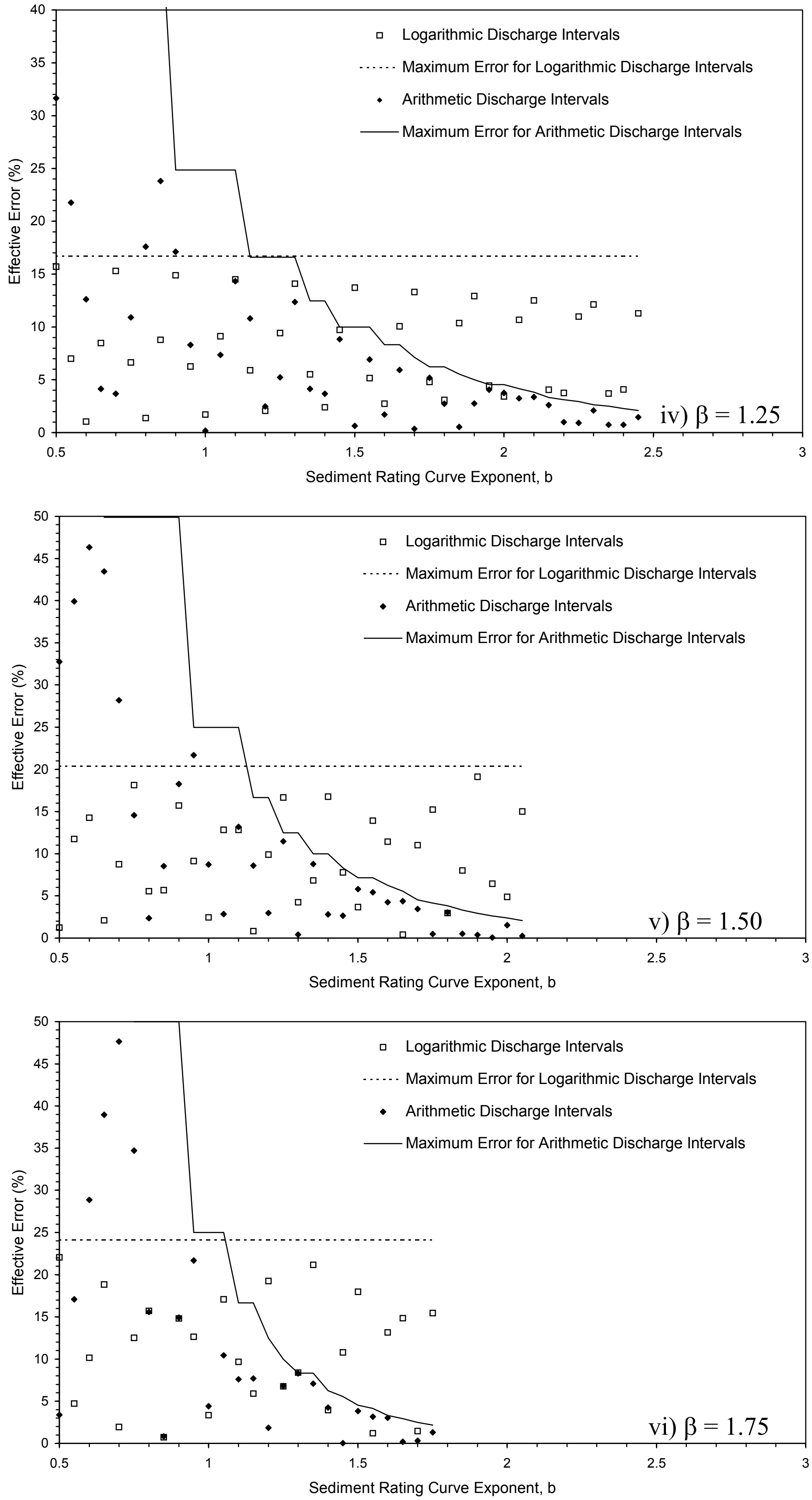


Figure 4.2 Concluded.

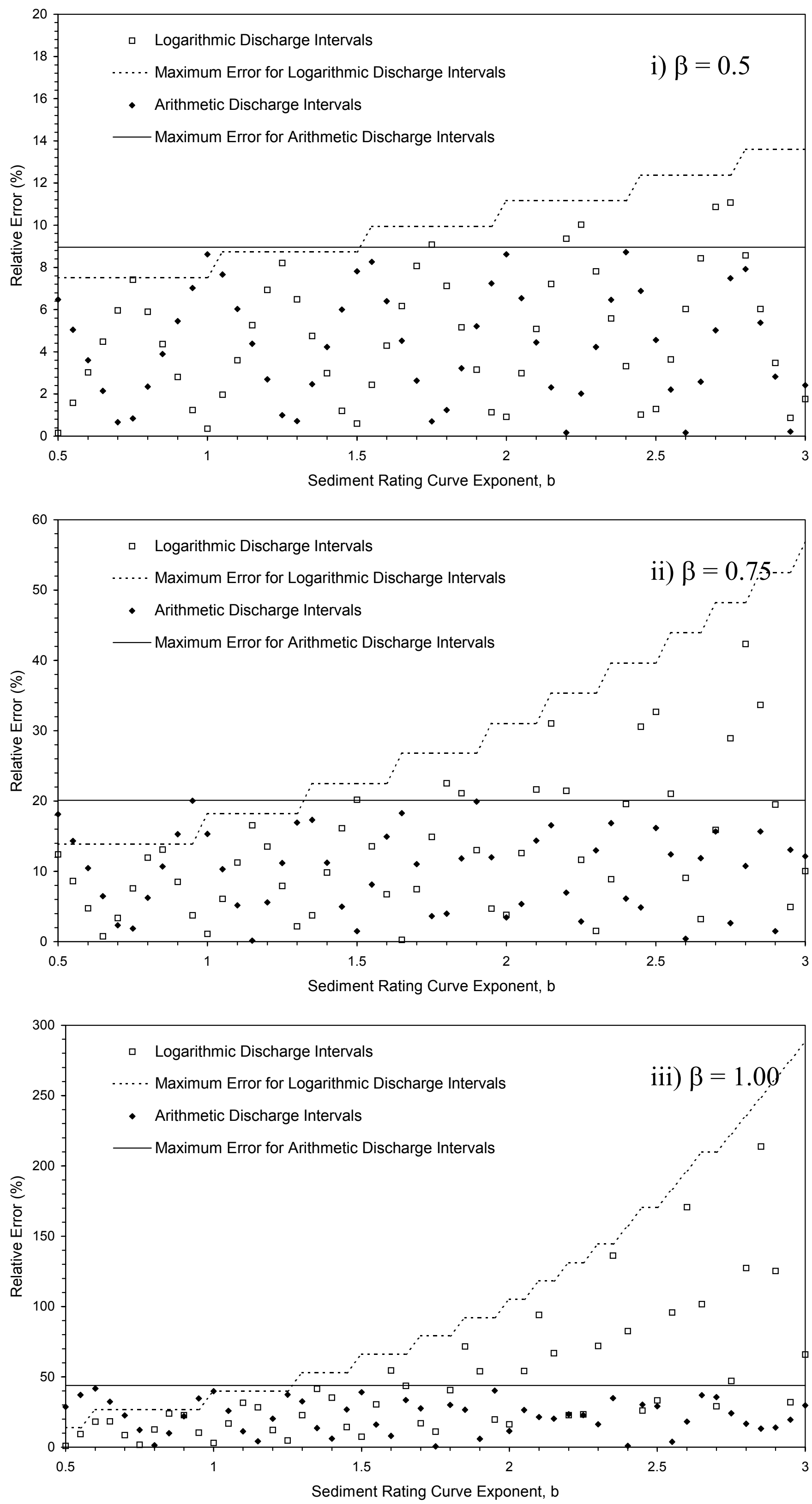


Figure 4.3 Relative Error, E_r as a function of sediment rating exponent, b, and log-standard deviation, β, for a log-normal flow distribution. Only values are shown if the effective discharge is less than the maximum discharge (corresponding to 0.1 percent exceedance).

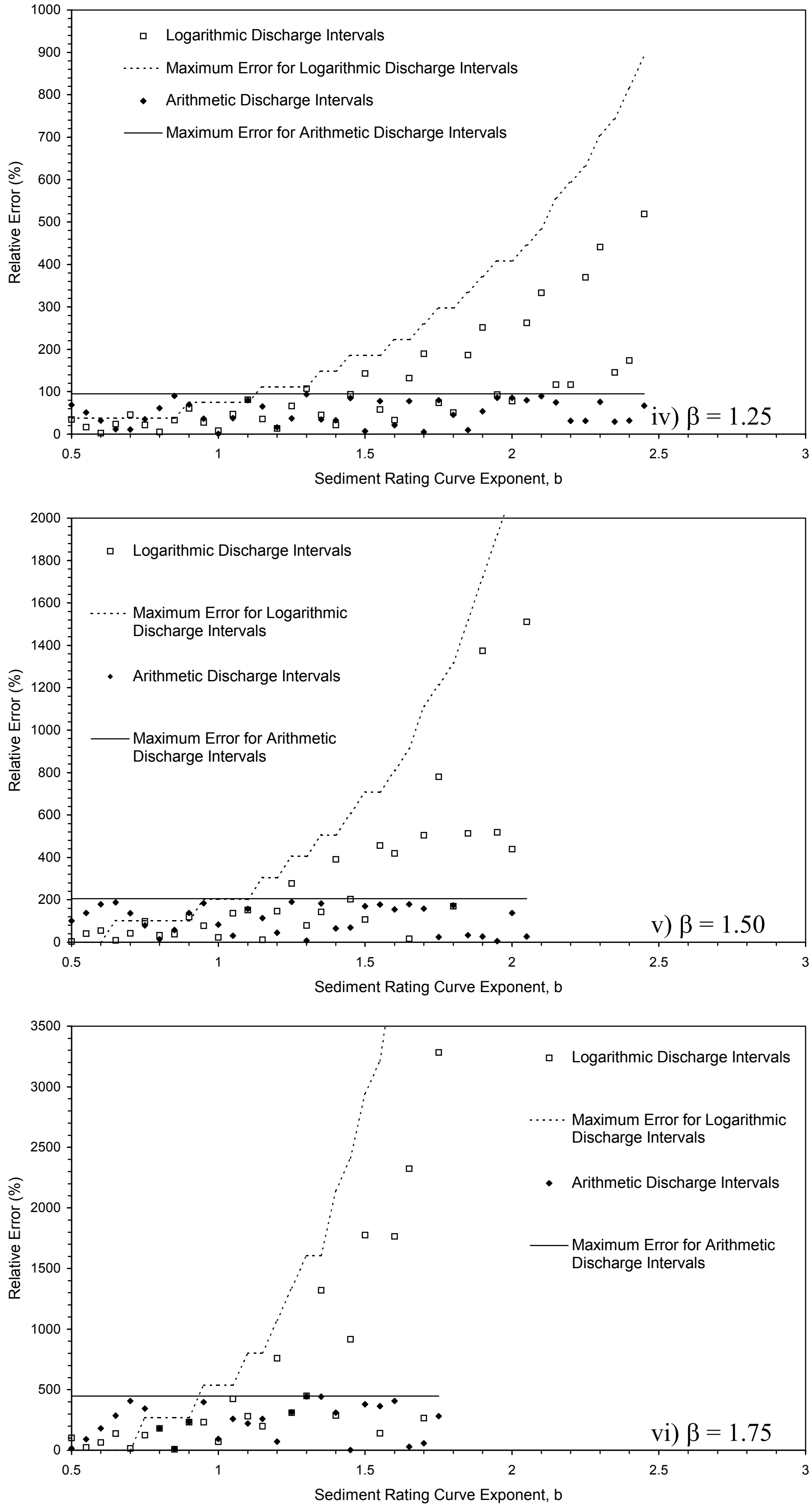


Figure 4.3 Concluded.

Using the law of probability, the notion of there being a *unique* effective discharge is flawed because the occurrence probability of an exact discharge tends toward zero with increasing precision. To overcome this limitation, MFA is based on the observed distribution of discharge *ranges*, rather than exact values. It follows that a *statistically* acceptable definition of the effective discharge is the *small range of discharge*, or δQ, which transports the most sediment over a period of years. Within a flow distribution, δQ may only approach zero in the ideal case of an infinite number of instantaneous measurements. Since the record of flows recorded at a gauge is only a sample of the actual population distribution experienced by the river, δQ is always significantly greater than zero.

For an *unbiased* estimation of the *actual* effective discharge (based on the population data set of continuous discharge measurements during the period of record), δQ cannot vary in size. If the discharge interval systematically increases, as in a logarithmic scale, then the resultant sample frequency distribution is incorrectly skewed in the negative direction (or misrepresented by exaggeration). As a direst result, the product of sediment load and frequency will tend to follow a similar trend. This is intuitive because in MFA, the sediment load transported by the mean discharge of a class is multiplied by a frequency corresponding to the probability of falling within that class. This probability increases with class size. With logarithmic class intervals, the systematic increase in the size of class interval with increasing discharge will overestimate the effective discharge.

Nash (1994) incorrectly used the log-normal probability density function. By differentiating the cumulative probability function (CPDF) with respect to the parameter concerned, in this case discharge, $F(Q)$, (Equation 4.29) Chow (1964, pp. 8-17) and Shahin et al. (1993, p. 104) showed that the frequency of discharge occurrence, $f(Q)$, is determined by Equation 4.30:

$$F(Q) = \frac{1}{\beta\sqrt{2\pi}} \int_{-\infty}^{Q} \mathrm{e}^{\frac{-(\,\ln Q\text{-}\alpha)}{2\beta^2}} \, \mathrm{d}Q \qquad (4.29)$$

$$f(Q) = \frac{1}{Q\beta\sqrt{2\pi}} \mathrm{e}^{\frac{-(\,\ln Q\text{-}\alpha)}{2\beta^2}} \qquad (4.30)$$

where 'α' and 'β' are the mean and standard deviation of the natural logarithm of discharge respectively. While the cumulative probability distributions of discharge and the logarithm of discharge are identical (Shahin et al., 1993), Equation 4.30 differs from the frequency of occurrence of the logarithm of discharge, $f(\ln Q)$, assumed by Nash:

$$f(Q) = \frac{1}{Q} f(\ln Q) \tag{4.31}$$

This indicates that the effective discharge may be markedly different if a logarithmic, rather than arithmetic, discharge scale is used. In turn, the modal (peak) and mean of the $f(Q)$ and $f(\ln Q)$ distributions are different and this implicates the effective discharge calculation (Table 4.2).

	Mean	Mode (Peak)
$f(Q)$ distribution	$e^{\alpha+\beta^2/2}$	$e^{\alpha-\beta^2}$
$f(\ln Q)$ distribution	e^{α}	e^{α}

Table 4.2 Mean and mode of a logarithmic normal discharge distribution in terms of the frequency of discharge, $f(Q)$, and the frequency of the natural logarithm of discharge, $f(\ln Q)$.

The mean of $f(Q)$ was given by Shahin et al. (1993, p. 105). The mode of $f(Q)$ was calculated by solving for discharge at the maximum of the derivative of Equation 4.30 and corresponds to the discharge giving the maximum gradient in the cumulative distribution (Equation 4.29). Using Equation 4.30 instead of Equation 4.1, the effective discharge is defined as

$$Q_e = e^{(b-1)\beta^2+\alpha} \tag{4.32}$$

If $Q_{e,a}$ represents the arithmetic-based effective discharge calculated by $f(Q)$ in Equation 4.30 and $Q_{e,l}$ represents the logarithmic-based effective discharge calculated by $f(\ln Q)$ in Equation 4.1, then the degree of bias incurred from using a logarithmic discharge scale is given by

$$Q_{e,l} = Q_{e,a}\ e^{\beta^2} \tag{4.33}$$

Therefore, log-based MFA will always overestimate the true effective discharge. The degree of misrepresentation error increases with flow variability such that the bias is minimal in canals and maximum in streams with very flashy flow regimes. The bias is not a function of the sediment rating. Notably, β must be as low as 0.25 before the error is less than the effective approximation error given in Equation 4.21.

The degree of bias suggested in Equation 4.33 can be demonstrated by using theoretical flow data. For this study, a log-normal PDF is assumed. The frequency associated with the mean of the *j*th discharge class is calculated as the probability of occurrence of any discharge within that class, or the area beneath the theoretical flow distribution curve (Equation 4.30) between the minimum and maximum class discharges. For arithmetic class intervals:

$$f(Q) = \frac{1}{\beta\sqrt{2\pi}} \int_{Q_{\min}+j(i_a-1)}^{Q_{\min}+ji_a} e^{\frac{-(\ln Q\text{-}\alpha)}{2\beta^2}} \tag{4.34}$$

where 'i_a' is the arithmetic class interval (Equation 4.13) and 'α' and 'β' are the natural logarithmic mean and natural logarithmic standard deviations of the distribution, respectively. Similarly, for logarithmic class intervals:

$$f(Q) = \frac{1}{\beta\sqrt{2\pi}} \int_{e^{\ln(Q_{\min})+j(i_l-1)}}^{e^{\ln(Q_{\min})+ji_l}} e^{\frac{-(\ln Q\text{-}\alpha)}{2\beta^2}} \tag{4.35}$$

where 'i_l' is the logarithmic class interval (Equation 4.19).

For this hypothetical example, the minimum and maximum discharges in the flow record were defined as the 99.9 percent and 0.1 percent exceedance probabilities and 25 class intervals were used. For a range of 'β' from 0.5 to 1.75 and 'b' from 0.5 to 3, a sediment frequency histogram was derived from the product of Equations 4.2 and 4.34. The arithmetic-based effective discharge, $Q_{e,a}$, was defined as the arithmetic mean discharge of the class with the greatest product. Similarly, a sediment-frequency histogram was derived from the product of Equations 4.2 and 4.35. The logarithmic-based effective discharge,

$Q_{e,l}$, was defined as the geometric mean discharge of the class with the greatest product. The predicted misrepresentation error, $Q_{e,l}/ Q_{e,a}$, was calculated in each case and plotted as a function of 'β' and 'b' (Figure 4.4). The deviation between the calculated $Q_{e,l} / Q_{e,a}$ and the theoretical value in Equation 4.33, shown by the error bars in Figure 4.4, is due to approximation error (an inevitable result of discretisation). The approximation error increases with flow variability as increasing 'β' increases the size of the class intervals. For all scenarios, the misrepresentation error is greater than unity and significantly increases with 'β' as Equation 4.33 indicates. The graphs also show the range of 'b' for which $Q_{e,a}$ falls within the first discharge class and the range of 'b' for which the $Q_{e,l}$ falls within the last discharge class (this should be clear from the square and cross symbols in the figure). When beta exceeds 1.5, there is a range of b for which $Q_{e,a}$ falls within the first class and (in italics) $Q_{e,l}$ falls within the last discharge class (this is shown where the square and cross symbols overlap). This clearly demonstrates the degree of bias from using a logarithmic discharge scale for streams with highly variable flow regimes. When 'β' is at least 1.25, there appears to be a decrease in misrepresentation at high values of 'b'. This is because $Q_{e,l}$ is in the last class and cannot further increase with 'b' whereas $Q_{e,a}$ continues to increase.

A database of 55 sites from U.S. rivers was compiled to further examine the degree of exaggeration in the logarithmic-based effective discharge using real data (Appendix Table A1). The database consists of sites previously used to determine the effective discharge by Biedenharn and Thorne (1994), Nash (1994) and Watson et al. (1997). These sites were selected from available data in the literature on the basis that they represent a wide range of physiographic and hydrological regions and have measured sediment load data with derived rating curves that permit the calculation of effective discharge without cross-sectional information. Two other Mississippi sites were analysed by Biedenharn and Thorne (1994) but are not included in this study. A further 11 sites documented by Nash (1994, p. 84) were not included on the basis of the following criteria: i) the flow record was reported to contain greater than 2 percent no flow days; ii) the record was considerably fragmented, or; iii) the gauge was discontinued more than 10 years ago. For sites studied by Biedenharn and Thorne (1994) and Watson et al. (1997), the period of flow record corresponds to that used in the original analyses. Periods of record are not given in the Nash (1994) data set, therefore a 30-year period was selected to represent the flow distributions. Mean daily flow data for the Biedenharn and Thorne (1994) and Nash

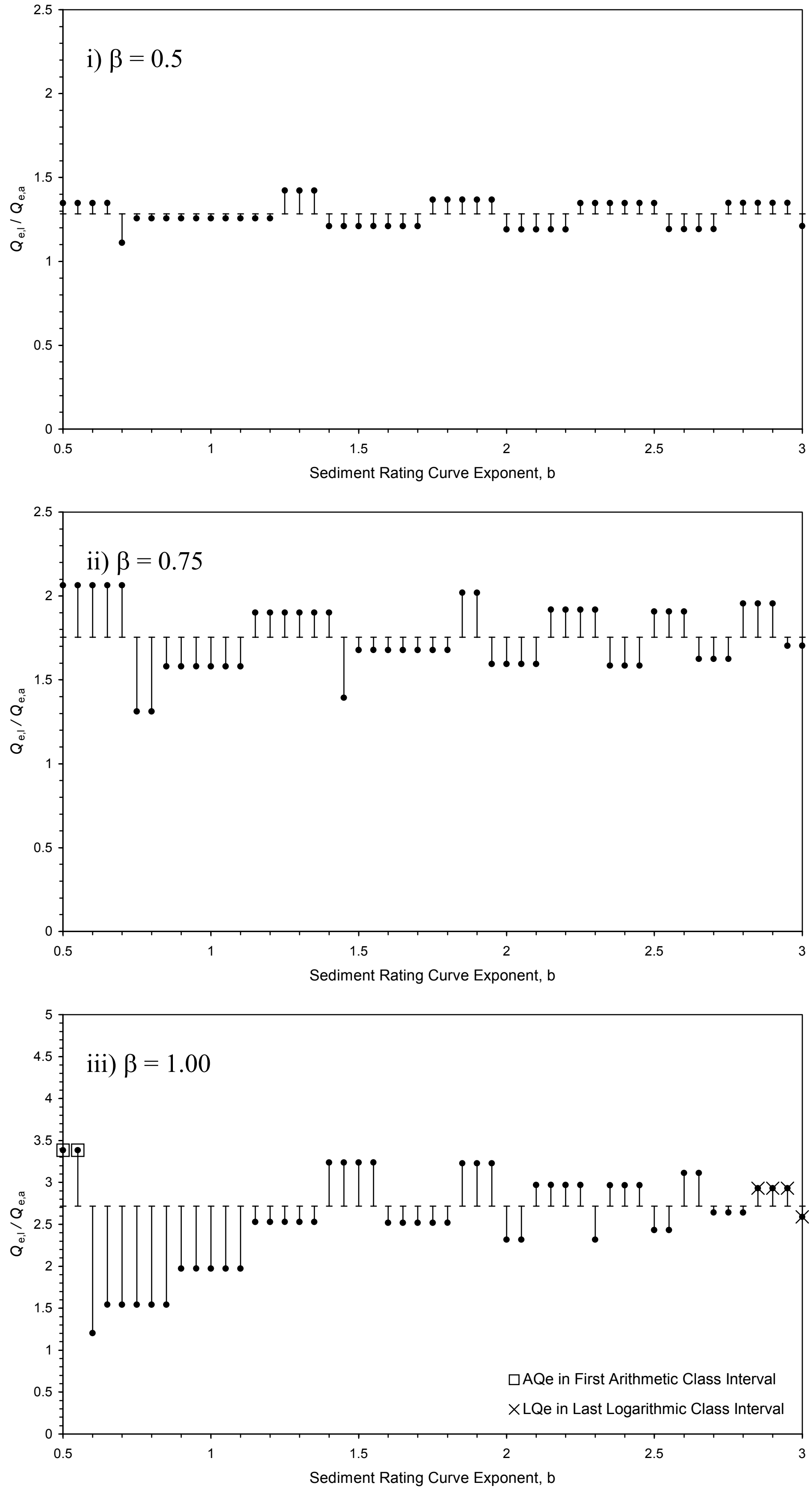


Figure 4.4 Misrepresentation Error (logarithmic-based effective discharge / arithmetic-based effective discharge) as a function of sediment rating exponent, b, and log-standard deviation, β, for a log-normal flow distribution.

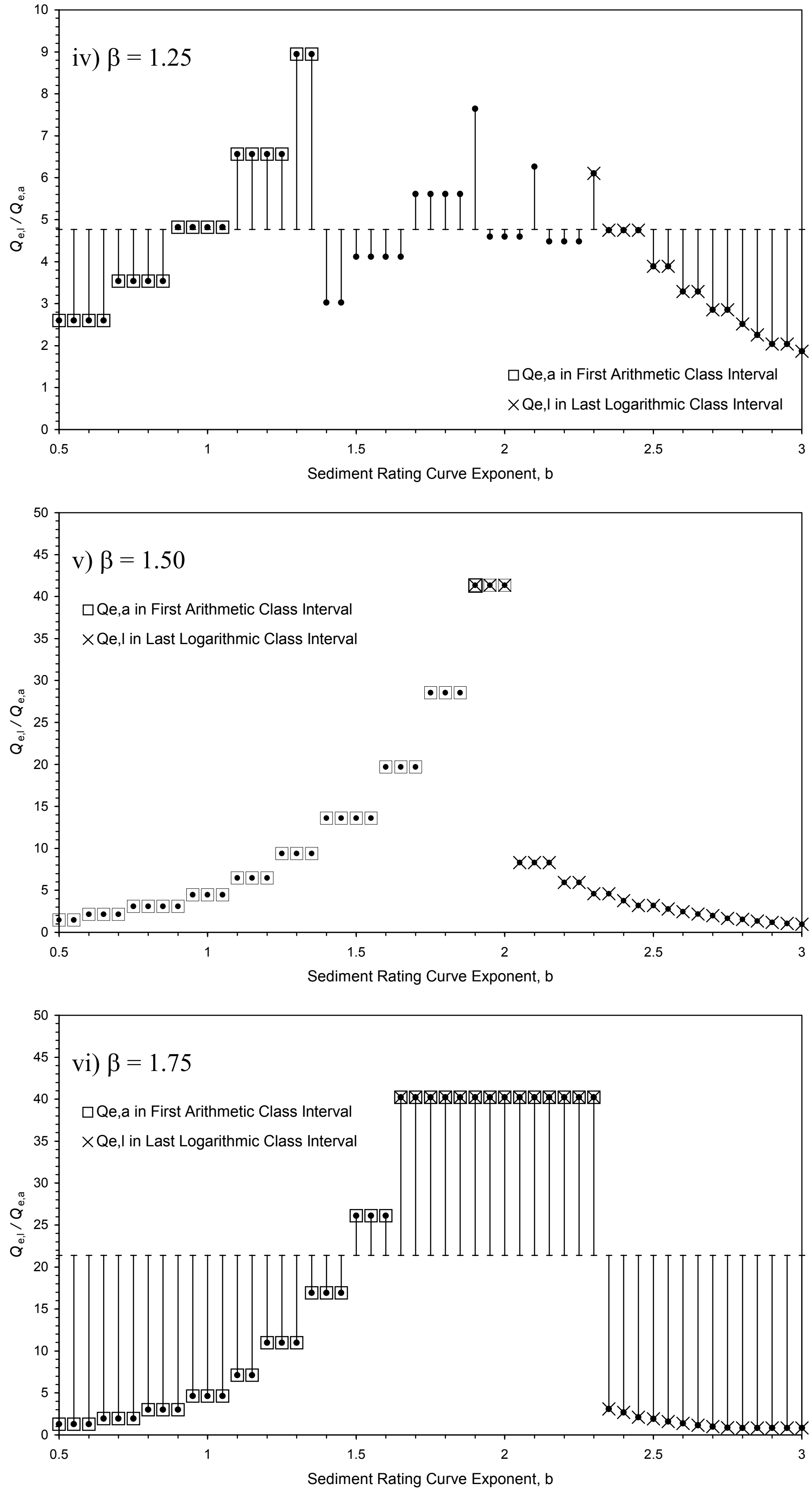


Figure 4.4 Concluded.

(1994) sites were supplied by the USGS and 15-minute flow data were supplied by Colorado State University for the Mississippi sites studied by Watson et al. (1997). In Appendix Table A1, 'a' and 'b' correspond to the coefficient and exponent in Equation 4.2 with sediment discharge measured in metric tonnes per day and discharge in cumecs. It is assumed that the suspended sediment load approximates the total bed material load at each site. The effective discharge calculations for each site were based on 25 discharge class intervals and the Flow-Frequency Histogram method described in Section 4.5.3.

The database was used to examine three hypotheses: i) the ratio of logarithmic-based effective discharge to arithmetic-based effective discharge is considerably greater than unity for the majority of sites; ii) the theoretical arithmetic-based effective discharge given by Equation 4.32 is only an average condition, with considerable variability as the flow distribution deviates from the log-normal density function, and; iii) the frequency of the effective discharge is highly variable as a direct result of ii and not confined to an 'intermediate' discharge as envisaged by Wolman and Miller (1960). This natural variability is examined in Section 4.5.3.

Figure 4.5 reveals the magnitude of bias in the effective discharge as a result of using logarithmic discharge intervals and Figure 4.6 is a cumulative plot of the percentage of sites with less than or equal to a given ratio of logarithmic-based effective discharge to arithmetic-based effective discharge. Only 9 percent of sites have a ratio less than or equal to unity, 25 percent of sites have a ratio not exceeding 1.3, 50 percent of sites have a ratio not exceeding 8.4 and 75 percent of sites have a ratio not exceeding 15.8. This degree of variability suggests that in most cases a logarithmic-based effective discharge will overestimate the dominant discharge. This may have significant implications for stable channel design. For example, an overestimated effective discharge may lead to the designed bankfull width also being an overestimate, which could result in sedimentation and, in extreme cases, braiding.

Figure 4.7 compares the arithmetic-based effective discharge (Q_e) with the theoretical value (Pred Q_e) given by Equation 4.32, which assumes a theoretical (predicted) log-normal flow distribution. A best-fit power function regression line gives the following relationship:

$$\text{Pred}\, Q_e = 1.19\, Q_e^{0.93} \tag{4.36}$$

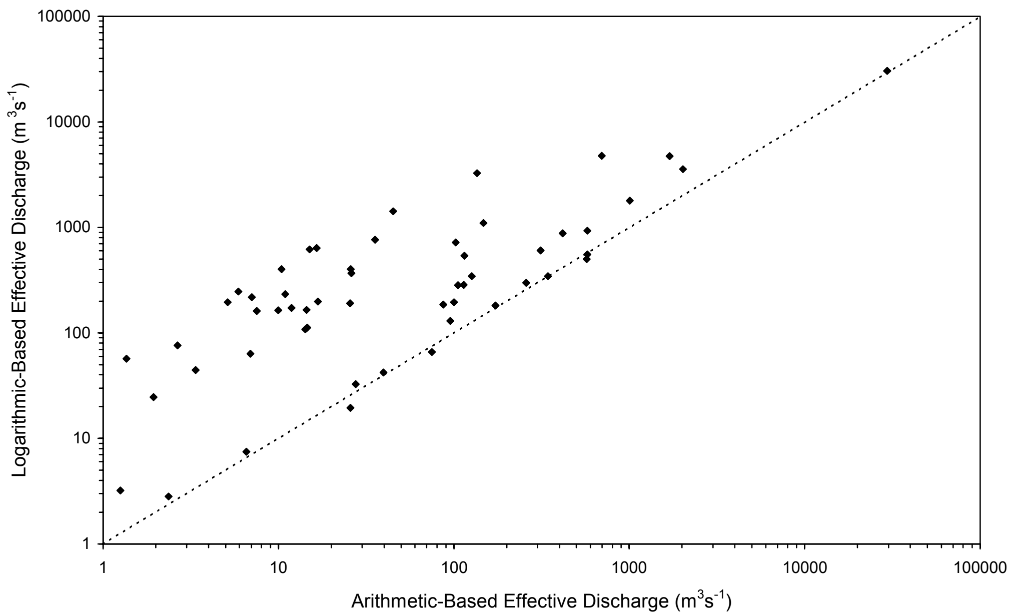


Figure 4.5 Comparison of log-based effective discharge with arithmetic-based effective discharge (dotted line represents equality) (data set of 55 U.S. sites).

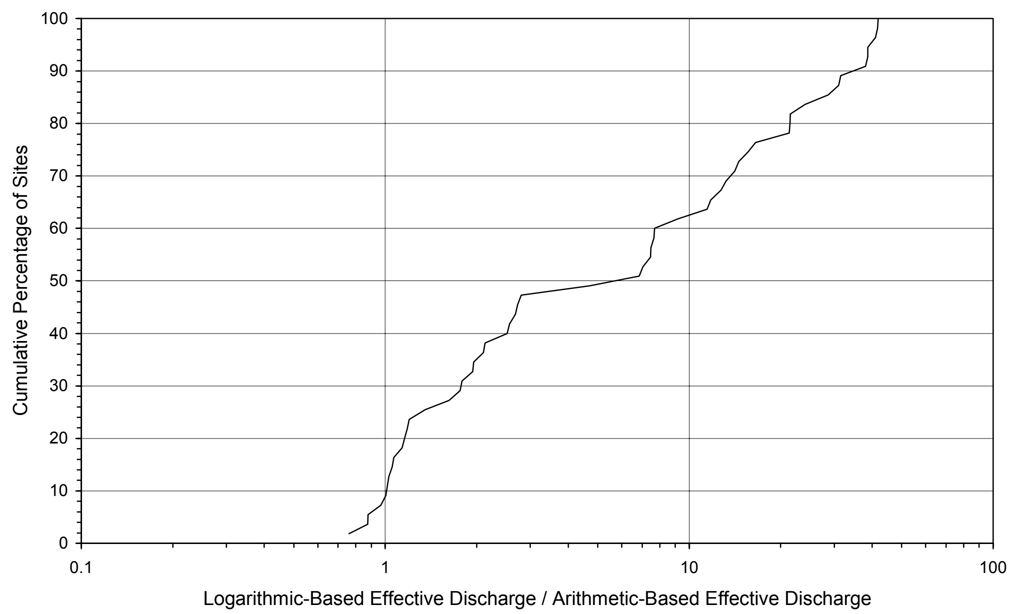


Figure 4.6 Cumulative plot of the percentage of sites not exceeding a given ratio of logarithmic-based effective discharge to arithmetic-based effective discharge (data set of 55 U.S. sites).

This relationship is near the line of equality (dotted line in Figure 4.7), with greatest deviation at high discharges. Therefore, Equation 4.32 represents an average condition for a range of different streams, however the considerable variability shown in Figure 4.7 demonstrates that real flow data should be used to calculate the effective discharge rather than the theoretical flow distribution. The greatest difference between the two discharges is caused by significant deviations from the smooth log-normal distribution. From an analysis of skewness and kurtosis of the logarithm of discharge (Appendix Table A1), in several cases a platykurtic (flat) and/or negatively skewed distribution with a high standard deviation tends to lower the frequency of the intermediate discharges and elevate the effectiveness at high discharges (for example, Eel River at Fort Seward, CA, and Thomes Creek at Paskenta, CA). Conversely, in several cases a leptokurtic (peaky) and or positively skewed distribution with a low standard deviation tends to elevate the effectiveness of the more frequent flows (for example the northern Mississippi streams studied by Watson et al., 1997). Although these trends explain some of the variability, the smoothness of the flow distribution also affects the magnitude and frequency of the effective discharge. While the theoretical log-normal distribution is smooth and unimodal, many of the sites in Appendix Table A1 exhibit more erratic distributions with multiple peaks. Figure 4.8 is a cumulative plot of the percentage of sites with less than or equal to a given ratio of predicted arithmetic-based effective discharge to calculated arithmetic-based effective discharge. Twenty-five percent of sites have a ratio not exceeding 0.35, 50 percent of sites have a ratio not exceeding 0.75 and 75 percent of sites have a ratio not exceeding 1.32.

When using an arithmetic scale for streams that are base flow dominated and therefore, positively skewed, the effective discharge often falls within the first discharge class. This is the case for 25 of the 55 sites in Appendix Table A1. When this occurs, Hey (1997a, p. 12) recommended subdividing the first class for a more accurate estimate of the true effective discharge (to yield a ‘hybrid scale’, Simon, 1999, pers. comm.). However, this method can also result in overestimating the effective discharge as with a logarithmic scale. This occurs because continuous subdivision of the first class will systematically reduce the frequency in each sub class until the modal (peak) class is shifted into the second class with a greater mean discharge.

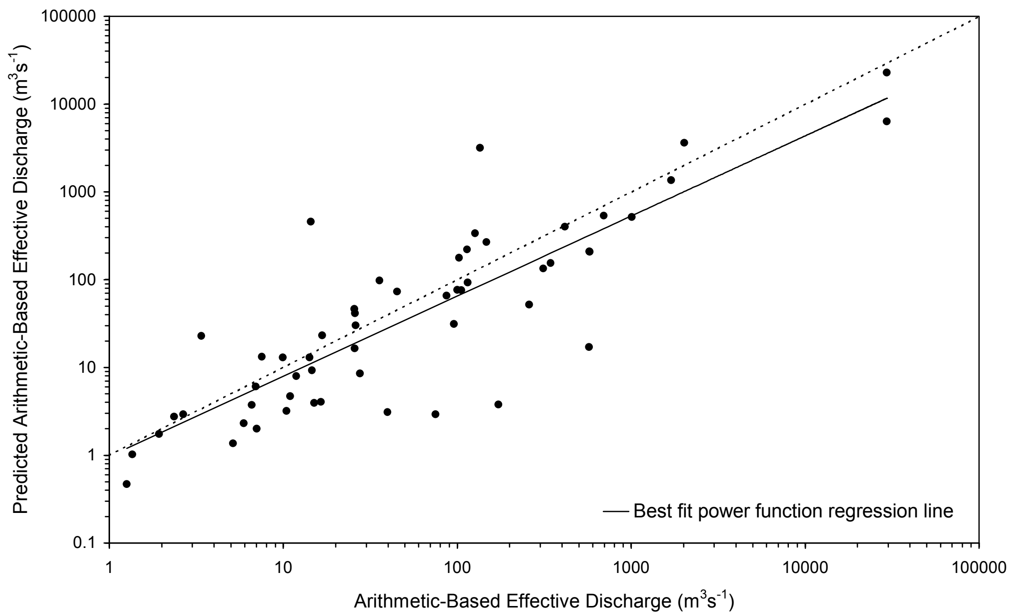


Figure 4.7 Comparison of arithmetic-based effective discharge with that predicted assuming a log-normal flow distribution (data set of 55 U.S. sites).

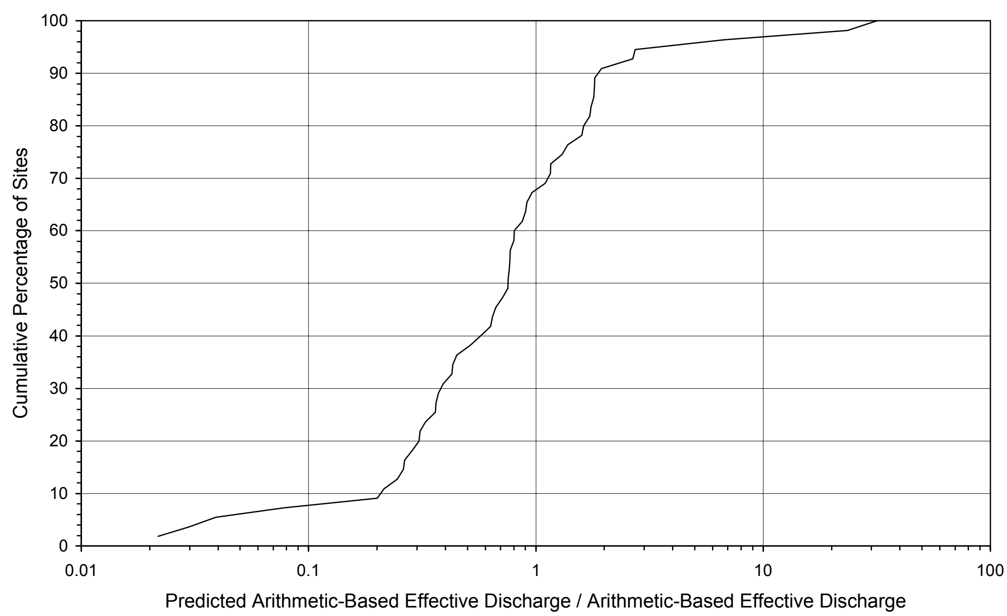


Figure 4.8 Cumulative plot of the percentage of sites not exceeding a given ratio of predicted arithmetic-based effective discharge (assuming a log-normal flow distribution) to calculated arithmetic-based effective discharge (data set of 55 U.S. sites).

In gravel-bed streams where measured load is available, Equation 4.2 may be replaced by

$$Q_s = a\left(Q - Q_c\right)^b \tag{4.37}$$

where Q_c is the critical discharge for incipient sediment motion. Setting the derivative of the product of Equations 4.30 and 4.37 to zero and solving for discharge, the effective discharge in a gravel-bed stream with a log-normal flow distribution satisfies the following equation for which an iterative procedure is required to solve for Q_e

$$Q_e b \beta^2 = \left(Q_e - Q_c\right)\left(\ln Q_e + \beta^2 - \alpha\right) \tag{4.38}$$

Equations 4.32 and 4.38 can *only* be used to predict Q_e if the empirical distribution of flows conforms to the theoretical log-normal PDF. Nash found that the log-normal distribution was sufficient for describing the frequency for most rivers studied with most divergence at low frequencies. However, this is not always the case as shown in Figures 4.7 and 4.8. For example, Goodwin et al. (1998) found that the distribution of flows in Reynolds Creek, Idaho, compared well with a theoretical normal probability density function (not log-transformed) and based on a similar approach to that used by Nash (1994), showed that the effective discharge, based on the Engelund and Hanson sediment transport equation (1967), can be determined analytically as the positive root of a quadratic expression:

$$Q_e = \frac{1}{2}\left(Q_m + \sqrt{Q_m^2 + 6 \cdot 8\sigma^2}\right) \tag{4.39}$$

where ‘Q_m’ is the mean discharge and ‘σ’ is the standard deviation of the flows.

It must be concluded that the degree of bias caused by using a logarithmic scale suggests that an arithmetic scale should be applied in all cases. Furthermore, the practice of subdividing the lowest discharge class to reduce approximation error may also result in overestimating the effective discharge and therefore, should not be carried out.

4.5.1.3 Number of Discharge Class Intervals

The number of discharge classes can influence the effective discharge calculation. Intuitively, it might be expected that the smaller the class interval and, therefore, the greater the number of classes, the more accurate would be the outcome. However, if too small an interval is used, discontinuities appear in the discharge frequency distribution. These, in turn, produce an irregular sediment load histogram, with multiple peaks. Therefore, the selected class interval should be small enough to accurately represent the frequency distribution of flows, but large enough to produce a continuous distribution, with no classes having a frequency of zero (Biedenharn and Thorne, 1994, p. 242). This may result in several attempts at calculating the effective discharge until a satisfactory result is produced.

There are no definite rules for selecting the most appropriate interval and number of classes, but Yevjevich (1972) stated that the class interval should not be larger than $s/4$, where '*s*' is an estimate of the standard deviation of the sample. For hydrological applications, he suggested that the number of classes should be between 10 and 25, depending on the sample size. Hey (1997a) found that 25 classes with equal, arithmetic intervals produced a relatively continuous flow-frequency distribution and a smooth sediment load histogram with a well defined peak in many cases, indicating an effective discharge which corresponded closely with bankfull flow. A smaller interval, and correspondingly larger number of classes, produced anomalous results.

4.5.1.4 Discharge Time Base

Mean daily discharges are conventionally used to construct the flow-duration curve. Although this is convenient, given the ready availability of mean daily discharge data, it can, in some cases, introduce error into the calculations. This arises because mean daily values can under-represent the occurrence of short-duration, high-magnitude flow events that occur within the averaging period, while over-representing effects of low flows. On large rivers, the use of the mean daily values is acceptable because the difference between the mean and peak daily discharges is negligible. However, on smaller streams, flood events may last only a few hours, so that the peak daily discharge is much greater than the corresponding mean daily discharge. Excluding the flood peaks and the associated high sediment loads can result in underestimation of the effective discharge. Rivers with highly variable flow regimes are most likely to be affected. To avoid this problem, it may be necessary to reduce the time base for discharge averaging from 24 hours (mean daily) to 1 hour, or even 15 minutes on flashy streams.

In practice, mean daily discharge data may be all that are available for the majority of gauging stations and these data may be perfectly adequate. However, caution must be exercised when using mean daily data for watersheds with flashy runoff regimes and short-duration hydrographs. The use of 15-minute data to improve the temporal resolution of the calculations should be seriously considered whenever the available flow records allow it. Watson et al. (1997) found that for 10 Demonstration Erosion Control (DEC) streams in northern Mississippi, the effective discharge computed from 15-minute instantaneous flow data are consistently greater than that computed from the conventional mean daily flow data. Mean daily flow-duration has less variability than the 15-minute data, which are capable of accurately representing peak flows in a flood hydrograph. On average, for the ten DEC streams, the mean annual sediment load calculated using mean daily flow data is underestimated by 58 percent of the actual total load.

Wolman and Miller (1960, p. 58) found that the greater the variability of the runoff, the larger the percentage of the total load which is likely to be carried by infrequent events. As runoff becomes increasingly variable as drainage area is reduced, Wolman and Miller suggest that the smaller the drainage area, the larger will be the percentage of sediment transported by infrequent flows. This is supported by Wolman and Gerson (1978) who suggest that peak discharge per square mile and the ratio of a high-magnitude event to mean annual flood (both measures of flow 'flashiness') decrease with increasing drainage area.

4.5.1.5 Period of Flow Record

The period of record must be sufficiently long to include a wide range of morphologically-significant flows, but not so long that changes in the climate, land-use or runoff characteristics of the watershed produce significant changes with time in the data. If the period of record is too short, there is a significant risk that the effective discharge will be inaccurate because of the occurrence of unrepresentative flow events. Conversely, if the period is too long, there is a risk that the flow and sediment regimes of the stream at the beginning of the record may be significantly different to current conditions.

A reasonable minimum period of record for an effective discharge calculation is about 10 years, with 20 years of record providing more certainty that the range of morphologically significant flows is fully represented in the data. Records longer than 30 years should be examined carefully for evidence of temporal changes in flow and/or sediment regimes. If the period of record at a gauging station is inadequate, consideration should be given to synthesising an effective discharge based on regional estimates of the flow-duration as outlined in the following Section 4.5.1.6.

4.5.1.6 Hydrological Data at Ungauged Sites

At ungauged sites and gauged sites where records are found to be unrepresentative of the flow regime, it is necessary to synthesise a flow distribution. There are three possible methods of doing this, using records from nearby gauging stations within the same catchment or hydrological region: i) catchment flow-duration curve method; ii) regional flow-duration curve method, and; iii) arithmetic manipulation of flow-duration curves. These methods are discussed in turn.

Catchment Flow-duration Curve Method (or Basin-Area method)

This method relies on the availability of gauge data from a number of stations along the same river as the ungauged study site. First, flow-duration curves for each gauging station are derived for the common period of record. Provided there is a regular downstream decrease in the discharge per unit watershed area, then a graph of discharge for a given exceedance duration against upstream basin area will produce a power function best-fit regression line with negligible scatter. An example of this method is given by Hey (1975). The equations generated by this method enable the flow-duration curve at an ungauged site on that river to be determined as a function of its upstream watershed area. Flow frequencies for selected discharge classes may then be extracted from the flow-duration curve for the ungauged site and the effective discharge calculated using the Flow-Duration Curve method outlined in Section 4.5.4

On streams with only one gauging station, flow-duration curves can be estimated for ungauged sites provided that the streams are tributaries to rivers where the relation between discharge and basin area conforms to a known power function. Estimates of the contributing flow to the main stem can be obtained from the difference between discharges on the main stem above and below the tributary junction. Discharge-basin area relations can then be derived for the tributary given the flow-duration curve at the gauging station and the predicted curve at its confluence with the main stem. However, this technique should not be used if there are distinct and abrupt downstream changes in the discharge per unit area for the watershed, due to tributaries draining different hydrological regions. In this case it would be preferable to use the Regional Duration Curve method described next.

Regional Flow-duration Curve Method

An alternative to the use of watershed area to generate a flow-duration curve for an ungauged site is to use a regional-scaling method based on data from watersheds with similar characteristics. For example, Emmett (1975) and Leopold (1994, p. 95) suggest using the ratio of discharge to bankfull discharge (Q/Q_b) as a non-dimensional index with which to transfer flow-duration relationships between basins with similar characteristics. However, bankfull discharge does not necessarily have either a consistent duration or return period. To overcome this problem, a non-dimensional discharge index was proposed by Watson et al. (1997) using a regional estimate of the 2-year discharge to normalise discharges (Q/Q_2).

For ungauged sites, the 2-year discharge may be estimated from regionalised discharge frequency relationships (e.g. Institute of Hydrology, 1993, for U.K. streams), which are based on regression relationships between the drainage area, channel slope, slope length and other basin characteristics. The dimensionless discharge index (Q/Q_2) can be used to transfer a flow-duration relationship to an ungauged site from a nearby, gauged site. The gauged site may either be within the same basin, or an adjacent catchment. The steps involved in developing a regional flow-duration relationship are:

i) Select several gauging stations and divide the discharge values of the flow-duration relationship for each station by the respective Q_2 for that gauge.

ii) Plot these ratios on a log-log graph. Discharges less than 1 percent of the Q_2 and with a probability of less than 1 percent, are insignificant morphologically and may be ignored.

iii) A flow-duration curve for any ungauged site may then be computed by substituting the regional Q_2 for that site. Flow frequencies for selected discharge classes may then be extracted from the flow-duration curve for the ungauged site.

Watson et al. (1997) tested the accuracy of these approaches. They found that the average error in bed material sediment yields at all the ungauged sites tested was only 2.8 percent when the method was used to transfer a flow-duration relationship within a watershed, and 5.5 percent when it was used to develop a regional flow-duration relationship.

Arithmetic Manipulation of Flow-duration Curves

In many cases a significant tributary input between a gauge and the project site prevents the direct use of gauge data to calculate the effective discharge. Figure 4.9 shows a simple tributary confluence configuration. If both reaches A and B are gauged with measured sediment data or stable cross section sites, then it is possible to combine the two flow distributions to synthesise a flow-duration relationship and sediment frequency histogram, and in turn, the effective discharge for the project reach C downstream from the confluence. If the tributary is not gauged, then either the Basin-Area or Regional Duration Curve methods must be used. This technique requires deriving standardised duration curves (SDC) for the gauged reaches for the same stratified set of exceedance probabilities. Then the discharges from both SDCs can be added for each probability to synthesise a SDC for reach C. This type of arithmetic manipulation assumes that both reaches A and B are similar in character and experience the same rainfall-run-off events.

The effective discharge for reach C can then be estimated by constructing a flow-frequency histogram from the SDC of reach C for a specific number of arithmetic class intervals. The average annual sediment load for the mean of each discharge class, Q_i, is then defined as the sum of the average annual sediment loads in reaches A and B transported by discharges of the same exceedance probability as the required mean discharge, Q_i. In gravel-bed rivers, the minimum discharge in the synthesised flow-frequency histogram for reach C should correspond to the greater of the exceedance probabilities of the critical discharges for incipient sediment transport between reaches A and B. This ensures that there are no zero frequencies in the sediment frequency histogram.

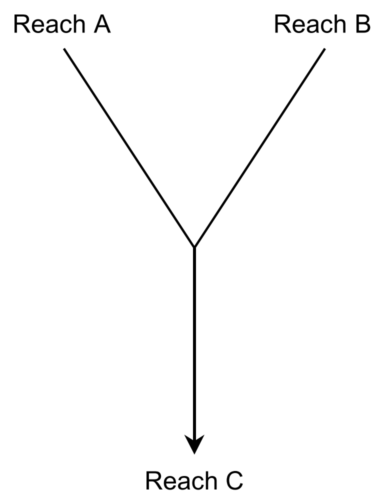


Figure 4.9 Simple tributary confluence configuration to demonstrate the Arithmetic Manipulation of Duration Curves method of deriving effective discharge at an ungauged site.

The recommended procedure to construct a SDC involves two stages:

i) Generate an event-based flow-duration curve from all recorded discharge measurements. As this usually involves manipulating a database comprising of thousands of measurements, a computer program is required. A class-based duration curve may be used to simplify the calculations, whereby a flow-frequency histogram with n classes is constructed first (where n is significantly smaller than the actual number of different discharges recorded), however the true form of the cumulative distribution may not be adequately represented.

ii) Generate a second (standardised) flow-duration curve between 0.01 and 100 percent discharge exceedance. To maintain the form of the distribution derived in i, a large number of exceedance values should be used, otherwise the tails of the distribution will be misrepresented. A computer program can facilitate the construction of the SDC based on thousands of exceedance probabilities. In base-flow-dominated streams, it may be necessary to reduce the lower bound probability in the SDC to 0.001 percent to adequately represent peak flows. The required discharge for each standardised probability is derived by linear interpolation of the event true-based duration curve.

This type of analysis can be modified for other scenarios: e.g., i) combine flow-duration relationships from several tributaries or, ii) if there is a gauge in close proximity to the project reach but neither measured sediment data nor a stable cross section below the next upstream tributary is available. In the latter case, the effective discharge for the project reach may be estimated if the upstream tributary is also gauged and stable cross sections can be found upstream of the confluence in both the main stem and the tributary. This scenario was used in the case study described in Chapter 8.

4.5.2 Compilation of Sediment Transport Data

4.5.2.1 Nature of the Sediment Load

The total sediment load of a stream can be broken down on the basis of measurement method, transport mechanism or source (Table 4.3). Nash (1994) and Hey (1997a) have found that effective discharges have generally been calculated for rivers which are characterised by suspended load after demonstrating that bed load did not affect the calculation (Nolan et al., 1987), or that the bed load was less than 10 percent of the total load (Walling and Webb, 1982; Biedenharn et al., 1987; Wolman and Miller, 1960), or bed load was proportional to suspended load (Benson and Thomas, 1966). The bed load contribution was simply disregarded in the calculations by Fisk (1977) and Ashmore and Day (1988). For gravel bed rivers, effective discharge based on bed load only has been calculated by Pickup and Warner (1976) Carling (1988), Hey (1997a) and Soar et al., (1999).

<table>
<tr><th>Measurement Method</th><th>Transport Mechanism</th><th>Sediment Source</th></tr>
<tr><td>Unmeasured Load</td><td>Bed Load</td><td rowspan="2">Bed Material Load</td></tr>
<tr><td rowspan="2">Measured Load</td><td rowspan="2">Suspended Load</td></tr>
<tr><td>Wash Load</td></tr>
</table>

Table 4.3 Classification of the total sediment load (after Thorne et al., 1998).

When bed load and suspended load are both significant fractions of the bed material load, then the effective discharge should be based on the total load for an unbiased estimate. When measured load is available, the suspended load fraction can be added to calculated bed load, using an appropriate bed load function (Andrews, 1980; Lyons et al. (1992).

4.5.2.2 Sediment Transport Data at Gauged Sites

In most alluvial streams, the major features of channel morphology are principally formed in sediments derived from the bed material load. It is, therefore, the bed material load that should be used in an effective discharge calculation. At gauged sites the measured load usually represents the suspended load, but excludes the bed load. Under these circumstances, the coarse fraction of the measured load (generally the sand load, that is particles larger than 0.063 mm) should be used to derive a bed material load rating curve. If available, bed load data should be combined with the coarse fraction of the measured load to derive a bed material load rating curve.

Where a significant proportion of the bed material load moves as bed load (such as in gravel-bed rivers), but no measurements of bed load are available, it may be necessary to estimate the bed load. This may be achieved using a suitable bed load transport equation or the SAM hydraulic design package (Thomas et al., 1996). Similarly, at gauging stations with no measured sediment load data at all, a bed material sediment rating curve may be generated using appropriate sediment transport equations, or the SAM package.

Sediment rating curves can often underestimate the actual load transported because of the nature of the power function. This is because of greater scatter of data points above the rating curve regression line than below the line on arithmetic scales. Ferguson (1987) presented a correction method to overcome the inherent bias in this type of log-transformed equation. A suitable correction factor, F, is discussed in Chapter 3 (Equation 3.17) and Equation 3.17 is repeated here:

$$F = \mathrm{e}^{\left(s^2/2\right)} \tag{3.17}$$

The effective discharge is generally independent of the coefficient 'a' in Equations 4.2 and 4.37. Emmett (1985) supported this based on findings from bed load measurements in Snake and Clearwater Rivers, Idaho. He showed that the rating curve exponent 'b' is the same for the suspended load and the bed load and the coefficient 'a' is independent of discharge and, thereby fixed. On this basis, Q_e must be the same for both sediment loads and the total load. If the effective discharge is independent of the coefficient 'a', then a correction factor for predicted measured load is not required in the calculation, as suggested by Hey (1997a). If the effective discharge of the bed load and the suspended load are unequal then correction of the measured load is necessary, as applied by Ashmore and Day (1988) for the Saskatchewan River basin in western Canada.

Measured load is usually expressed as a single-power function of discharge. If measured load data are available and the error variance of a sediment rating curve is not constant (using logarithmic scales), then consideration should be given to using different rating curves for different ranges of discharge. Experience has shown that a single rating curve significantly overestimates sediment load at low discharges and underestimates at high discharge (Kuhnle et al., 1999). Hey (1997a) used separate rating curves for in-bank and over-bank flows. Nash (1994) also stressed that a single rating relationship is inaccurate at high discharges.

Finally, measured data must be representative of annual conditions as sediment rating is strongly influenced by hysteresis caused by different seasons and whether sediment load is measured during the rising or falling limb of the hydrograph (Nash, 1994).

4.5.2.3 Sediment Transport Data at Ungauged Sites

At ungauged sites it will be necessary to use an appropriate sediment discharge function as no rating curve will be available. The application of a suitable sediment transport equation is vital and the SAM package is helpful because it includes guidance on the selection of equations best-suited to the type of river and bed material in question (Raphelt, 1990). Furthermore, Pickup and Warner (1976) demonstrated that although the use of two different equations encompasses a wide range of variability, they tend to yield near identical recurrence intervals for the effective discharge.

Theoretical bed load equations were used to calculate the effective discharge by Marlette and Walker (1968), Prins and de Vries (1971), Pickup and Warner (1976), Andrews (1980), Hey (1997a) and Goodwin et al. (1998).

4.5.3 Flow-Frequency Histogram Method

The recommended procedure to determine the effective discharge is executed in three steps. In Step 1, the flow-frequency distribution is derived from available flow-duration data (Figure 4.10). In Step 2, sediment data are used to construct a bed material load rating curve, or curves, or select an appropriate sediment transport function (Figure 4.11). In Step 3, the flow-frequency distribution and bed material load rating relationship or function are combined to produce a sediment load histogram (Figure 4.12), which displays sediment load as a function of discharge for the period of record and is a measure of effectiveness. The arithmetic mean of the discharge class in the histogram with the peak frequency is the effective discharge (definitions of the flowchart symbols used in Figures 4.10 to 4.12 are given in Appendix Figure D1).

Step 1 – Flow-Frequency Distribution (Figure 4.10)

1) Evaluate Flow Record

The flow record is a historic record of discharges at a gauging station. The record from a single gauging station can be used to develop the flow-frequency distribution if the gauge is in close proximity to the study site and the discharge record at the gauge is representative of the flow regime there. If a gauging record is either unavailable or unrepresentative, the flow-frequency distribution can be derived using either the Basin-Area or Regional Duration Curve method.

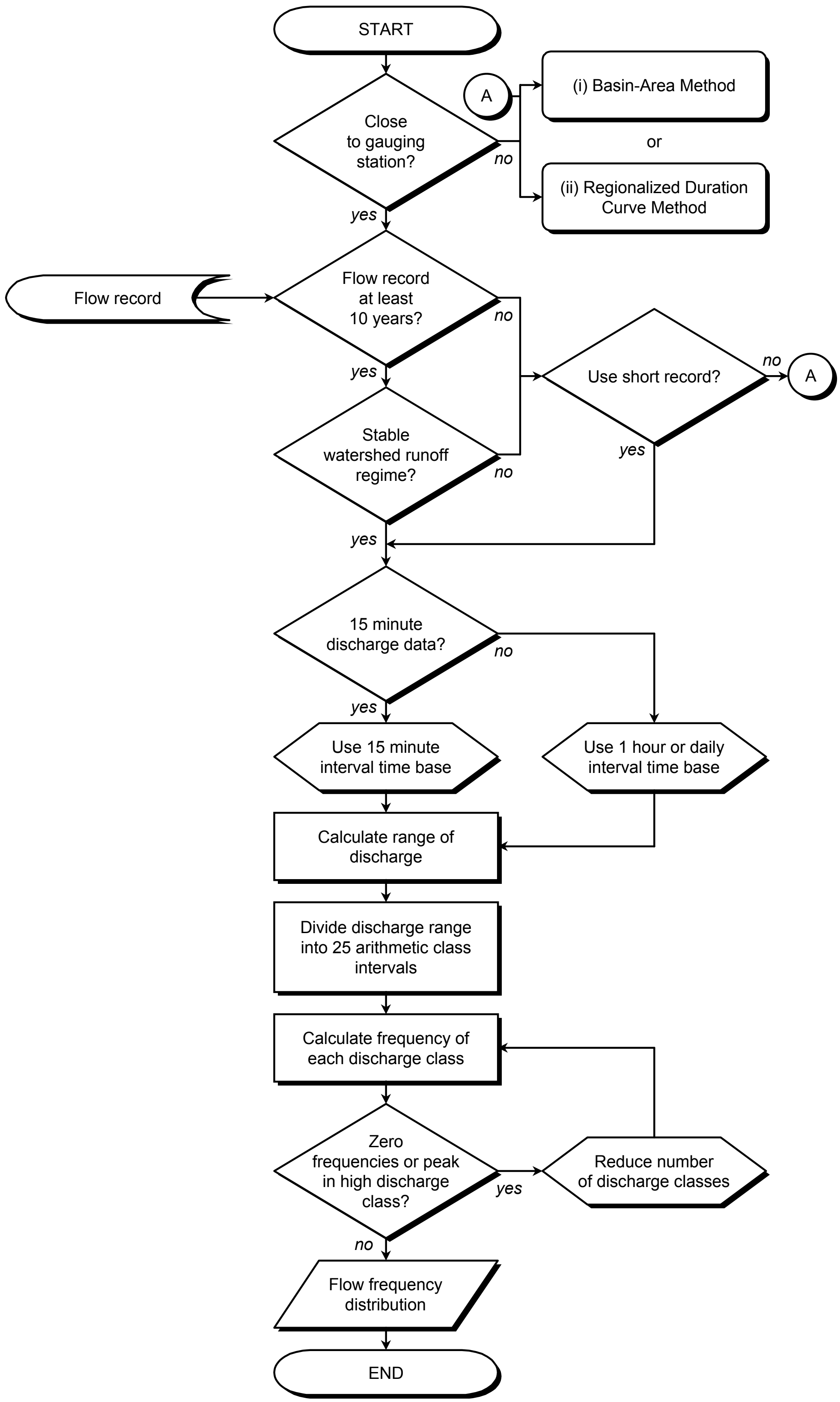


Figure 4.10 Procedure for generating a flow-frequency histogram.

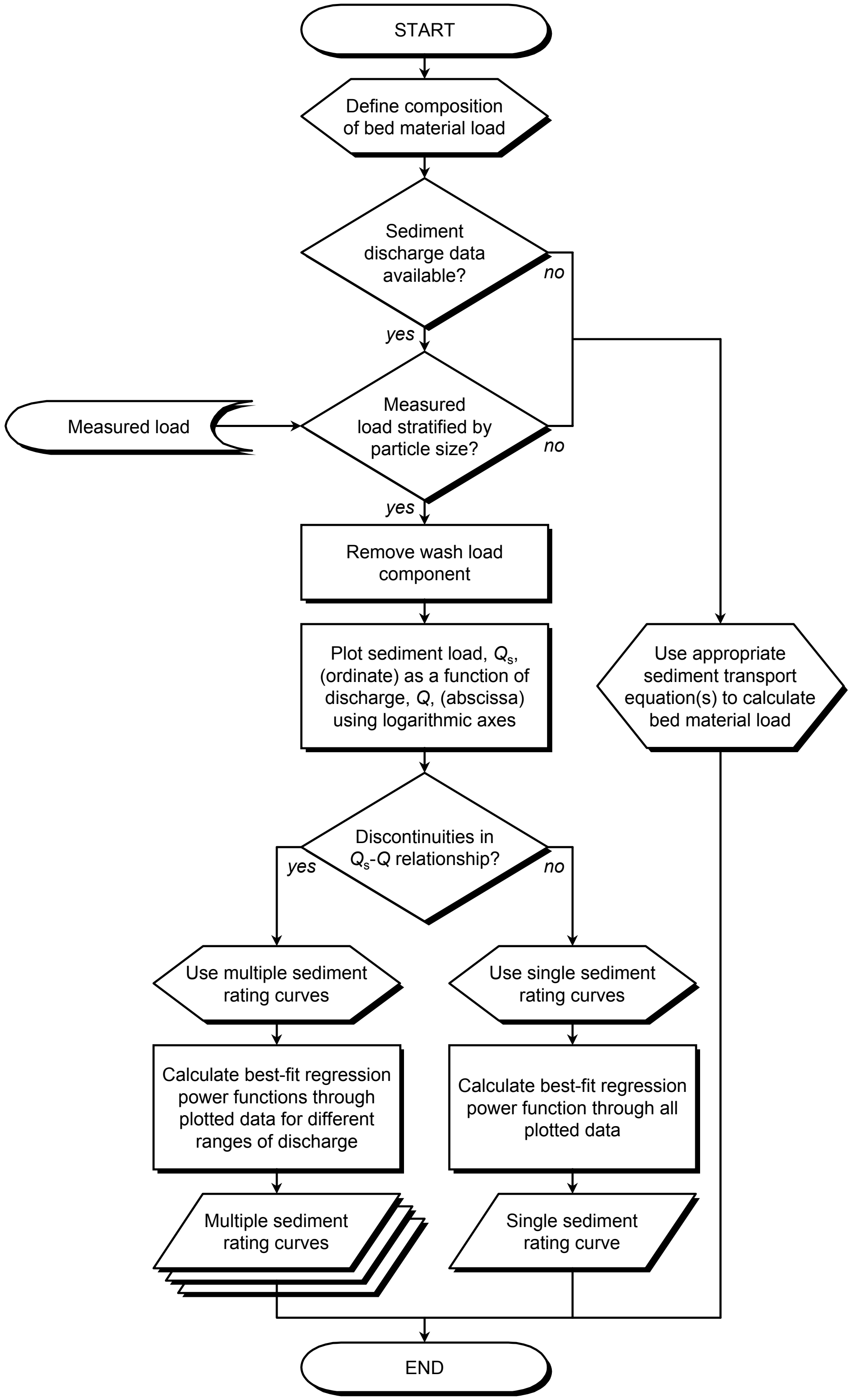


Figure 4.11 Procedure for generating a bed material load rating curve(s).

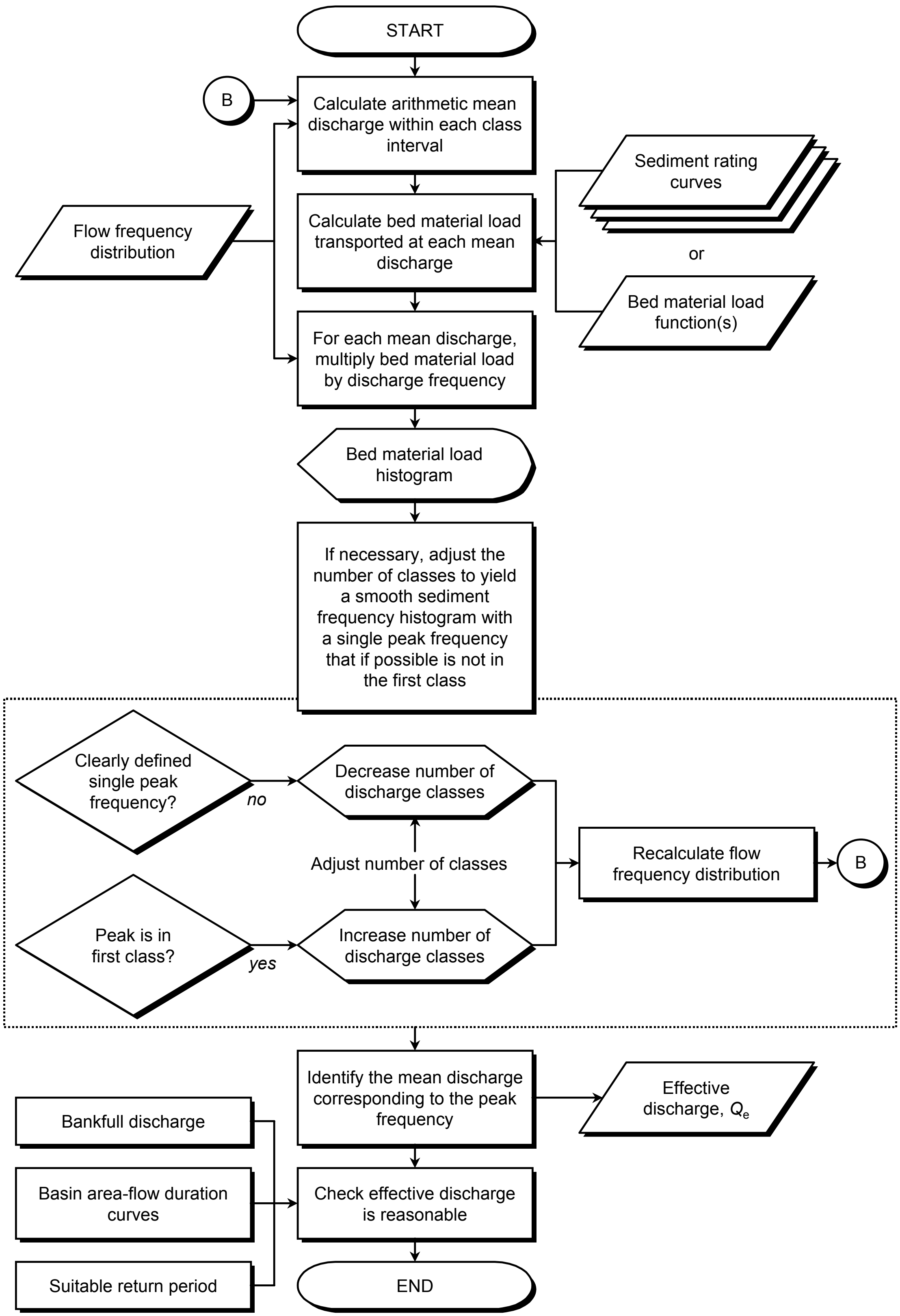


Figure 4.12 Procedure for generating a bed material load histogram.

2) Check the Period of Record and Stability of Run-off Regime

It is recommended that the length of period of record be at least 10 years and that measurements be continuous to the present day. Discharge data can still be used if there are short gaps in the record, but caution must be exercised when collecting data from a discontinuous record or disused gauge. The flow-frequency curve will not be representative of the natural sequence of flows over the medium-term if the length of record is less than 10 years or if the record has been influenced by changes in the watershed run-off regime. If this is the case, a flow-duration curve should be developed using either the Basin-Area or Regional-Duration Curve method.

3) Determine the Discharge-Averaging Time Base

To construct the flow-frequency distribution, the time base should be sufficiently short to ensure that short-duration, high-magnitude events are properly represented. If 15-minute data are unavailable, then either 1-hour or mean daily data can be used, but caution must be exercised when using mean daily data to develop a flow-frequency distribution for a stream which exhibits a flashy regime.

4) Calculate Discharge Range

The range of discharges is calculated by subtracting the minimum discharge in the flow record from the maximum discharge.

5) Calculate Discharge Class Interval

It is recommended that the initial attempt to construct the flow-frequency distribution should use 25 arithmetic class intervals. Therefore, the class interval is the discharge range, calculated in Step (4), divided by 25. The class interval should not be approximated by rounding. The relative proportions of the bed material load moving in suspension and as bed load should be estimated during site reconnaissance. For rivers in which the bed material load moves predominantly as suspended load, the first discharge class goes from zero to the class interval, the second class is the class interval to twice the class interval, and so on until the upper limit of the discharge range is reached. For gravel-bed rivers, where bed material load moves predominantly as bed load, the minimum discharge used in generating the flow-frequency distribution should be set equal to the critical discharge for the initiation of bed load transport.

6) Calculate Flow-frequency Distribution

The frequency of occurrence of each discharge class is determined from the record of observed flows. The calculation is simplified if flow-frequency is expressed as *percentage* occurrence. If a regional flow-duration curve has been developed for an ungauged site, then the frequency for each discharge class must be calculated using the equation for the curve, which is usually a power function. This can be achieved using the Flow-duration Curve Method (Section 5.4)

7) Check For Extreme Flow Events

It is recommended that all discharge classes display flow frequencies greater than zero and that there are no isolated peaks in individual classes at the high end of the range of observed discharges. If this is not the case, it is likely that either the class interval is too small for the discharge range, or the period of record is too short. Both zero frequencies and extreme flow events (outliers) can be eradicated by incrementally reducing the number of classes. Steps 5 and 6 are repeated for the new class intervals until a satisfactory flow-frequency distribution is produced. Caution must be exercised when reducing the number of intervals as a small number of classes may represent a considerable loss in empirical information.

Step 2 - Bed Material Load Rating Curve (Figure 4.11)

1) Define Composition of Bed Material Load

It is recommended that wash load (particles less than 0.063 mm) be excluded from the data set used to develop the sediment rating curve. If the bed material load moves both as bed load and suspended load, then bed load and suspended load measurements are required to determine the bed material load. If measured load data are insufficient, appropriate equations available in a hydraulic design package, for example SAM (Thomas et al., 1996), can be used to generate bed material loads for selected discharges.

2) Determine Sediment Data Availability

Sediment transport data are required to generate the bed material load rating curve. These data may be obtained from measurements at a gauging station if the gauge is in close proximity to the study site and the sediment record at the gauge is representative of the sediment load there. Otherwise, sediment transport data must be derived for the study site.

3) Plot Bed Material Load Data

The bed material load (y-axis) is plotted as a function of discharge (x-axis) in a scatter plot, with both axes on logarithmic scales.

4) Determine Sediment Rating Curve, or Curves

A power function best-fit regression line should be fitted to the data in the scatter plot to produce a bed material load function of the form given in Equation 4.2 as it is repeated here:

$$Q_s = \mathrm{a}\,Q^{\mathrm{b}} \tag{4.2}$$

where, 'Q_s' is bed material load discharge, 'Q' is water discharge and 'a' and 'b' are constants. Usually, discharge is given in m^3s^{-1} and sediment discharge is given in tonnes per day (ft^3s^{-1} and imperial tons if metric units are not used). If there are discontinuities in the relationship indicated by heterogeneous variance (often the relationship appears to underestimate at low discharge and overestimate at high discharges), then multiple sediment rating curves should be used for different ranges of discharge.

Step 3 - Bed Material Load Histogram (Figure 4.12)

1) Calculate Representative Discharges

The discharges used to generate the bed material load histogram are the arithmetic mean discharges in each class of the flow-frequency distribution.

2) Construct the Bed Material Load Histogram

The bed material transport rate for each discharge class is found from the equation(s) of the rating curve, or curves. This load is multiplied by the frequency of occurrence of that discharge class to find the average annual bed material load transported by that discharge class during the period of record. This method facilitates the calculation of average annual sediment yield, which is the sum of the sediment loads in each discharge class. The results are plotted as a histogram. Alternatively, the histogram may be expressed as total load rather than load per year by multiplying the frequencies by the number of years in the flow record. Originally, Wolman and Miller (1960) expressed sediment load frequency as a non-dimensionless frequency (modified from Wolman and Miller, 1960, p. 56):

$$P_i = \frac{100\left(Q_{s,i}\ F_i\right)}{\sum_{j=1}^{n}\left(Q_{s,j}\ F_j\right)}\% \qquad (4.40)$$

where '$Q_{s,i}$' is the rate of sediment load transported by flow 'Q_i', 'F_i' is the frequency of occurrence of flow Q_i and 'n' is the number of classes in the flow-frequency histogram. While this method has merit, it makes the annual sediment yield, which is frequently required from the analysis, more difficult to calculate.

3) Find the Effective Discharge

The bed material load histogram should display a continuous distribution with a single mode (peak). If this is the case, the effective discharge corresponds to the mean discharge of the modal class (that is the peak of the histogram). Greater accuracy may be achieved by incrementally increasing the number of classes and repeating the whole procedure, providing the distribution remains a smooth, unimodal function. If the modal class cannot be readily identified, the effective discharge can be estimated by drawing a smooth curve through the tops of the histogram bars and interpolating the effective discharge from the peak of the curve. Alternatively, the number of discharge classes may be decreased until a smooth unimodal distribution is produced.

A cumulative plot from the sediment frequency histogram is recommended to examine the sensitivity of effectiveness (the major sediment transporting flows) as a function of discharge and derive a *range* of effective flows (after Biedenharn and Thorne, 1994).

4) Check Effective Discharge is Reasonable

At the end of the procedure, it is important to check that the effective discharge is a reasonable value for the project reach. The return period for the effective discharge is expected to vary between sites as its value reflects the flow and sediment transport regime of the individual river or reach. For sites where annual maximum series flood flow data are available, it is sensible to check the return period of the calculated effective discharge to ensure that it lies within acceptable bounds. Suitable ranges of the dominant flow return period have been reported in the literature. Predicted effective discharge return periods outside the range of 1 to 3 years should be queried.

Alternatively, and if peak flow data are unavailable, the exceedance probability of the effective discharge should be calculated. Nixon (1959) showed that the bankfull discharge is equalled or exceeded on average 0.6 percent of the time using data from 29 rivers in England and Wales. While this may be used as a guideline, the range of effective discharge exceedance probabilities varies considerably in nature. Figure 4.13 shows the extent of this variability using the arithmetic-based effective discharges in Appendix Table A1 and Figure 4.14 is a cumulative plot of the percentage of sites for which a given effective discharge is equalled or exceeded. At least 25 percent of sites have exceedance probabilities less than or equal to 5.1, 50 percent of sites have exceedance probabilities less than or equal to 11.2 percent and 75 percent of sites have exceedance probabilities less than or equal to 15.8 percent. Less than 10 percent of sites have probabilities less than or equal to the 0.6 percent given by Nixon (1959). Predicted effective discharge exceedance probabilities greater than 10 percent should be queried and the flow distribution examined for deviations from log-normality. For example, the six sites in Figure 4.13 with exceedance probabilities greater than 30 percent all have kurtosis greater than 0.5 or skewness greater than 0.46 (both based on the logarithm of discharge) and all have log-standard deviations less than 1.0. These criteria tend to increase the effectiveness of high frequency discharges.

A further check is to compare the duration of the effective discharge with basin area-flow-duration curves. The percentage of the time that the effective discharge is equalled or exceeded should be compared to the expected range of values reported in the literature. Graphs which express duration as a function of drainage area can be used to assess whether the duration of the effective discharge is comparable to the results of other studies (e.g. Watson et al., 1997, based on data from several U.S. streams, after Andrews, 1980, 1984; Biedenharn et al., 1987). It is not intended that these types of graph be used to predict effective discharge as large errors are likely to result because of the considerable variability of data points.

Finally, a morphological check should be undertaken to compare the effective discharge to the bankfull discharge. This is best performed by identifying the bankfull reference level at a stable cross section and calculating the corresponding discharge, for example, by using the stage-discharge relationship at a nearby gauging station.

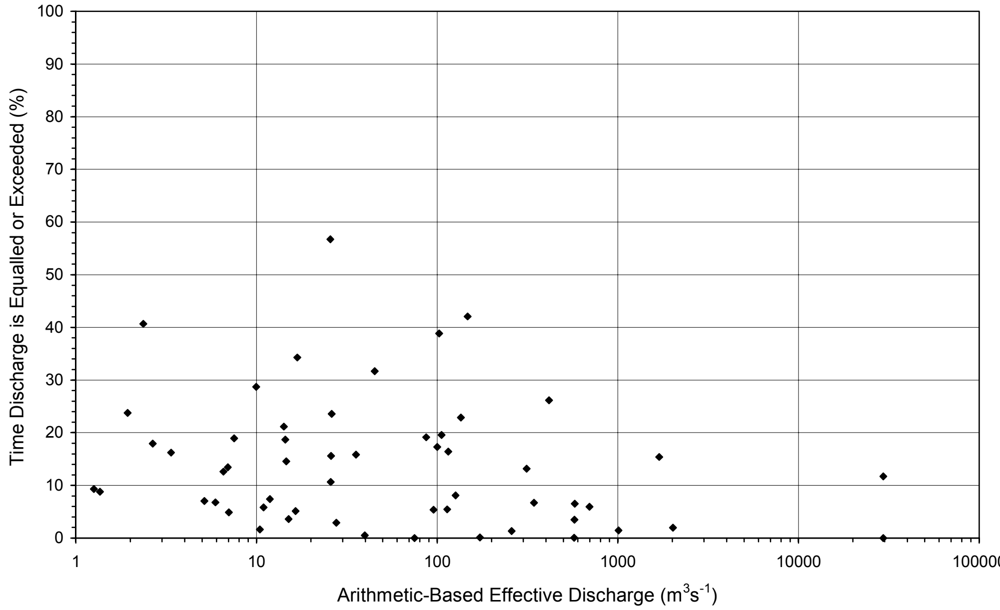


Figure 4.13 Variability of effective discharge exceedance probabilities (data set of 55 U.S. sites).

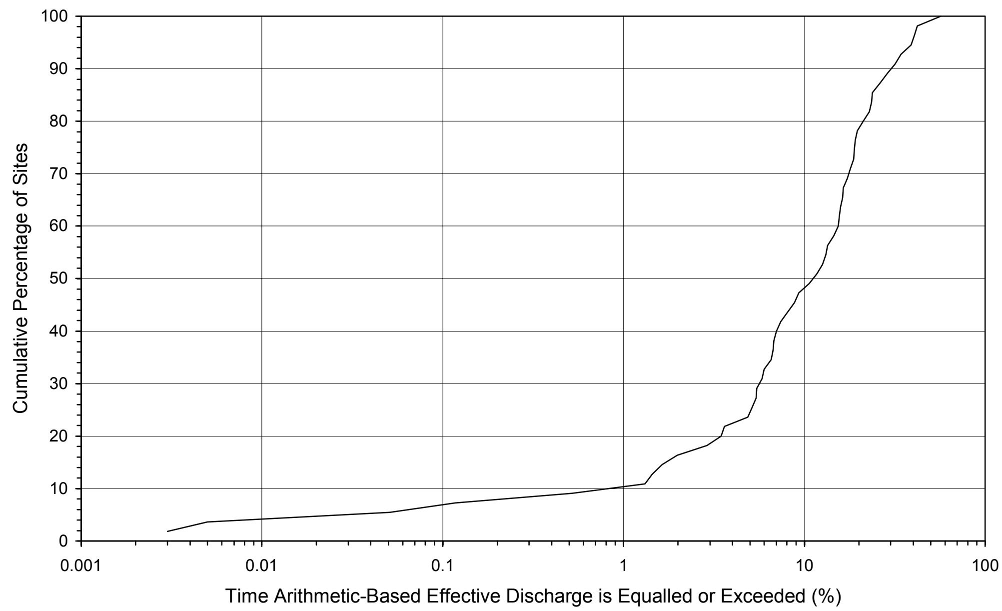


Figure 4.14 Cumulative plot of the percentage of sites less than or equal to a given effective discharge exceedance probability (data set of 55 U.S. sites).

4.5.4 Flow-duration Curve Method

This technique derives a flow-frequency histogram from a flow-duration curve, which is a relationship between discharge and the cumulative frequency of discharge occurrence over a period of flow record. Often a flow-duration curve is the only flow data obtainable since the historical flow data used in the Flow-frequency method is not readily available. The frequency of each discharge class is calculated as the percentage exceedance of the lower bound discharge of the class subtracted by that of the upper bound discharge. Once the flow distribution has been calculated, the remaining stages in the calculation of the effective discharge are the same as the Flow-frequency Method (Section 4.5.3). The advantage of this method is that the flow-duration curve is based on continuous data rather than discrete data (used in a frequency distribution). The frequency distribution that is derived from a flow-duration curve will tend to be smoother than that based on the database of flow measurements. This method was used to calculate the effective discharge of 10 northern Mississippi streams by Watson et al. (1997).

4.5.5 Recent Methodological Developments

Recently, several methodological developments have attempted to improve the effective discharge calculation in an attempt to alleviate the approximation error caused when in assigning the effective discharge to the mean of the modal class, rather than some other value in that class. Orndorff and Whiting (1999) recognised that the effective discharge is highly dependent on the level of discretisation (using a set number of classes) involved in the Flow-frequency Histogram method and that fitting a statistical distribution, such as the log-normal probability density function assumed by Nash (1994), may misrepresent the empirical distribution of flows, particularly with bimodal or polymodal distributions. They suggest using statistical software to derive an ‘empirically based’ (rather than theoretical) probability density function (PDF) of stream flow from the actual flow record, which is then multiplied by a sediment rating relationship of the form given in Equation 4.2, or Equation 4.37 for a gravel-bed river.

Similarly, following an examination of magnitude-frequency characteristics in the Fraser River, British Columbia, Sichingabula (1999) recommended calculating an ‘event-based’, rather then the conventional ‘class-based’ effective discharge, defined as the maximum sediment load in tonnes of individual events *without* dividing them into classes. Sichingabula found that a different effective discharge is obtained for every different class size used in its calculation and concluded that 'the problem with the conventional method of determining the effective discharge is the need to discretise the time series and pull out an isolated range rather than recognise the overall variability and episodic nature of sediment transport events' (p. 1371). Furthermore, Appendix Table A1 reveals that the exceedance probability of effective discharges falling in the first discharge class may be very low (for example, Long Creek near Pope, MS, at 3.6 percent), which means that the arithmetic mean of the first class may represent a very high percentage of the range of flows experienced by the river. Significantly decreasing the size of discharge interval results in greater precision at low discharges and may, therefore, alleviate this type of approximation problem.

Although computationally possible, there are two potential problems with a truly event-based effective discharge. The first problem is based on the statistical precept that the occurrence frequency of an 'exact' discharge tends towards zero with increasing precision. Secondly, a cumulative probability distribution based on individual events over a specific time period will exhibit considerable variability (white noise) superimposed on the general shape of the curve (trend). This is the inevitable result of a sample flow distribution based on a specific sampling time base. Assuming stationarity in drainage basin controls, this empirical variability is stochastic and will not be repeated. While this method can more accurately describe an empirical distribution of effectiveness (the product of magnitude and frequency) in some cases, further research is required to convert this mathematically rigorous method into a statistically sound, practical procedure suitable for the end user community.

To overcome these limitations, it is possible to use very small discharge intervals and calculate class frequency from an event-based empirical *cumulative* flow distribution (flow-duration curve), to simulate an event-based frequency distribution and give a quasi-event-based effective discharge. Also, the effective discharge (and average annual sediment yield) should be based on the general trend of the frequency distribution, rather than the empirical variability that is specific to the period of record only. This can be achieved by one of many statistical techniques for 'removing the trend'. Two of the simplest and most effective methods are 'polynomial regression smoothing' and 'moving average smoothing', both widely used in time series analysis (Box and Jenkins, 1994). Polynomial regression smoothing involves fitting a polynomial to a cumulative flow distribution. As Chow (1964, p. 8-5) notes: 'For practical purposes, however, it is sometimes necessary to treat arbitrarily the discrete variables as continuous variables by fitting a continuous function to the variates'. This has been undertaken by Oldenburger (1996), who used a quadratic equation in logarithmic space with some success to represent the flow-duration relationship of Napa Creek, California. Although quadratic and cubic expressions are often used in time series analysis, they do not fit all flow distributions and increasing the order of the regression, while improving the interpolation quality, imposes global assumptions about the nature of the data and often produces artefacts (Diggle, 1990, p. 25).

The advantage of the moving average smoothing operation is that the technique can be applied to all distributions, including polymodal distributions, without such global assumptions. Nash (1994) found that only 31 percent of rivers studied had unimodal flow-frequency distributions, while 52 percent exhibited bimodal distributions and a further 16 percent had polymodal distributions (greater than two dominant peaks). As the form of the empirical distribution deviates from the ideal unimodal case, a flow-frequency histogram with 'n' classes may not represent the true distribution. A useful technique to gradually reveal the general trend in an empirical distribution is to use successive applications of a simple (non-weighted) moving average of order 3 (the repeated mean of the Q_{i-1}, Q_i and Q_{i+1} frequencies) (Diggle, 1990, p. 23). Applying a higher-order moving average may result in considerable loss of information within bimodal and polymodal distributions. As the effective discharge is always greater than the modal discharge, only frequencies of discharges greater than the peak discharge should be smoothed. If all frequencies are smoothed then the modal discharge frequency may be significantly underestimated. Based on limited experience while developing the method, a suitable smoothing guide for greater than 25 classes is to use int((n-n_m) /25) smoothing repetitions, where 'n' is the number of classes, 'n_m' is the modal discharge class and int(x) refers to the integer part of the quotient 'x'.

Using this technique, it is possible to improve the Flow-Duration Curve method to accurately represent the form of the empirical flow occurrence distribution with a smooth histogram and thereby, significantly reduce approximation error to within acceptable limits for the capacity of the channel. This technique may involve developing a cumulative distribution from thousands of discharge measurements (depending on the sampling time base) and interpolation of the resultant flow-duration curve for hundreds or thousands of classes (appropriate to the size of the database and capacity of the channel). Therefore, handling this quantity of data requires a computer program, which probably explains why the method has not been developed previously. An example of this technique is shown in Figure 4.15 for the Delaware River at Trenton, NJ (Appendix Table A1). For this case study, an event-based frequency distribution is approximated by using 1000 discharge classes, giving a class size of only 3.68 m^3s^{-1} compared to 147.18 m^3s^{-1} if the conventional 25 classes were used.

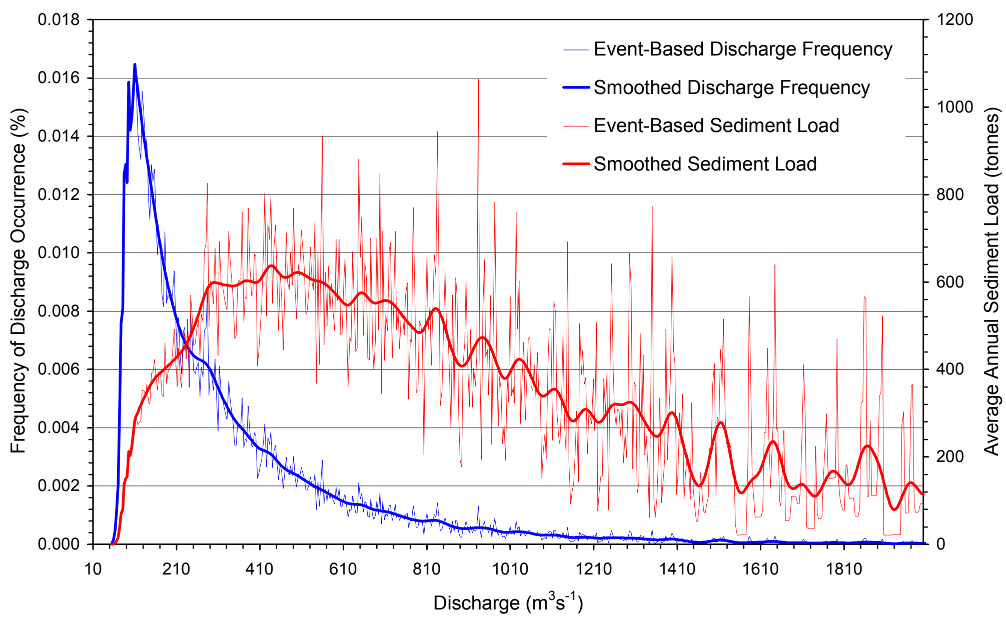


Figure 4.15 Flow and sediment frequency distributions for Delaware River at Trenton, NJ. Event-based distributions are based on 1000 arithmetic discharge classes. Smoothed distributions are based on a repeated moving average of order 3.

The effective discharge of the event-based distributions is given as 933.4 m^3s^{-1}. By extracting the trend of this distribution, the effective discharge is given as 436.5 m^3s^{-1}. The elevated event-based effective discharge is a function of the empirical frequency variability at intermediate to high discharges, which has a large impact on the variability of sediment load. While a discharge 933.4 m^3s^{-1} appears to be the most effective discharge for the period of record, the peak is an isolated event and not representative of the general trend of effectiveness. The probability of this discharge being the effective discharge for another period of record (such as a post-project monitoring period for a restored river) is very low. The effective discharge of the smoothed distribution is more realistic of the general form of the distribution and, if used to restore stable channel dimensions, has the highest probability of being the most effective discharge in the restored channel (assuming stationarity in drainage basin controls). The effective discharge calculated using only 25 arithmetic classes is 416 m^3s^{-1} (see Appendix Table A1). Therefore, by significantly alleviating the problem of discretisation (and therefore, approximation error), the quasi-event-based effective discharge gives a greater degree of accuracy in discharge effectiveness. In the case of the Delaware River site, the conventional 25-class frequency distribution underestimated the effective discharge by approximately 20 m^3s^{-1}, however Figure 4.15 shows that there is a range of discharges between about 300 m^3s^{-1} and 575 m^3s^{-1} that transport similar magnitudes of sediment load.

4.6 CONCLUSION: EFFECTIVE DISCHARGE TIME-EVENT COMPRESSION

Pickup and Warner (1976) disputed the equivalence of bankfull and effective discharges. They calculated an effective discharge with a recurrence interval between 1.15 and 1.4 years, less than the most probable annual flood of Dury (1973, 1976) and Hey (1975) and significantly less than the measured bankfull discharge with a frequency ranging between 4 and 10 years. This inequality is supported by Benson and Thomas (1966), Webb and Walling (1982), Nolan et al. (1987) and Lyons et al. (1992). All these observations indicate that discharges more frequent than the modal discharge are the most effective sediment-transporting events, while more extreme events are probably responsible for width adjustment. The importance of the rarer events is stressed by Baker (1977) who found that it is these events which are more 'effective' at controlling channel form when the flow record depicts a high proportion of large events and the channel has resistant boundaries. Therefore, it follows that there may be two distinct groups of discharge that control the overall channel form: intermediate to extreme flows which control channel capacity, and; more frequent flows which convey the majority of the sediment load and shape resulting bedforms and instream features. The effective discharge involves time-event compression, which represents a time series by a unique flow with a unique frequency. However, based on 21 streams in the Saskatchewan basin, Ashmore and Day (1988) concluded that that no generalisation can be made as to the recurrence frequency of the effective discharge. Nash (1994) supported this for 55 rivers across America covering a wide range of physiographic regions. In light of this, the research by Pickup and Warner (1976), and others, suggests that each flow produces its own particular channel response that depends on the magnitude of the event and the initial state of the channel (Pickup and Warner, 1976, p. 366).

The term dominant discharge suggests that other flows are minor or negligible, when in fact they may have a large influence on channel morphology as demonstrated by Biedenharn and Thorne (1994). Despite the assertion by Richards (1977) that 'no single discharge frequency is equally important as 'channel forming' at all positions', it can also be argued that the channel forming flow is a socially constructed concept that may occur in nature for any significant period. The concern that a single dominant discharge is too severe a simplification of the actual flow regime suggests a need to move toward multiple dominant discharges. This concept was developed by Kennedy (1972) and Pickup and Reiger (1979) who stated that every competent flow event exerts some influence on channel form so that the shape and size of the channel at any time is a weighted sum of effects of all discharges occurring prior to this time. They inferred that the conventional 'dominant discharge-static regression' approach is a valid approach only when temporal variations in discharge are small or when the channel geometry parameter of interest is insensitive to variations in discharge (Pickup and Reiger, 1979, p. 1).

From a sediment control perspective, Biedenharn and Thorne (1994) suggested that better analysis of the cumulative sediment transport curve is necessary to identify the range (rather than single event) which transports the dominant sediment load. They identified thresholds within the curve which appeared to correspond to sedimentary features of an alluvial channel in regime. Magnitude-frequency analysis of the lower Mississippi revealed that the dominant discharge corresponded to 'bar-full' discharge and that an effective range of channel-forming flows occurs between the stage that just tops mid-channel bars and that which significantly overtops the banks. This range is bounded by the 40 percent and 3 percent exceedance probabilities and is responsible for transporting approximately 70 percent of the sediment load in the Lower Mississippi. Despite advocating the use of dominant discharge, Wolman and Miller (1960, p. 65) also stressed that the channel shape is affected by a range of flows rather than a single discharge. Furthermore, Carlston (1965) concluded that no single flow event is responsible for meander wavelength. In response to this finding, Wolman (pers. comm. to Carlston, 1965, p. 880) suggested that there is possibly a range of effective flows between mean annual and bankfull discharge that is responsible for controlling meander planform dimensions.

River morphology is controlled and shaped by both destructive and constructive processes such that the effectiveness of a destructive event depends upon the force exerted, the return period of the event (Wolman and Miller, 1960) *an*d the magnitude of the constructive or restorative processes which are more frequent and occur during the intervening intervals (Wolman and Gerson, 1978, p. 190). Hence, river regime must represent a balance between destructive and restorative processes and, therefore, infrequent and frequent flows. On this basis, it is recommended that further research into effective discharge should focus on identifying a range of effectiveness which is causally linked to channel morphology and, therefore, can be used in channel restoration design to restore instream sedimentary features and physical habitat as well as the basic stable channel dimensions. Identifying a range of effective flows will require a more event-based approach to magnitude-frequency analysis, rather than the conventional class-based technique, and further research should be directed towards testing the methods discussed in Section 4.5.5 for a wide range of rivers with different types of flow-frequency distributions and sediment rating relationships.

Magnitude-Frequency Analysis (MFA) has been well documented in the engineering and geomorphological literature since the 1960 paper by Wolman and Miller. However, step-by-step guidance on undertaking the analysis has not been made available. At the onset of this research, the objective of this chapter was to develop a standardised procedure for carrying out MFA and obtaining an objective estimate of the effective discharge. This discharge could then be used as the channel-forming discharge for channel restoration design on the basis that using the bankfull discharge is too subjective as identification of the bankfull reference level from field indicators is problematic. However, through a detailed examination of the components of MFA it became apparent that the type and number of discharge class intervals used in the procedure results in different estimates of the effective discharge. In light of this finding, a quasi-event-based approach to the analysis has been formulated which overcomes the class-related problems of the conventional approach. The new methodological development uses very small discharge classes to provide a very detailed sediment-frequency histogram from which a smoothed-frequency distribution can be derived, thereby providing a more accurate representation of discharge effectiveness.

Calculating the channel-forming discharge is a critical stage in the channel design procedure developed in Chapter 3 as the channel slope, cross section dimensions and meander planform geometry (described in later chapters) are related either directly to the channel-forming discharge, or indirectly through its relationship with channel width or sediment transport. In particular, accurate estimation of the channel-forming discharge is imperative in hydraulic geometry analysis. In the following chapter a series of enhanced width equations are developed for channels with different types of bed and bank characteristics and levels of uncertainty. These empirical equations are essential to overcome the indeterminacy problem in stable channel design, as bankfull discharge and width provide the input parameters for the analytical determination of depth, slope and sinuosity discussed in Chapter 6.

CHAPTER 5

Enhanced Width Equations

5.1 INTRODUCTION: BACKGROUND AND RESEARCH OBJECTIVES

The design and engineering of stable alluvial channels with mobile bed materials may be approached through theoretical and empirical analyses. However, to date no completely theoretical approach capable of designing channels with the morphological complexity of natural rivers has been derived. Reliance on empirical methods makes the acquisition and analysis of field data from stable channels with natural attributes of paramount importance. An important component of empirical approaches to stable channel design rests on downstream hydraulic geometry analysis. The approach employs a statistical treatment of data sets linking flow regime, sediment characteristics and resulting channel forms under dynamically stable conditions (Chapter 2). The principle of downstream hydraulic geometry is a central tenet of channel restoration design (Chapter 3).

The hydraulic geometry equations required in the channel restoration design procedure are those expressing bankfull width as a function of bankfull discharge, for different types of bed and bank characteristics, and a generic meander wavelength equation expressed as a function of bankfull width. These relationships exhibit the least variability as opposed to other combinations of the dependent and independent variables (for examples see Hey and Thorne, 1986, and Williams, 1986). This chapter summarises a series of investigations aimed at improving existing equations by combining data sets, where appropriate, 'typing' equations according to bed and bank characteristics and incorporating natural variability into estimates of stable width. Meander wavelength is discussed in Chapter 7.

There is a wealth of gravel-bed data documented in the geomorphology and engineering literature which describe both static and mobile bed downstream hydraulic geometry in straight and meandering rivers (for example, Leopold and Madock, 1953; Leopold and Wolman, 1957; Nixon, 1959; Nash, 1959; Charlton et al., 1978; Andrews, 1984; Hey and Thorne, 1986). In contrast, morphological equations for stable sand-bed channels usually refer to either alluvial canals in regime (e.g. Lacey, 1930; Simons and Albertson, 1960) or laboratory channels (e.g. Ackers, 1964). While the extensive regime studies of alluvial canals in America, Egypt, India and Pakistan between the 1930s and 1960s generated an exhaustive set of morphological equations for straight alluvial canals, the hydraulic geometry of stable meandering rivers with sand beds is relatively poorly understood. Furthermore, there is a paucity of hydraulic geometry analyses in the geomorphology and engineering literature, which address stable channel dimensions in sand-bed rivers with different bank characteristics.

Acquisition of a data set originally collected by J. C. Brice, United States Geological Survey (USGS), the 'Brice Data Collection', by UAEWES/ERDC in 1998 and further field data collected from a selection of the Brice sites between 1998 and 1999 made possible development of improved equations for the stable width of alluvial channels with sand-bed materials.

One of the most extensive alluvial river data sets has been assembled by Osterkamp and Hedman (1982) and describes downstream hydraulic geometry pertaining to the 'active channel' width, rather than the conventional bankfull width, for North American Rivers. This data set is examined together with other existing data sets in terms of their applicability to channel restoration design.

The objectives of this chapter are to: i) review existing downstream hydraulic geometry equations for the stable bankfull width of mobile sand-bed and gravel-bed rivers; ii) review the Brice data collection and summarise the field and analytical methods adopted to update the database; iii) derive new bankfull width equations based on hydraulic geometry analyses for sand-bed and gravel-bed rivers from existing and new data which incorporate uncertainty in their estimates, and; iv) examine the extensive Osterkamp and Hedman data set and associated hydraulic geometry equations and discuss their utility value as design equations.

Natural variability equations suitable for engineering application are given at the end of chapter together with corrections for bias due to logarithmic transformation of variables required in hydraulic geometry analysis.

5.2 DIMENSIONAL AND DIMENSIONLESS EQUATIONS

Regime-type equations are often expressed as dimensional models. For example, the coefficient in a metric width-discharge expression has dimensions $m^{1-3b}s^{b}$ and the coefficient in a meander wavelength-width expression has dimensions m^{1-b}, where 'b' is the relevant exponent. The dimensional regime model has been criticised for being 'unscientific' (Inglis, 1948) and lacking a rational basis consistent with Froude and geometric similarity (Parker, 1982). According to Church and Mark (1980), this can lead to incorrect interpretation and comparison of coefficients. Bray (1982), Parker (1982) and Andrews (1984) have developed dimensionless hydraulic geometry equations for gravel-bed rivers. The types of dimensionless groupings adopted are given in Table 5.1.

	Width	Discharge
Bray (1982)	$\frac{Wg^{0\cdot2}}{Q^{0\cdot4}}$	$\frac{d_{50}\,g^{0\cdot2}}{Q^{0\cdot4}}$
Parker (1982) and Andrews (1984)	$\frac{W}{d_{50}}$	$\frac{Q}{(G_s-1)^{0\cdot5}g^{0\cdot5}d_{50}^{2\cdot5}}$

Note: W = width; Q = discharge; d_{50} = median sediment size; G_s = specific gravity of sediment; g = acceleration due to gravity.

Table 5.1 Dimensionless groupings used in non-dimensional width equations based on downstream hydraulic geometry analysis.

Hey and Heritage (1988) compared the two types of dimensionless groupings in Table 5.1 using empirical and rationally based data sets and revealed considerable differences between their reduced forms, expressing width as a function of discharge and median sediment size. The discrepancies were attributed to covariance in the non-dimensional models. The Bray model produces little variance in dimensionless width because width and discharge are positively correlated, resulting in a suppressed R^2 value. Conversely, the Parker and Andrews model produces a large variance in dimensionless width because width and median sediment size are often negatively correlated (Section 5.4), albeit not strongly, resulting in an elevated R^2 value. This spurious correlation compromises the basic assumption of the linear regression model and is complicated by the fact that Q appears in both sides of the Bray model and d_{50} appears in both sides of the Parker and Andrews model and act as scaling factors (Rhoads, 1992). Furthermore, the conventional dimensional width model is bivariate, in that discharge is the only independent variable in the equation. However, the reduced form of the non-dimensional model contains median particle size as a second parameter. As the coefficient of determination, R^2, always increases with the number of regressor variables (regardless of their significance), the reduced dimensionless equations can appear misleadingly to account for more of the variance than their dimensional counterparts. In light of these implications, non-dimensional regime-type relationships are considered unsuitable for developing design equations for bankfull width and wavelength and the usual dimensional form of the equations are assumed from this point forward.

5.3 SINGLE AND COMPOSITE DATA SETS

In general, data sets used in hydraulic geometry analysis are regionally based and apply to a particular locality, rather than 'type' of channel. Consequently, applying the resultant morphological equations beyond the parent region must be exercised with extreme caution (Burns, 1998). However, as there are insufficient equations to represent adequately the wide range of physiographic regions and hydrological provinces found in nature, alternative solutions are required.

One such solution is to combine data sets pertaining to stable channels with similar types of bed and/or bank characteristics and derive more generic equations from the composite data sets. Arguably, this method represents a significant improvement on existing equations since the composite equations are more applicable on the basis that: i) the parent data set is less regional and more closely associated with actual channel morphology; ii) hydraulic geometry equations have greater statistical significance with increasing sample size, and; iii) it is likely that the parent data set has greater ranges of discharge, slope, sinuosity, bed material size, etc, than the individual data sets. The latter point is particularly important as width equations derived from data sets with small discharge ranges tend to have exponents which deviate significantly from the expected value of 0.5, based on existing regime, hydraulic geometry and analytical equations (Chapter 2). In light of these points, data sets pertaining to similar types of channel have been combined where possible for the purpose of formulating a set of channel design equations suitable for restoring river channels.

5.4 MINIMUM SAMPLE SIZE FOR WIDTH EQUATIONS

When available design equations are inapplicable because the stream conditions at a project site are markedly different from the sites used to derive the equations, it is recommended that nearby reference reaches should be used to derive new morphological relationships. To achieve this, significant fieldwork effort may be required and it is essential, therefore, that time is used effectively during data collection. A frequently posed question concerns the number of sites required to derive a statistically significant relationship. A general orthodoxy is that a minimum of 30 points is required if statistical assumptions are not to be significantly compromised. However, an investigation using U.K. gravel-bed data reveals that this may be an over-approximation if a suitable typing scheme is adopted when deriving a relationship.

The investigation used the Hey and Thorne (1986) data set for mobile gravel-bed rivers in the U.K. This data set has 62 sites comprising 29 sites with riparian vegetation density less than or equal to 5 percent (erodible banks) and 33 sites with riparian vegetation density greater than 5 percent (resistant banks). The statistical procedure adopted is outlined below:

i) Based on previous work (Chapter 2), it was assumed that the exponent in the power relationship, $W = aQ^b$, between bankfull width, W, and bankfull discharge, Q, can be fixed at 0.5 with minimal error. By using this principle, the coefficient, a, was calculated for each site. As hydraulic geometry requires logarithmic transformation of the variables, the natural logarithm of each coefficient, r, was calculated for each site, where $r = \ln(a)$.

ii) Using a computer algorithm, two random values of r were chosen and the mean value calculated and transformed back onto an arithmetic scale to derive the mean coefficient, a, for the two sites.

iii) Step ii was repeated 1000 times and the percentage of trials for which the mean coefficient, a, fell within the 95 percent confidence limits of the mean coefficient for the whole data set was recorded. This parameter is termed by the authors as the *hydraulic geometry representation* (percent) and is essentially a measure of how well the equation derived from fewer samples approximates the equation derived from all the sample data.

iv) Steps ii and iii were then repeated for increasing sample sizes, n, until $n = 62$ (the full data set).

Theoretically, hydraulic geometry representation should increase as sample size increases and the true equation approached. This is shown graphically in Figure 5.1 with sample size on the abscissa and percentage representation on the ordinate.

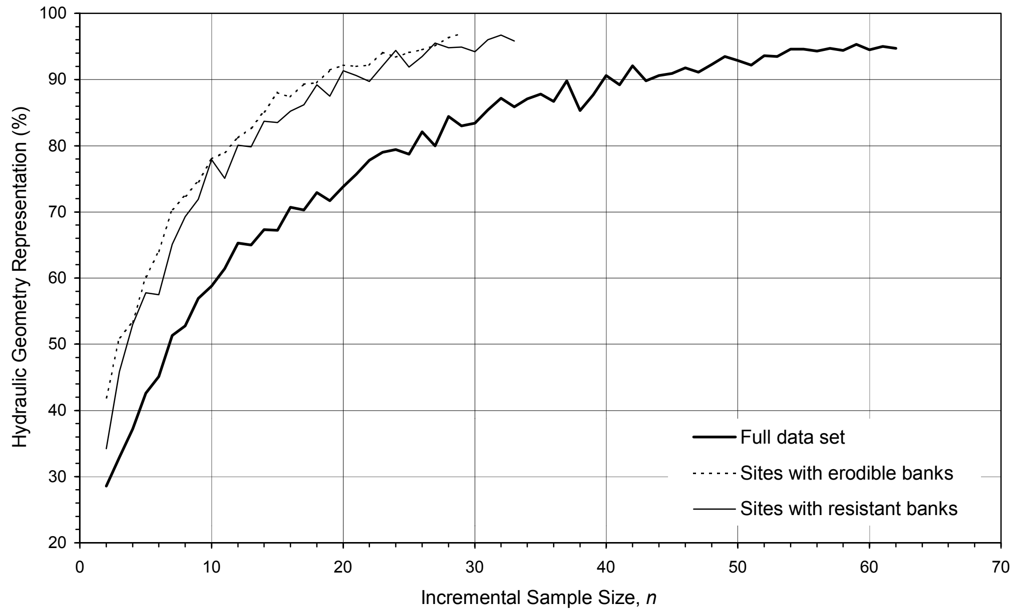


Figure 5.1 Effect of sample size on the ability to represent a true hydraulic geometry relationship of the form $W = aQ^{0.5}$ for different stream types. Hydraulic geometry representation (ordinate) is the percentage frequency that the mean coefficient, a, for a specific sample size (abscissa), is expected to fall within the 95 percent confidence limits of the mean coefficient for the whole data set, with repeated sampling. Source data: Hey and Thorne (1986).

For the non-typed data set, the equation produced from 30 random samples represents approximately 85 percent of the true relationship with all 62 samples. Below 30 samples the hydraulic geometry representation decreases rapidly. Therefore, an adequate equation could have been produced with approximately 25 to 35 samples, whereby further samples do not significantly improve the quality of the published equation.

When the data are typed by a simple bank classification scheme, the degree of variability expressed by outlying data points is considerably reduced and a greater percentage of sites plot near the regression line. In terms of sample size, Figure 5.1 reveals that fewer sites than the non-typed case are required to derive a statistically acceptable relationship. In fact, 90 percent hydraulic geometry representation is achieved with approximately 18 sites. Even 10 random sites yield a value of approximately 78 percent representation for the two typed data sets.

In summary, when deriving a regional or typed hydraulic geometry relationship that expresses bankfull width as a function of bankfull discharge, the minimum number of sites required to yield a statistically acceptable relationship is greatly reduced if streams are ‘typed’ by bank characteristics. For the example case, only 10 to 20 sites are required to construct a relationship which mimics the true equation by approximately 80 to 90 percent.

5.5 CONTROLS ON WIDTH-DISCHARGE RELATIONSHIPS OF SAND-BED AND GRAVEL-BED RIVERS

Sand-bed channels are usually found in the middle and lowland zones of a river system and are characterised by well-developed alluvial floodplains. The median size of bed material is usually taken to be between 0.063 mm and 2 mm in diameter and channel slopes tend to be less than 0.001 (Bathurst, 1997). Both bed load and suspended load transport occur over a bed characterised by ripples and dune features. The bank material is predominantly comprised of cohesive or sandy material. Cohesive-bed channels have a median sediment size of less than 0.063 mm in diameter and behave differently to sand-bed channels due to pore water pressures in the channel boundary. Cohesive-bed rivers are not considered in this report.

Gravel-bed channels are usually found in the piedmont zone of a river system, although mixed-bed streams of sand and gravel-bed material may be found in the middle and lower zones. The median size of bed material is usually taken to be between 2 mm and 64 mm in diameter and channel slopes tend to lie in the range 0.0005 to 0.005 (Bathurst, 1997). The bed material is mainly transported as bed load. Ripples and dune features are generally not found in gravel-bed rivers, and the dominant channel feature is the pool-riffle couplet. The bank material is generally either cohesive sediment or a composite structure of finer sediment overlying gravel (Thorne and Tovey, 1981). Cobble-bed channels have a median sediment size greater than 64 mm in diameter. Boulder-bed and bedrock rivers behave different to alluvial gravel-bed rivers and are not considered in this report. As Church and Rood (1983, p. 1) noted, there is no reason to expect channels with non-erodible boundaries to exhibit consistency in channel form.

Theoretically, there should be different morphological relationships for sand-bed and gravel-bed rivers. Although the same general flow processes operate in both channel types, the interaction between the processes and resultant forms are markedly different. Gravel-bed rivers are characterised by macro bedforms scaled on the channels’ dimensions such as pools and riffles, and are generally absent of smaller scale ripple and dune features. In light of these differences, Hey and Thorne (1986, p. 671) argued that the hydraulic geometry of gravel-bed and sand-bed rivers are different. However, there are no general formulae which relate sediment properties to relative erosional resistance and hence to channel shape (Richards, 1982, p. 161). For example, the bed and side factors of Blench (1969, 1970) are in fact flow-based indices and correlate poorly with perimeter sediment properties (Kellerhalls, 1967).

Discharge explains the most variability in width but if rivers are typed by bed and bank material for width equations derived from hydraulic geometry analyses, there are three kinds of influence that must be considered: i) the influence of bed material on width; ii) the influence of bank material and vegetation on width, and iii); the influence of bed material load on width.

The influence of the bank character on width is determined by the stability of the banks, as channels with more resistant banks (cohesive and/or dense bank vegetation) tend to be narrower than those with erodible banks (non-cohesive material and/or thin vegetation). However, the effects of perimeter sediments and bank vegetation on channel shape and size are difficult to quantify (American Society of Civil Engineers (ASCE) Task Committee, 1998a, b). Schumm (1960) and Ferguson (1973b, using Schumm's data) used a weighted index of the silt-clay content in the channel perimeter to demonstrate that channels with silty (cohesive) banks are generally characterised by narrow and deep sections, while a greater percentage of coarser material in the banks tend to be associated with more wider and shallower sections.

Several investigations have reported the strong control of riparian vegetation on bank stability (Hickin, 1984; Thorne, 1990) and, therefore on channel dimensions and in particular channel width. While several researchers have argued that this influence is scale-dependent, whereby vegetation has a dominant control in small streams but negligible effect in wide streams (e.g. Zimmerman et al., 1967; Keller and Swanson, 1979; Keller and Tally, 1979), other observations have revealed that the influence of vegetation on channel morphology is independent of scale and affects downstream hydraulic geometry relationships (e.g. Charlton et al., 1978; Andrews, 1984; Hey and Thorne, 1986; Huang and Nanson, 1997, 1998). In the latter case, it is argued that the local influence of bank characteristics explains a large percentage of the unexplained variance of width as a function of discharge alone, which can be accounted for by using broad vegetation categories in hydraulic geometry analyses. Furthermore, Hey and Thorne (1986) showed that the influence of bank vegetation can override the inherent strength of the bank materials as rooting systems improve toe protection which impedes gravel removal, thereby stabilising the upper stratigraphy from mass failure.

The influence of bed material size has been extensively examined, yet results have been somewhat contradictory. Given the same bank material and flow regime, the general consensus is that the width of a sand-bed channel is expected to be wider than that of a gravel-bed channel. This can be explained from basic principles by neglecting sediment load and using tractive force theory, discussed in Chapter 2. From Equation 2.71, it is clear that the width-to-depth ratio in a threshold channel increases as the friction angle of bed material decreases and therefore, increases for smaller and less angular grains (Henderson, 1963; Li et al., 1976). Furthermore, the majority of regime and hydraulic geometry data tend to show that streams are wider if sandy bed material is present. However, Kellerhalls (1967) inferred that bed material exerts no influence on width and Schumm (1960) suggested that the coarse material in gravel-bed rivers tends to inhibit scouring and encourage widening to maintain capacity, although they may also dissipate flow energy through greater roughness, which limits the potential for bank erosion (Richards, 1982, p. 167).

Blench (1970) demonstrated that the width of sand-bed channels increases with the concentration of bed material load, such that

$$W_{\mathrm{m}} = \left[\frac{F_{\mathrm{b}} \left(1 + 0{\cdot}12\mathrm{C} \right)}{F_{\mathrm{s}}} \right]^{0\cdot5} Q^{0\cdot5} \tag{5.1}$$

where ‘W_{m}’ is the mean width, ‘F_{b}’ and ‘F_{s}’ are Blench’s bed and side factors, respectively, ‘C’ is the bed material load concentration in parts per million and ‘Q’ is the bankfull discharge. This is not surprising as braided rivers are characterised by high sediment loads and very high width-to-depth ratios. This is supported by experimental results of Khan (1971), who demonstrated an increase in width-to-depth ratio with increasing bed load transport. However, including sediment discharge in hydraulic geometry equations is problematic because of its strong correlation with water discharge, which compromises the non-covariance assumption of the linear regression model.

For mobile gravel-bed rivers, Hey and Thorne (1986) and Hey (1997c) showed that the value of the sediment discharge coefficient in width relationships is so small that its influence can be neglected. This is probably because large changes in discharge and depth can be accommodated in many rivers by small changes in width, especially in those with resistant banks with steep bank slopes.

Based on the above considerations, it may be hypothesised that for a given bankfull discharge, sand-bed channels are wider than gravel-bed rivers, all other factors being equal, and that the character of the bank exerts a significant influence on width. For stable channel design, it is recommended that sediment discharge should not be included in the prediction of stable width because of its interrelationship with the rate of flow, but reserved for the analytical determination of depth and slope. Assuming a fixed discharge exponent, b, of 0.5 in the equation $W = aQ^b$, van den Berg (1995) recommended an average value of the coefficient, a, of 3 for gravel-bed rivers (after Ferguson, 1981, p. 114) and 4.7 for sand-bed rivers, which is very close to the value of 4.8 originally proposed by Lacey (1930). These values serve only as a guideline and warrant a more detailed investigation into the differences between sand-bed and gravel-bed hydraulic geometry and their respective natural variability.

5.6 WIDTH-DISCHARGE RELATIONSHIPS IN SAND-BED RIVERS

5.6.1 Review of Existing Sand-Bed Data

With a paucity of sand-bed equations available to predict stable channel morphology, best practice methods still rely on equations originally developed for designing stable canals. For mobile sand-bed channels, Hey (1988, 1997c) suggested using Blench's (1969, 1970) equations or those derived by Simons and Albertson (1960). As rivers only behave as canals in very rare conditions, further research is necessary to develop equations specifically for dynamically stable rivers with mobile sand beds.

Six published data sets were initially considered for representing the relationship between bankfull width, W, and bankfull discharge, Q_b. The composition of the combined sand-bed data set is given below (Figure 5.2, Table 5.2):

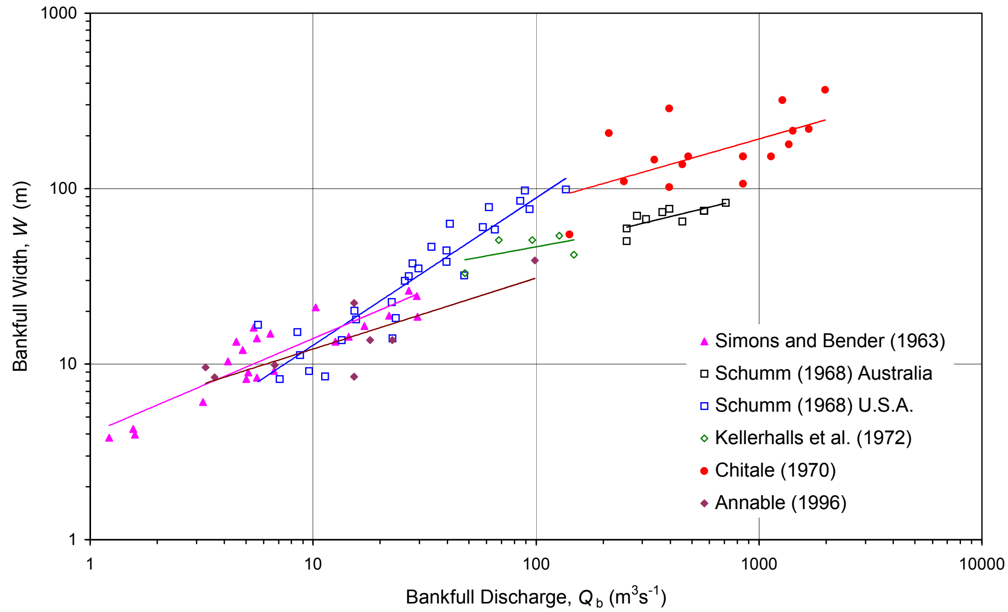


Figure 5.2 Downstream width-discharge relationships in sand-bed streams based on data from various sources.

i) Simons and Bender (Simons and Albertson, 1960): data from regime canals in Wyoming, Colorado and Nebraska, U.S.A. (22 sites). Discharges range from 1.2 m^3s^{-1} to 29 m^3s^{-1} and slopes are less than 0.00039. No information on sinuosity is available. A wide range of bank material and vegetation types is included in the data set. This regime data set is used to compare the width-discharge relationships derived from canal data with the width equations derived from river data.

ii) Schumm (1968): data from sites on the Riverine Plains of new South Wales, Australia (10 sites). Discharges range from 255 m^3s^{-1} to 708 m^3s^{-1}, slopes are less than 0.00028 and sinuosities range from 1.6 to 2.3.

iii) Schumm (1968): data from sites located in the semiarid to subhumid regions of the Great Plains of the U.S.A. (28 sites). Discharges range from 5.7 m^3s^{-1} to 136 m^3s^{-1}, slopes are less than 0.00026 and sinuosities range from 1.05 to 2.5. The bed material in all of the Australia and U.S. channels contains more than 90 percent sand and bankfull discharge was calculated from the cross-sectional area of the channel (Schumm, 1968, p. 49), although no information concerning the exact method of calculation was given.

iv) Chitale (1970): data from large alluvial rivers, including U.S.A. (17 sites). Only data with discharges below 2000 m^3s^{-1} were used as larger rivers in the data set include braided channels. Discharges exceed 142 m^3s^{-1}, slopes are less than 0.00125 and sinuosities range from 1.0 to 2.3.

v) Kellerhalls et al. (1972): data from streams in Alberta, Canada (5 sites). The bankfull discharge corresponds to the flow with a recurrence interval of two years (Bray, 1975, 1982). The full data set comprises mainly gravel-bed rivers of which Church and Rood (1983) rejected 34 of the sites as unsuitable for regime studies. Only stable, sand-bed sites with a dominant single channel were selected (including those with minor secondary channels) from the Church and Rood (1983) database. Discharges range from 48 m^3s^{-1} to 148 m^3s^{-1}, slopes are less than 0.00051 and sinuosities range from 1.3 to 2.0.

vi) Annable (1996): data from southern Ontario, Canada (8 sites). This is a subset of a larger database of morphological characteristics. Discharges range from 3.3 m^3s^{-1} to 99 m^3s^{-1}, slopes are less than 0.0053 and sinuosities range from 1.1 to 2.0.

Only data which include bankfull width and bankfull discharge were used in the analysis on the basis that downstream hydraulic geometry relationships describe *causal* relationships between process and channel form, rather than purely statistical correlations. Therefore, sand-bed data were excluded if the recorded discharge was the mean annual (time-average) discharge (e.g. Leopold and Maddock, 1953). A data set presented by Huang and Nanson (1997) for small streams in southeastern Australia was also considered. The data set consists of 30 sites, of which 14 were reported to have sand-beds and included semi-quantitative information on the type of bank vegetation. However, the exponents in the general width-discharge relationships for sand-bed and gravel-bed rivers were given as 0.35 and 0.23, respectively, which deviate very strongly from the theoretical 0.5 value and lie outside of the world-wide variations in exponents given by Park (1977) and Rhodes (1987). On this basis, Huang and Nanson (1997) considered that channel geometry is multivariate controlled but for the remainder of their investigation, proceeded to fix the discharge exponent to 0.5. Although it is unclear why the Australian data should yield relationships markedly different from other published equations, possible explanations include: i) the relationships were derived from small samples; ii) willows and shrubs on the bed were found to significantly influence channel roughness and deflect flow in sand-bed channels to the extent of overriding the morphological influence of bank material and vegetation, and; iii) the streams commonly show a downstream *decrease* in estimated bankfull discharge in their lower reaches (Huang and Nanson, 1997, p. 238). In light of these complications, the data were not included in this study.

Bivariate hydraulic geometry equations for bankfull width were derived from the selected data sets and are given in Table 5.2. The individual relationships show marked differences, in terms of coefficient, exponent and natural variability. These differences are partly attributable to the small sample sizes and small discharge ranges covered by each data set. Therefore, the equations in Table 5.2 are not suitable as design equations and should be applied with caution. Research into width-discharge relationships (both empirical and theoretical) has indicated that a discharge exponent of 0.5 is appropriate for stable channel design. However, only the Simons and Bender data for regime canals has an exponent approximating this value.

The General Linear Hypothesis was applied to examine whether the exponents significantly differ from 0.5. Table 5.2 shows that the significance level is below 95 percent in all cases except for the Schumm data from U.S.A. which include some ephemeral channels. For these data sets, the fixed exponent model is acceptable. For each data set, the range of coefficients within 95 percent confidence limits (on the mean coefficient) is also given for the fixed exponent model. In light of the differences between

Data Source	**n**	**a**	**b**	**R^2**	**$P_{b\neq0.5}$ (percent)**	**A^***
Simons and Bender (Simons and Albertson, 1960)	22	4.02	0.54	0.81	49.0	3.48 (3.20-3.78)
Schumm (1968) Aus.	10	11.01	0.31	0.58	92.2	5.68 (4.84-6.67)
Schumm (1968) USA	28	1.85	0.84	0.86	>99.9	6.45 (5.31-7.85)
Chitale (1970)	17	15.58	0.36	0.38	73.7	4.36 (3.90-4.87)
Kellerhalls et al. (1972)	5	15.96	0.23	0.28	60.7	4.80 (3.69-6.24)
Annable (1996)	8	4.81	0.40	0.69	59.6	3.77 (2.88-4.92)
Composite data set	90	4.13	0.55	0.89	60.6	4.94 (4.56-5.35)

Note: n = samples in data set; a = discharge coefficient when exponent is not fixed; b = discharge exponent; R^2 = coefficient of determination; $P_{b\neq0.5}$ = significance level of rejecting the null hypothesis that the exponent equals exactly 0.5; a^* = discharge coefficient when exponent is fixed at 0.5 (values in parentheses are 95 percent confidence limits on the mean coefficient value).

Table 5.2 Width-discharge relationships based on downstream hydraulic geometry analysis derived from different sand-bed data sets.

the data sets, an equation was derived for the composite data set of 90 sites with a discharge range between 1 m^3s^{-1} and 2000 m^3s^{-1}, which revealed a width equation very similar to that derived from the Simons and Bender data with an R^2 value of 0.89. The composite equation based on existing data from channels with sand beds is also given in Table 5.2.

Figure 5.3 applies confidence limits around the composite sand-bed data set to display the degree of natural variability (unexplained variance) exhibited by the data. The General Linear Hypothesis was used to test if the exponent was significantly different from the theoretical 0.5 value. Because of the considerable variability in Figure 5.3, the null hypothesis is rejected at only the 61 percent significance level. Therefore, using the conventional criteria to test statistical significance, the exponent can be fixed at 0.5 at the 95 percent level. Confidence limits applied to the fixed exponent model are given in Figure 5.4.

A degree of the variability shown in Figures 5.3 and 5.4 is probably attributable to instability at some of the sites and differences in bank characteristics within the data set. To reduce the width of the confidence bands (and increase R^2), further data are required from stable sand-bed sites. Such a data set would considerably improve the knowledge base for stable channel restoration design.

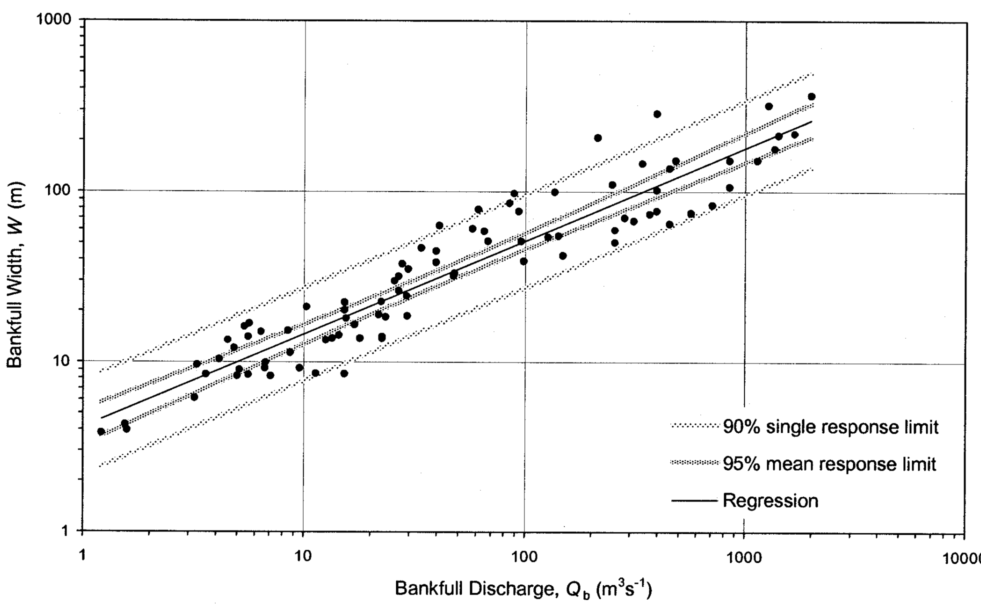


Figure 5.3 Confidence intervals applied to the width-discharge relationship $W = 4 \cdot 13 Q_b^{0 \cdot 55}$ based on a composite data set of 90 sand-bed sites.

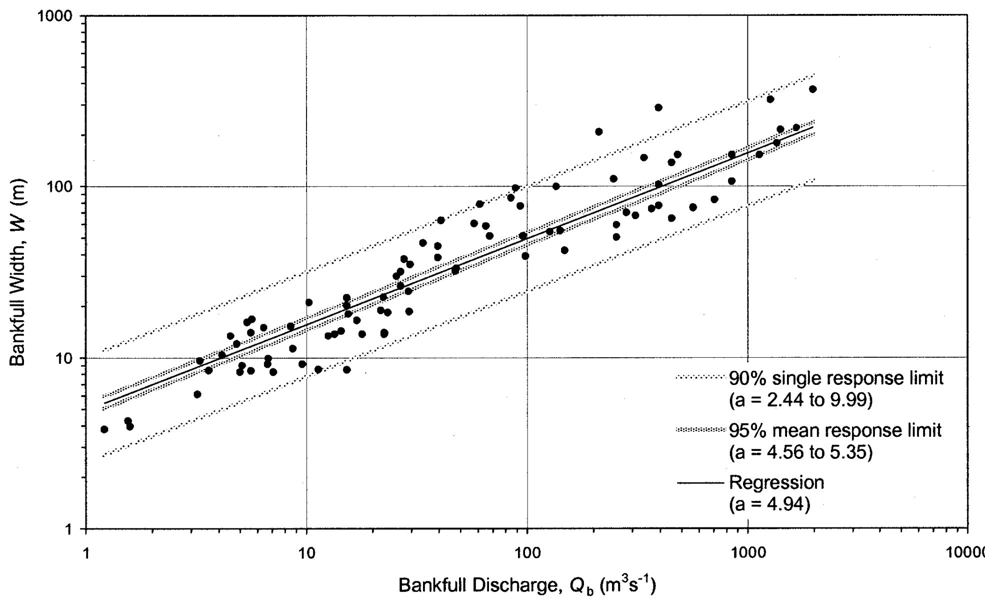


Figure 5.4 Confidence intervals applied to the width-discharge relationship with fixed discharge exponent $W = \mathrm{a}\,Q_{\mathrm{b}}^{0\cdot 5}$ based on a composite data set of 90 sand-bed sites.

5.6.2 The Brice Data Collection

A compilation of data from approximately 350 American alluvial streams has been collected and analyzed by J. C. Brice of the United States Geological Survey (USGS). The data were collected to support research reported in a series of papers and reports written in the 1970s and 1980s which presented methods of identifying and interpreting alluvial forms and features from aerial photographs and maps. These data sources were assembled during the period 1970-1974 under a grant from the U.S. Army Research Office. By analysing the air photographs from different dates and using measurement and analytical methods developed by Brice (1984), the rate and direction of planform changes could be estimated to a high degree of accuracy. Therefore, the data proved particularly useful for analysing channel stability, in terms of the risk of planform shift, at the locations of bridges and other structures.

The most complete accounts of Brice's classification, analysis and interpretation of the original data are reported in two United States Federal Government Publications which are not widely available outside the U.S.A. (Brice, 1975, 1982). Brice also presented a short overview of planform classification at the Rivers '83 conference (Brice, 1984).

Of the total number of streams documented, Brice made a field visit to 174 meandering and braided streams, for which at least two air photographs with different dates were available. It is these sites which are the most useful for channel stability studies. A further 176 (estimated) sites have a single air photograph archived with the database. The historical period covered by the air photographs and maps includes various dates generally between the 1920s and the 1970s.

The Brice Collection is representative of the types of alluvial channels and morphological conditions found in different regions of the U.S.A. In total, 30 U.S. states are represented in the database. The criteria for selecting the rivers in the original collection were that:

i) The rivers are representative of many geographic environments within the United States.
ii) River reaches are mapped on a suitable scale and contour interval with suitable airphoto coverage.
iii) Air photographs were available from different years mostly between the mid-1930s and the late 1960s for identifying and quantifying planform changes in river channel pattern and bankline movement though time.
iv) Air photographs exhibit high resolution and minimum tilting distortion.
v) The rivers include a wide assemblage of fluvial features and are classified by a range of river types.
vi) The reaches selected for examination are close to USGS gauging stations with long periods of record.

Brice (1984) used the collection to develop geomorphic methods of stream channel stability assessment for use by bridge and highway engineers. This involved measuring channel width from the air photographs.

Under a research project at Johns Hopkins University (JHU), sponsored by the UAEWES/ERDC, the data set was used to evaluate methods for forecasting incremental planform change due to bankline migration in meandering channels (Cherry et al., 1996). As part of this work, the data set was inventoried and derived parameters for selected sites were tabulated and categorized on the basis of planform shift and the Brice stream classification (Brice, 1975). The JHU study of planform change was limited to the sites with two air photographs.

Re-examination of the air photographs in light of Brice's classification of the streams revealed that some of the 60 streams classified by Brice as braided actually exhibited only localized braiding in the form of chute channels or minor secondary channels. Using an updated classification developed by Cherry et al. (1996), the numbers of meandering and braided rivers were defined as 133 and 41, respectively. Under a research project at the University of Report, U.K., to examine the potential use of the data set in developing a methodology for channel restoration design, also sponsored by UAEWESERDC, the tabulated database was extended to include other parameters including a revised stream classification, flow data obtained from the nearby USGS gauging stations and a simple vegetation classification based on inspection of the air photographs. The full inventoried data set was presented by Thorne and Soar (1997).

Of the 133 meandering streams investigated by Cherry et al. (1996), Brice classified the bed material type of each site as: sand (93); gravel (16); sand-gravel (22), or; sand-silt (1). The bed material of one site was not recorded. The dominant type of riparian vegetation for each site was classified by Thorne and Soar (1997), on the basis of areal coverage, from the air photographs as: trees (71); grass (19); grass-trees (23); shrubs-trees (6), or; shrubs-grass (4). Ten sites are excluded because the riparian vegetation class could not be identified using photography alone.

The Brice Collection was assembled for a stability analysis associated with the natural planform migration of stable alluvial rivers. The inventoried data set provides a useful starting point for deriving new hydraulic geometry relationships for stable sand-bed sites.

5.6.3 Preliminary Analysis Using Existing Brice Data

The data set of 133 meandering rivers inventoried by Cherry et al. (1996) was used to derive a preliminary morphological equation for channel width. The bed material information recorded by Brice during the 1960s and 1970s was cross-referenced by information supplied by USGS district offices and several disparities were revealed. For this analysis, sand-bed sites were selected based on the recent information supplied by the USGS. It was assumed that the widths measured by Brice from air photographs (1:24000), using analytical techniques described by Brice (1984), correspond to an average bankfull width in the study reaches. The Brice data set includes no measurements of the channel-forming flow, such as bankfull discharge; therefore, a statistical procedure was adopted to derive a surrogate flow for hydraulic geometry analysis.

Flow data were obtained from the USGS and recurrence intervals were calculated for sites with at least 30 years flow record from annual maximum flow series. For each site, the distribution of flow frequencies was compared with a theoretical Gumbel EV1 distribution and sites were rejected if the peak flow data failed to conform to the theoretical distribution using a 90 percent correlation coefficient as a threshold. This technique was adopted to filter out watersheds which exhibit erratic flow distributions, rather than stable runoff regimes. The filtered data set consisted of 81 sites.

For each discharge frequency, the discharge which minimised the degree of data scatter about the regression line between log-transformed width and discharge and yielded the maximum coefficient of determination, R^2, was selected as the dominant discharge. This type of statistical procedure has been used previously by Bray (1982) for hydraulic geometry analysis of gravel-bed rivers in Alberta, Canada, and relies on the assumption that the bankfull width of the channel is most closely associated with the dominant discharge and the greatest association is that which minimises the variability in width as a function of discharge. Although a statistical 'fix', the technique is reasonable given data limitations.

This minimisation corresponded to the discharge with a recurrence interval of 1.6 years, $Q_{1.6}$, with an R^2 value of 0.69 (Figure 5.5). This flow frequency is within the range of 1 to 2 years as suggested by Leopold et al. (1964). The frequency also approximates the most probable flood, $Q_{1.58}$, which was equated to bankfull flow by Dury (1973) and the discharge with a recurrence interval of 1.5 years, $Q_{1.5}$, which was equated to bankfull flow by Hey (1978) for gravel-bed rivers in the U.K. The resultant width equation is given in Table 5.3.

The resulting width relationship is shown in Figure 5.6 within confidence bands. By fixing the exponent at exactly 0.5, the coefficient is modified very slightly to 2.95. Confidence bands are applied to the fixed exponent model in Figure 5.7.

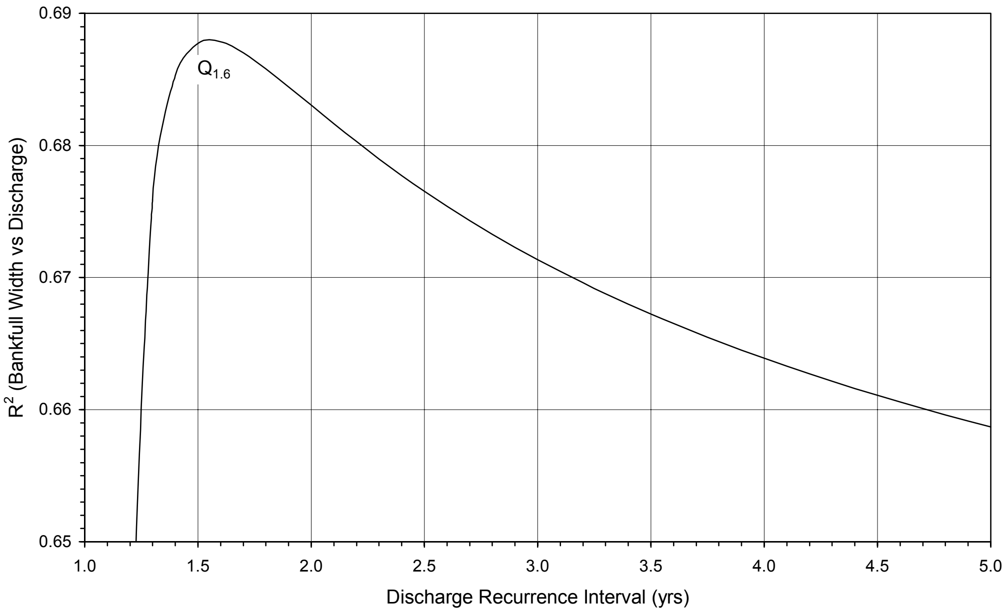


Figure 5.5 Variability of R^2 in the relationship $W = aQ^b$ with discharge frequency, using data from selected Brice sites with sand beds.

Data Source	n	a	b	R^2	$P_{b\neq0.5}$ (percent)	a^*
Cherry et al. (1996)	81	2.96	0.50	0.69	0	2.95 (2.68-3.24)

Note: n = samples in data set; a = discharge coefficient when exponent is not fixed; b = discharge exponent; R^2 = coefficient of determination; $P_{b\neq0.5}$ = significance level of rejecting the null hypothesis that the exponent equals exactly 0.5; a^* = discharge coefficient when exponent is fixed at 0.5 (values in parentheses are 95 percent confidence limits on the mean coefficient value).

Table 5.3 Width-discharge relationship for U.S. sand-bed streams: Width data compiled by Cherry et al. (1996).

The statistical procedure used to derive the equation in Table 5.3 revealed that discharges of different frequencies are highly correlated, therefore the distribution of R^2 in Figure 5.5 is very platykurtic (the distribution of R^2 as a function of discharge recurrence interval is not 'peaky'), therefore the adopted 1.6-year frequency is not particularly well defined and the range of R^2 varies only slightly for discharges between 1.5-year and 2-year recurrence intervals. Moreover, the method has limited applicability because the width-discharge relationship is merely a statistical predictor unlike the causal relationship between bankfull width and bankfull discharge. This problem introduces uncertainty because it is highly unlikely that a common recurrence interval describes the channel forming discharge at all sites. A further concern is the uncertainty in the assumption that Brice's width measurements corresponded to the bankfull width. The associated error is likely to be greatest for sites with dense overhanging bank vegetation which may lead to systematic underestimation of channel width.

These findings warrant the collection of new data from sand-bed rivers in order to validate the form of the width-discharge relationships in Table 5.3 and address whether the wide confidence bands in Figures 5.6 and 5.7 are true representations of the natural variability inherent in stable sand-bed rivers.

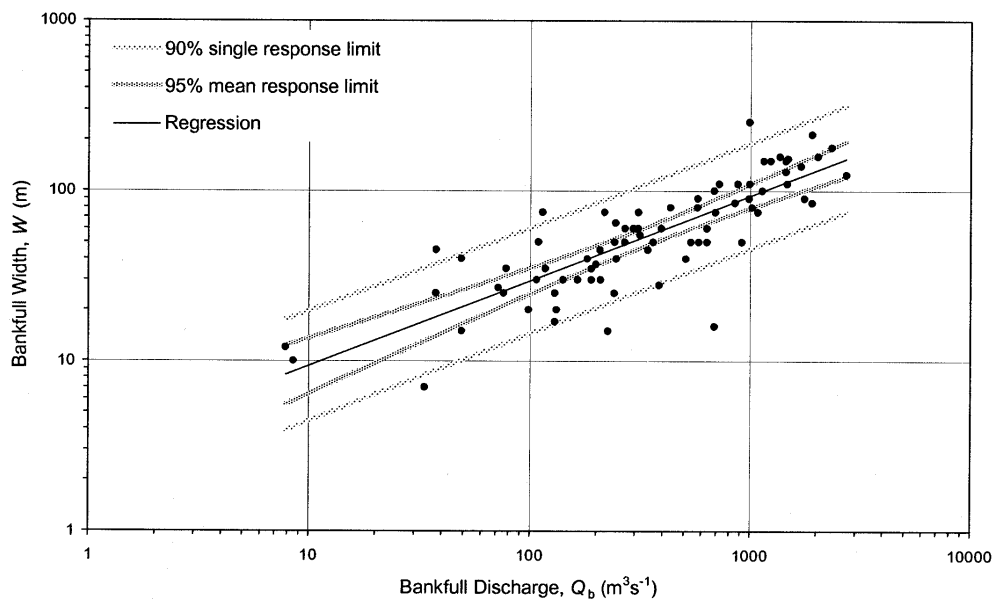


Figure 5.6 Confidence intervals applied to the width-discharge relationship $W = 2 \cdot 96 Q_{1 \cdot 6}^{0 \cdot 50}$ based on a subset of the Brice database for channels with sand beds.

5.7.4 Updating the Brice Collection

Under the guidance of UAEWES/ERDC, a fieldwork campaign was initiated in 1998 to update the Brice Collection with an enhanced hydraulic geometry database for sand-bed rivers including cross section surveys, descriptions of bed and bank characteristics and general reconnaissance and photography of each site visited. The criteria for site selection were as follows:

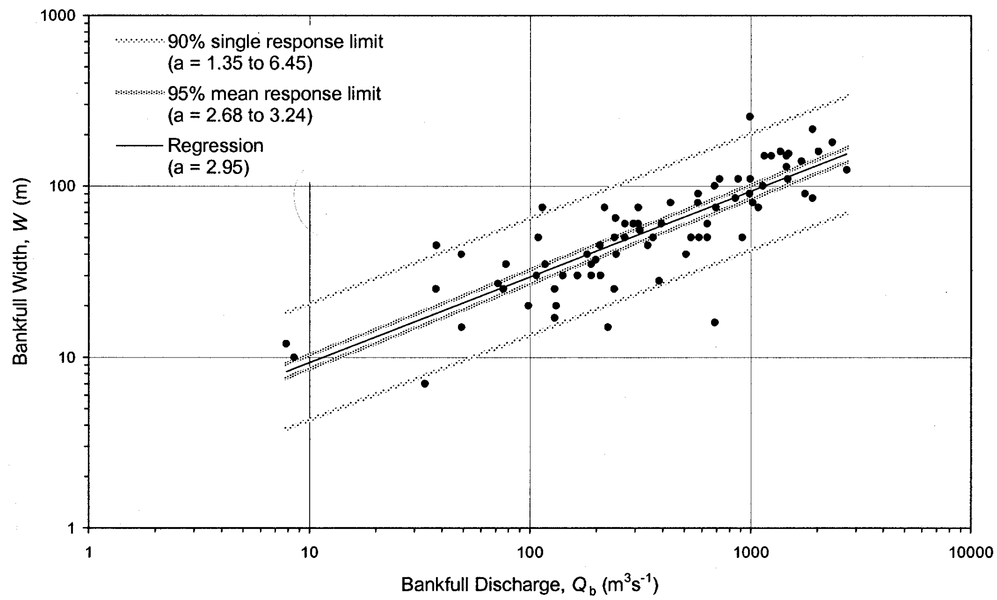


Figure 5.7 Confidence intervals applied to the width-discharge relationship with fixed discharge exponent $W = \mathrm{a}\,Q_{1\cdot 6}^{0\cdot 5}$ based on a subset of the Brice database for channels with sand beds.

i) *The bed of the channel should be formed predominantly in alluvial sand-bed material*. The Brice data set already contained information on bed material type. This was substantiated by descriptions given by USGS district staff. Brice described the sites identified for survey as sand-bed and in several cases sand/gravel-bed. Gravel-bed rivers were avoided.

ii) *The channel should be free from constraints such as bed outcrops or training structures*. When Brice compiled the data set, he tried to identify reaches which were relatively homogeneous and natural. This was essential for subsequent research on natural planform typology (Brice, 1975, 1982).

iii) *The channel should be stable and in natural equilibrium with the flow regime and sediment supply. There should be no significant aggradation, degradation or changes in width. Dynamic equilibrium, such as lateral migration of the channel, whilst maintaining constant reach average sinuosity and cross-sectional shape and size, is acceptable*. Historical air photographs were inspected for evidence of changes in sinuosity, width and channel type over time. Reaches were assessed in the field for stability using geomorphological techniques to identify any scouring, sedimentation and lateral instability on a reach-scale basis.

iv) *The site should be near a USGS gauging station.* All Brice sites are near gauging stations, although some stations have now been discontinued. Most of the sites have long periods of historical flow data.

v) *The flow regime should not be severely affected by the operation of reservoirs or inter-basin transfers*.

vi) *There should not have been significant changes in land use during the period of record which could have affected the discharge record*. The air photographs generally suggest a high degree of stationarity in catchment land use.

Site selection was also dictated by the availability of assistance from UAEWES/ERDC and USGS staff during the fieldwork period.

Within each stable reach delineated by Brice, suitable sites for cross section survey were identified at an observed point of inflexion between successive meander bends from 7.5 min USGS Quadrangle Topographic Surveys. This location often corresponds to the position where the thalweg crosses over the channel centreline, the ‘crossing’, although in an actively migrating river, the crossing lies somewhat downstream from the geometric inflexion. The inflexion point was selected as the geomorphological reference for the data set for several reasons. The cross section exhibits the most homogeneous morphology and in many cases, approximates a trapezoidal shape because secondary currents are at a minimum. While this shape also facilitates field survey, the simplicity of the cross section is most appropriate for analysis. This is because research into fluid flow and sediment transport has generally been undertaken in straight, trapezoidal (usually laboratory) channels with uniform bed material. Hence, flow resistance and sediment transport equations are more suited to the inflexion point than to locations around bendways which have complex, poorly understood flow patterns and high degrees of asymmetry. Furthermore, many of the sites were visited at relatively high stages and it would have been impractical to survey cross sections at pools where depths and point velocities were greatest. The inflexion point was also preferred because the absence of gentle bank gradients facilitated the identification of the bankfull reference level. For these reasons, the inflexion point is likely to be a common survey location in many of the published hydraulic geometry data sets.

General reconnaissance of each reach was undertaken while travelling to the sites by boat and at each cross section location. Particular attention was given to describing the nature of the channel banks and floodplain vegetation. Photographs of each reach and cross section were also taken for the purpose of specifying channel 'type' and for archiving with the Brice Collection.

A cross section was selected as representative of the Brice reach in terms of instream morphology and riparian vegetation character. At several of the sites, with low or irregular sinuosity, a relatively straight reach was identified for survey. All sites were selected as close to the gauging station as possible to minimise the influence of tributary inputs. In rare cases, large tributary inputs required identifying a cross section beyond the reach delineated by Brice. A single cross section was surveyed at each site. At regular increments across a tag line, soundings were measured with both a survey staff and in deep water by an echo sounder attached to a boat. In the larger rivers, the boat was attached to a second tag line to ensure that the incremented line remained perpendicular to the banks. The banks were surveyed using a series of different length rules and a clinometer. Major breaks of slope were identified between the channel bed and floodplain. The water surface slope was not measured in the field and values were obtained from the original Brice data set, which are reach average gradients for the study reaches.

To derive an average bed material size gradation, three bed material samples were obtained at equal distances across the bed of each section using a 'Ponar' bed material sampler. Bank material samples from each bank were retrieved from mid-bank locations. Bed and bank samples were sieved by the UAEWES/ERDC and USGS staffs.

For each site, the effective discharge was calculated using the quasi-event-based magnitude frequency method with 500 classes (Chapter 4), which produced relatively smooth frequency distributions in most cases. In the absence of measured bed material load and stage-discharge relationships, Brownlie's flow resistance (Brownlie, 1981, 1983) and total load equations (Brownlie, 1981) were used in the calculations (Chapter 6 equations). Ideally, bankfull discharge should be measured in the field by surveying the channel profile at the bankfull elevation back to the gauging station and reading the bankfull discharge from the stage-discharge curve. However, in the Brice sites, this was unfeasible as the distance between site and gauge was often in the order of several kilometres. Therefore, bankfull discharge was calculated based on well-defined bankfull reference levels identified in the field and from the cross sections and photographs. A modified equal velocity method for compositing discharge was adopted to calculate depth at a given discharge as a function of bed and bank roughness (see Chapter 6 for method). As the majority of channels were tree-lined (although of varying density according to region), a constant Manning 'n' value of 0.08 was assigned to the channel banks in each case based on guidance given by Chow (1959) for channels with medium to dense tree/ shrub cover and bed roughness was calculated from the Brownlie (1981, 1983) equations. Brownlie's equations incorporate the contribution to flow resistance from bed forms as well as that due to grain roughness and are the recommended equations in the SAM hydraulic design package for stable channel design (Thomas et al., 1996; Copeland, 1994).

Cross section measurements were calculated at both the effective and bankfull elevations and the full database is given in the Appendix Tables B1 and B2.

5.7.5 Effective Discharge Investigation

Research at UAEWES/ERDC (Hey, 1997a) has indicated that the dominant, 'channel forming' flow is best represented by the effective discharge, which is the flow transporting the greatest quantity of sediment during the period of flow record. Furthermore, research has shown that the observed bankfull elevation frequently corresponds to the flow stage at the effective discharge (Chapter 4). However, the results discussed below indicate that this equivalence is not a general condition in stable sand-bed rivers. Figure 5.8 shows cumulative distributions of two ratios for the new data set: i) bankfull discharge, Q_b, over effective discharge, Q_e, and; ii) the long-term sediment load transported by discharges not exceeding bankfull discharge, Y_b (percentage), over the long-term sediment load transported by discharges not exceeding the effective discharge, Y_e (percentage).

The two distributions in Figure 5.8 are very similar and reveal that Q_b/Q_e is greater than unity at approximately 86 percent of sites and between unity and 10 at approximately 67 percent of sites. This suggests that as a general rule, bankfull discharge is an upper limit to the effective discharge in this data set, and the effective discharge is a high, in-bank flow. Biedenharn and Thorne (1994) also demonstrated for the Mississippi River that the upper limit of the range of effective flows forms an upper boundary to the top-bank elevations and Pickup and Warner (1976), Lyons et al. (1992) (Chapter 4) showed that the effective discharge occurs more frequently than the bankfull discharge at their study sites.

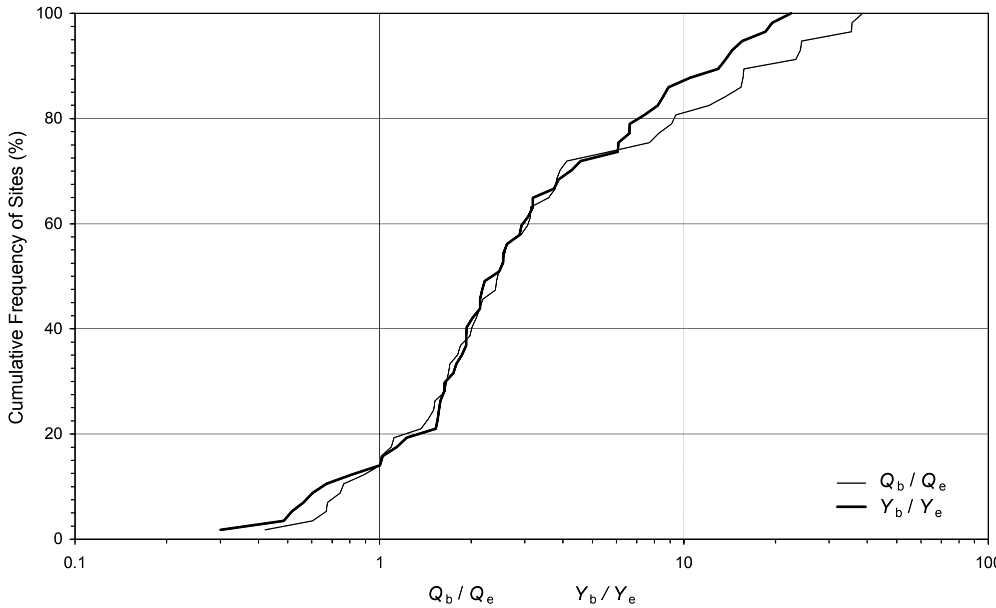


Figure 5.8 Cumulative distributions of the ratio between bankfull discharge, Q_b, to effective discharge, Q_e, and the ratio between the long-term sediment load transported by discharges not exceeding bankfull discharge, Y_b (percent) to the long-term sediment load transported by discharges not exceeding the effective discharge, Y_e (percent).

From a numerical basis, this can be partially explained by considering the cumulative distribution of sediment load as a function of discharge which is derived from the bed material histogram used to estimate the effective discharge. The peak in the histogram defines the effective discharge and this translates to the steepest gradient in the S-shaped cumulative distribution. However, the bankfull stage usually defines a marked discontinuity in a river's sediment rating curve due to the break in bank slope, increased resistance over the floodplain and exchange of momentum between in-bank and over-bank flows. As the frequency of over-bank flows usually decreases with increasing stage, bankfull discharge coincides in many cases with the upper break point in the cumulative sediment curve, at a moderate to high frequency (Figure 5.9).

Numerically, the peak in the sediment histogram (effective discharge) cannot correspond to the upper break point in the cumulative sediment curve (bankfull discharge), although they may be similar in certain cases. In fact, it is unlikely that the modal (effective) discharge in sand-bed rivers will exceed the median discharge (50 percentile in the cumulative curve). This is demonstrated in the following discussion.

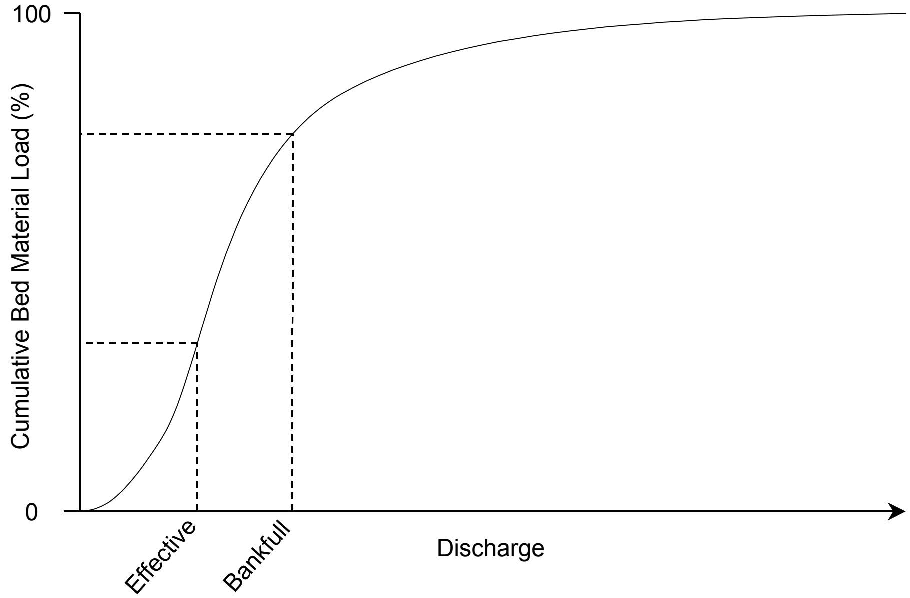


Figure 5.9 Hypothetical cumulative sediment curve showing the locations of the effective discharge at the inflexion point and the bankfull discharge at the upper break point.

Figure 5.8 also shows that the ratio Q_b/Q_e is highly variable. Examination of the distribution of flows at each site revealed that the observed differences in Q_b/Q_e are partially attributable to differences in flow variability. Harvey (1969), Stevens et al. (1975) and Baker (1977) argued that when flows are highly variable, channel characteristics may exhibit disequilibrium with the prevailing flow regime, rather than fluctuating about some mean condition, because rivers have a memory for past events and may be shaped by high-magnitude, low-frequency flows rather than flows of intermediate frequency as suggested by Wolman and Miller (1960). Harvey (1969, p. 94) noted that: ‘since bankfull discharge has variations in frequency, both on any one stream and between streams, it appears that stream channels may be adjusted to different hydrologic regimes in different ways’. Flow variability is often expressed in terms of a ratio of some peak flow to the mean annual (time averaged) discharge, Q_m (Schumm, 1977; Knighton, 1984; Pizzuto, 1986). The bankfull discharge recurrence interval was calculated for each site using the annual maximum flow series. In approximately 83 percent of sites, the bankfull discharge had a recurrence interval of between 1 and 2 years, therefore the 2-year flow event, Q_2, formed an approximate upper boundary for the majority of sites. In light of this, the ratio, Q_2/Q_m was used to define flow variability in the study sites and used in a regression analysis to partly explain the observed variability in Q_b/Q_e (Figure 5.10). The best-fit relationship is a power function which explains 73 percent of the variance in Q_b/Q_e, and is given by

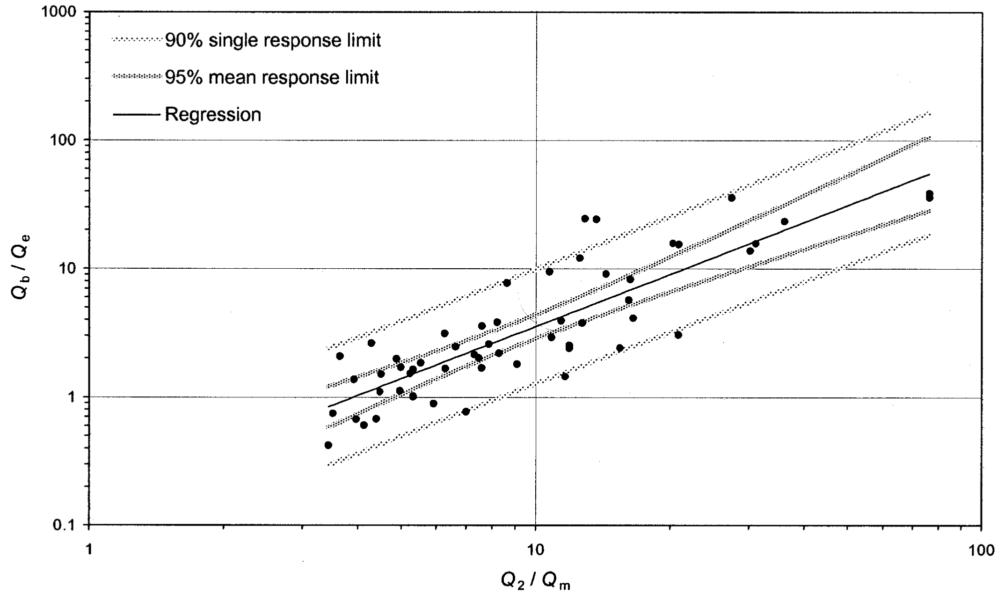


Figure 5.10 Ratio between bankfull discharge, Q_b, and effective discharge, Q_e, expressed as a function of flow variability defined as the ratio between the 2-year recurrence interval flow, Q_2, and the mean annual (time-averaged) discharge, Q_m.

$$\frac{Q_b}{Q_e} = 0 \cdot 16 \left(\frac{Q_2}{Q_m} \right)^{1 \cdot 35} \tag{5.2}$$

The variability in Figure 5.10 is partly a result of surveying only one cross section per site, but is also attributable to other factors which control the effectiveness of very frequent discharges and shape the lower tail of the cumulative sediment curve. For example, the shape of the cross section influences the stage-discharge relationship and therefore the sediment rating relationship. On this basis, it can be shown that the ratio Q_b/Q_e varies strongly with the percentage of the long-term sediment load transported by discharges not exceeding the effective discharge, Y_e (Figure 5.11). The best-fit power function explains 80 percent of the variance in Q_b/Q_e, and is given by

$$\frac{Q_b}{Q_e} = 121 \cdot 75\, Y_e^{-1 \cdot 19} \tag{5.3}$$

Figure 5.11 shows that bankfull and effective discharges may be equal for values of Y_e between approximately 25 and 55 (within the lower 90 percent confidence limit on a single response). The degree of variability in Equation 5.3 is less than that in Equation 5.2, because Y_e is controlled by both flow variability and the cross-sectional morphology. These dimensionless equations can be used as tentative design guidance for

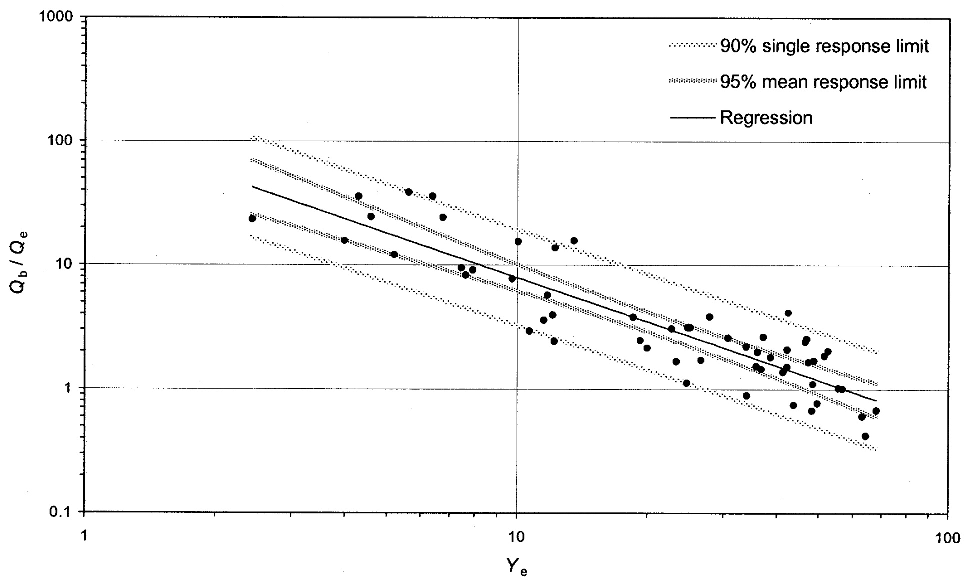


Figure 5.11 Ratio between bankfull discharge, Q_b, and effective discharge, Q_e, expressed as a function of the percentage of the long-term sediment load transported by discharges not exceeding the effective discharge, Y_e.

channel restoration design in sand-bed rivers, although further research is recommended to further develop these initial findings.

It is hypothesised that in base flow-dominated streams with infrequent flood flows, the small, high-frequency discharges that prevail in between infrequent eroding events are highly effective sediment transporting flows in terms of their channel restoring (or reforming) capabilities, thereby maintaining average cross-sectional geometry over the medium- to long-term. Conversely, in rivers with non-flashy flow regimes, the effective discharge is large enough and occurs frequently enough to scour the bed and/or banks and small flows have negligible influence on the medium- to long-term channel morphology. In such cases, the effective discharge may occur near the bankfull stage. Interestingly, in channels with non-variable flow regimes, such as alluvial canals with near constant full supply discharge, bankfull discharge must be the effective discharge.

To demonstrate the influence of flow variability on the difference between effective and bankfull discharges, bed material load histograms and cumulative sediment curves are given for three sites in Figure 5.12. When flow variability is high, for example in the East Nishnabotna River at Red Oak, IA, the effective discharge is a very frequent flow, while the bankfull discharge is not very effective at transporting sediment. When flow

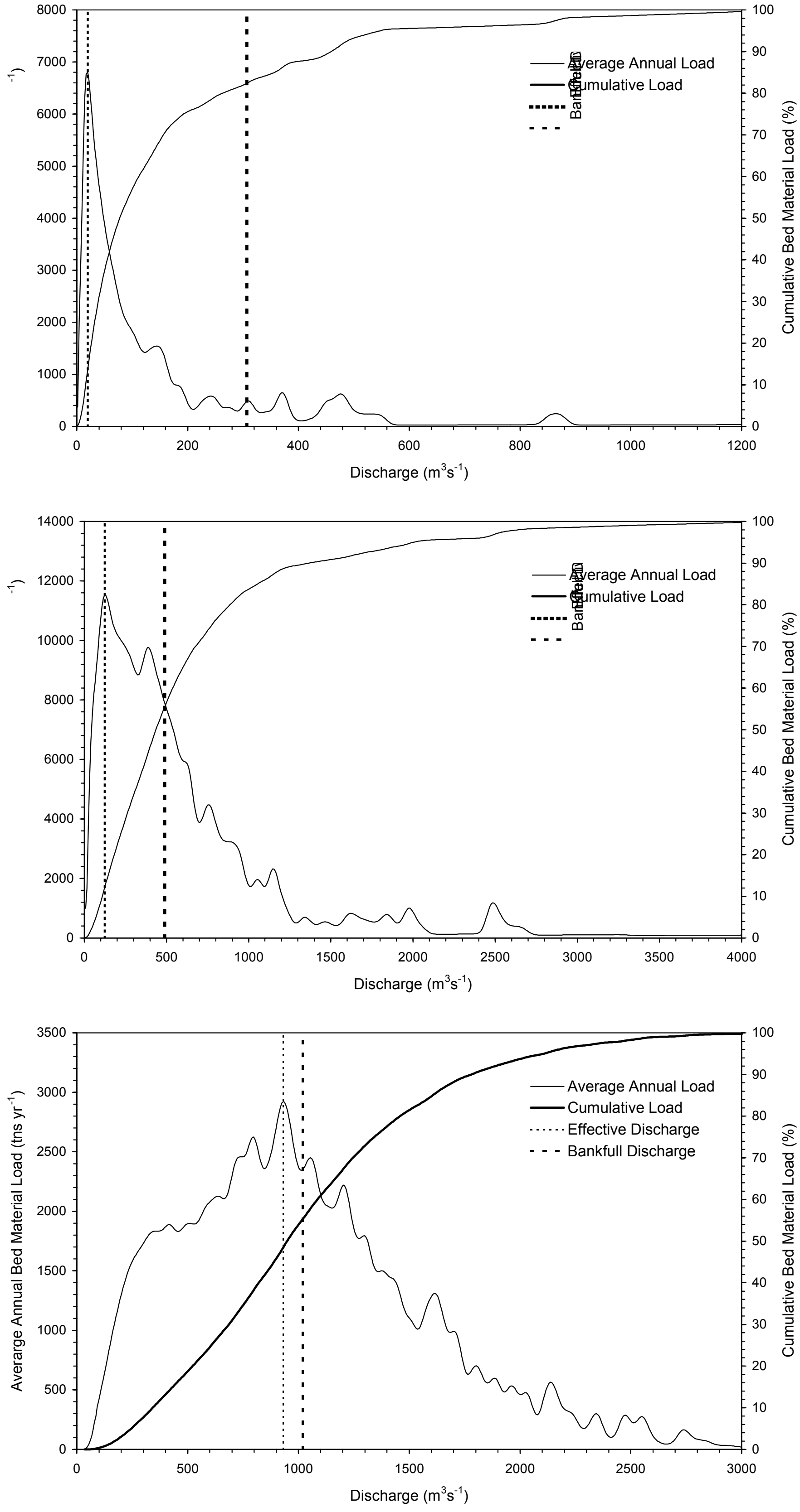


Figure 5.12 Bed material load histograms and cumulative sediment curves for East Nishnabotna at Red Oak, IA (top, $Q_2/Q_m = 20.3$), Tombigbee near Amory, MS (middle, $Q_2/Q_m = 11.4$) and Wabash at Riverton, IN (bottom, $Q_2/Q_m = 4.5$).

variability is moderate, for example in the Tombigbee River near Amory, MS, the cumulative sediment curve is very linear below bankfull discharge and defines a wide range of effective flows. When flow variability is low, for example in the Wabash River at Riverton, IN, the effectiveness of the small discharges is suppressed and a greater percentage of the long-term sediment load is transported by intermediate to high flows, which significantly reduce the cumulative frequency of the bankfull discharge to a value close to the effective discharge.

The equivalence between effective and bankfull discharges in mobile gravel-bed rivers, as demonstrated by Andrews (1980) and Hey (1997a) is possibly a result of the high shear stresses required to mobilise gravel bed material which render the very frequent modal flows very ineffective at transporting sediment. Consequently, there is only a small range of in-bank flows capable of moving the bed material, and the bankfull discharge is a highly effective flow. It is hypothesised in gravel-bed rivers that the variability in Q_b/Q_e will not exhibit a strong relationship with flow variability. Further research is required to investigate the above statements.

Hey (1997a) demonstrated that the effective discharge does not have a fixed frequency but is influenced by both the nature of the sediment load and the flow regime. The U.S. sand-bed data have revealed that both sediment rating and flow variability control the magnitude and variability of the ratio between bankfull and effective discharges. These findings present a potential dilemma for river managers in terms of defining a dominant discharge. Bankfull stage is usually difficult to determine, particularly when a channel has incised, and the effective discharge, albeit a more objective flow, only corresponds to the bankfull discharge in certain cases. Further research is required to examine the morphological significance of the effective discharge and whether a range of effectiveness can be quantified with an upper-bound corresponding to the bankfull condition. It is envisaged that using cumulative sediment curves to determine a range of effective flows will have considerable application potential. For the 58 U.S. sites, the ratio Q_e/Q_m exceeds unity in approximately 79 percent of sites, therefore as a general rule in sand-bed rivers the mean annual discharge and the bankfull discharge form lower and upper bounds, respectively, to the range of effective discharge, while the 2-year flow is an upper bound to the range of bankfull discharge.

5.7.6 Updated Hydraulic Geometry Relationships

Hydraulic geometry analysis based on the effective discharge did not yield a significant relationship for either width or depth because of the considerable variability in the ratio between effective and bankfull discharges. However, new morphological equations for bankfull width as a function of bankfull discharge, albeit calculated from a flow resistance equation, were significant and warrant further discussion here.

Using the full data set of 58 U.S. sites, bankfull discharge explains 76 percent of the variance in bankfull width. However, by using a simple typing system based on bank vegetation density, at least 85 percent of the variance in width could be explained. The type of river bank is best described using two categories: Type T1 = less than 50 percent tree cover on the banks, and; Type T2 = at least 50 percent tree cover on the banks. All of the sites were tree-lined to some degree, therefore an equation for grass-lined or thinly vegetated banks could not be derived. The percentage of silt-clay in the banks was not a significant variable in the prediction of width, possibly because the root-binding properties of tree roots are overriding the effect of cohesion at these sites. Hey and Thorne (1986) also demonstrated no relationship between measured strength of bank material and cross-sectional dimensions. The new width equations are given in Table 5.4 and Figures 5.13 to 5.17.

Data Source	**n**	**a**	**b**	R^2	$P_{b\neq0.5}$ **(percent)**	a^*
All sand-bed sites	58	3.76	0.52	0.76	40.7	4.24 (3.90-4.60)
Type T1 (<50 percent tree cover)	32	4.88	0.51	0.87	22.5	5.19 (4.78-5.63)
Type T2 (≥50 percent tree cover)	26	3.27	0.50	0.85	3.3	3.31 (3.04-3.60)

Note: n = samples in data set; a = discharge coefficient when exponent is not fixed; b = discharge exponent; R^2 = coefficient of determination; $P_{b\neq0.5}$ = significance level of rejecting the null hypothesis that the exponent equals exactly 0.5; a^* = discharge coefficient when exponent is fixed at 0.5 (values in parentheses are 95 percent confidence limits on the mean coefficient value).

Table 5.4 New width equations derived from U.S. sand-bed river data.

For 15 sand-bed streams in midwestern U.S.A., Pizzuto (1986) demonstrated that bankfull depth increased with flow variability and explained the relationship in terms of a floodplain accretion model. For the 58 streams examined here, neither bankfull depth nor width varies significantly with flow variability, as defined by Q_2/Q_m.

The equations given in Table 5.4 are a significant improvement on the statistically based equation using $Q_{1.6}$ as dominant discharge (see Section 5.6.3) because bankfull discharge has been shown to have a variable frequency and typing the character of the bank further reduces the unexplained variance. The General Linear Hypothesis confirmed that the new equations could assume a fixed discharge exponent of 0.5 at the 95 percent significance level. The two equations typed by bank vegetation are also significantly different at the 95 percent level (null hypothesis of equivalence rejected at greater than the 99.9 percent significant level). Assuming a fixed exponent of 0.5, the coefficient in the equation from all data (4.24) is considerably greater than in the equation based on $Q_{1.6}$ (2.96). This may be due to the original width measurements by Brice underestimating the true bankfull widths because of overhanging bank vegetation in the air photographs. Interestingly, the composite equation from various data sets given in Table 5.2 is similar to the equation derived from the new data.

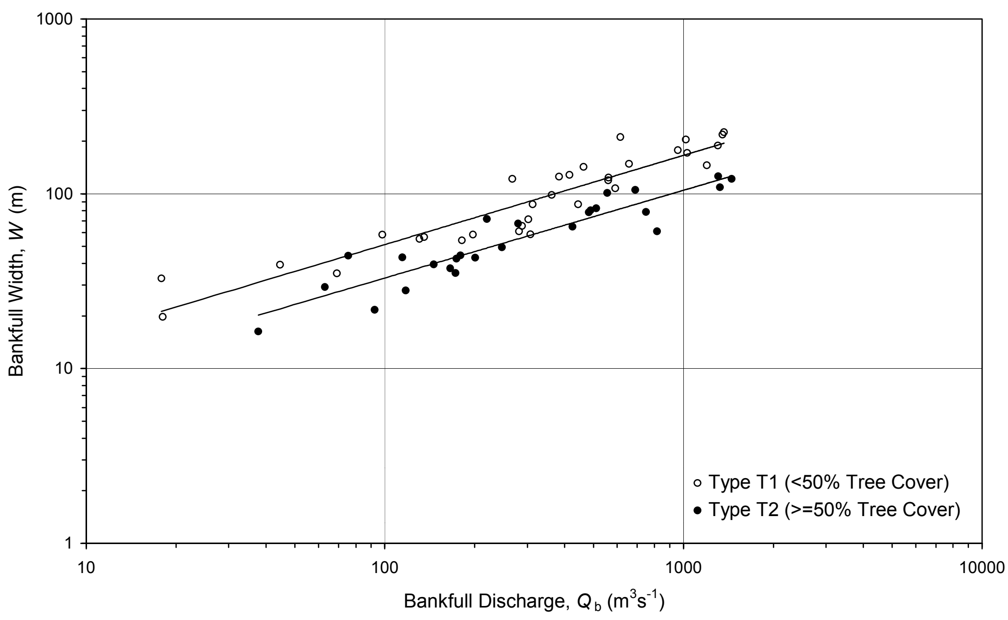


Figure 5.13 Best-fit width-discharge relationships for U.S. sand-bed rivers with banks typed according to density of tree cover.

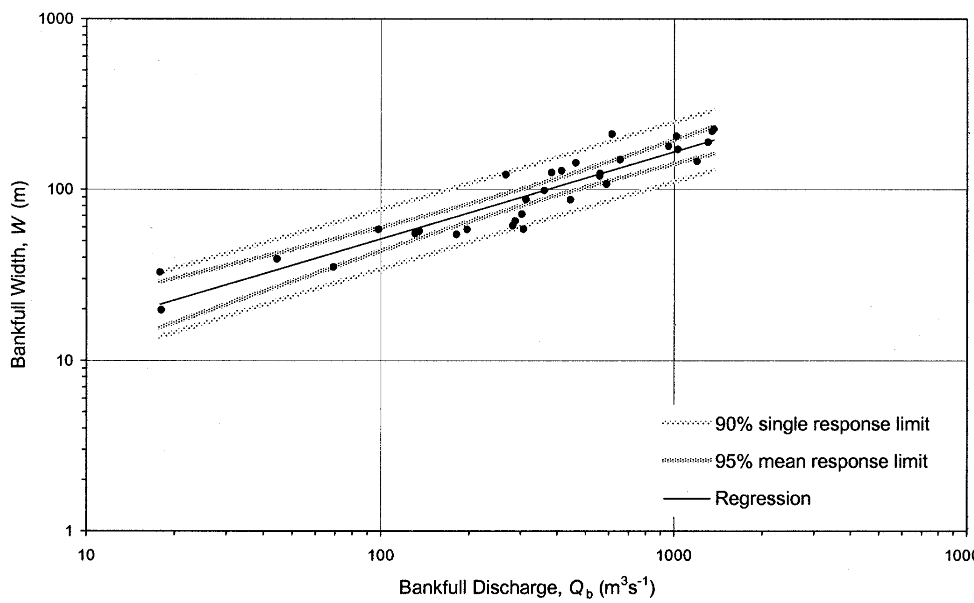


Figure 5.14 Confidence intervals applied to the width-discharge relationship $W = 4 \cdot 88 Q_b^{0 \cdot 51}$ based on 32 U.S. sand-bed sites with less than 50 percent tree cover on the banks.

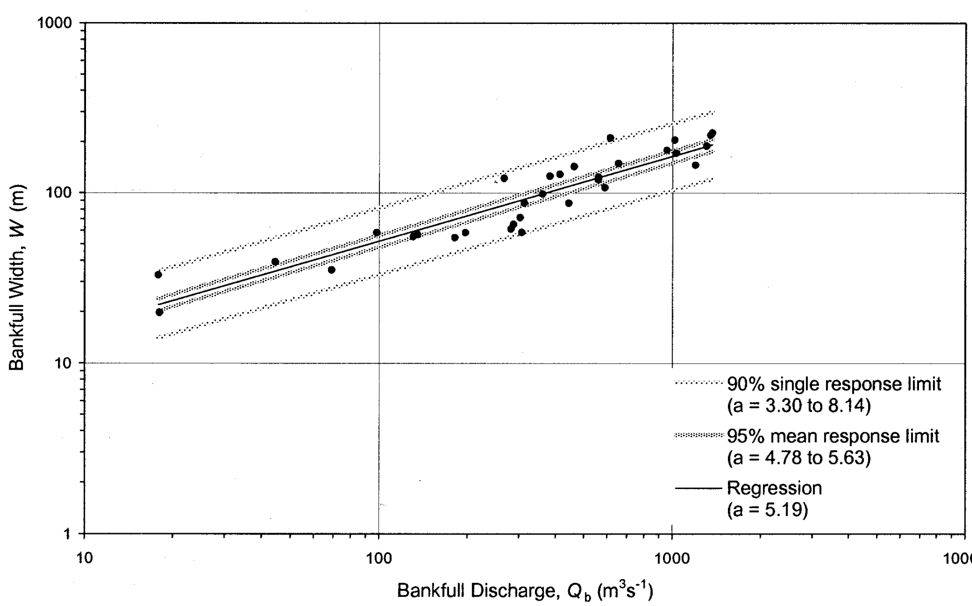


Figure 5.15 Confidence intervals applied to the width-discharge relationship with fixed discharge exponent $W = \mathrm{a}\, Q_b^{0 \cdot 5}$ based on 32 U.S. sand-bed sites with less than 50 percent tree cover on the banks.

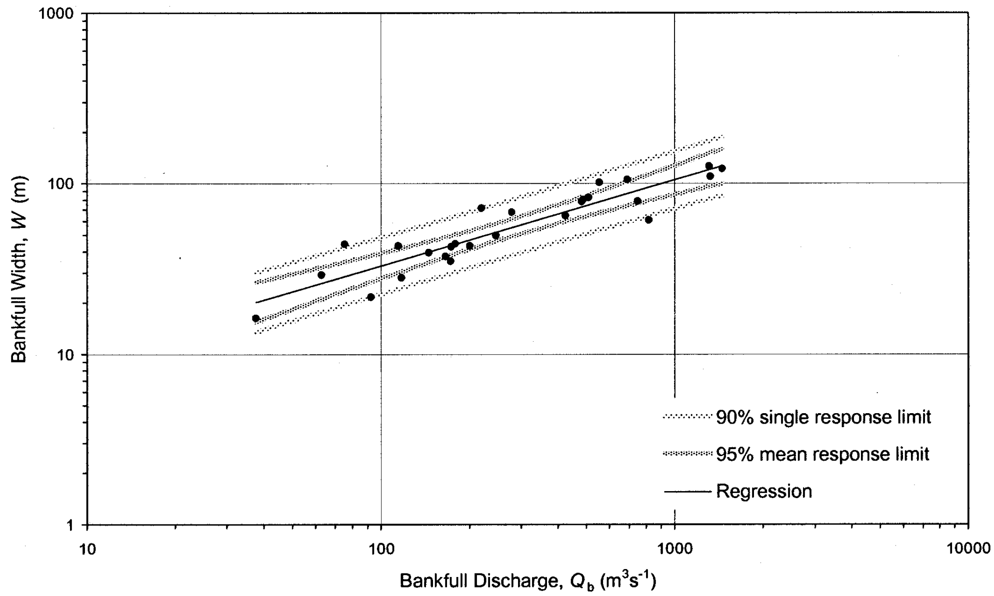


Figure 5.16 Confidence intervals applied to the width-discharge relationship $W = 3 \cdot 27 Q_b^{0 \cdot 50}$ based on 26 U.S. sand-bed sites with at least 50 percent tree cover on the banks.

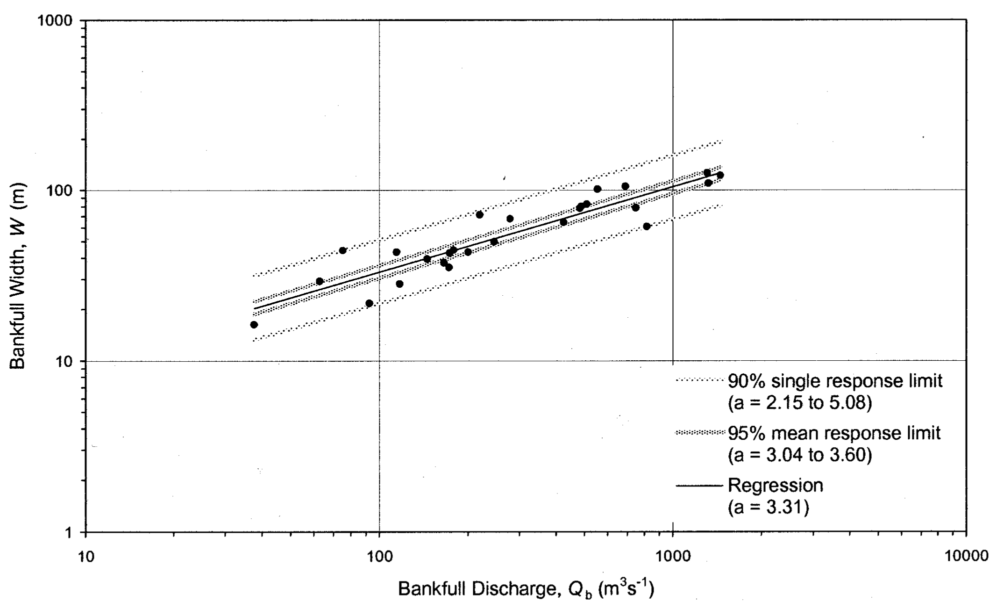


Figure 5.17 Confidence intervals applied to the width-discharge relationship with fixed discharge exponent $W = \mathrm{a}\, Q_b^{0 \cdot 5}$ based on 26 U.S. sand-bed sites with at least 50 percent tree cover on the banks.

While the equations given in Table 5.4 may be used for design purposes, they are subject to several limitations. In the absence of stage-discharge relationships at each site, the equations are based on flow resistance considerations. As cross-sectional geometry was used to calculate discharge, discharge is not truly independent of width in this analysis. Furthermore, only one cross section was measured at each site to maximise the size of the data set, and identification of the bankfull reference level is always subject to a degree of uncertainty even when based on fields experience and geomorphic criteria, as it was for this study. These factors contribute to the observed variability in the width relationships. Finally, small rivers are not well represented in the data set, and extrapolation is required if the equations are to be applied when discharge is less than 17 m^3s^{-1} in type T1 channels and less than 38 m^3s^{-1} in type T2 channels.

5.8 WIDTH-DISCHARGE RELATIONSHIPS IN GRAVEL-BED RIVERS

5.8.1 Existing Dimensional Equations

Since the inception of the hydraulic geometry concept by Leopold and Maddock in 1953, numerous morphological equations have been derived pertaining to stable channels with quasi-fixed and mobile gravel beds. Notable published contributions have been made from researchers working in the U.K. and North America, and a summary of best-fit downstream hydraulic geometry equations for channel width are given in Table 5.5. Interestingly, only the Bray (1973, 1982), Charlton et al. (1978) and Hey and Thorne (1986) equations are multivariate, but the exponents of the secondary variables are very small and can be neglected in most cases with negligible effect on width.

Other significant equations include dimensionless equations by Bray (1982), Parker (1982) and Andrews (1984) for North American rivers and hydraulic geometry of the wetted perimeter, P, as a function of the 1.5-year recurrence interval flood, $Q_{1.5}$, by Hey (1982) for U.K. rivers.

Reference		Data Source	Coefficient	Exponents			
				Q_b	Q_s	S	d_{50}
Nixon (1959)		U.K.	2.99	0.5			
Nash (1959)		U.K. U.S.A.	2.39	0.54			
Kellerhalls (1967)		U.S.A Canada Switzerland and Laboratory	3.26	0.5			
Bray (1973, 1982)		Canada	3.83	0.53*			-0.07
Emmett (1975)		U.S.A.	2.86	0.54			
Charlton et al. (1978)	type-A	U.K.	3.74	0.45			
	type-A_G		3.37-4.86	0.45			
	type-A_T		2.62-4.11	0.45			
	type-B		2.43	0.41		-0.098	
Parker (1982)		U.K.	3.73	0.45			
Parker (1982)		Canada	5.86	0.44			
Hey and Thorne (1986)	all data	U.K.	3.67	0.45			
	type-I		3.98	0.52	-0.01		
	type-II		3.08	0.52	-0.01		
	type-III		2.52	0.52	-0.01		
	type IV		2.17	0.51	-0.01		

Note: Q_b = bankfull discharge (m^3s^{-1}) (* 2-year recurrence interval flood equated with bankfull discharge); Q_s = bed load transport (kg s^{-1}); S = slope; d_{50} = median particle size of bed material (m); type-A = low sediment load; type-A_G = low sediment load and grass-lined banks; type-A_T = low sediment load and tree-lined banks; type-B = appreciable sediment load; type-I = grassy banks with no trees or shrubs; type-II = 1 to 5 percent tree/shrub cover; type-III = 5 to 50 percent tree/shrub cover; type-IV = greater than 50 percent tree-shrub cover or incised into floodplain.

Table 5.5 Existing dimensional width equations for gravel-bed rivers.

Hey (1988, 1997c) forwarded the Hey and Thorne (1986) equations for stable channels with mobile beds as best practice design equations. These equations are most appropriate to the type and size range of gravel-bed rivers found in England and Wales, in particular to channels with composite banks of gravel deposits overlain by cohesive sediments, and may be less applicable to rivers in other regions as cautioned by Burns (1998). Using published data available to the authors, a composite width equation has been derived from a compiled database of 187 sites with a discharge range between 1 m^3s^{-1} and 1000 m^3s^{-1}, and further equations for U.K. and North American gravel-bed rivers have been derived from subsets of the database.

5.8.2 Revised Equations Based on Existing and Composite Data Sets

Eleven published data sets were initially considered for examining the relationship between bankfull width, W, and bankfull discharge, Q_b, in gravel-bed rivers with mobile beds and discharges in the broad range of 1 m^3s^{-1} to 1000 m^3s^{-1}. For several of the data sets described below, site descriptions given in a catalogue of alluvial channel regime data compiled by Church and Rood (1983) aided in the selection of suitable sites for this study:

i) Wolman (1955): data from Brandywine Creek, Pennsylvania (5 sites). Hydraulic geometry data were extracted from Leopold and Wolman (1957). Banks were classified as cohesive but are composed of fine material overlain gravel deposits (Church and Rood, 1983). Discharges range from 22 m^3s^{-1} to 71 m^3s^{-1}, slopes range from 0.00062 to 0.0037 and median sizes of bed material range from 0.022 m to 0.104 m. Information is not available on sinuosity, although relatively straight reaches were identified for collecting data. At each site, a single cross section was surveyed and bankfull stage was identified from a marked break in a plot of width-to-depth ratio against stage.

ii) Nixon (1959): data from U.K. rivers (7 sites). The full data set consists of 27 width-discharge measurements for different types of bed material, but only sites labelled as gravel-bed were selected. The bank material in these streams is variable and includes sites with clay, clay-silt, silt-gravel and sand banks. Discharges range from 40 m^3s^{-1} to 333 m^3s^{-1} and slopes range from 0.00036 to 0.0025. Information is not available on sinuosity, particle size of bed material, the method used to identify the bankfull reference level or the number of cross sections surveyed at each site.

iii) Emmett (1972): data from Alaskan steams, south of the Yukon River (3 sites). Church and Rood (1983) classified the majority of sites in the full data set as anastomosed, braided or split channels with occasional islands, leaving only three sites with meandering planforms (including sites with minor secondary channels). These sites have gravel banks. Discharges range from 8.5 m^3s^{-1} to 510 m^3s^{-1}, slopes range from 0.0042 to 0.0156 and median sizes of bed material range from 0.012 m to 0.021 m. Information is not available on sinuosity. Only one cross section was measured at each site and bankfull stage corresponds to the elevation of the active floodplain.

iv) Kellerhalls et al. (1972): data from rivers in Alberta, Canada (21 sites). The original data set of 108 reaches was used to derive the equations given by Bray (1973, 1982). According to Bray (1975), bankfull discharge at these sites corresponds to the 2-year recurrence interval flow. Church and Rood (1983) rejected 34 of the original sites as unsuitable for regime studies. Only stable sites with a dominant single channel (including those with minor secondary channels) were selected for this study based on site descriptions given by Church and Rood (1983). Discharges range from 3.7 m^3s^{-1} to 368 m^3s^{-1}, slopes range from 0.0005 to 0.018, sinuosities range from 1.1 to 2.4 and median sizes of bed material range from 0.023 m to 0.14 m. The data are reach average measurements from several cross sections. Channel dimensions correspond to the elevation of the 2-year flow.

v) Emmett (1975): data from sites in the upper Salmon River area of Idaho (25 sites). Out of the full data set, only sites with meandering planforms and exhibiting stable behaviour were selected. Church and Rood (1983) rejected one site as it was located at a bridge and the channel width was fixed. Three of the sites have cohesive banks, a further three have sand banks and the remainder of the sites have gravel banks. Discharges range from 1.1 m^3s^{-1} to 145 m^3s^{-1}, slopes range from 0.0009 to 0.006 and median sizes of bed material range from 0.011 m to 0.058 m. Information is not available on sinuosity. Only one cross section was measured at each site and bankfull stage corresponds to the elevation of the active floodplain.

vi) Charlton et al. (1978): data from U.K. rivers (17 sites). These sites are meandering channels, classified as irregularly sinuous by Church and Rood (1983). The full data set contains a further 6 sites classified as split channels with occasional islands. The bank material is sand or gravel. The banks are characterised as ‘tree-lined’ for 10 of the sites and ‘grass-lined’ for the remaining 7 sites. Discharges range from 2.7 m^3s^{-1} to 157 m^3s^{-1}, slopes range from 0.0009 to 0.0137, sinuosities range from 1 to 1.65 and median sizes of bed material range from 0.033 m to 0.113 m. Information is not available on the method used to define bankfull stage. The data are reach average measurements from several cross sections. Several of the sites have negligible bed load transport.

vii) Williams (1978a): data from Colorado, New Mexico, Oregon, Pennsylvania, Tennessee, Utah, West Virginia and Wyoming (20 sites). This data set was originally used to examine methods of calculating bankfull discharge. These sites are a subset of the full data set for which bankfull stage corresponds to the elevation of the active floodplain. Church and Rood (1983) considered the data from several of the original sites to be unreliable as they conflicted with data reported in a different paper (Williams, 1978b). Only data from meandering channels (including those with minor secondary channels) have been selected for this study. Bank material is non-cohesive and varies between sites. Discharges range from 1.7 m^3s^{-1} to 89 m^3s^{-1}, slopes range from 0.001 to 0.0237 and median sizes of bed material range from 0.003 m and 0.19 m. Sinuosity is only available for 6 of the sites and ranges from 1.1 to 1.8 (Church and Rood, 1983). The data are reach average measurements from several cross sections.

viii) Griffiths (1981a): data from the Buller River catchment, South Island, New Zealand (7 sites). These data were originally used to test theoretical design equations and pertain to channels with relatively straight planforms and low bed load transport rates. Discharges range from 70 m^3s^{-1} to 650 m^3s^{-1}, slopes range from 0.0008 to 0.0066 and median sizes of bed material range from 0.02 m to 0.11 m. Information is not available on the method used to define bankfull stage. Several of the sites have negligible bed load transport. No information is available on the type of bank material or the number of cross sections surveyed at each site. Bankfull stage corresponds to the elevation of the active floodplain.

ix) Andrews (1984): data from rivers in the Rocky Mountain region of Colorado (23 sites). Church and Rood (1983) classified all of these sites as either meandering or irregularly sinuous. Bank material is either composed of gravel or cohesive material. Bank vegetation is characterised as ‘thick’ for 9 of the sites and ‘thin’ for the remaining 14 sites. Discharges range from 1.9 m^3s^{-1} to 255 m^3s^{-1}, slopes range from 0.0009 to 0.021 and median sizes of bed material range from 0.023 m to 0.122 m. Sinuosity data are only available for 18 of the sites and range from 1.1 to 2.0 (Church and Rood, 1983). The data are reach average measurements from several cross sections. Bankfull stage corresponds to the elevation of the active floodplain.

x) Hey and Thorne (1986): data from U.K. rivers (62 sites). These sites are characterised by composite banks of fine sediments overlain by gravel deposits and cover a wide range of riparian vegetation in terms of density. Discharges range from 3.9 m^3s^{-1} to 424 m^3s^{-1}, slopes range from 0.0012 to 0.021, sinuosities range from 1 to 2.5 and median sizes of bed material range from 0.014 m to 0.176 m. The data are reach average measurements from two riffles and two adjacent pool sections. Bankfull stage corresponds to the elevation of the active floodplain.

xi) Annable (1996): data from streams in Alberta, Canada (18 sites). This is a subset of a larger database of morphological characteristics. The sites exhibit a wide variety of bank material and riparian vegetation types. Discharges range from 2.3 m^3s^{-1} to 49 m^3s^{-1}, slopes range from 0.0008 to 0.023, sinuosities range from 1.2 to 2.5 and median sizes of bed material range from 0.004 m to 0.064 m. These data are reach average measurements from several cross sections. Bankfull stage was determined at each site based on various field indicators.

Other data sets not used in this study include that of Kellerhalls (1967) from sites with armoured gravel beds and Huang and Nanson (1997) from streams in southeastern Australia. The width-discharge equation derived from the latter data set was found to be significantly different to other published equations. To validate their analytical regime equations, Julien and Wargadalam (1995) used data sets compiled by Griffiths (1981c) from New Zealand rivers, Colosimo et al. (1988) from river reaches in Calabria, Southern Italy, and Higginson and Johnston (1988) from stable sites in Northern Ireland rivers. These three data sets were originally used to develop flow resistance equations for gravel-bed rivers. However, the Griffiths (1981c) data did not conform to the conventional power function of the width-discharge equation, the Colosimo et al. (1988) data pertain to a low stage, rather than bankfull, discharge and the data set by Higginson and Johnston (1988) includes bankfull velocities but not cross-sectional area data, therefore bankfull discharge cannot be accurately calculated. As a result, these data were considered unsuitable for hydraulic geometry analysis.

Bivariate hydraulic geometry equations were derived from the selected data sets and are given in Table 5.6. The General Linear Hypothesis was applied to examine whether the exponents significantly differ from 0.5. In general, deviation from the 0.5 value is greatest for the smaller data sets, which was not unexpected. Table 5.6 shows that the significance level is below 95 percent in all cases, indicating that the fixed exponent model is acceptable for the individual data sets. For each data set, the range of coefficients within 95 percent confidence limits (on the mean coefficient) is also given for the fixed exponent model.

Reference	n	a	b	R^2	$P_{b\neq0.5}$ (percent)	a^*
Wolman (1955)	5	5.68	0.36	0.66	58.4	3.33 (2.82-3.93)
Nixon (1959)	7	1.59	0.64	0.86	72.2	3.27 (2.58-4.14)
Emmett (1972)	3	2.97	0.57	0.84	17.3	3.98 (1.03-15.34)
Kellerhalls et al. (1972)	21	5.47	0.49	0.97	41.5	5.24 (5.00-5.45)
Emmett (1975)	25	3.34	0.49	0.82	16.4	3.29 (2.96-3.65)
Charlton et al. (1978)	17	4.32	0.40	0.76	89.4	2.96 (2.59-3.38)
Williams (1978a)	20	3.55	0.53	0.52	19.5	3.85 (3.09-4.80)
Griffiths (1981a)	7	2.10	0.64	0.59	33.9	2.86 (2.09-3.93)
Andrews (1984)	23	3.71	0.52	0.97	67.8	3.94 (3.71-4.19)
Hey and Thorne (1986)	62	3.67	0.45	0.79	90.0	2.97 (2.78-3.17)
Annable (1996)	18	2.45	0.66	0.61	75.7	3.83 (3.18-4.61)

Note: n = samples in data set; a = discharge coefficient when exponent is not fixed; b = discharge exponent; R^2 = coefficient of determination; $P_{b\neq0.5}$ = significance level of rejecting the null hypothesis that the exponent equals exactly 0.5; a^* = discharge coefficient when exponent is fixed at 0.5 (values in parentheses are 95 percent confidence limits on the mean coefficient value).

Table 5.6 Width-discharge relationships derived from different gravel-bed data sets.

The mean coefficient in the fixed exponent model varies between 2.86 and 5.24, which is a significant range considering all data were collected from gravel-bed rivers. However, by excluding the Kellerhalls et al. (1972) data the upper bound in this range is considerably reduced. This data set is the only one where bankfull discharge was assumed to correspond to a flow with a specific recurrence interval, the 2-year flow, and bankfull stage was not identified in the field but corresponds to the stage of the 2-year flow. While the high coefficient may be partly attributable to weak bank sediments (all sites have non-cohesive banks), it is likely that the fixed recurrence interval flow is not an adequate representation of bankfull discharge in all cases and furthermore, recorded width values may not have any morphological significance. In light of these considerations, the Kellerhalls et al. (1972) data were excluded from further analysis.

Composite equations for width have been derived from three composite data sets in the discharge range 1 m^3s^{-1} and 1000 m^3s^{-1} comprising: i) all selected data; ii) U.K. data, and; iii) North American data (Table 5.7) and reveal for the same discharge that the width in a gravel-bed river is on average significantly less than the width in a sand-bed river given by Table 5.4, assuming other conditions are equal. The composite data sets include some cobble-bed streams. A multiple regression analysis on the full data set of 187 sites revealed that the median particle size of bed material is not a significant parameter at the 95 percent level. On this basis, cobble-bed streams were not excluded.

Data Source	**n**	**a**	**b**	**R^2**	**$P_{b\neq0.5}$ (percent)**	**a^***
All data	187	3.83	0.46	0.80	99.0	3.31 (3.17-3.47)
North America	94	3.39	0.53	0.80	73.2	3.68 (3.45-3.94)
U.K.	86	3.52	0.46	0.80	89.7	2.99 (2.83-3.16)

Note: n = samples in data set; a = discharge coefficient when exponent is not fixed; b = discharge exponent; R^2 = coefficient of determination; $P_{b\neq0.5}$ = significance level of rejecting the null hypothesis that the exponent equals exactly 0.5; a^* = discharge coefficient when exponent is fixed at 0.5 (values in parentheses are 95 percent confidence limits on the mean coefficient value).

Table 5.7 Width-discharge relationships derived from composite gravel-bed data sets, including North American and U.K. data.

The two equations derived from all data and U.K data share the same exponent value of 0.46. This is partly because they are both strongly influenced by the Hey and Thorne (1986) data, which comprise the largest number of sites out of the individual data sets and yield an exponent value of 0.45 (Table 5.6). The exponent value of 0.46 is significant at the 95 percent level for the combined data set of 187 sites, therefore caution must be exercised when applying the fixed exponent model in this case. The significance of this equation is elevated due to the large sample size. Interestingly, the 0.46 value is identical to the theoretical exponent derived from tractive force theory appropriate to threshold channels (Chapter 2), despite the data collected by Hey and Thorne pertaining to mobile bed channels.

Table 5.7 reveals that North American gravel-bed rivers are generally wider than those found in the U.K. rivers, assuming discharge and other conditions are equal. This is generally the case for the individual data sets in Table 5.6, although the equations derived from Pennsylvania streams studied by Wolman (1955) and Idaho streams studied by Emmett (1975) are more similar to the equations derived from U.K. data. The difference is further exemplified by confidence limits applied to the coefficient in the fixed exponent models (Table 5.7), whereby the confidence bands for the North American and U.K. data sets lie above and below the confidence band for all data respectively.

The General linear Hypothesis confirmed that the two regional equations are significantly different at the 95 percent level. Figure 5.18 shows all selected data and the fixed exponent width-discharge relationships for gravel-bed rivers in the U.K. and North America, and confidence limits are applied to the composite data sets in Figures 5.19 to 5.22.

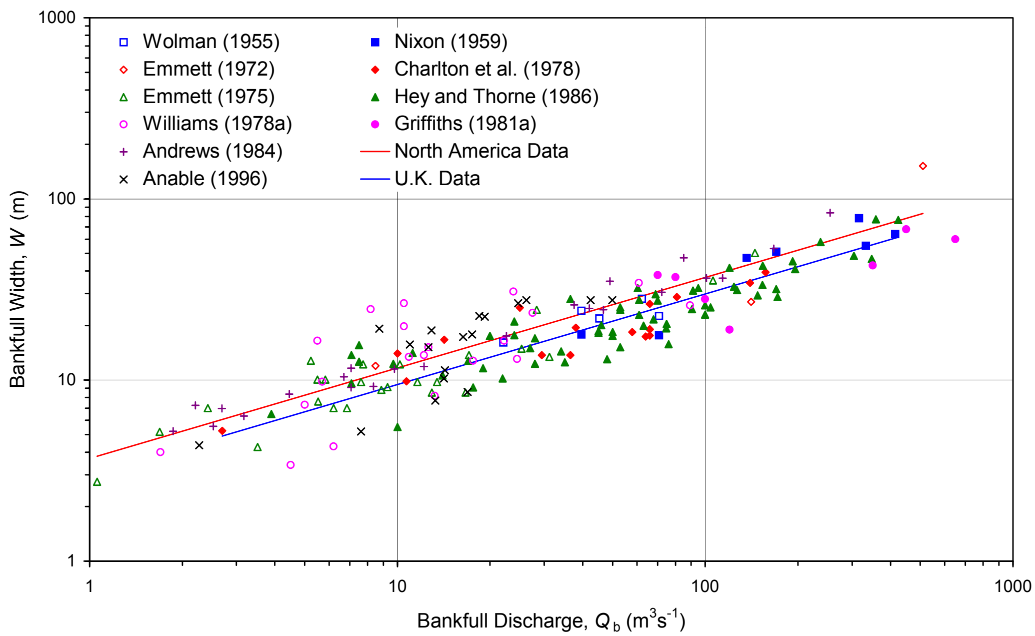


Figure 5.18 Width-discharge relationships with fixed discharge exponent for North American rivers, $W = 3{\cdot}68 Q_b^{0{\cdot}5}$, and U.K. gravel-bed rivers, $W = 2{\cdot}99\, Q_b^{0{\cdot}5}$.

The difference between the width relationships cannot satisfactorily be explained using the site descriptions given in original publications and the catalogue of alluvial data compiled by Church and Rood (1983). A possible explanation is that the U.K. sites have on average more resistant banks than the North American sites. However, many of the Andrews (1984) sites have cohesive banks, yet the fixed exponent model for this data set has a very high coefficient value of 3.94. A plausible explanation is that width in mobile gravel-bed streams varies with flow variability. This was not the case in sand-bed channels (Section 5.7.5). The high-frequency flows in streams with very variable flow regimes have the potential to deposit sediment on channel banks in between more destructive peak flow events which erode the banks, thereby maintaining equilibrium cross-sectional geometry over the medium- to long-term. However, bed material transport is negligible in gravel-bed streams during low flows, and it is unlikely that bar deposition is significant when low flows prevail, unless there is a significant sand fraction in the bed material which is readily mobilised.

Therefore, the medium- to long-term channel width in gravel-bed streams is likely to be more responsive to flashy flow regimes than in sand-bed streams, provided the bank-lines are not held by very resistant material or bank vegetation. This might also partly explain why the streams studied by Wolman (1955) that are in a catchment with low variability in annual rainfall are narrower for the same discharge than the Rocky Mountain streams studied by Andrews (1984) where annual precipitation is highly variable. Further research is required to validate this hypothesis.

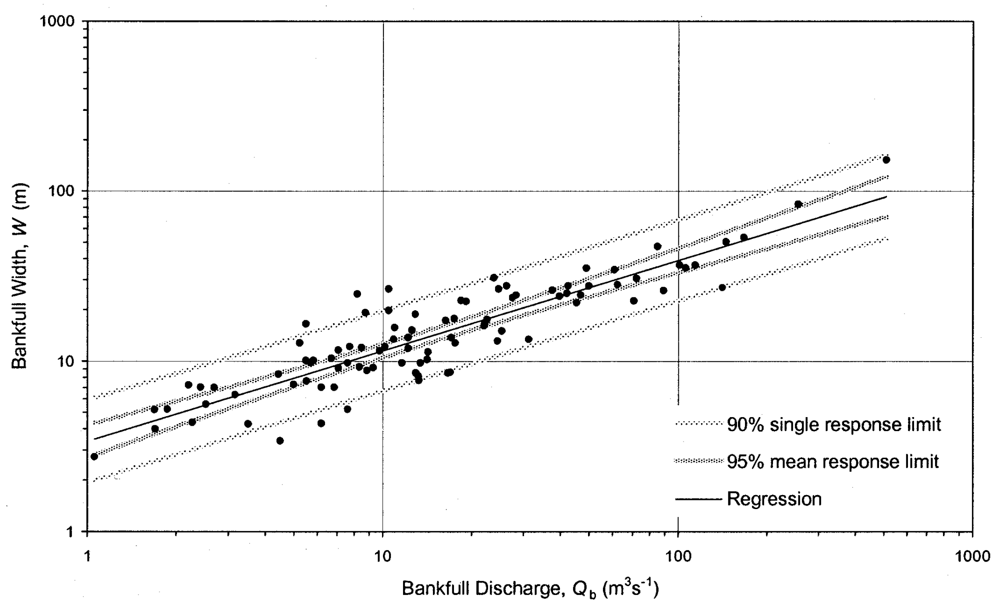


Figure 5.19 Confidence intervals applied to the width-discharge relationship $W = 3{\cdot}39 Q_b^{0{\cdot}53}$ based on 94 sites in North American gravel-bed rivers.

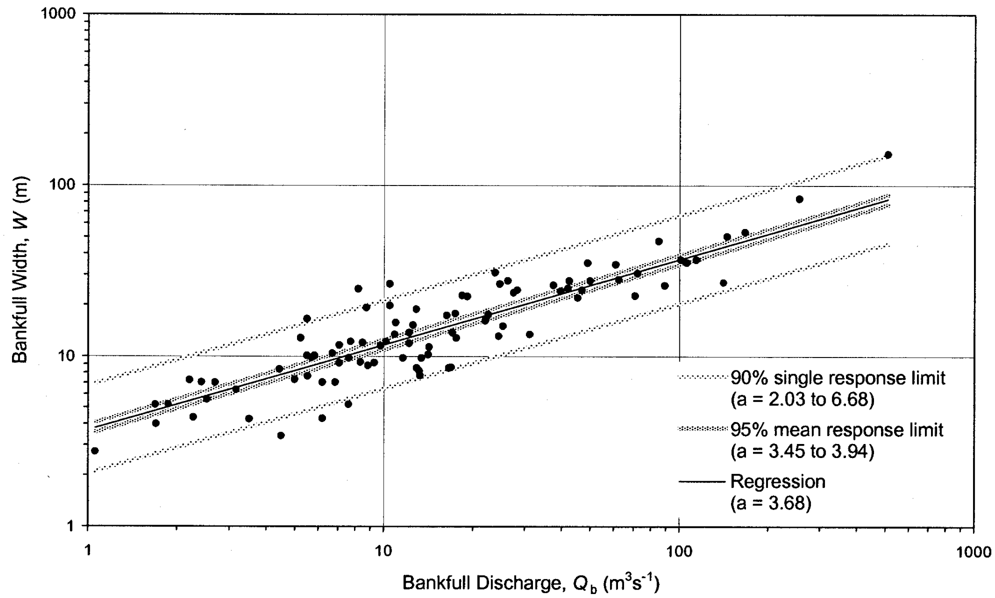


Figure 5.20 Confidence intervals applied to the width-discharge relationship with fixed discharge exponent $W = \mathrm{a}\,Q^{0\cdot 5}$ based on 94 sites in North American gravel-bed rivers.

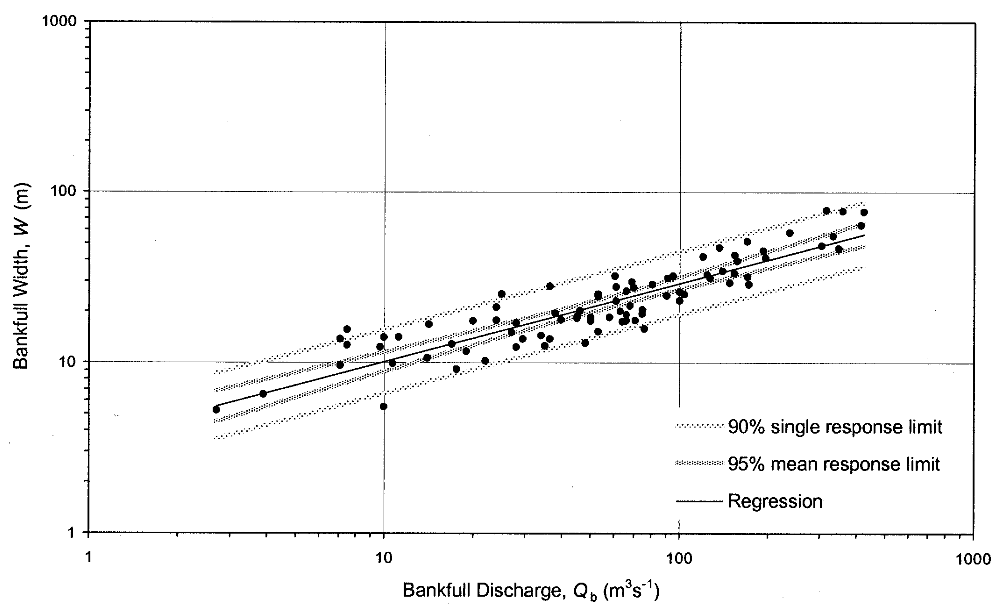


Figure 5.21 Confidence intervals applied to the width-discharge relationship $W = 3{\cdot}52 Q_b^{0\cdot 46}$ based on 86 sites in U.K. gravel-bed rivers.

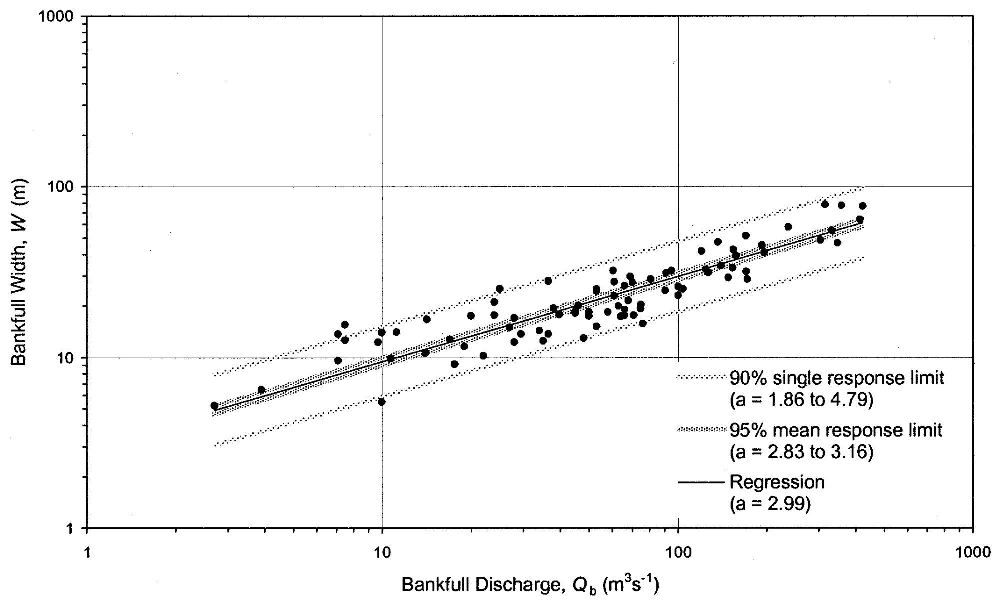


Figure 5.22 Confidence intervals applied to the width-discharge relationship with fixed discharge exponent $W = \mathrm{a}\,Q_{\mathrm{b}}^{0.5}$ based on 86 sites in U.K. gravel-bed rivers.

5.8.3 Gravel-Bed Rivers with 'Typed' Banks

The gravel-bed river data described in Section 5.8.2 comprise a wide range of bank material types (e.g. cohesive, sand, gravel and composite banks of various strata). Limited qualitative information on the composition of bank material is available in some of the original papers and in the catalogue of alluvial regime data compiled by Church and Rood (1983), however different width-discharge relationships based on different types of material could not be derived from this limited information alone. While in certain cases there may be a causal link between bank material type and width, in many cases it is likely that the strength of the bank fabric is controlled by the type, density and location of the riparian vegetation, as demonstrated by Hey and Thorne (1986). Qualitative information on the character of bank vegetation is available in the data sets of Charlton et al. (1978) for U.K rivers and Andrews (1984) for intermontane rivers in the Rocky Mountains of Colorado. In both of these data sets, two vegetation categories were used. Hey and Thorne (1986) also recorded semi-quantitative information on vegetation density using four categories (see Table 5.5). These data would have more general applicability if they are combined into composite data sets and typed according to whether sites have 'erosive' banks (low density of trees) or 'resistant' banks (high density of trees). The Hey and

Thorne data were reclassified for this analysis using two distinct groups, since it was considered that the original typing system was difficult to apply in practice because of the subjectivity in distinguishing between four ranges of bank vegetation density. This revised classification also improves the significance of width-discharge equations because sample sizes are larger and enables better comparison with the equations derived from the data compiled by Charlton et al. (1978) and Andrews (1984). The bank typing systems are given in Table 5.8 and the width-discharge relationships for discharges in the range 1 m^3s^{-1} to 1000 m^3s^{-1} based on these systems are defined in Table 5.9.

	Bank Type	
Reference	**Erodible**	**Resistant**
Charlton et al. (1978)	Grass-lined	Tree-lined
Andrews (1984, 1999 pers. comm.)	Thin: almost entirely grass with a light to moderate coverage	Thick: predominantly trees and bushes with 100 percent coverage
Hey and Thorne (1986)*	Less than 5 percent tree/shrub cover, grassy banks or incised into floodplain	At least 5 percent tree/shrub cover

Note: * modified from original definitions by combining the original bank types

Table 5.8 Definitions of erodible and resistant bank types based on categories of riparian vegetation from three data sets.

Data Source	**n**	**a**	**b**	**R^2**	**$P_{b\neq0.5}$ (percent)**	**a^***
Erodible Banks						
Charlton et al. (1978)	10	4.25	0.46	0.91	43.6	3.74 (3.09-4.51)
Andrews (1984)	9	4.18	0.50	0.95	7.2	4.13 (3.78-4.51)
Hey and Thorne (1986)	29	4.25	0.46	0.92	86.4	3.69 (2.46-5.55)
Resistant Banks						
Charlton et al. (1978)	7	2.76	0.48	0.85	25.8	2.51 (2.27-2.77)
Andrews (1984)	14	3.88	0.46	0.96	61.5	3.66 (3.43-3.91)
Hey and Thorne (1986)	33	1.85	0.57	0.93	98.0	2.45 (2.33-2.58)

Note: n = samples in data set; a = discharge coefficient when exponent is not fixed; b = discharge exponent; R^2 = coefficient of determination; $P_{b\neq0.5}$ = significance level of rejecting the null hypothesis that the exponent equals exactly 0.5; a^* = discharge coefficient when exponent is fixed at 0.5 (values in parentheses are 95 percent confidence limits on the mean coefficient value).

Table 5.9 Width-discharge relationships derived from different gravel-bed data sets with typed bank vegetation.

The General Linear Hypothesis was used to examine the similarity in the equations given in Table 5.9 and whether discharge exponents could be fixed at 0.5 without compromising statistical significance. The results of the analysis are given in two Venn diagrams (Figure 5.23) which portray simultaneous significance levels where two or all three data sets were compared. The values represent the significance levels of *rejecting* the null hypothesis that: i) equations are the same (upper values), and: ii) discharge exponents are exactly 0.5 (lower values). For all three data sets separately, the equations for the erodible and resistant bank types were significantly different at the 95 percent level.

For the erodible bank type, at the 95 percent level all three width-discharge equations are not significantly different from each other, and all equations can assume a fixed exponent value of 0.5, whether considered individually or collectively. The results also show that the two equations from U.K. data sets are almost identical, while the equation from the Colorado data, with a higher coefficient in the fixed model, is slightly (although not statistically) different.

For the resistant bank type, a fixed exponent model at the 95 percent level cannot adequately represent the Hey and Thorne equation, with a high exponent value of 0.57. Therefore, fixing the exponent at 0.5 for the purpose of developing practical engineering equations (as undertaken by Hey and Thorne, 1986), will underestimate and overestimate the best-fit power function at high discharges and low discharges, respectively. The equation derived from Colorado data is also significantly different to the equations derived from U.K. data. These U.K. data equations show marked similarities and reflect the finding in the previous section that the selected North American sites are wider on average for the same discharge than the selected U.K. sites.

The differences between the equations and their respective variabilities can be visualised by displaying confidence ellipses applied to the coefficients and exponents simultaneously (Figure 5.24). For the erodible bank type, there is considerable overlap of confidence regions, the difference in size reflecting differences in variability of data points about the best-fit power functions and different sample sizes. However, for the resistant bank type, the location and orientation of the ellipse described by the Colorado data are significantly different from that of the two ellipses described by the U.K. data, which entirely overlap despite their different origins.

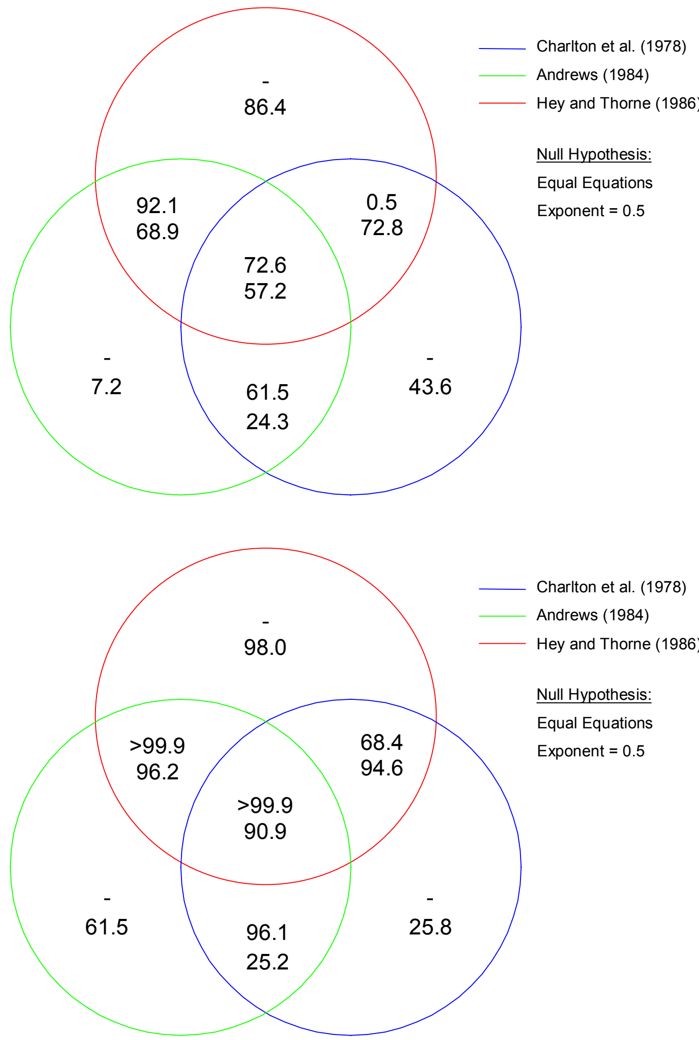


Figure 5.23 Venn diagrams showing the results of applying the General Linear Hypothesis to width-discharge relationships typed by 'erosive' banks (upper figure) and 'resistant' banks (lower figure), according to bank vegetation categories. Values are significance levels of *rejecting* the null hypothesis that: i) equations are the same (upper values), and: ii) discharge exponents exactly equal 0.5 (lower values). Simultaneous significance levels for comparing two or all three equations pertaining to each bank type are defined where circles overlap.

These techniques demonstrate that the two U.K. data sets can be combined to give the data more general applicability. As the resistant bank equation derived from the Colorado data is significantly different from the equations for U.K. rivers with resistant banks, it is recommended that the Andrews data should not be combined with the U.K. data. In general, the Colorado streams with tree-lined banks have similar widths, for the same discharges, to those in the U.K. streams with grass-lined/low tree-density banks. In the absence of further North American gravel-bed river data with typed riparian vegetation, the Andrews equations are limited as practical design equations and caution must be exercised when applying them beyond the range of conditions found in the Rocky Mountain region of Colorado. Based on the above analysis, composite equations for U.K. gravel-bed rivers with confidence bands are shown in Figures 5.25 to 5.28.

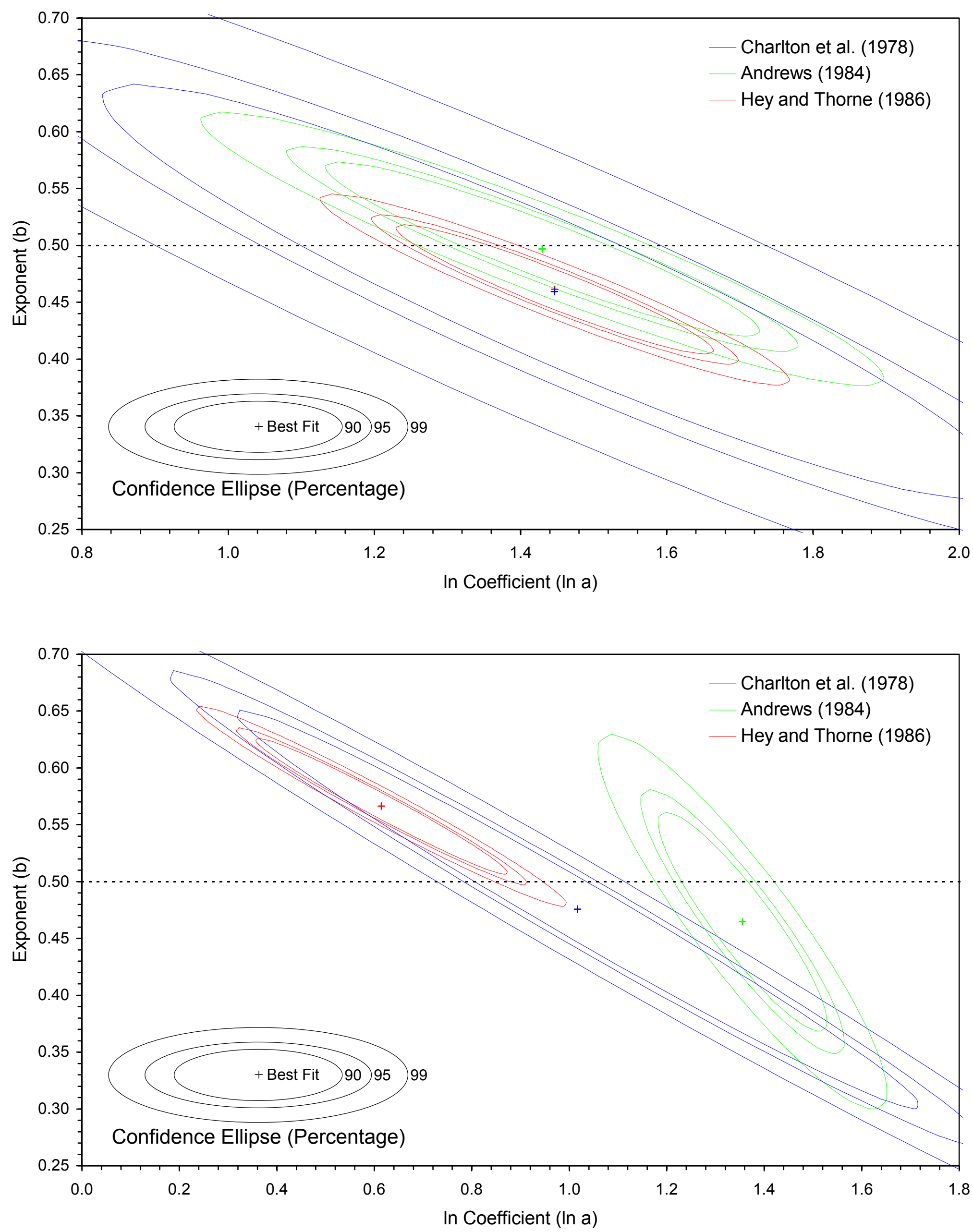


Figure 5.24 Confidence ellipses applied to the coefficient and exponent in width-discharge equations typed by 'erosive' banks (upper figure) and 'resistant' banks (lower figure), according to bank vegetation categories.

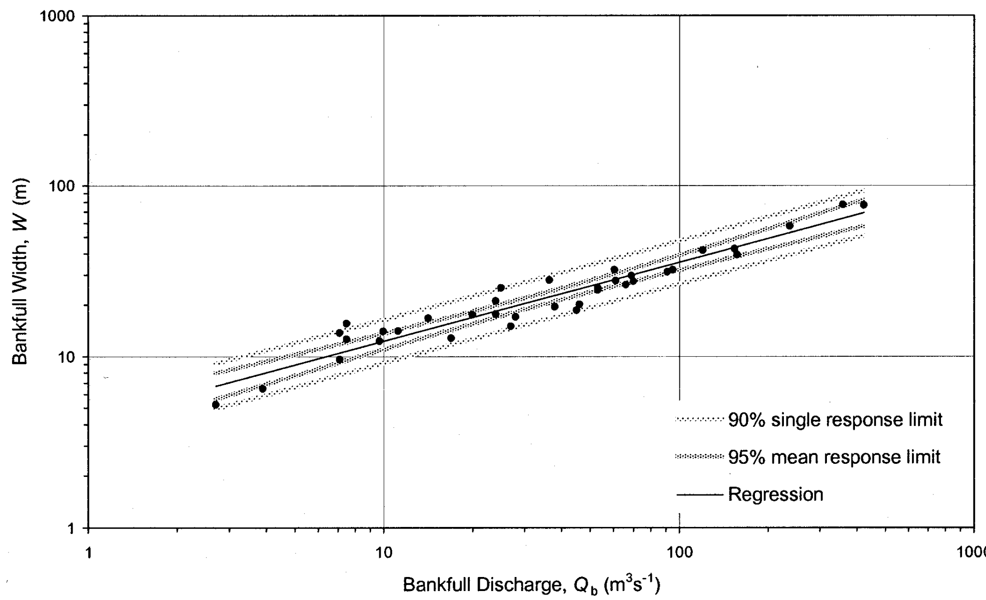


Figure 5.25 Confidence intervals applied to the width-discharge relationship $W = 4 \cdot 25 Q_b^{0 \cdot 46}$ based on 36 sites in U.K. gravel-bed rivers with 'erodible' banks.

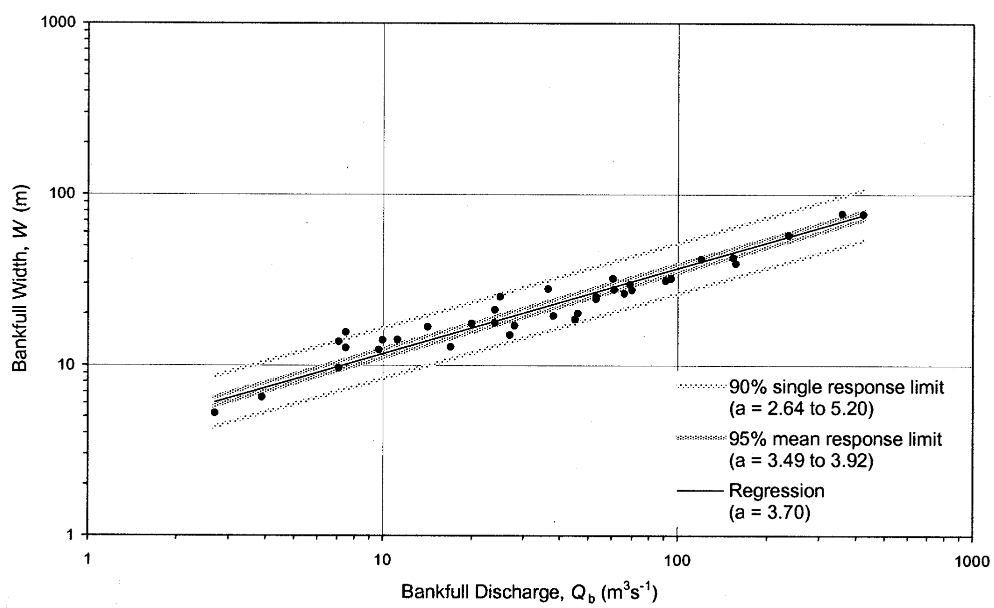


Figure 5.26 Confidence intervals applied to the width-discharge relationship with fixed discharge exponent $W = \mathrm{a}\, Q^{0 \cdot 5}$ based 36 sites in U.K. gravel-bed rivers with 'erodible' banks.

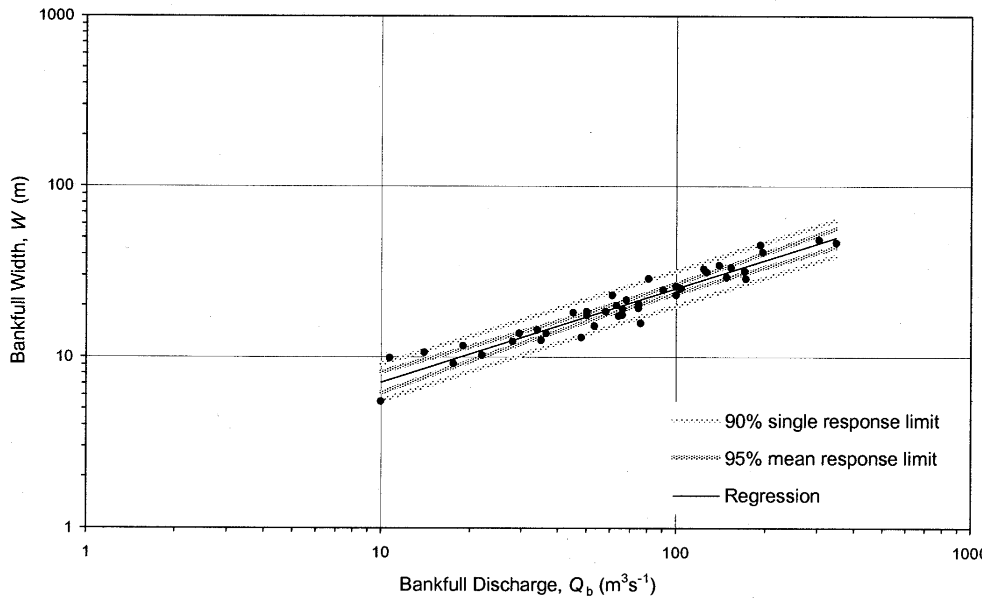


Figure 5.27 Confidence intervals applied to the width-discharge relationship $W = 2\cdot 00\, Q_\mathrm{b}^{0\cdot 55}$ based on 43 sites in U.K. gravel-bed rivers with 'resistant' banks.

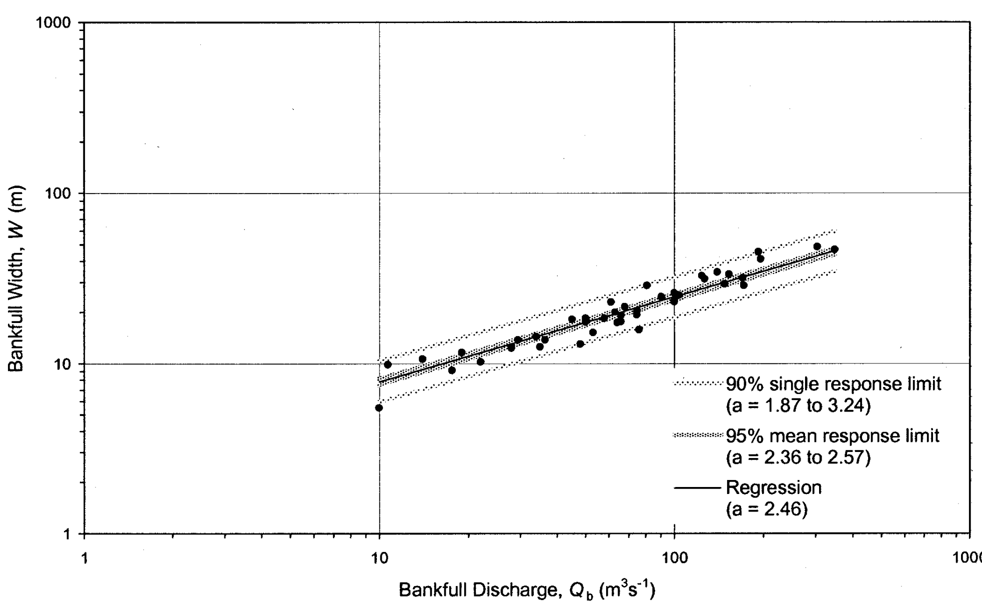


Figure 5.28 Confidence intervals applied to the width-discharge relationship with fixed discharge exponent $W = \mathrm{a}\, Q_\mathrm{b}^{0\cdot 5}$ based on 43 sites in U.K. gravel-bed rivers with 'resistant' banks.

5.9 CHANNEL GEOMETRY ANALYSIS BASED ON THE OSTERKAMP AND HEDMAN DATA SET

5.9.1 Concept and Purpose

The concept of 'channel geometry' was first employed by Langbein (1960) to estimate the mean flow condition from average width for the purpose of extrapolating flow data. Direct measurement of flood discharge at ungauged sites is problematic (Wharton, 1995a) and calls for indirect techniques of flood discharge estimation for design and appraisal purposes. The method offers a quick and inexpensive reconnaissance-level approach to flood estimation and an improvement over traditional slope-area methods which require slope and roughness information (Osterkamp and Hedman, 1979). The technique was defined by Osterkamp and Hedman (1982, p. 1) as 'an indirect means of evaluating streamflow characteristics at a site' and, through manipulation of the equations, has potential application for river channel design and management (Wharton, 1995a).

5.9.2 Technique

Channel geometry equations are the antithesis of conventional hydraulic geometry equations, since discharge is the dependent (predicted) variable and channel geometry attributes are the independent, measured variables. Since it is the river's flow regime which dictates the channel geometry of a natural river, as represented in hydraulic geometry equations, the technique does not link cause and effect. The technique is simply a statistical method to predict, rather than explain, the magnitude of flood discharges in natural channels (Osterkamp and Hedman, 1979). Using similar techniques to hydraulic geometry, 'channel geometry' equations are developed from data pertaining to natural channel systems by relating stream flow data from gauging stations to river channel dimensions measured in the vicinity of those stations using regression techniques (Wharton, 1995a, p. 650). Using channel geometry as a modification of the hydraulic geometry concept was first proposed by Moore (1968) in Nevada and later developed by Hedman (1970) for California streams. The equations take the form of power functions and generally relate discharge of some statistical frequency to either width or cross-sectional area, measured at a specified geomorphic reference stage (Wharton, 1995a). Detailed guidelines for undertaking the channel geometry method, including selecting suitable reaches, measuring cross sectional dimensions and computing the flood discharges are given by Wharton (1992, 1995a) and Osterkamp and Hedman (1982).

Because of the considerable variance in depth-discharge and slope-discharge relationships, most channel geometry equations are limited to channel width as the independent variable, such that

$$Q_{\mathrm{f}} = \mathrm{a}\,W^{\mathrm{b}} \tag{5.4}$$

where 'a' is a coefficient and 'Q_{f}' is a measure of flow frequency, such as mean annual discharge or a flood discharge of a specific recurrence interval such as the 2-year recurrence interval flood, Q_2.

5.9.3 Channel Geometry Reference Levels

The geomorphic reference level employed in the work of Osterkamp and Hedman is the 'active channel' level, which was first proposed by Hedman et al. (1974) to predict flood frequencies and was described by Osterkamp and Hedman (1977, p. 256) as '…a short-term geomorphic feature…actively, if not totally sculptured by the normal processes of water and sediment discharge'. This differs from the conventional definition of the bankfull reference level which was described by Williams (1978a, p. 1141) as 'an overflow surface that is periodically constructed and possibly eroded by the river but is undergoing net growth during the 'present time' (past ten years or so)'. The bankfull stage is usually equated to the level of the active floodplain or 'valley flat' (Williams, 1978a) but has also been referenced as the lowest level of perennial vegetation (Schumm, 1960; Nunnally, 1967). The interface of perennial vegetation and exposed bank surface also befits the active channel definition given by Hedman and Kastner (1977, p. 286): 'The reference level used to measure the geometry of the active channel is selected where the banks abruptly change to a more gently sloping surface. This level is associated with the stabilising influence of riparian vegetation. Hence the break in slope identifying the active-channel reference level is generally coincident with the lower limit of perennial vegetation. In perennial streams the active channel is exposed between 75 and 94 percent of the time' (Hedman and Osterkamp, 1982, p. 3).

In baseflow-dominated rivers in humid environments, the active channel and active flood plain levels may coincide (Wharton, 1995a, p. 654). According to Osterkamp (1998, pers. comm.), 'the widths…for the active channel ordinarily are only slightly less, as a percentage, than are channel widths measured from the bankfull reference level. As a fairly decent generalization, with (1) decreasing channel size and drainage-basin area, (2) increasing coarseness of bed sediment, and (3) increasing channel gradient, differences (and the significance) between active channel and bankfull morphologies increase'. Therefore, the active channel morphology approximates the bankfull condition in relatively large streams with well-developed floodplains. Conversely, in high gradient channels bedded with alluvial cobbles and boulders, the active channel reference level may be well defined but the absence of an identifiable floodplain inhibits the use of the bankfull level. As the channel restoration design procedures usually target lowland, meandering streams, the bankfull width may be tentatively equated with the active channel width, the difference being inversely proportional to stream size.

Furthermore, Osterkamp et al. (1983) commented that geometry data collected at the floodplain level might be related to flood discharges with recurrence intervals of two years, Q_2. This flood flow has been approximated to the bankfull discharge in previous research (Bray, 1975, 1982; Biedenharn et al., 1987; Watson et al., 1997) and is often used in channel geometry equations, including those derived by Osterkamp and Hedman (1982).

5.9.4 The Channel Geometry Data Collection

The extent of channel geometry research is described by Osterkamp and Hedman (1982) and Wharton (1995a, 1995b). Equations to predict mean annual discharge, Q_m, from the active channel width have been derived by Osterkamp and Hedman (1977) for 32 high gradient streams in Montana, Wyoming, South Dakota, Colorado and New Mexico, Osterkamp and Hedman (1982) for 252 streams in the Missouri and from bankfull width by Wharton (1992) for 75 sites in England, Wales and Scotland. However, the mean annual discharge occurs at a stage considerably below the bankfull reference level, therefore such equations have limited applicability for channel restoration design which is based on conveying the bankfull discharge through a reach with stable bankfull dimensions.

A large data set has been assembled which predicts statistical flow frequencies, including Q_2 from channel width for a wide range of conditions found in streams in the U.S.A. The morphological relationships are documented in three main publications between 1977 and 1982: i) Hedman and Osterkamp (1982) for 151 streams in western U.S.A.; ii) Hedman and Kastner (1977) for 131 streams in the Missouri Basin, and; iii) Osterkamp and Hedman (1982) for 252 streams in the Missouri Basin.

The data collected by Hedman and Osterkamp (1982) pertain to channel sites in arid to semiarid areas of western U.S.A., and the streams surveyed had mostly ephemeral flow regimes. Some perennial stream data are included in the data set but generally refer to mountainous areas where snowmelt and high magnitude rainfall events sustain a constant runoff regime. The equations relating active channel width to Q_2 for four different physioclimatic environments in western U.S.A. are given in Table 5.10 (recalculated to make width the dependent variable and given in metric units).

Physioclimatic Region		Equation
Alpine and pine-forested	$W_A = 2 \cdot 25 Q_2^{0 \cdot 61}$	(5.5)
Northern plains and intermontane areas east of Rocky Mountains	$W_A = 1 \cdot 06 Q_2^{0 \cdot 63}$	(5.6)
Southern plains east of Rocky Mountains (subject to intensive precipitation events)	$W_A = 0 \cdot 74 Q_2^{0 \cdot 59}$	(5.7)
Plains and intermontane areas west of Rocky Mountains	$W_A = 1 \cdot 76 Q_2^{0 \cdot 59}$	(5.8

Note: W_A = active channel width; Q_2 = 2-year recurrence interval flow.

Table 5.10 Width-discharge relationships for streams in different physioclimatic regions in western U.S.A (derived from channel geometry equations of Hedman and Osterkamp, 1982).

The above hydraulic geometry equations are not particularly appropriate for channel design of perennial meandering rivers because most streams in the data set have zero discharge for at least 20 percent of the time. Furthermore, the equations are typed according to regional characteristics rather than the physical character of the streams themselves.

Channel geometry research on perennial streams in the Missouri Basin by Hedman and Kastner (1977) also adopted a regional approach for typing morphological equations. Recognising the limitations for application based on the physioclimatic typing methods used in previous research, a later study in the Missouri basin by Osterkamp and Hedman (1982) presented an alternative typology based on a data set comprising 252 streams by recognising that intrinsic sediment characteristics have a measurable effect on geometry-discharge relationships. This comprehensive study proposed seven stream types based on the sediment characteristics of the channel bed and banks for the purpose of predicting mean annual discharge and fixed frequency discharges of the 2-, 5-, 10-, 25-, 50- and 100-year floods for perennial streams in the Missouri basin. The typology uses the proportion of silt and clay in the bed for the first three groups, a combination of bed and bank characteristics for the next two groups and bed sediment characteristics for the remaining two groups (Table 5.11).

Stream Type		Bed d_{50} (mm)	Bed silt-clay content (percent)	Bank silt-clay content (percent)
i)	High silt-clay bed	<2.0	61 to 100	n/a
ii)	Medium silt-clay bed	<2.0	31 to 60	n/a
iii)	Low silt-clay bed	<2.0	11 to 60	n/a
iv)	Sand bed, silt banks	<2.0	1 to 10	70 to 100
v)	Sand bed, sand banks	<2.0	1 to 10	1 to 69
vi)	Gravel bed	2.0 to 64.0	n/a	n/a
vii)	Cobble bed	>64.0	n/a	n/a

Note: d_{50} = median particle size of bed material. Bank silt-clay content is the higher value of two samples taken from the upper and lower bank. n/a = data not available.

Table 5.11 Channel ‘typing’ scheme adopted by Osterkamp and Hedman to describe active channel geometry of perennial streams in the Missouri basin (modified from Osterkamp and Hedman, 1982, p. 8).

According to Osterkamp and Hedman (1982, p. 8), sediment characteristics were not used in multiple regression equations because they would have proved too complex for general use or would be oversimplified or inaccurate. Equations relating Q_2 to active channel width for the seven channel types are derived here from the original published data using regression analysis and are given in Table 5.12.

Stream Type		n	a	b	R^2	$P_{b\neq 0.5}$ (percent)	a^*
i)	High silt-clay bed	15	1.32	0.51	0.88	7.7	0.80 (0.69-0.93)
ii)	Medium silt-clay bed	17	2.48	0.38	0.39	68.4	1.50 (1.10-2.04)
iii)	Low silt-clay bed	30	1.33	0.53	0.60	30.3	1.53 (1.21-1.93)
iv)	Sand bed, silt banks	33	0.51	0.79	0.86	>99.9	2.24 (1.87-2.67
v)	Sand bed, sand banks	96	2.14	0.58	0.72	97.1	3.03 (2.65-3.46)
vi)	Gravel bed	42	1.30	0.64	0.67	94.6	2.15 (1.79-2.59)
vii)	Cobble bed	19	1.63	0.60	0.81	83.7	2.34 (1.87-3.05)

Note: n = samples in data set; a = coefficient when exponent is not fixed; b = exponent; R^2 = coefficient of determination; $P_{b\neq 0.5}$ = significance level of rejecting the null hypothesis that the exponent equals exactly 0.5; a^* = coefficient when exponent is fixed at 0.5 (values in parentheses are 95 percent confidence limits on the mean coefficient value).

Table 5.12 Width-discharge relationships expressing active channel width as a power function of the 2-year recurrence interval flow for seven stream types in the Missouri basin based on bed and bank sediment characteristics (Source data: Osterkamp and Hedman, 1982).

The range of active widths in the Osterkamp and Hedman data set range from 0.762 to 430 m and slopes range from 0.000060 to 0.028. The channel material characteristics of streams sampled in the Missouri basin range from those with as much as 92 percent silt-clay content in the bed sediment to median particle sizes as great as 250 mm in alpine streams.

The results confirmed previous assertions by Osterkamp (1980) and showed that for streams of similar discharge, narrowest channels occur when the sediment load is entirely composed of silt and clay, as cohesive banks are generally assured and are not easily eroded by large flood events. Channel widths are larger in sand-bed channels, reaching a peak for streams that transport a medium- to coarse-grained sand load. For streams with a medium particle size of sediment load greater than coarse sand, the bed and banks become protected and stabilised because of armouring and the resultant channels are narrower (Osterkamp and Hedman, 1982). In Table 5.12 the exponent tends to vary with the magnitude of bed material load. The lowest exponent values are associated with silt-clay bed channels with negligible bed load. The exponent increases significantly for channels with both sand beds and banks and Osterkamp (1980) remarked that the exponent may be as high as 1.0 for braided streams with appreciable sediment loads. In gravel- and cobble-bed streams, bed armouring controls bed material load and the exponent values are lower than in sand-bed channels. These findings favour a variable exponent model (Osterkamp et al., 1983; Rhoads, 1991) for the width-discharge relationships of natural channels and question previous research which suggests that it is the coefficient of hydraulic geometry equations which is a function of channel type and the exponent tends to be fixed at approximately 0.5. Table 5.12 shows that only the sand-bed types iv and v have exponent values different to 0.5 at the 95 percent significance level. These findings also conflict with the composite sand-bed and gravel-bed equations given in Sections 5.7.6 and 5.8.2 where the discharge exponent approximates 0.5 at the 95 percent significance level. As the significance levels are sensitive to the coefficient of determination, R^2, the exponents in the other equations do not statistically differ from 0.5 at the 95 percent significance level despite a wide range of exponent values.

Kolberg and Howard (1995) used statistical methods to assess similarity within the Osterkamp and Hedman typology for midwestern U.S.A. streams using mean annual discharge data, Q_m, rather than the 2-year flood, Q_2, used herein. Their approach involved comparing each of the channel geometry equations for ‘equivalence’ of regression equations and ‘parallelism’ of regression slopes. The results showed that seven distinct types were not statistically significant at the 0.25 (75 percent) level and only three types based on sediment characteristics were necessary to differentiate between the sample sites. The following types were proposed (Kolberg and Howard, 1995, p. 2358): i) high silt and clay-bed streams (silt-clay content greater than 60 percent); ii) gravel-bed and cobble-bed streams, and; iii) sand-bed or silt-bed channels with sand, silt or clay banks. These results confirmed the assertion by Howard (1980) that thresholds in the hydraulic geometry of sand-bed and gravel-bed streams do exist. The consistency of discharge-width relationships in sandy alluvial channels, types ii to v, was further noted by Rhoads (1991).

Kolberg and Howard (1995) also showed that a downstream hydraulic geometry typology is only required for lowland rivers. Using a data set from Virginia and North Carolina, they found no conclusive evidence that discharge-width relationships are controlled, in part, by sediment characteristics in piedmont streams, possibly as a result of the relatively homogeneous conditions found in the piedmont zone. This is in accordance with other findings from different regions (Osterkamp and Hedman, 1982; Schumm, 1960; Nanson and Hickin, 1986; Miller and Onesti, 1979). Furthermore, Kolberg and Howard (1995) demonstrated that the channel geometry relationships of sand-bed and gravel/cobble-bed streams are only statistically different for active channel widths greater than 10 m wide.

Using the General Linear Hypothesis, a similar analysis to that of Kolberg and Howard was undertaken to compare the hydraulic geometry equations derived from the Osterkamp and Hedman data (Table 5.12), using Q_2, rather than Q_m. The results of these diagnostic tests are given in Table 5.13.

Type	i	ii	iii	iv	v	vi	vii
i		32.7	10.8	97.6	>99.9	98.1	97.9
ii			42.6	99.9	>99.9	98.6	98.5
iii				98.7	>99.9	98.0	97.4
iv					>99.9	94.6	99.1
v						95.7	55.2
vi							20.8

Table 5.13 Results of applying the General Linear Hypothesis to test equivalence between width-discharge relationships of seven stream types defined by Osterkamp and Hedman (1982). Values refer to the percentage significance level of rejecting the null hypothesis of equal equations.

The results do not corroborate the findings of Kolberg and Howard (1995) but indicate that the equations for types i, ii and iii are not significantly different at the 95 percent level. This suggests that once the silt-clay content in the bed exceeds 10 percent, channels tend to exhibit the same width-discharge relationships. In contrast, the sand-bed types iv and v have very similar morphological relationships at the 95 percent level (the equivalence of types iv and vi marginally failing at 94.6 percent). This suggests that for sand-bed channels in the data set, the nature of the bank material has a strong influence on the magnitude of channel width. The results also reveal that the gravel-bed and cobble-bed streams in the data set do not have different width-discharge relationships at the 95 percent level. The only unexpected result in the equivalence tests is the marked similarity between the equations of types v (sand bed and banks) and vii (gravel bed). This may be a result of poor data representation in the equations or the fact that sand-bed and cobble-bed streams both have significant form roughness due to dunes and large particle elements respectively which may exert similar influences on the hydraulic geometry of these sites.

The results in Table 5.13 suggest that only four channel types are necessary to represent the Osterkamp and Hedman Missouri basin data: i) streams with greater than 10 percent silt-clay content in the bed; ii) sand-bed streams with silt banks; iii) sand-bed streams with sand banks, and; iv) gravel-bed and cobble-bed streams (median bed material greater than 2.0 mm).

5.9.5 Modified Width Equations Based on Four Stream Types

Morphological equations expressing active channel width as a function of the 2-year flow for four stream types are given in Table 5.14. The results of applying the General Linear Hypothesis to test equivalence between these relationships are given in Table 5.15. At the 95 percent significance level, the results confirm that the new stream types have statistically different width-discharge relationships and the fixed exponent model is only appropriate to channels with silt-clay beds. The high discharge exponents conflict with the equations for sand-bed and gravel-bed rivers given in Sections 5.7.6 and 5.8.2, where the exponent approximates 0.5 at the 95 percent level. While the active channel and bankfull reference levels may be similar, it is likely that the fixed frequency flow used in the analysis does not correspond to the bankfull discharge in many cases and may partly explain the observed deviations from the theoretical 0.5-exponent value.

In summary, the absence of bankfull measurements in the Osterkamp and Hedman (1982) data set limits its application for hydraulic geometry analysis. Osterkamp (1980) explained the high discharge exponents, especially that pertaining to sand-bed channels, on the basis of differences in sediment load. However, the U.S. data set of 58 sand-bed rivers discussed in Section 5.7.6 suggests that the 0.5-exponent value is statistically significant for streams with different types of bank characteristics. Despite the wide range

Stream Type	n	a	b	R^2	$P_{b\neq0.5}$ (percent)	a^*
a) Low to high silt-clay bed	62	1.41	0.48	0.53	25.9	1.30 (1.06-1.60)
b) Sand bed, silt banks	33	0.51	0.79	0.86	>99.9	2.24 (1.76-2.86)
c) Sand bed, sand banks	96	2.14	0.58	0.72	97.1	3.03 (2.54-3.61)
d) Gravel or cobble bed	61	1.41	0.63	0.72	98.4	2.22 (1.83-2.70)

Note: n = samples in data set; a = coefficient when exponent is not fixed; b = exponent; R^2 = coefficient of determination; $P_{b\neq0.5}$ = significance level of rejecting the null hypothesis that the exponent equals exactly 0.5; a^* = coefficient when exponent is fixed at 0.5 (values in parentheses are 95 percent confidence limits on the mean coefficient value).

Table 5.14 Width-discharge relationships expressing active channel width as a power function of the 2-year recurrence interval flow for four stream types in the Missouri basin based on bed and bank sediment characteristics (Source data: Osterkamp and Hedman, 1982).

Type	i	ii	iii	iv
i		32.7	10.8	97.6
ii			42.6	99.9
iii				98.7
iv				

Note: Values refer to the percentage significance level of rejecting the null hypothesis of equal equations.

Table 5.15 Results of applying the General Linear Hypothesis to test equivalence between width-discharge relationships of four stream types based on data collected by Osterkamp and Hedman (1982).

of stream types compiled in the Osterkamp and Hedman data set, it is recommended that morphological equations derived from the data set should not be used for channel restoration design and reserved for examining the relationships between the active channel geometry and discharges with fixed frequencies until further site data are collected.

5.10 DESIGN EQUATIONS FOR CHANNEL WIDTH INCORPORATING NATURAL VARIABILITY

The general form of the width-discharge relationship, corrected for bias (Chapter 3) is given by

$$W = F\,\mathrm{a}\,Q^{\mathrm{b}} \tag{5.4}$$

where 'W' is bankfull width, 'Q' is the bankfull discharge and 'a' and 'b' are defined by ordinary least squares regression of natural logarithmic transformed variables and 'F' is a correction factor to account for bias as a result of the transformation. Using modified versions of the confidence interval equations given in Chapter 3, bankfull width within 100(1-p) percent confidence limits, W_p, is given by

$$W_p = F\,\mathrm{e}^k \tag{5.5}$$

$$k = \ln\left(\mathrm{a}Q^{\mathrm{b}}\right) \pm \mathrm{c}_{1,p}\sqrt{\frac{1}{\mathrm{c}_2} + \frac{\left[\ln(Q) - \mathrm{c}_3\right]^2}{\mathrm{c}_4}} \tag{5.6}$$

where values of $\mathrm{c}_{1,p}$, c_2, c_3 and c_4 define natural bankfull width variability, at 'p' probability (Table 5.16).

An expression for bankfull width based on the fixed exponent model (discharge exponent=0.5), corrected for bias, is given by

$$W = F^*\,\mathrm{a}^*\,Q^{0\cdot 5} \tag{5.7}$$

where a^* is the coefficient in the fixed exponent model and equals the exponential of the mean of the natural logarithm of $W/(Q^{0.5})$ and F^* is a correction factor to account for bias as a result of the logarithmic transformation. Using modified versions of the confidence interval equations given in Chapter 3, bankfull width within 100(1-p) percent confidence limits, W_p, with discharge exponent fixed at 0.5, is given by:

$$W_p = F^*\,\mathrm{e}^k\,Q^{0\cdot 5} \tag{5.8}$$

$$k = \mathrm{c}_5 \pm \mathrm{c}_{6,p}\,\mathrm{c}_7 \tag{5.9}$$

where values of c_5, $\mathrm{c}_{6,p}$, and c_7 define natural width variability, at 'p' probability (Table 5.17).

Channel Type			Best-Fit Equation			Confidence Limits					
Bed	Bank	Source	*F*	a	b	$c_{1,0.01}$	$c_{1,0.05}$	$c_{1,0.1}$	c_2	c_3	c_4
S	all	U.S.A.	1.051	3.76	0.52	0.841	0.631	0.528	58 (0.983)	5.694	65.123
S	E_1	U.S.A.	1.026	4.88	0.51	0.631	0.469	0.390	32 (0.670)	5.757	40.522
S	R_1	U.S.A.	1.023	3.27	0.50	0.600	0.443	0.367	26 (0.963)	5.617	24.322
G	all	U.S.A.	1.054	3.39	0.53	0.853	0.644	0.539	94 (0.989)	2.700	136.011
G	E_2	U.S.A.	1.013	4.18	0.50	0.488	0.348	0.285	14 (0.933)	3.642	24.372
G	R_2	U.S.A.	1.004	3.88	0.46	0.300	0.203	0.162	9 (0.900)	1.629	5.148
G	all	U.K.	1.033	3.52	0.46	0.669	0.505	0.422	86 (0.989)	3.973	104.89
G	E_3	U.K.	1.014	4.25	0.46	0.458	0.341	0.284	36 (0.973)	3.528	53.481
G	R_3	U.K.	1.009	2.00	0.55	0.369	0.276	0.230	43 (0.977)	4.165	29.543
G	E_4	U.K.	1.014	4.25	0.46	0.454	0.336	0.279	29 (0.967)	3.620	41.664
G	R_4	U.K.	1.009	1.85	0.57	0.366	0.272	0.226	33 (0.971)	4.231	24.580

Note: G = Gravel; S = Sand; E_1 = <50 percent tree cover; R_1 = ≥50 percent tree cover; E_2 = 'thin' vegetation (Andrews, 1984); R_2 = 'thick' vegetation (Andrews, 1984); E_3 = <5 percent tree/shrub cover or 'grass-lined' banks; R_3 = ≥5 percent tree/shrub cover or 'tree-lined' banks; E_4 = <5 percent tree/shrub cover (Hey and Thorne, 1986); R_4 = ≥5 percent tree/shrub cover (Hey and Thorne, 1986); Values given refer to mean response confidence limits. Values in parentheses are used to calculate single response confidence limits.

Table 5.16 Constant values used to derive an unbiased bankfull width expression with confidence bands based on the best-fit power function of bankfull discharge. Coefficients pertaining to the 99, 95 and 90 percent confidence limits are given.

Natural variability equations for sand-bed and gravel-bed channels and different types of bank type have been derived from existing and new data. Equations from the Andrews (1984) data set are also given in Tables 5.16 and 5.17 as composite equations for North American gravel-bed rivers with different bank characteristics could not be derived because of limited data. With only small sample sizes, the Andrews equations should be applied with caution. New equations derived from the Hey and Thorne (1986) data are also given, as this is a popular data set that has been applied widely by both engineers and

Channel Type			Fixed Exponent		Confidence Limits				
Bed	Bank	Source	F^*	a^*	c_5	$c_{6,0.01}$	$c_{6,0.05}$	$c_{6,0.1}$	c_7
S	all	U.S.A.	1.050	4.24	1.444	0.110	0.082	0.069	1 (8.616)
S	E_1	U.S.A.	1.026	5.19	1.646	0.110	0.082	0.068	1 (6.657)
S	R_1	U.S.A.	1.022	3.31	1.200	0.115	0.085	0.070	1 (6.099)
G	all	U.S.A.	1.054	3.68	1.304	0.088	0.066	0.056	1 (10.695)
G	E_2	U.S.A.	1.011	4.12	1.416	0.121	0.087	0.071	1 (4.742)
G	R_2	U.S.A.	1.003	3.66	1.298	0.095	0.065	0.053	1 (4)
G	all	U.K.	1.033	2.99	1.095	0.073	0.055	0.046	1 (10.274)
G	E_3	U.K.	1.015	3.70	1.309	0.078	0.058	0.049	1 (7.000)
G	R_3	U.K.	1.010	2.46	0.901	0.058	0.044	0.036	1 (7.557)
G	E_4	U.K.	1.014	3.69	1.307	0.086	0.064	0.053	1 (6.385)
G	R_4	U.K.	1.010	2.45	0.895	0.068	0.051	0.042	1 (6.745)

Note: G = Gravel; S = Sand; E_1 = <50 percent tree cover; R_1 = ≥50 percent tree cover; E_2 = 'thin' vegetation (Andrews, 1984); R_2 = 'thick' vegetation (Andrews, 1984); E_3 = <5 percent tree/shrub cover or 'grass-lined' banks; R_3 = ≥5 percent tree/shrub cover or 'tree-lined' banks; E_4 = <5 percent tree/shrub cover (Hey and Thorne, 1986); R_4 = ≥5 percent tree/shrub cover (Hey and Thorne, 1986); Values given refer to mean response confidence limits. Values in parentheses are used to calculate single response confidence limits.

Table 5.17 Constant values used to derive an unbiased bankfull width expression and confidence bands based on a linear function of the square root of bankfull discharge. Coefficients pertaining to the 99, 95 and 90 percent confidence limits are given.

geomorphologists. Notably the threshold between erodible and resistant banks in sand-bed and gravel-bed rivers is not the same. Based on the distribution of data points, 50 percent tree cover delineated two distinct groups in the sand-bed data set and 5 percent tree cover delineated two distinct groups in the gravel-bed data.

Using mean bands of uncertainty for the erodible and resistant bank types, practical width equations for sand-bed and gravel-bed rivers can be derived. Within 95 percent confidence limits on the mean response, the equation derived from the U.S. sand-bed data is given by

$$W = \left(3 \cdot 38 + 1 \cdot 94 V\right) Q^{0 \cdot 5} \, \mathrm{e}^{\pm 0 \cdot 083} \tag{5.10}$$

where ‘e’ is exponential and the binary variable ‘V’ has a value of unity if tree cover over the banks is less than 50 percent and a value of zero if tree cover over the banks is at least 50 percent. Similarly, the equation derived from the U.K. gravel-bed river data is given by

$$W = \left(2 \cdot 48 + 1 \cdot 27 V\right) Q^{0 \cdot 5} \, \mathrm{e}^{\pm 0 \cdot 051} \tag{5.11}$$

where ‘e’ is exponential and the binary variable ‘V’ has a value of unity if banks are ‘grass lined’ with less than 5 percent tree/shrub cover and a value of zero if banks are ‘tree-lined’ with at least 5 percent tree/shrub cover.

In summary, downstream hydraulic geometry analysis has been used to derive a series of width-discharge relationships for different types of bed and bank characteristics and levels of statistical uncertainty. The gravel-bed river equations have been derived from existing data sets, while equations for sand-bed rivers required the collection of new regime-type data. By fixing the discharge exponent in these equations to 0.5, as recommended in the literature and overview of methods presented in Chapter 2, a comprehensive set of practical engineering equations has been developed. Estimation of width from these equations, together with the channel-forming discharge derived following guidance given in Chapter 2, provides the necessary input parameter to make the fluvial system determinate and to facilitate the analytical determination of depth and slope (and sinuosity, given the valley gradient) that are described in the next chapter.

The analysis of the new sand-bed data has revealed that the equivalence between bankfull and effective discharges, which was assumed in the procedure outlined in Chapter 3, only holds true under certain conditions. For example, in base-flow dominated rivers with infrequent, high magnitude flow events and when sand material is readily mobilised at low stage flows, the effective discharge has significantly underestimated the bankfull discharge. As a general rule in sand-bed rivers, the mean annual discharge and the bankfull discharge appear to form lower and upper bounds, respectively, to the range of effective discharge, while the 2-year flow is an upper bound to the range of bankfull discharge. Morphological relationships derived from the new data have been derived which provide further guidance for estimating the channel-forming discharge when certain conditions prevail. However, despite the advances made in this chapter, the relationship between channel-forming, effective and bankfull discharges, especially in gravel-bed rivers, remains equivocal.

CHAPTER 6

Analytical Channel Design

of Depth, Slope and Sinuosity

6.1 INTRODUCTION: BACKGROUND THEORY

Stable design of cross section width, depth and slope based on analytical equations of flow resistance and sediment transport (assuming flow continuity) is indeterminate. Without a further process-based equation that relates cross-sectional dimensions and slope, theoretically there are an infinite number of stable design solutions that could convey a steady discharge and sediment load through the system. This dilemma is demonstrated graphically in Figure 6.1, whereby any point on the stable slope-width curve (and width-depth curve, not shown) is theoretically stable for an input flow and sediment discharge. Above the stable curve, combinations of width, depth and slope would destabilise the system through erosion due to excess stream power. Conversely, combinations below the curve indicate potential sedimentation, as there is insufficient energy per unit width to transport the input sediment load. Figure 6.1 is a simplification of the fluvial system as channel dimensions are not fixed along a meander path and input flow and sediment discharge are not steady but highly variable in nature. Therefore, this one-dimensional approach assumes that a single flow can determine the stability status of the channel. Ideally, this flow should be the geomorphologically important channel-forming discharge which is responsible for shaping and sizing the bankfull configuration and sedimentary features.

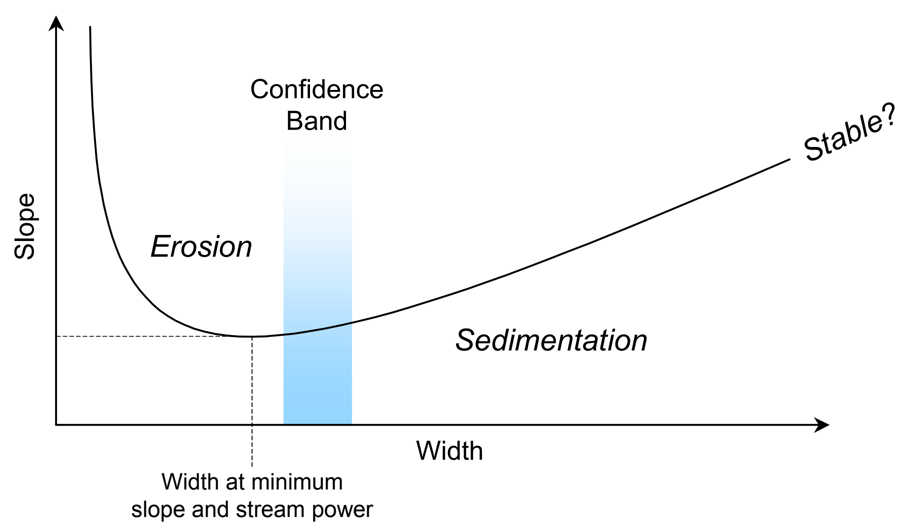


Figure 6.1 Analytical channel design of stable depth and slope

In reality, the stable slope-width curve is bounded by the valley slope as an upper limit, when the design alignment is straight, and the minimum stream power condition as a lower limit, when the design sinuosity is greatest, represented by the turning point of the curve. Furthermore, floodplain constraints and hard points in the channel bed may restrict the range of possible widths and depths. Several researchers have suggested that the minimum slope in Figure 6.1 best represents an ideal, stable channel configuration (Chapter 2), although there is no rational basis to suggest that stability is maximised at this point. Observations in actively meandering channels indicate that maximum possible sinuosity (defined by the minimum slope) promotes cut-offs and is, therefore, not a stable state on a reach scale. Furthermore, the authors have found that not all combinations of input flow and sediment discharge produce a turning point on the width-slope curve. This is often the case in small channels with gentle bank slopes and coarse beds that require a large bankfull width to transport the input sediment. Also, when a minimum slope can be defined, it is experience of the authors that width-discharge relationships based on hydraulic geometry rarely give widths that bisect the turning point. From a statistical viewpoint, this is not surprising since channel width is highly variable in nature and the probability of there being a unique solution on the curve tends towards zero with increasing precision. The shape of the rising limb of the slope-width curve is highly sensitive to the magnitude of the sediment load, such that the gradient of the curve decreases with decreasing sediment load and tends toward a small range of slope in fixed-bed channels, for which there is no minimum. In light of the above considerations, an alternative criterion for specifying analytical channel dimensions is required.

The recommended procedure is to use an appropriate width-hydraulic geometry equation to make the channel design problem determinate and to provide a range of stable depths and slopes (and therefore, sinuosities given the valley slope) defined by a width confidence band. In Figure 6.1, stable values of slope are given where the confidence band intercepts the stable slope-width curve. The output from the procedure should be used to design cross sections at meander inflexion points and the average bed slope and sinuosity through a project reach.

This Chapter presents an overview of an existing stable channel design procedure applicable to sand-bed streams developed by Copeland (1991, 1994) at the UAEWES/ERDC, Vicksburg, Mississippi, through a Flood Control Channels Research

Program (FCCRP) and the development of a complementary approach for mobile gravel-bed rivers that is a component of a computer program developed at the University of Nottingham. The dimensional equations in this chapter are all given in metric units.

6.2 SAND-BED RIVERS: COPELAND ANALYTICAL METHOD

The Copeland analytical method is a module in the SAM (Stable channel Analytical Method) hydraulic design package (Thomas et al. 1996) that provides practical assessments of stability in alluvial channels and may be used to guide end-users in the design of stable channel geometry in straight channels with sand beds. The approach is aimed at the planning and preliminary design stages of flood-control projects as a practical alternative to detailed numerical and physical modelling that are usually unfeasible in river management projects. The procedure determines the design variables of width, depth and slope that satisfy flow resistance and sediment transport equations, given input values of discharge and sediment inflow (defined from the upstream supply reach), bed material composition and bank roughness. Unlike extremal hypotheses, the end-user is presented with a suite of other suitable design solutions as well as that defined by the minimum stream power. The Copeland method involves a one-dimensional representation of a trapezoidal cross section and assumes steady, uniform flow conditions. Despite these simplifications, the method accounts for bank roughness, n_s, as well as bed roughness, n_b, in a composite flow resistance equation and is, therefore, particularly suited to small streams which are usually those targeted for restoration.

The process-based equations used in the method are those of Brownlie (1981, 1983) which are multiple regression equations on dimensionless variables based on an extensive database of sand-bed channels. The resistance equations account for both grain and form roughness due to bed features. Since the type and roughness of bed features change with the nature of the flow, it was necessary to produce two resistance equations: one for lower regime flow (ripple and dunes) and another for upper regime flow (plane bed, standing waves and antidunes), with a transitional regime when dunes give way to a mobile flat bed. These equations are expressed in the form of stage-discharge predictors with hydraulic radius as the independent variable, which Brownlie assumed to equal depth with negligible error. In the Copeland method, the Brownlie resistance equations are used to account for bed roughness only, therefore the hydraulic radius is that associated with the bed, R_b:

Lower regime $$R_b = 0\cdot 2836\, d_{50}\, q_*^{0\cdot 6539}\, S^{-0\cdot 2877}\, \sigma^{0\cdot 0813} \tag{6.1}$$

Upper regime $$R_b = 0\cdot 3742\, d_{50}\, q_*^{0\cdot 6248}\, S^{-0\cdot 2542}\, \sigma^{0\cdot 1050} \tag{6.2}$$

where 'd_{50}' is the median grain size of the bed material, 'S' is the water surface slope (equals bed slope), 'σ' is the geometric standard deviation of bed particle sizes and 'q_*' is a dimensionless unit discharge, given by

$$q_* = \frac{Q\,D}{A\left(g\, d_{50}^{3}\right)^{0\cdot 5}} \tag{6.3}$$

where 'g' is acceleration due to gravity, 'Q' is the input design discharge, 'D' is the depth in the central strip of a trapezoidal cross section and 'A' is the cross-sectional area. The σ parameter is given by

$$\sigma = \left(\frac{d_{84\cdot 1}}{d_{15\cdot 9}}\right)^{0\cdot 5} \tag{6.4}$$

where '$d_{15.9}$' and '$d_{84.1}$' are sizes of bed particles for which 15.9 percent and 84.1 percent are finer in the cumulative distribution.

Rearranging Equations 6.1 and 6.2 to make discharge the independent variable gives the following flow resistance equations

Lower regime $$Q = \frac{A\left(g d_{50}^{3}\right)^{0\cdot 5}}{D}\left(\frac{R_b\, S^{0\cdot 2542}}{0\cdot 372\, d_{50}\, \sigma^{0\cdot 1050}}\right)^{1/0\cdot 6539} \tag{6.5}$$

Upper regime $$Q = \frac{A\left(g d_{50}^{3}\right)^{0\cdot 5}}{D}\left(\frac{R_b\, S^{0\cdot 2877}}{0\cdot 2836\, d_{50}\, \sigma^{0\cdot 0813}}\right)^{1/0\cdot 6248} \tag{6.6}$$

Brownlie defined the type of flow regime by comparing the grain Froude number, F_g, with a threshold grain Froude number, F_g'. These parameters are defined by

$$F_{\mathrm{g}} = \frac{Q}{A\left[\mathrm{g}\left(G_{\mathrm{s}}-1\right)d_{50}\right]^{0\cdot 5}} \tag{6.7}$$

where 'G_s' is the specific gravity of bed material, and

$$F_{\mathrm{g}}^{'} = \frac{1\cdot 74}{S^{1/3}} \tag{6.8}$$

According to Brownlie, lower regime flow occurs if F_g is less than $0.8F_g$' and upper regime flow occurs when F_g is greater than $1.25F_g$' or if the slope is greater than 0.006. When the grain Froude number is within these limits, the flow is in the transitional regime. In the Copeland method, the threshold between lower and upper regimes is distinguished by F_g equalling F_g'.

The hydraulic radius associated with the side slopes, R_s, is calculated using the Manning equation with user-defined values of the bank roughness coefficient, n_s, such that

$$R_{\mathrm{s}} = \left(\frac{Q\,n_{\mathrm{s}}}{A\,S^{0\cdot 5}}\right)^{1\cdot 5} \tag{6.9}$$

The roughness associated with the bed, n_b (accounted for in Brownlie's equation), and banks, n_s (user-defined), are composited into a single roughness value using the 'equal velocity' method proposed by Horton (1933), Einstein (1942, 1950) and Einstein and Banks (1950). This method attempts to compensate for the negligible influence of bank roughness when side slopes are steep, and discharge is calculated conventionally as the sum of discharges in vertical panels across the channel. This is demonstrated using a hypothetical example in the SAM User's Manual (Thomas et al., 1996). Using the equal velocity method, the water area is divided up imaginatively into two sub-areas, one associated with the bed wetted perimeter, P_b, with roughness n_b, and one associated the wetted perimeter of the side slopes, P_s, with roughness n_s, such that each sub-area has the same mean velocity as the whole cross section. A composite Manning n value is then defined by

$$n = \left(\frac{P_{\mathrm{b}}\,n_{\mathrm{b}}^{1\cdot 5} + P_{\mathrm{s}}\,n_{\mathrm{s}}^{1\cdot 5}}{P}\right)^{2/3} \tag{6.10}$$

From Equation 6.10 it can be shown that hydraulic parameters are partitioned according to the expression (Einstein, 1950)

$$A = R_{\mathrm{b}} P_{\mathrm{b}} + R_{\mathrm{s}} P_{\mathrm{s}} \tag{6.11}$$

From the geometry of a trapezoidal cross section, Equation 6.11 can be rearranged to give an expression for R_{b} as a function of bank roughness, as given by

$$R_b = \frac{A - \mathrm{a_R}\, D A^{-1\cdot 5} S^{-0\cdot 75}}{B} \tag{6.12}$$

where 'B' is the bed width of the trapezoid and the resistance constant $\mathrm{a_R}$ is given by

$$\mathrm{a_R} = 2\left(Q n_{\mathrm{s}}\right)^{1\cdot 5}\left(1 + Z^2\right)^{0\cdot 5} \tag{6.13}$$

where 'Z' is the side slope (1 vertical : Z horizontal). The relationship between water elevation and discharge in a trapezoidal channel is then given by substituting Equation 6.12 into either Equation 6.5 or 6.6.

The Brownlie sediment transport equation is based on the same data set used to derive the resistance equations and is expressed as concentration of the total load (bed load and suspended load), C, in parts per million (by mass). Like the resistance equations, the sediment transport relationship can be used with any consistent set of units and is given as a function of R_{b} (rather than R) in the Copeland method by

$$C = 9021\cdot 82\left(F_{\mathrm{g}} - F_{\mathrm{g_o}}\right)^{1\cdot 978} S^{0\cdot 6601}\left(\frac{R_{\mathrm{b}}}{d_{50}}\right)^{-0\cdot 3301} \tag{6.14}$$

where 'F_{g}' is the grain Froude number (Equation 6.7) and '$F_{\mathrm{g_o}}$' is the critical grain Froude number for incipient particle motion given by

$$F_{g_o} = 4\cdot 596\, \tau_{*\mathrm{c}}^{0\cdot 5293} S^{-0\cdot 1405} \sigma^{0\cdot 1606} \tag{6.15}$$

where 'τ_{*_c}' is the critical Shields parameter for incipient particle motion, which was given by Brownlie as a new regression relationship, such that

$$\tau_{*c} = 0 \cdot 22\,Y + 0 \cdot 06\left(10\right)^{-7 \cdot 7\,Y} \tag{6.16}$$

where 'Y' is a function of the grain Reynolds number, R_{g}, and is given by

$$Y = \left[R_{\mathrm{g}}\left(G_{\mathrm{s}} - 1\right)^{0 \cdot 5}\right]^{-0 \cdot 6} \tag{6.17}$$

$$R_{\mathrm{g}} = \frac{\left(\mathrm{g}\,d_{50}^{3}\right)^{0 \cdot 5}}{\nu} \tag{6.18}$$

where 'ν' is the kinematic viscosity of water.

In the analytical method bed material is assumed to move over the bed portion of the channel only and not over the channel sides. This requires a scaled reduction in sediment calculated concentration to give an average concentration, C_{m} (parts per million by mass) for the whole channel. A suitable adjustment is accomplished by

$$C_{\mathrm{m}} = \frac{CBD}{A} \tag{6.19}$$

Furthermore, sediment discharge is usually expressed in terms of mass per unit time (kilograms per second, tonnes per day, etc). By modifying Equation 6.19 a more practical expression is given by

$$Q_{\mathrm{s}} = \frac{\rho\, C_{\mathrm{m}}\, Q}{10^{6}}\,(\mathrm{kg\ s^{-1}}) \tag{6.20}$$

where 'ρ' is the density of water (1000 kg $\mathrm{m^{-3}}$ when Q has $\mathrm{m^3 s^{-1}}$ units). Q_{s} can be expressed in metric tonnes per day when Equation 6.20 is multiplied by 86.4. In the Copeland method, a modified version of Equation 6.20 is used with variables measured in imperial units.

The analytical method solves Equations 6.1 or 6.2 and 6.20 simultaneously given input values of the channel forming discharge, Q, sediment discharge conveyed by the channel forming discharge, Q_s, (both defined from the upstream supply reach) with user-defined values of bank roughness, n_s, side slopes, Z (based on geotechnical surveys), and sediment characteristics, d_{50} and ρ. The numerical procedure uses Newton-Raphson iteration in two dimensions to derived approximations for slope, S, and depth, D, for specified values of the channel bed width, B. The program in SAM derives a suite of 20 slope-width solutions for a range of widths with an increment of $0.1B$ on either side of a regime value prescribed by the expression

$$\text{metric: } B = 3\cdot 6\,Q^{0\cdot 5} \quad \text{imperial: } B = 2\cdot 0\,Q^{0\cdot 5} \tag{6.21}$$

A stability curve similar to Figure 6.1 is then plotted from these width-slope values and the solution at the minimum stream power is also obtained in the model. Comprehensive details of the actual programming are not given here. The Copeland method has been shown to be applicable to both high-energy ephemeral streams and low-energy meandering streams in the U.S.A. (Copeland, 1991, 1994).

The method is applicable to the range of conditions in the database compiled by Brownlie, as given in Table 6.1.

Variable	Range
Slope, S	0.000003 to 0.037
Discharge, Q	0.003 to 19992 $m^3\ s^{-1}$ 0.11 to 706000 $ft^3\ s^{-1}$
Median particle size of bed material, d_{50}	0.088 to 2.8 mm
Hydraulic radius, R	0.025 to 17.07 m 0.082 to 56 ft
Temperature	0 to 63 °C
Width-to-depth ratio, W / D	>4
Geometric standard deviation of bed particle sizes, σ	≤5

Table 6.1 Ranges of channel variables used in the Brownlie (1981, 1983) flow resistance and sediment transport equations and ranges of application of the Copeland analytical method for stable channel design in sand-bed rivers.

The Copeland analytical method represents a significant improvement on existing one-dimensional approaches to stable channel design. The method is still under development and current research is centred on developing improved guidance on stable width based on hydraulic geometry with uncertainty in estimates, as portrayed in Figure 6.1. The program has been reworked for this project. Several modifications have been made and these are summarised briefly here.

The original computer code required input of design variables in imperial units. In the enhanced program, all variables and equations are expressed in metric units. Also, the stability curves in the original method relate to bed width (input variable). While the bed width is a geomorphologically important parameter in terms of bed load transport, the uncertainty bands for width, derived from hydraulic geometry analyses in Chapter 5, are given for bankfull width and not bed width, which may be significantly different in small streams with gentle bank slopes. Therefore, it is sensible to derive *bankfull* width-slope stability curves for a given design (channel-forming) discharge. In the enhanced program, this required finding the partial derivative of Equations 6.5, 6.6 and 6.20 with respect to bankfull width in the iteration procedure.

In many stable channels, the bank-lines exhibit different characteristics, in terms of stability and vegetation cover, which control side slope angles and contribute to roughness. In the enhanced procedure provision has been for the end-user to specify different values of side slope angles and Manning coefficient for the left and right banks if required. This required a revised expression for the hydraulic radius associated with the bed, (as a function of *bankfull* width and depth) derived by modifying Equations 6.12 and 6.13 to give

$$R_b = \frac{A - \mathrm{a_R}\, D A^{-1\cdot 5} S^{-0\cdot 75}}{W - D\left(Z_\mathrm{l} + Z_\mathrm{r}\right)} \tag{6.22}$$

where the modified resistance constant a_R is given by

$$\mathrm{a_R} = Q^{1\cdot 5}\left[\left(1 + Z_\mathrm{l}^2\right)^{0\cdot 5} n_\mathrm{l}^{1\cdot 5} + \left(1 + Z_\mathrm{r}^2\right)^{0\cdot 5} n_\mathrm{r}^{1\cdot 5}\right] \tag{6.23}$$

where 'Z_l' and 'n_l' are the side slope and roughness of the left bank, respectively, and 'Z_r' and 'n_r' are the side slope and roughness of the right bank, respectively.

One of the main problems of one-dimensional sediment transport approaches is defining appropriate hydraulic parameters from complex channel geometries to use in the equations. In particular, there is much uncertainty as to appropriate definitions of the hydraulic radius associated with sediment transport. In the Copeland method, the hydraulic radius associated with the bed, R_b, (used in the flow resistance equations) is used to calculate sediment discharge. However, sediment transport is driven by excess boundary shear stress, which is controlled by the depth of the entire flow rather than that associated with bed roughness alone. Three alternative assumptions can be used to specify suitable values of depth, width and hydraulic radius associated with sediment transport:

i) Sediment transport occurs over the bed portion of the channel only and the hydraulic radius has a value equal to the depth over the bed, with negligible error.

ii) Sediment transport occurs over the bed portion of the channel only and the hydraulic radius is calculated as the ratio of the channel area above the bed to the wetted perimeter of this sub-area, defined as the sum of the bed width and twice the depth. Approaches i and ii are suitable in large rivers or in narrow streams with very steep side slopes.

iii) Sediment transport occurs over both the bed and banks of the channel and is related to an effective width, W_e and an effective depth, D_e. Flume experiments at Colorado State University (Gessler et al., 1994) on sand-bed channels with side slopes, Z, between 1 and 3, revealed that up to 10 percent of sediment transport occurred over the channel banks in 95 percent of the test runs. As expected, this percentage decreases with increasing width-to-depth ratio but in narrow streams, with width-to-depth ratios less than 10, transport over the side slopes may be as much as 20 to 50 percent of the total load. Clearly, in channels with irregular geometries sediment concentration may vary significantly across the section. However, most sediment transport equations, including those of Brownlie, were developed from wide channels with relatively uniform depth. Composite parameters of effective width and effective depth were developed during research at UAEWES/ERDC and define an equivalent rectangular cross section that transports the same quantity of sediment as the irregular cross section. The parameters are given by Thomas et al. (1996) as

$$D_{\mathrm{e}} = \frac{\sum_{\mathrm{i}=1}^{n} A_{\mathrm{i}} D_{\mathrm{i}}^{5/3}}{\sum_{\mathrm{i}=1}^{n} A_{\mathrm{i}} D_{\mathrm{i}}^{2/3}} \tag{6.24}$$

$$W_{\mathrm{e}} = \frac{\sum_{\mathrm{i}=1}^{n} A_{\mathrm{i}} D_{\mathrm{i}}^{2/3}}{D_{\mathrm{e}}^{5/3}} \tag{6.25}$$

where ‘i’ corresponds to vertical panels above the cross section between surveyed elevations, ‘n’ is the total number of vertical panels and ‘D_{i}’ and ‘A_{i}’ are the average depth and area, respectively, of panel i. In a trapezoidal cross section, there are three panels, one rectangle above the bed and one triangle above each side slope, and the effective width and effective depth are defined by

$$D_{\mathrm{e}} = \frac{D}{2}\left(\frac{2^{8/3} B + \mathrm{a}_{\mathrm{R}}\ D}{2^{8/3} B + \mathrm{a}_{\mathrm{R}}\ D}\right) \tag{6.26}$$

$$W_{\mathrm{e}} = \frac{\left(2^{5/3} B + \mathrm{a}_{\mathrm{R}}\ D\right)^{8/3}}{\left(2^{8/3} B + \mathrm{a}_{\mathrm{R}}\ D\right)^{5/3}} \tag{6.27}$$

where ‘D’ is the depth above the central strip of the cross section and ‘B’ is the bed width, which is defined geometrically by $W - D\,(Z_{\mathrm{l}}+Z_{\mathrm{r}})$.

As the hydraulic design package, SAM, already incorporates the effective parameters in other modules which use sediment transport calculations, it is recommended that they should be used as the default option in the analytical channel design method, especially when modelling small sand-bed channels with appreciable sediment loads.

These alternatives have been coded into the working model at the University of Nottingham. This model also includes a complementary method for the design of stable dimensions in gravel bed rivers with mobile beds, which is a significant advance on the original model and gives the approach wider application potential. An overview of the analytical method for gravel-bed rivers is given in the next section.

6.3 GRAVEL-BED RIVERS

The general framework for the method that applies to gravel-bed rivers is the same as that for sand-bed rivers. The essential difference is in the governing equations of flow resistance and sediment transport that are appropriate for channels with predominantly gravel bed material. The method assumes that suspended load is a negligible component of the total load and can be ignored in sediment transport considerations.

In a mobile gravel-bed river, flow resistance due to form drag is not very pronounced because bed features are much more rounded in form (Raudkivi, 1990, p. 126). In general, grain roughness (or skin friction) controls flow resistance above a straight gravel bed, although when roughness elements are sufficiently large (large cobbles or boulders) energy losses resulting from accelerations and decelerations within the flow column (spill resistance) may be significant (Bathurst, 1978, 1982, 1985, 1997). In a gravel- bed river, Hey (1979, p. 366) considered spill resistance to be only of local significance and concluded that it can be neglected without appreciable error. There is a plethora of flow resistance equations available for gravel-bed rivers, although most are variant forms of the theoretical uniform flow equation for the mean velocity in open channels with rough boundaries and turbulent flow developed by Keulegan (1938), given in natural logarithm form as

$$V = 2 \cdot 50\, V_* \ln\left(\frac{\mathrm{a}R}{k}\right) \tag{6.28}$$

where 'V_*' is the shear velocity, $(\mathrm{g}RS)^{0.5}$, 'a' is a coefficient and 'k' is an equivalent sand roughness height. From a study of available data the coefficient, a, was found by Keulegan to span a wide range of values, with a mean value of 12.22. Hey (1979) remarked that the value of k reflects the type and strength of flow resistance, which varies between sand-bed and gravel-bed rivers, while the coefficient, a, is a function of cross-sectional shape. For different channel cross-sectional shapes, obtained from Keulegan's original analysis, Hey (1979) showed that the coefficient, a, is expected to range between 11.1, for infinitely wide channels, to 13.46, for semi-circular channels. Using the data presented by Hey, best-fit expressions for the coefficient as a function of the width-to-depth ratio, W/D_m, for rectangular and trapezoidal channels can be derived:

Rectangular channels:

$$W/D_m \leq 20 \qquad a = 2\cdot 83\, e^{-0\cdot 17(W/D_m)} + 11\cdot 45 \tag{6.29}$$

$$W/D_m > 20 \qquad a = 11\cdot 33 \approx 11\cdot 3 \tag{6.30}$$

Trapezoidal channels (60° side slopes)

$$W/D_m \leq 20 \qquad a = 4\cdot 07\, e^{-0\cdot 22(W/D_m)} + 11\cdot 40 \tag{6.31}$$

$$W/D_m > 20 \qquad a = 11\cdot 37 \approx 11\cdot 3 \tag{6.32}$$

Equations 6.29 and 6.31 have R^2 values of 0.9998 and 0.9934, respectively. The coefficient values in Equations 6.30 and 6.32 correspond to the mean W/D_m ratio for ratios above 20.

Representative values of k for gravel-bed rivers have been derived from several flow resistance equations and are given in Table 6.2. The equation given by Hey (1979) has a coefficient, a, value of 11.75 (when k is $3.5d_{84}$), which corresponds to a W/D_m ratio of 13.2 in a rectangular channel (from Equation 6.29) and 11.2 in as trapezoidal channel with 60° side slopes (Equation 6.31). The Hey equation yields approximately average velocity estimates between the equations of Leopold *et al*. (1964) and Limerinos (1970). According to Raudkivi (1990, p. 127), the Hey equation produces reasonable values when D_m/d_{84} is greater than about six. The expression derived by Bathurst (1985) is suitable for mountain streams with slopes greater than 0.02. In his analysis, the Hey equation tended to underestimate resistance, due to the additional form and spill resistance in upland rivers. Rearranging the original equation to the form of Equation 6.28 and replacing flow depth, D_m, by the hydraulic radius, R, yields an equivalent roughness k value approximately 50 percent greater than in the Hey equation.

Reference	K
Leopold, Wolman and Miller (1964)	a $d_{84}/3{\cdot}11$
Limerinos (1970)	a $d_{84}/3{\cdot}74$
Hey (1979)	a $d_{84}/3{\cdot}36$
Bathurst (1985)	a $d_{84}/4{\cdot}96$

Table 6.2 Values of the roughness height, *k*, in the general Keulegan (1938) relationship, derived from various flow resistance equations for gravel-bed rivers

The Keulegan equation is used as a template in the analytical method and the end-user can specify which *k* value to use according to preference of flow resistance equation given in Table 6.2. In general, the Hey equation is recommended for U.K. gravel-bed rivers and the Limerinos equation is recommended for North American gravel-bed rivers.

Meyer-Peter and Müller (1948) developed a dimensionless bed load discharge equation for gravel-bed rivers based on excess shear stress considerations. The equation was derived from a data set covering a wide range of conditions, with slopes between 0.004 and 0.2, mean bed particle sizes between 0.4 and 30 mm and depths up to 1.2 m. The equation has been used extensively in Europe (Simons and Senturk, 1977, p. 516). Sediment discharge in the original equation was expressed in units of submerged weight per unit time per unit channel width and has been rearranged here to make sediment discharge the independent variable in units of kilograms per second of dry sediment load.

$$Q_s = \frac{8\rho g^{0\cdot5} W_s}{G_s - 1}\left[K^{1\cdot5} R_s S - 0\cdot047\left(G_s - 1\right)d_{50}\right]^{1\cdot5} \text{ (kg s}^{-1}\text{)} \qquad (6.33)$$

where 'G_s' is the specific gravity of bed material (approximates 2.65 for quartz), 'ρ' is the density of water (1000 kg m^{-3}), 'g' is acceleration due to gravity (9.81 m s^{-2}), 'd_{50}' is the median size of bed particles (m), which is assumed to equal the mean size used in the original equation and 'S' is the water surface slope. The parameters, 'W_s' and 'R_s' are the width and hydraulic radius associated with sediment transport, where W_s may be equated with the bed width or effective width depending on the preference of the end-user. The dimensionless parameter 'K' is a roughness factor given by

$$K = \frac{d_{90}^{1/6}\, Q}{26\, A\, S^{0\cdot5}\, R_s^{2/3}} \tag{6.34}$$

where 'Q' is the discharge, 'A' is the cross section area and 'd_{90}' is the size of bed particle for which 90 percent are finer in the cumulative distribution. In general, K varies between 0.5 and 1.0 and decreases as form roughness increases. In gravel-bed rivers, K has an upper value in this range. Notably, if K has a value of 1.0 and bed load is zero, Equation 6.33 reduces to the Shields stress equation with a Shields parameter of 0.047, which is often cited as appropriate criterion for incipient particle motion in gravel-bed rivers.

The analytical method for gravel-bed rivers is based on the simultaneous solution of Equations 6.28 and 6.33. The method is only applicable to rivers with mobile beds. The approach is in development and recent improvements include the incorporation of other sediment transport functions into the method, thereby giving the end-user a choice of design equations depending on preference and an alternative to the Meyer-Peter and Müller (1948) equation when suspended load is significant in fine gravel-bed rivers.

6.4 LIMITATIONS FOR CHANNEL RESTORATION DESIGN

The analytical methods described in Sections 6.2 and 6.3 require a simplification of the fluvial system to provide practical solutions. Channel geometry is represented by very regular cross sections and flow is assumed to be steady and uniform. Furthermore, stable channel dimensions are derived for a single flow event. In natural meandering rivers, cross sections, even at inflexion points, may be very irregular and shaped by the whole range of flows experienced by the river rather than a single event. On the basis of these considerations, the channel dimensions output from the approach should be treated as an approximate guide for the project reach. Furthermore, as the output stable design is a function of the channel-forming discharge only, the likely instability due to other flow events in the long tem record should be examined through a sediment impact assessment, and modifications made accordingly (Chapter 3). The quality of the output data from the one-dimensional approach should be sufficient when comparing various sites in terms of their restoration potential and in some cases should provide the necessary information to warrant more detailed modelling.

The output dimensions from the analytical methods are a function of the flow resistance and sediment transport equations used in their derivation. Sediment transport equations, in particular, are associated with a high degree of uncertainty as most theoretical treatments are based on some idealised and simplified assumptions and are supported by limited laboratory data and, in only a few cases, field data. Calculated results from the various published equations differ considerably and rarely show close agreement with measured data. This was demonstrated by Yang (1996) from a comprehensive review of sediment transport functions. In channel restoration design, miscalculation of the true input sediment load can lead to instability. Therefore, in the absence of measured sediment data, it is recommended that the same sediment transport equation be used both to determine the supply load from upstream and in the analytical design method, thereby cancelling any systematic errors produced in its application.

In terms of flow resistance, the equal velocity method for composite channel roughness tends to underestimate discharge during high stage flows (Thomas et al., 1996, p. 2d-7), particularly in a cross section with steep lower banks (usually below the lower limit of perennial vegetation) and very gradual upper banks. Further research is required to examine this problem and identify alternative methods to composite roughness in these cases. One potential technique, although subjective, would be to define a notable break of slope on each bank profile and apply the equal velocity method only to the portion of the cross section between these bank locations and calculate discharge over the upper banks using the conventional approach of dividing the flow area into independent panels.

Often, it is desired to restore the natural amenities of a river, for example, fisheries, recreation or conservation, while also preserving flood defence and land drainage functions. Under these circumstances, a widely used approach is to design a channel with a compound, or multi-stage cross-section. The smaller, inner channel then has the attributes of a natural, regime channel, while the larger channel that surrounds it is able to convey flood flows without causing property damage on the surrounding floodplain. This type of configuration is difficult to design analytically because of the significant shear stresses at the interface between over-bank flow and main channel flow due to lateral momentum exchange. Failure to account for this momentum exchange, and the resulting energy losses, in discharge calculations may result in considerable overestimation of the channel capacity for a given depth of flow. However, there is a paucity of knowledge concerning the physics of fluid flow in compound channels, the mechanics of sediment transport, and the relationships between flow regime, channel geometry and channel stability of the inner channel and the floodway. To account for these energy losses using an empirical approach, current research by the authors and at UAEWES/ERDC aims to develop a roughness multiplier applied to the main channel composite Manning coefficient and to assist in establishing a range of uncertainty determinations. These advances should facilitate in the restoration of more natural cross sections.

In summary, this chapter has presented a short overview of the analytical component of the channel design framework. Using estimates of discharge and width, following guidance from the preceding chapters, estimates of depth and slope (and sinuosity, given valley gradient) can be determined though the simultaneous solution of flow resistance and sediment transport equations for sand-bed or gravel-bed rivers. The components of the procedure described in Chapters 4 to 6 relate to the specification of stable channel slope and reach-average cross section dimensions. The output estimate of sinuosity, together with bankfull width, can then be input to determine meander planform geometry, as described in Chapter 7. The objective of the following chapter is to provide the necessary guidance to allow the end-user to complete the Channel Design stage of the design framework presented in Chapter 3. This is achieved through the development of simple design equations to specify reach average meander planform and a series of morphological relationships for introducing local morphological variability around meander bends, as found in natural channels.

CHAPTER 7

Planform Geometry
and Morphological Variability

7.1 INTRODUCTION

Restoring meander planform geometry using the historical approach (Chapter 2), based on analysing meander traces either from historical maps or floodplain meander scars, assumes stationarity in drainage basin controls, whereby the nature of the flow and sediment regimes that moulded the historical channels are considered representative of flow and sediment inputs to the restored channel. This is likely to be unrealistic if there have been significant changes in catchment land-use, such as progressive urbanisation. Examining meander traces from reference reaches with stable geometries, preferably in the same river system, is a more suitable approach if channel conditions are similar to that in the restored reach. Where possible, a wavelength-width relationship should be derived if a number of stable sites have been identified within the catchment. However, in most disturbed catchments where river restoration is a suitable management strategy, identifying reference reaches with natural meandering planforms to use as design templates is problematic and an alternative method is required.

The advantage of calculating stable channel slope analytically on the basis of sediment transport, before predicting the meander planform, is that sinuosity is given as the ratio of measured valley slope to channel slope and is, therefore, directly related to the sediment regime which controls channel stability. Once sinuosity has been derived, only wavelength is required to solve the sine-generated curve equation and define the target reach-average planform. Improved guidance for specifying planform geometry using practical techniques is a prerequisite to further the trend towards restoring meandering rivers and 're-naturalising' the riverine landscape. For example, in Denmark, where environmental legislation has allowed rivers to be restored on a wide scale, only 0.1 percent of all straightened rivers have been re-meandered (Madsen, 1995).

This chapter is divided into the three essential stages necessary to lay out the basic planimetric geometry of a restored channel according to the procedural framework given in Chapter 3 (Figure 3.6): i) determination of reach-average meander wavelength from channel width with uncertainty in estimates; ii) reach-average planform layout according to the sine-generated curve, and; iii) determination of local morphological variability around meander bendways with uncertainty in estimates. Through this sequence there is an increase in the level of design detail. In practice, there is also a stage iv, which should be left to the river, to embroider the intricate forms and features that cannot be engineered nor constructed into the channel mould.

7.2 MEANDER WAVELENGTH

Although several relationships expressing meander wavelength, L_m, as a power function of bankfull width, W, have been documented in the literature (Chapter 2) and exhibit marked similarities in terms of best-fit regression equations, individual scattergrams of width and wavelength observations portray only limited natural variability. This is the result of a combination of small sample sizes and the regional nature of the individual data sets, thereby limiting the range of environmental controls found in nature that are represented in the data set and cause observed wavelengths to deviate from predicted values. To examine the natural variability in the relationship between width and wavelength and give data more general applicability, individual data sets should be combined. A composite relationship has been derived from nine available data sets, described below, consisting of 438 sites (Figures 7.1 and 7.2). Only sites with sinuosities of at least 1.2 and bankfull widths between 1 m and 1000 m were selected. Within these constraints, meander wavelengths range from 10.4 m to 19,368 m and sinuosities range from 1.2 to 5.3:

i) Leopold and Wolman (1957) data from U.S. rivers (21 sites).
ii) Leopold and Wolman (1960) data compiled from various sources and including rivers in France (1 site), U.S.A. (34 sites) and one model river (total of 36 sites).
iii) Carlston (1965) data from U.S. rivers (29 sites).
iv) Schumm (1968) data from midwestern U.S. rivers (25 sites).
v) Chitale (1970) data from large alluvial rivers in Africa (1 site), Canada (1 site), India (16 sites), Pakistan (2 sites) and U.S.A. (1 site) (total of 21 sites).
vi) Williams (1986) data compiled from various sources and including rivers in Australia (2 sites), Canada (7 sites), Sweden (17 sites), Russia (1 site), U.S.A. (16 sites) and one model river (total of 43 sites).

vii) Thorne and Abt (1993) data from various sources including measurements from the Red River 1966 (35 sites) and 1981 (39 sites) hydrographic surveys between Index, Arkansas, and Shreveport, Louisiana, and rivers in India (12 sites), Netherlands (1 site), U.K. (48 sites) and U.S.A. (18 sites) (total of 153 sites).

viii) Annable (1996) data from streams in Alberta, Canada (30 sites)

ix) Cherry et al. (1996) data from U.S. rivers with predominantly sand beds (79 sites)

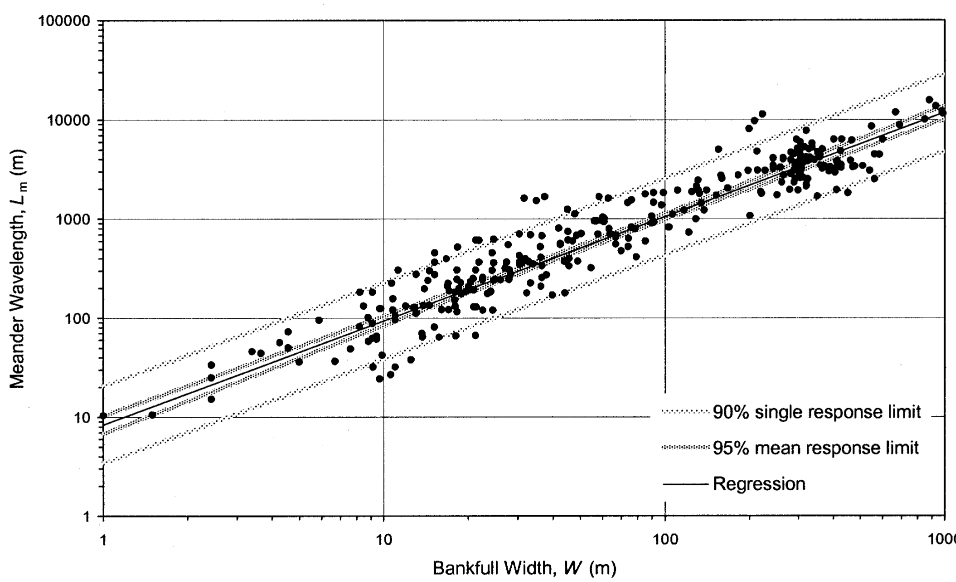


Figure 7.1 Confidence intervals applied to the relationship between meander wavelength and bankfull width, $L_{\mathrm{m}} = 8{\cdot}36 W^{1{\cdot}05}$, based on a composite data set of 438 sites.

Figures 7.1 and 7.2 show that the 95 percent mean response confidence limits are very narrow due to the large sample size, whereas the 90 percent single response limits provide almost an order of magnitude in wavelength for a given width. The single response limits are strongly influenced by outlying data points which might reflect environmental controls absent from the rest of the data set that result in deviations from the best-fit relationship. The general form of the meander wavelength equation, corrected for bias (Chapter 3) is given by

$$L_{\mathrm{m}} = F \mathrm{a} W^{\mathrm{b}} \tag{7.1}$$

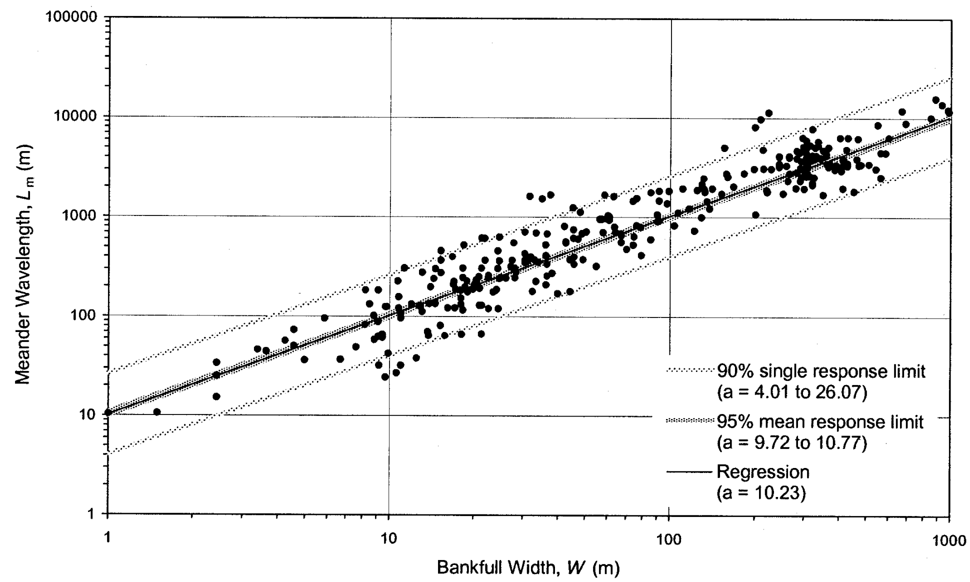


Figure 7.2 Confidence intervals applied to the relationship between meander wavelength and bankfull width with width exponent fixed at 1.0, $L_{\mathrm{m}} = 10 \cdot 23W$, based on a composite data set of 438 sites.

where 'a' and 'b' are defined by ordinary least squares regression of natural logarithmic transformed variables and 'F' is a correction factor to account for bias as a result of the transformation. Using modified versions of the confidence interval equations given in Chapter 3, meander wavelength within 100(1-p) percent confidence limits, $L_{\mathrm{m},\,p}$, is given by

$$L_{\mathrm{m},\,p} = F\mathrm{e}^{k} \tag{7.2}$$

$$k = \ln\left(\mathrm{a}W^{\mathrm{b}}\right) \pm \mathrm{c}_{1,\,p}\sqrt{\frac{1}{\mathrm{c}_2} + \frac{\left[\ln\left(W\right) - \mathrm{c}_3\right]^2}{\mathrm{c}_4}} \tag{7.3}$$

where values of $\mathrm{c}_{1,p}$, c_2, c_3 and c_4 define natural wavelength variability, at 'p' probability (Table 7.1).

The General Linear Hypothesis was used to test the null hypothesis that the best-fit power function relationship is not significantly different from a linear relationship, as suggested by research including Leopold and Wolman (1960), Dury (1976), and Richards (1982). This was achieved by comparing the width exponent with a theoretical exponent of unity.

Because of the large data set, the results showed that the null hypothesis could not be rejected at less than the 99.1 percent significance level, thereby confirming that a linear relationship is statistically different from the best-fit relationship in this case. However, for practical engineering purposes deviations from the best-fit power function appear to be very similar to those produced by the linear function (Figure 7.3).

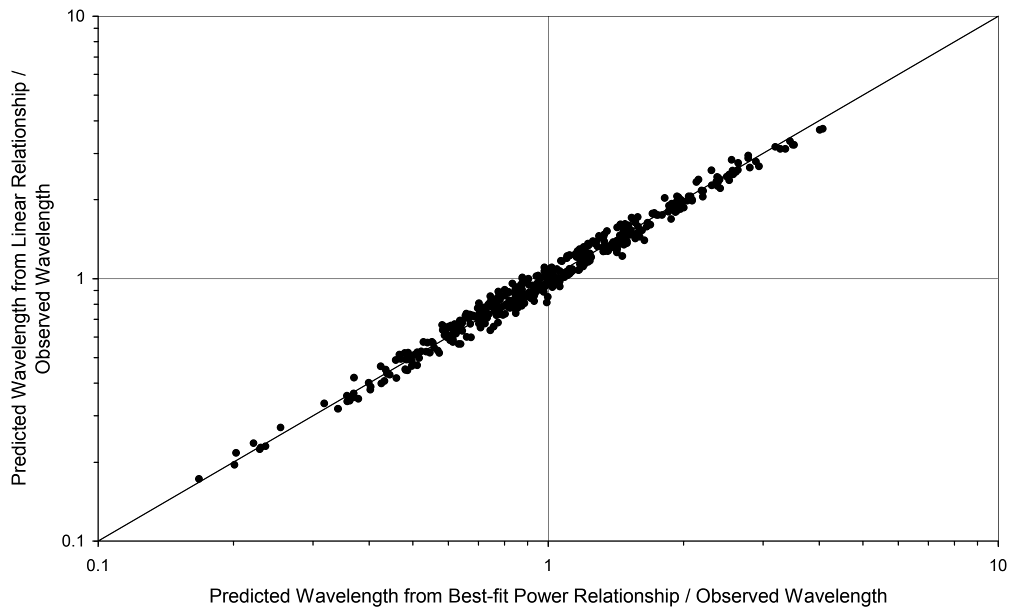


Figure 7.3 Accuracy of the best-fit relationship, expressing meander wavelength as a power function of bankfull width, compared with that of the fixed exponent model, expressing meander wavelength as a linear function of bankfull width.

A linear wavelength-width relationship was derived from logarithmic transformed data. This transformation was necessary to normalise the error variance relative to the mean response. An expression for meander wavelength based on the fixed exponent model (width exponent=1), corrected for bias, is given by

$$L_{\mathrm{m}} = F^{*}\mathrm{a}^{*}W \tag{7.4}$$

where a^{*} is the coefficient in the fixed exponent model and equals the exponential of the mean of the natural logarithm of $L_{\mathrm{m}}/(W^{1})$ and F^{*} is a correction factor to account for bias as a result of the logarithmic transformation. Using modified versions of the confidence interval equations given in Chapter 3, meander wavelength within 100(1-p) percent confidence limits, $L_{\mathrm{m},\,p}$, with width exponent fixed at 1, is given by:

$$L_{\mathrm{m},p} = F^{*}\,\mathrm{e}^{k}\,W \tag{7.5}$$

$$k = \mathrm{c}_5 \pm \mathrm{c}_{6,p}\,\mathrm{c}_7 \tag{7.6}$$

where values of c_5, $c_{6,p}$ and c_7 define natural width variability at 'p' probability (Table 7.2).

Best-Fit Equation			Confidence Limits					
F	a	b	$c_{1,0.01}$	$c_{1,0.05}$	$c_{1,0.1}$	c_2	c_3	c_4
1.156	8.36	1.05	1.392	1.058	0.887	438 (0.998)	4.084	802.315

Note: Values given refer to mean response confidence limits. Value in parentheses is used to calculate single response confidence limits.

Table 7.1 Constant values used to derive an unbiased wavelength expression with confidence bands based on the best-fit power function of bankfull width. Coefficients pertaining to the 99, 95 and 90 percent confidence limits are given.

Fixed Exponent		Confidence Limits				
F^{*}	a^{*}	c_5	$c_{6,0.01}$	$c_{6,0.05}$	$c_{6,0.1}$	c_7
1.158	10.23	2.326	0.067	0.051	0.043	1 (21.928)

Note: Values given refer to mean response confidence limits. Value in parentheses is used to calculate single response confidence limits.

Table 7.2 Constant values used to derive an unbiased wavelength expression and confidence bands based on a linear function of bankfull width. Coefficients pertaining to the 99, 95 and 90 percent confidence limits are given.

The single response confidence band incorporates a spectrum of meandering from near-straight channels at the lower limit to tortuous meanders at the upper limit. Based on Table 7.2, an unbiased morphological expression for meander wavelength within 95 percent confidence limits on the mean response suitable for engineering design is given by

$$L_{\mathrm{m}} = (11\cdot 26 \text{ to } 12\cdot 47)W \tag{7.7}$$

According to Hey (1976) and Thorne (1997), twice the distance between successive riffles (or pools) in a straight channel equals 4π (12.57). This is based on the assumption that the average size of the largest macroturbulent eddies (or helical flow cell) is half the channel width. Equation 7.7 shows that the upper range of stable meander wavelengths is numerically very close to this value and similar to the coefficient of 12.34 given by Richards (1982). This corroborates the assertion by Leopold and Wolman (1957, 1960) that the matching of waveforms in bed topography and planform is related to the mechanics of the flow and, in particular, to the turbulent flow structures responsible for shaping the forms and features of meandering channels.

Once meander wavelength has been estimated and sinuosity calculated following the analytical determination of channel slope, the reach-average planform configuration can be specified by solving the sine-generated curve equation.

7.3 PLANFORM LAYOUT

The sine-generated curve is expressed in terms of channel direction relative to the meander belt axis, ϕ, at any distance, s, along the channel, such that

$$\phi = \omega \sin\left(\frac{2\pi s}{L_{\mathrm{m}} P}\right) \qquad (2.107)$$

where 'ω' is the maximum angle a meander loop takes relative to the meander belt axis, 'L_{m}' is meander wavelength and 'P' is sinuosity. From this expression it follows that sinuosity is the average reciprocal of $\cos\phi$ for values of ϕ between 0 and ω. Based on a set of trial values of ω, Langbein and Leopold (1966) developed an approximation for P, such that

$$P = \frac{4 \cdot 84}{4 \cdot 84 - \omega^2} \qquad (7.8)$$

The maximum value of ω is approximately 2.2 radians when meander loops intersect. Equation 7.8 can be rearranged to produce an expression for ω as a function of P, which facilitates solution of the sine-generated curve equation and laying out the planform, such that

$$\omega = 2 \cdot 2\sqrt{\frac{P-1}{P}} \tag{7.9}$$

The radius of curvature, R_c, is not constant in the sine-generated curve but ranges from a maximum value at the inflexion point to minimum curvature around the bend apex for approximately one sixth of the channel length, $\lambda P/6$, when curvature, ϕ, is an approximate linear function of channel distance, s. Through this linear range, ϕ varies approximately from $+0.5\omega$ to -0.5ω, and therefore subtends an arc angle of approximately ω with minimal error. It follows that the radius of curvature around a meander bend is approximated by

$$R_c = \frac{\lambda P}{6\omega} \tag{7.10}$$

Substituting Equation 7.9 into Equation 7.10 yields a simple expression of radius of curvature as a unique function of wavelength and sinuosity given by

$$R_c = \frac{\lambda P^{1 \cdot 5}}{13\sqrt{P-1}} \tag{7.11}$$

By combining Equations 7.7 and 7.11 and assuming an average sinuosity of 1.5 for a typical meandering channel, the radius of curvature within 95 percent confidence limits on the mean response is given by

$$R_c = (2 \cdot 25 \text{ to } 2 \cdot 49)W \tag{7.12}$$

By combining empirical meander relationships, Leopold and Wolman (1960) showed that the radius of curvature-to-width ratio approximates 2.4 in natural channels. This condition falls within the confidence band in Equation 7.12. From a physical basis, Bagnold (1960) suggested that a meander bend with an R_c/W of between 2 and 3 is associated with minimisation of energy losses because of asymmetry in the flow distribution. According to Bagnold, when R_c/W approximates 2, water filaments begin to separate from the inner bank resulting in a local decrease in inertia resistance and associated local increase in

R_c/W just downstream from the bed apex. In tighter bends, the separation zone breaks down because of the formation of large eddies and distortion of the free surface (spill resistance). Although Bagnold's theory was based on pipe flow considerations and not open channels, the range of R_c/W between 2 and 3 has also been found to be associated with greatest lateral migration rates of meander bends (Apmann, 1972; Hickin and Nanson, 1984, 1975; Nanson and Hickin, 1983, Begin, 1981) and Hey (1976) demonstrated that this range is associated with arc angles between 120 and 150 deg, corresponding to well-developed meander bends (although based on circular, rather than sine-generated, curves).

Of the 438 sites used to derive the wavelength-width relationships given in Section 7.2, radius of curvature is recorded for 263 of the sites. This subset was used to validate Equation 7.12 by analysing the distribution of radius of curvature-to-width ratio, R_c/W (Figure 7.4)

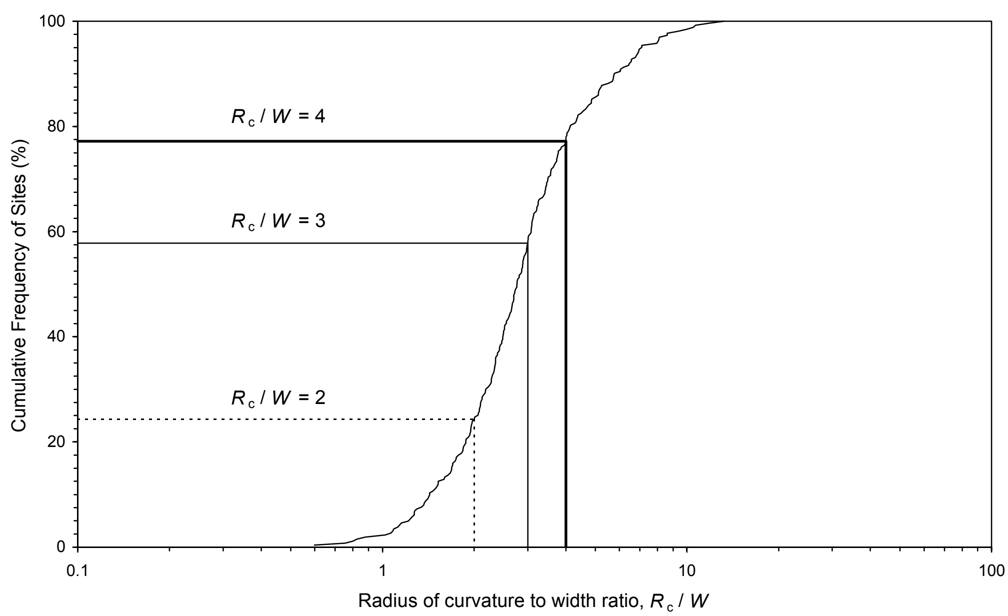


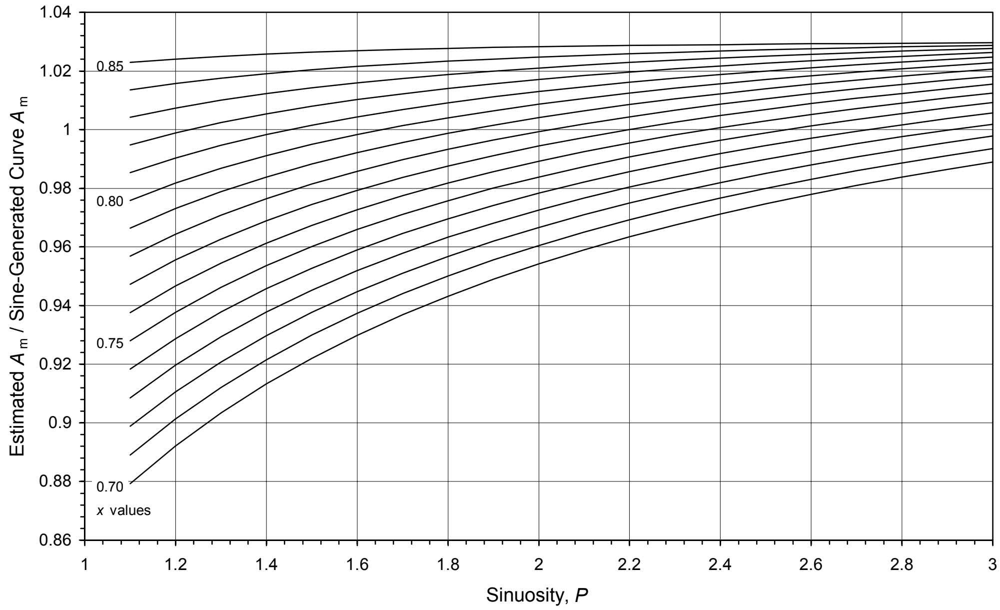
Figure 7.4 Cumulative distribution of radius of curvature-to-width ratio derived from a composite data set of 263 sites.

Figure 7.4 shows that 33.5 percent, 52.9 percent and 71.2 percent of the sites have R_c/W values between 2 and 3, 2 and 4 and 1.5 and 4.5, respectively. The modal value of R_c/W is 2.77, which is slightly greater than the range given by Equation 7.12. Although a wide range of R_c/W is represented by the data set, the range between 2 and 3, for which Equation 7.12 provides a reasonable approximation, describes the steepest section of the cumulative curve.

In many restoration schemes, resulting from site constraints, it would not be possible to re-establish the original corridor width and restoration would be confined to a narrow 'right of way'. Direct calculation of the desired meander amplitude, A_m, (meander belt width) from the sine generated curve equation is not possible and can only be accomplished once the planform has been drawn. However, an approximation can be made, thus enabling the restoration designer to make recommendations as to whether adjustments to the restored channel design are necessary to allow it to be accommodated within the available floodplain area. Based on Langbein and Leopold's (1966) assumption that Equation 7.10 is valid for one sixth of the distance along the channel within one wavelength, it can be shown that

$$A_m = \frac{L_m P}{3\omega}\left[1 - \cos\left(\frac{\omega}{2}\right) + \omega \sin\left(x\omega\right)\right] \tag{7.13}$$

where 'ω' is calculated from Equation 7.9 , given sinuosity, and 'x' is an adjustment factor such that $x\omega$ approximates the average channel direction angle through the near-straight reach between bend apices, over one-third of the distance along the channel within one wavelength. By comparing estimated meander amplitude with that measured from the sine-generated curve, the value of x can be shown to vary with sinuosity (Figure 7.5). For sinuosities between 1.1 and 3.0, suitable values of x range from 0.73 to 0.82 (where the associated lines cross the y-axis=1.0 grid line). However, Figure 7.5 shows that a value of 0.85 provides a reasonable approximation over this range within a small factor of safety.



Note: 'x' is an adjustment factor, such that $x\omega$ approximates the average channel direction angle through the near-straight reach between bend apices, over one-third of the distance along the channel within one wavelength, where 'ω' is the maximum angle the meander loop takes relative to the meander belt axis.

Figure 7.5 Comparison between estimated meander amplitude (Equation 7.13) with that measured from the sine-generated curve as a function of sinuosity.

The above discussion provides the necessary design equations to layout the sine-generated curve as an approximation of reach-average meander planform geometry. However, Carson and Lapointe (1983) criticised the sine-generated curve for its symmetrical configuration and forwarded a delayed inflexion model as a better representation of natural meander geometry. They proposed an asymmetry index, z:

$$z = 100\left(\frac{u}{u+d}\right) = 100\left(1 - \frac{R_e}{A_m}\left(1 - \cos\theta\right)\right) \tag{7.14}$$

where 'R_e' refers to the 'bend-entry' radius of curvature, 'θ' is the angle of the delayed inflexion point, relative to the meander belt axis and 'u' and 'd' are distances transverse to the meander belt axis between the delayed inflexion point and the upstream and downstream bend apices respectively, as shown in Figure 7.6. The path of the sine-generated curve can be modified in accordance with Equation 7.14 if it is assumed that R_c and ω defined by the sine-generated curve equal R_e and θ, respectively, in Figure 7.6. This modification does not change the direction angle at the inflexion point but moves the inflexion point to coincide with the entrance to the next downstream bend where radius of curvature is constant (see Figure 7.6) and the direction angle of the channel path is increased from ω/2 in the sine-generated curve to ω. Therefore, between bend apex and downstream infection point there is a non-linear decrease in curvature. However, the rate of change of curvature in this zone is undefined and the meander path must be completed by freehand.

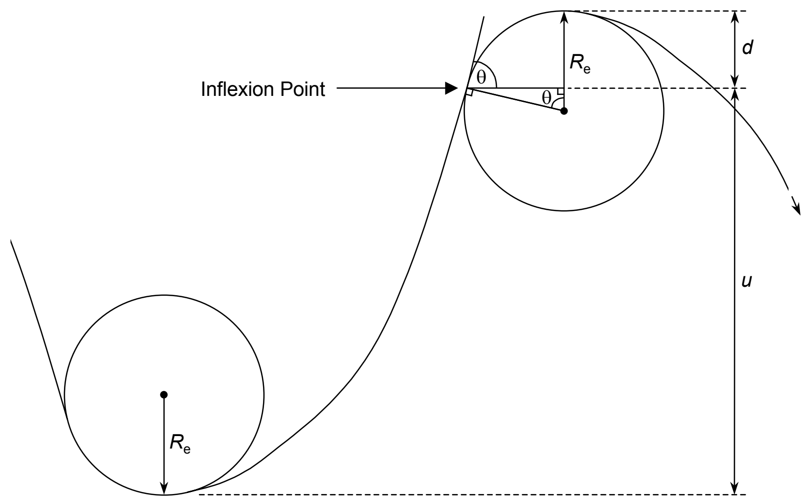


Figure 7.6 Asymmetrical meander geometry resulting from the down-channel delay in inflexion points (modified from Carson and Lapointe, 1983, p. 45).

As discussed in Chapter 3, the bankfull dimensions in a natural meandering river are not uniform but exhibit characteristic morphological variability along the meander path which is predominantly explained by the rhythmic growth and decay of helical flow cells, although other environmental controls can distort the general downstream trends. Notably, width is usually held constant in two- and quasi-three-dimensional computer models which route flow and sediment through curved channels. However, the oscillatory nature of bankfull width, location of pools and riffles along the meander path and the maximum scour depth adjacent to the outer bank in the pools are of particular importance for channel restoration design and are considered in the following section.

7.4 NATURAL VARIABILITY AROUND MEANDER BENDWAYS

In a study of scour prediction in river bends, Thorne (1988) compiled an empirical data set of cross section and planform dimensions from meander bends in the Red River between Index, Arkansas, and Shreveport, Louisiana. The data were measured from the 1981 hydrographic survey and include measurements of width, W_i, and mean depth, D_m, at meander inflexion points, maximum scour depth of pools, D_{max}, radius of curvature, R_c, meander wavelength, L_m, and sinuosity, P (given by Thorne and Abt, 1993). The Red River reach extends for approximately 1000 km through Arkansas and Louisiana and has a sinuosity of approximately 1.5 to 2. Bed material is composed of medium sand (d_{50} is 0.25 mm), and bank materials are fine sand and silt (d_{50} is 0.1 mm) (Thorne et al., 1991). Dimensions in the data set are referenced to the water surface profile for the 2-year recurrence interval flow, which is a high in-bank flow that was identified by Biedenharn et al. (1987) as a good guide to the channel-forming flow in this river and is equated with bankfull discharge in this study. This is a reasonable assumption in a river with moderate-to-high width-to-depth ratios as in-bank changes in discharge near the bankfull elevation correspond to negligible changes in cross section width.

From the data set, an empirical equation predicting the maximum scour depth during high, steady, in-bank flows was derived (Section 7.4.3). In a subsequent report (Thorne and Abt, 1993), the database was extended using various sources and used to compare scour depths synthesised by bend flow models with those estimated by the empirical approach. As all measured widths refer to a common location in the meander planform geometry, the inflexion point, these studies provided a useful baseline database for examining the variability of width around meander bends as required in the channel restoration design procedure discussed in Chapter 3.

To further the objectives of this study, the 1981 hydrographic survey was reanalysed and further width measurements were taken at the bend apices, W_a, and at pool locations, corresponding to the maximum scour depth, W_p, of 67 meander bends. For each bend, the channel distance between the bend apex and inflexion point, $Z_{a\text{-}i}$, and the channel distance between the bend apex and pool location, $Z_{a\text{-}p}$, were also recorded. While the apex and maximum scour locations were readily identified from the hydrographic survey, locating riffles (defined here as topographic highs, rather than on the basis of sediment properties) relative to the inflexion points was more problematic and less objective. This was because a single cross section could not adequately represent the topographic highs as they extended downstream with similar depth for some variable distance close to the inflexion points (probably related to the length of channel between successive bends). On this basis, estimates of riffle widths and 'riffle-offset' distances from the inflexion points were not included in the enhanced data set. The enhanced Red River data set is given in Appendix Table C1.

The hydrographic survey was also used to 'type' each bend according to the tripartite system identified in Chapter 3:

i) Equiwidth meanders - denoted as 'Type-e' (T_e) meanders

ii) Meanders with point bars - denoted as 'Type-p' (T_b) meanders

iii) Meanders with point bars and chute channels - denoted as 'Type-c' (T_c) meanders

Ranges of physical characteristics pertaining to each of the meander bend types are given in Figure 7.7 and Table 7.3.

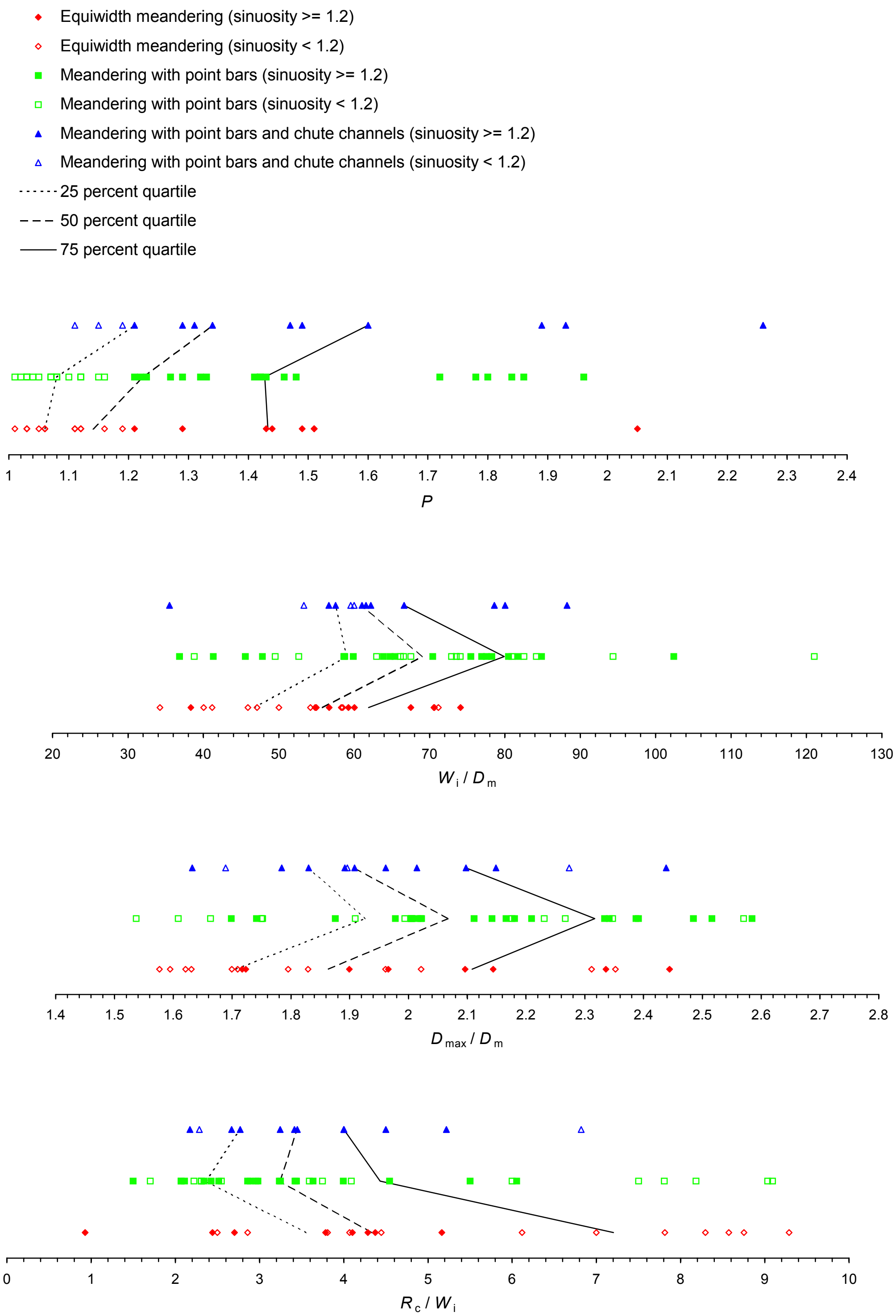
7.4.1 Width Variability between Bend Apex and Inflexion Point

In a study of downstream hydraulic geometry in U.K. gravel-bed rivers, Hey and Thorne (1986) demonstrated that the width at riffles, *RW*, and the width at pools, *PW*, deviate linearly from the mean width around the meander bend, *W*, according to the following dimensionless relationships:

$$RW = 1 \cdot 034W \tag{7.15}$$

$$PW = 0 \cdot 966W \tag{7.16}$$

These equations support the divergence and convergence of downstream isovels that are characteristic of riffles and pools, respectively. However, the relationships are based on bankfull measurements over the proportion of the channel where flow is moving in a downstream direction (Thorne, 1999, pers. comm.) and do not account for the body of



Note: P = sinuosity; W_i / D_m = inflexion point width-to-mean depth ratio; D_{max} / D_m = maximum scour depth in pool-to-mean depth at inflexion point; R_c / W_i = radius of curvature-to-inflexion point width ratio.

Figure 7.7 Ranges and distributions of physical characteristics found in different meander bend types identified from the 1981 Red River hydrographic survey between Index, Arkansas, and Shreveport, Louisiana.

	n	S (10^6)	P	W_i / D_m	D_{max} / D_m	R_c / W_i
Type-e	20 (8)	65 to 268 (133 to 268)	1.0 to 2.1 (1.2 to 2.1)	34.2 to 74.1 (38.3 to 74.1)	1.6 to 2.4 (1.7 to 2.4)	0.9 to 9.3 (0.9 to 5.2)
Type-b	34 (19)	76 to 294 (105 to 294)	1.0 to 2.0 (1.1 to 2.0)	36.8 to 121.0 (36.8 to 102.4)	1.5 to 2.6 (1.7 to 2.6)	1.5 to 9.1 (1.5 to 6.1)
Type-c	13 (10)	91 to 201 (91 to 201)	1.1 to 2.3 (1.2 to 2.3)	33.5 to 88.2 (33.5 to 88.2)	1.6 to 2.4 (1.6 to 2.4)	2.2 to 6.8 (2.2 to 5.2)

Note: n = number of meander bends studied; S = water surface slope; P = sinuosity; W_i / D_m = inflexion point width-to-mean depth ratio; D_{max} / D_m = maximum scour depth in pool-to-mean depth at inflexion point; R_c / W_i = radius of curvature-to-inflexion point width ratio. Values in parentheses refer to meander bends with sinuosity 1.2 or greater.

Table 7.3 Ranges of physical characteristics found in different meander bend types identified from the 1981 Red River hydrographic survey between Index, Arkansas, and Shreveport, Louisiana.

water above the point bar crest where water flows in a radial direction toward the outer bank or is static or flows upstream (Dietrich and Smith, 1983). In meandering rivers with well-developed point bars, the bankfull width at the bend apex is usually considerably wider than that at the inflexion point, as demonstrated by the Red River data used in this study. In terms of channel restoration design, the sheltered zone of flow over the inner bank of a meander bend has significant physical habitat value and should be restored to maximise the morphological diversity, and in turn the biodiversity, found in natural channels. As Equations 7.15 and 7.16 reveal only a 7 percent change in width along a meander path, the coefficients can assume values of unity with negligible error, and therefore, they have only limited practical use for restoring the form *and* function of meandering rivers.

Two dimensionless parameters have been devised to analyse the width variability around meander bends based on the enhanced Red River data set: i) the ratio of bend apex width to inflexion point width, W_a/W_i, and; ii) the ratio of pool location width (at maximum scour) to inflexion point width, W_p/W_i. Theoretically, these parameters adjust according to the degree of curvature and the type of meander bend. To derive new morphological relationships, sinuosity, P, was preferred as the independent variable rather than the radius of curvature-to-width ratio which would have resulted in width appearing on both sides of the regression equations. Notably, the authors found no statistical or morphological justification for using logarithmic relationships in any of the regression analyses undertaken.

7.4.1.1 Bend Apex Width

Two hypotheses were tested:

Hypothesis 7.1

The ratio of bend apex width to inflexion point width, W_a/W_i, is a positive function of sinuosity, P, according to the linear relationship

$$\frac{W_a}{W_i} = a + bP \tag{7.17}$$

where 'a' and 'b' are defined by ordinary least squares regression.

Hypothesis 7.2

The ratio of bend apex width to inflexion point width, W_a/W_i, is a function of bend type such that the ratio is smallest for 'e-type' (equiwidth) bends (approaching a value of unity) and greatest for 'c-type' (point bars with frequent chute channels) bends, according to Equation 7.17.

$$\frac{W_a}{W_i} = f(\text{bend type}), \text{ such that } \left(\frac{W_a}{W_i}\right)_e < \left(\frac{W_a}{W_i}\right)_b < \left(\frac{W_a}{W_i}\right)_c \tag{7.18}$$

where the subscripts 'e', 'b' and 'c' refer to the ratios of 'e-type', 'b-type' and 'c-type' bends, respectively. The best-fit morphological relationships are shown in Figure 7.8.

Using modified versions of the confidence interval equations given in Chapter 3 (for non-logarithmic transformed data), the W_a/W_i ratio within 100(1-p) percent confidence limits, $(W_a/W_i)_p$, is given by

$$\left(\frac{W_a}{W_i}\right)_p = a + bP \pm c_{1,p}\sqrt{\frac{1}{c_2} + \frac{(P - c_3)^2}{c_4}} \tag{7.19}$$

where values of $c_{1,p}$, c_2, c_3 and c_4 define natural W_a/W_i variability, at 'p' probability (Table 7.4).

	Best-Fit Equation			Confidence Limits					
	a	b	R^2	$c_{1,0.01}$	$c_{1,0.05}$	$c_{1,0.1}$	c_2	c_3	c_4
Type-e	0.85	0.13	0.13	0.254	0.186	0.153	20 (0.952)	1.249	1.256
Type-b	0.97	0.23	0.25	0.313	0.233	0.194	34 (0.971)	1.311	2.619
Type-c	2.09	-0.19	0.34	0.292	0.207	0.169	13 (0.929)	1.480	1.481

Note: Values given refer to mean response confidence limits. Value in parentheses is used to calculate single response confidence limits.

Table 7.4 Constant values used to derive an expression for the ratio of bend apex width to inflexion point width, W_a/W_i, with confidence bands based on the best-fit linear function of sinuosity and meander bend type. Coefficients pertaining to the 99, 95 and 90 percent confidence limits are given.

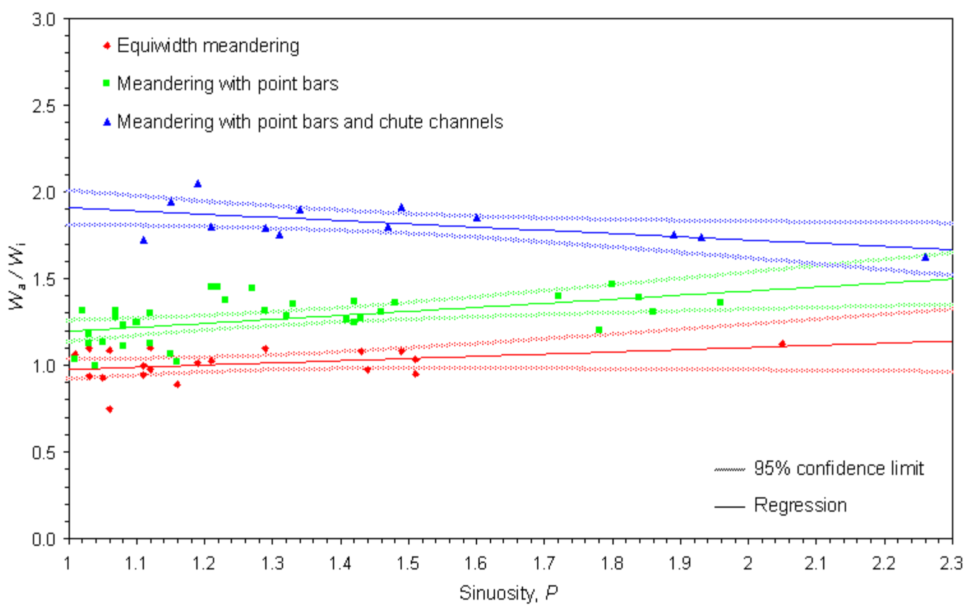


Figure 7.8 Ratio of bend apex width to inflexion point width, W_a/W_i, as a function of meander bend type and sinuosity, P. Confidence limits of a mean response are shown at the 95 percent level. Source data: 1981 Red River hydrographic survey.

Equation 7.17 can be expanded to yield a composite relationship expressing the ëmagnitudeí and ëbend type variabilityí of the three bend types. In Equations 7.20 and 7.21, the additive effects of ëe-typeí, ëp-typeí and ëc-typeí bends are represented by the binary parameters, T_e, T_b and T_c, respectively. The value of T_e has a value of ë1í for all three types of bend and represents the smallest planform width ratio. If point bars are present but chute channels are rare, then T_b is assigned a value of '1' and T_c is assigned a value of '0'. If point bars are present and chute channels are common, then both T_b and T_c are assigned values of '1'. Obviously T_c can only be given a value of '1' when T_b has a value of '1'.

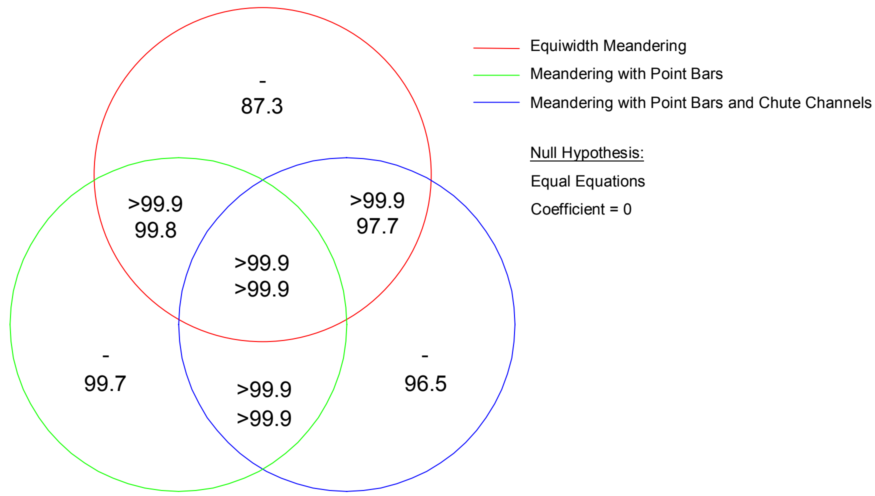
$$\frac{W_a}{W_i} = \left(0\cdot 85 + 0\cdot 13P\right)T_e + \left(0\cdot 12 + 0\cdot 10P\right)T_b + \left(1\cdot 13 - 0\cdot 42P\right)T_c \qquad (7.20)$$

or

$$\begin{aligned}\frac{W_a}{W_i} &= \left(0\cdot 85T_e + 0\cdot 12T_b + 1\cdot 13T_c\right) + \left(0\cdot 13T_e + 0\cdot 10T_b - 0\cdot 42T_c\right)P \\ &= \text{magnitude} + \text{bend type variability}\end{aligned} \qquad (7.21)$$

As R^2 values in Table 7.4 are low, two statistical techniques have been applied to test for significance of these morphological relationships: a) the General Linear Hypothesis, and; b) confidence ellipses of the regression coefficients (Chapter 3). The General Linear Hypothesis was used to examine whether sinuosity is significant in the relationships. The results of the analysis are given in a Venn diagram (Figure 7.9). The values represent the significance levels of *rejecting* the null hypothesis that: i) equations for different bend types are the same (upper values), and: ii) sinuosity coefficients are equal to 0 (lower values). The results indicate that only the e-type bend relationship in not a function of sinuosity at the 95 percent level. C-type bends are only slightly above the 95 percent significant level, despite the negative relationship in Figure 7.8. This is probably because there is a considerable degree of scatter about the best-fit line which reduces the statistical significance of the relationship. When any of the equations are compared simultaneously, all permutations suggest that sinuosity is significant and the equations are statistically different at the 95 percent level.

In Figure 7.10, confidence limits are applied to the regression constants, 'a' and 'b' (intercept and gradient) simultaneously, rather than to the equation itself. If sinuosity is not a significant parameter in Equation 7.17, then the confidence regions for each meander bend type would circumscribe a coefficient, 'b', value of zero (zero gradient in Figure 7.8). However, the graph shows that sinuosity is significant at the 99 percent level in the type-b equation and only just significant at the 95 percent level in the type-c equation. In general, the majority of the confidence regions lie either above or below the



Note: Values are significance levels of *rejecting* the null hypothesis that: i) equations for different bend types are the same (upper values), and: ii) sinuosity coefficients are equal to 0 (lower values). Simultaneous significance levels for comparing two or all three equations pertaining to each bend type are defined where circles overlap.

Figure 7.9 Venn diagram showing the results of applying the General Linear Hypothesis to the best-fit regression relationships predicting the ratio of bend apex width to inflexion point width as a function of sinuosity and meander bend type.

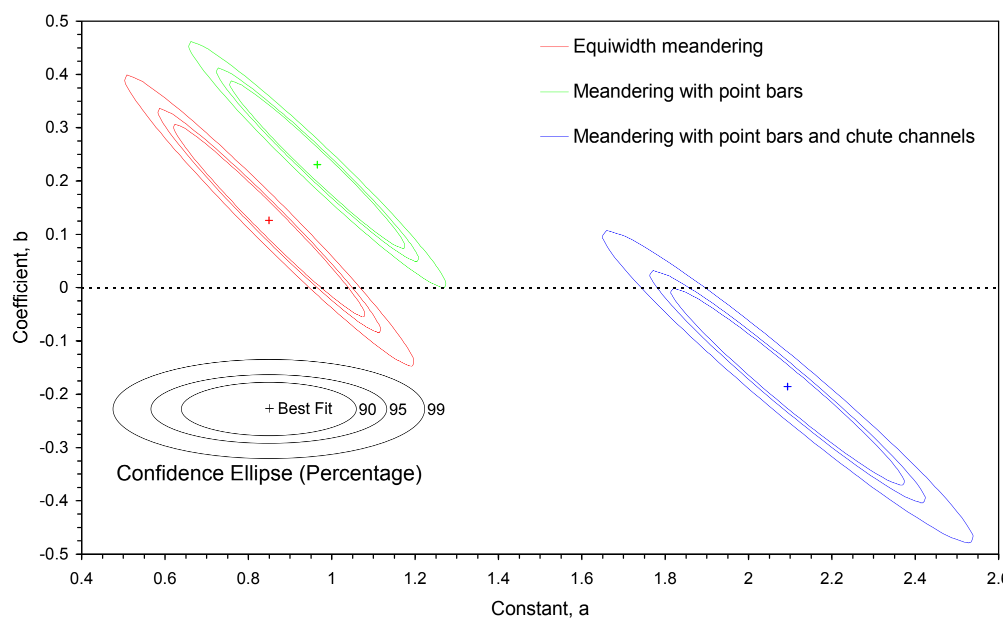


Figure 7.10 Confidence ellipses applied to the regression values of ‘a’ and ‘b’ in the best-fit relationship, $W_a/W_i = a + bP$, for different meander bend types, where ‘W_a’ is the bend apex width, ‘W_i’ is the inflexion point width and ‘P’ is sinuosity.

	Best-Fit Equation			Confidence Limits					
	a	b	R^2	$c_{1,0.01}$	$c_{1,0.05}$	$c_{1,0.1}$	c_2	c_3	c_4
Type-e	0.91	0.09	0.14	0.227	0.150	0.119	8 (0.889)	1.491	0.439
Type-b	1.41	-0.04	0.01	0.225	0.164	0.135	19 (0.950)	1.497	1.095
Type-c	2.06	-0.17	0.45	0.222	0.153	0.123	10 (0.909)	1.579	1.053

Note: Values given refer to mean response confidence limits. Value in parentheses is used to calculate single response confidence limits.

Table 7.5 Constant values used to derive an expression for the ratio of bend apex width to inflexion point width, W_a/W_i, with confidence bands based on the best-fit linear function of sinuosity and meander bend type, applicable to sites with sinuosity of at least 1.2. Coefficients pertaining to the 99, 95 and 90 percent confidence limits are given.

zero gradient line. Notably, sinuosity is least significant for the type-e meander bends. This is expected as the processes required to develop point bars and increase the W_a/W_i ratio are absent in this type of channel. The graph also clearly shows that c-type bends are inversely related to sinuosity.

Despite the significance tests indicating that W_a/W_i is a function of sinuosity in all but e-type meander bends, the results are biased by the non-normal distributions of sinuosity for each bend type. For example, the b-type bend exhibits a skewness of 1.50, compared to that of a perfectly normal distribution of zero. This is a likely outcome of this type of analysis when the independent variable is highly skewed in nature. Using logarithmic values of sinuosity did not overcome this problem, nor increase the coefficient of determination, R^2. Therefore, in order to reduce the bias, the morphological equations were reproduced for sites with sinuosity equal to or greater than 1.2, which can be considered to be a threshold between straight channels with only slight sinuosity and meandering channels with moderate to high sinuosity. The modified relationships applicable to sites with sinuosity of at least 1.2 are shown in Figure 7.11 (bends with sinuosity less than 1.2 are shown for comparison only) and the respective values in Equation 7.19 were recalculated and are given in Table 7.5.

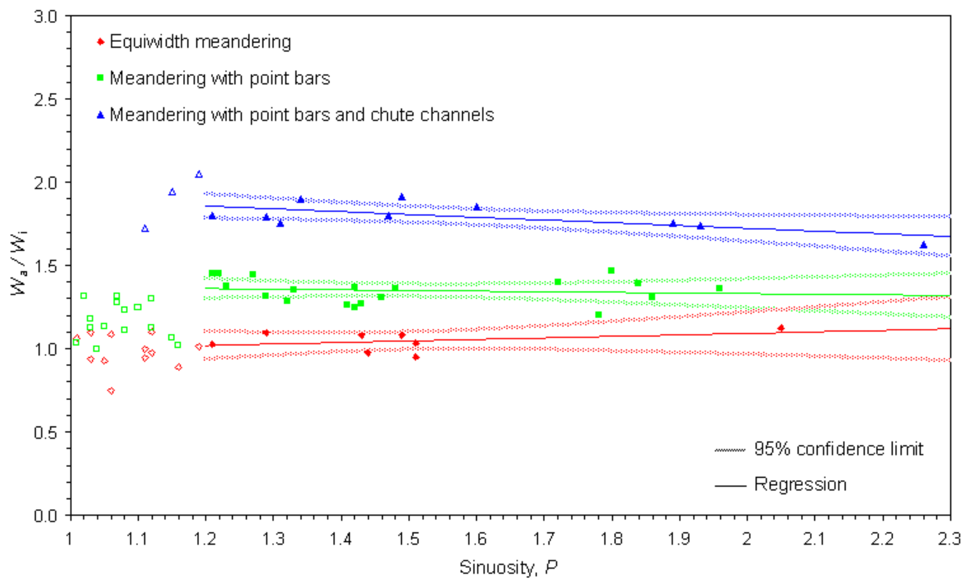
Similar to Equations 7.20 and 7.21, Equation 7.17 can be expanded to yield a composite relationship expressing the 'magnitude' and 'bend type variability' of the three bend types applicable to sites with sinuosities of at least 1.2:

$$\frac{W_{\mathrm{a}}}{W_{\mathrm{i}}} = (0 \cdot 91 + 0 \cdot 09P)T_{\mathrm{e}} + (0 \cdot 50 - 0 \cdot 05P)T_{\mathrm{b}} + (0 \cdot 65 - 0 \cdot 13P)T_{\mathrm{c}} \qquad (7.22)$$

or

$$\begin{aligned}\frac{W_{\mathrm{a}}}{W_{\mathrm{i}}} &= (0 \cdot 91T_{\mathrm{e}} + 0 \cdot 50T_{\mathrm{b}} + 0 \cdot 65T_{\mathrm{c}}) + (0 \cdot 09T_{\mathrm{e}} - 0 \cdot 05T_{\mathrm{b}} - 0 \cdot 13T_{\mathrm{c}})P \\ &= \text{magnitude} + \text{bend type variability}\end{aligned} \qquad (7.23)$$

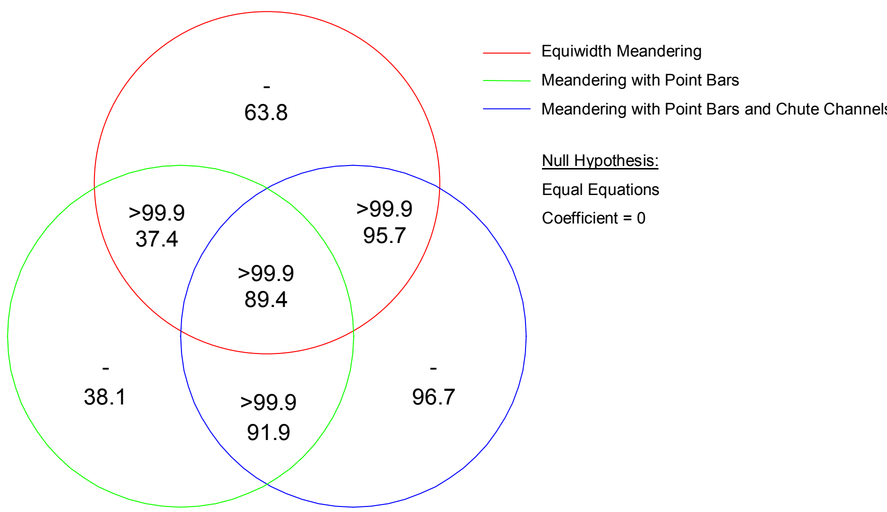
Figure 7.11 shows that meander bends with point bars (type-b) and low sinuosity (P<1.2) have a visibly lower W_a/W_i ratio than those of moderate to high sinuosity (P>=1.2). As straight channels with alternate bar features develop into meandering channels, the apex width increases relative to the upstream crossing width until sinuosity approaches a value of approximately 1.2. For meander bends with higher sinuosity, there appears to be no relationship. This may explain why sinuosity was shown to be a significant parameter in Figure 7.10.



Note: filled symbols = sinuosity of at least 1.2; empty symbols = sinuosity less than 1.2

Figure 7.11 Ratio of bend apex width to inflexion point width, W_a/W_i, as a function of meander bend type and sinuosity, P, for sinuosities of at least 1.2. Confidence limits of a mean response are shown at the 95 percent level. Source data: 1981 Red River hydrographic survey.

The same significance tests were applied to the subset of meander bends with sinuosity of at least 1.2 (Figures 7.12 and 7.13). The results of applying the General Linear Hypothesis technique demonstrate that both e-type and b-type meander bends are not a function of sinuosity at the 95 percent level, in fact the null hypothesis of a zero coefficient is still accepted at the 65 percent level, while the c-type bend only marginally fails the 95 percent significance test (significant at the 96.7 percent level). If all three bend types are tested simultaneously, then sinuosity is not a significant parameter *in all* the equations at the 95 percent level. Figure 7.13 shows that the revised confidence ellipses significantly overlap the zero coefficient line for e-type and b-type bends, indicating that sinuosity is not a significant parameter. The c-type bend ellipse is very similar to that in Figure 7.10 as this type of bend is generally only found when sinuosity is high. These results present a case for removing sinuosity from Equation 7.17 and rejecting Hypothesis 7.1 for moderate to high sinuosity, as portrayed in Figure 7.14.



Note: Values are significance levels of *rejecting* the null hypothesis that: i) equations for different bend types are the same (upper values), and: ii) sinuosity coefficients are equal to 0 (lower values). Simultaneous significance levels for comparing two or all three equations pertaining to each bend type are defined where circles overlap.

Figure 7.12 Venn diagram showing the results of applying the General Linear Hypothesis to the best-fit regression relationships predicting the ratio of bend apex width to inflexion point width as a function of sinuosity and meander bend type, for sinuosities of at least 1.2.

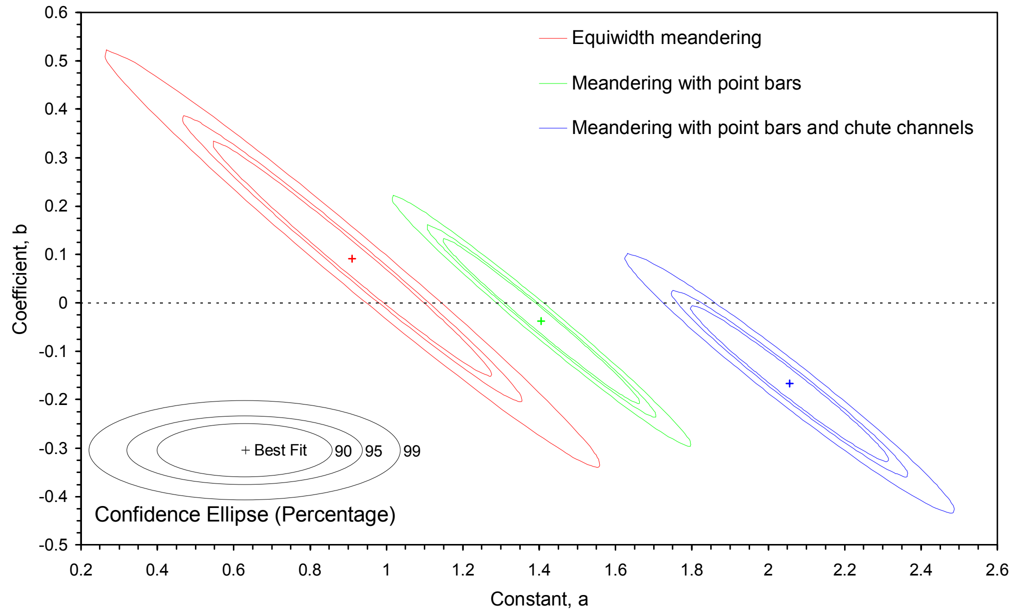
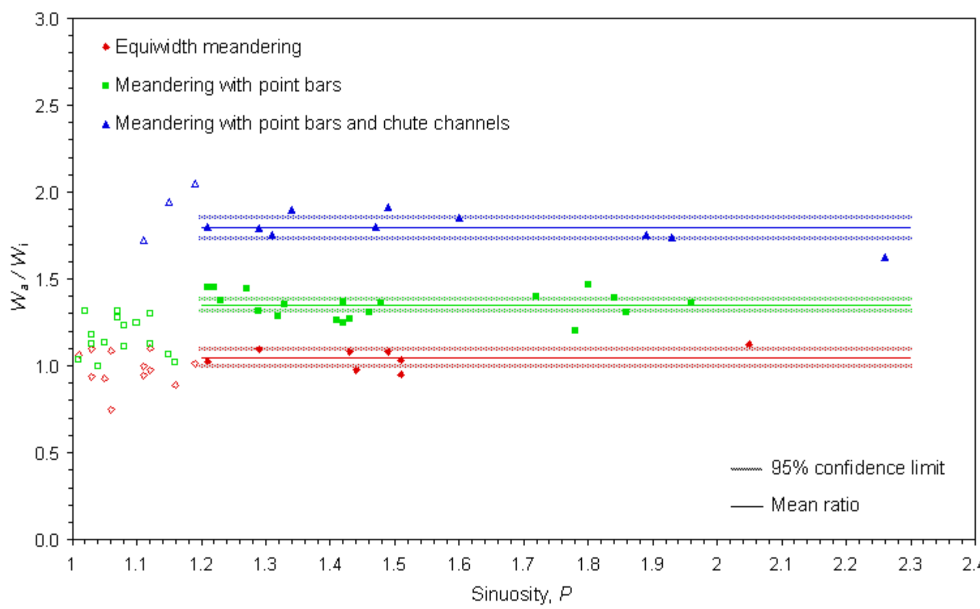


Figure 7.13 Confidence ellipses applied to the regression values of ëaí and ëbí in the best-fit relationship, W_a/W_i = a + bP, for different meander bend types, where: ëW_aí is the bend apex width; ëW_ií is the inflexion point width; ëPí is sinuosity of at least 1.2.



Note: filled symbols = sinuosity of at least 1.2; empty symbols = sinuosity less than 1.2

Figure 7.14 Ratio of bend apex width to inflexion point width, W_a/W_i, as a function of meander bend type only, for sinuosities of at least 1.2. Confidence limits of a mean response are shown at the 95 percent level. Source data: 1981 Red River hydrographic survey.

Revised estimates of W_a/W_i, independent of sinuosity, within 100(1-p) percent confidence limits, $(W_a/W_i)_p$, are given by

$$\left(\frac{W_a}{W_i}\right)_p = \mathrm{a} + \mathrm{u}_p \qquad (7.24)$$

where 'a' is the mean ratio and 'u_p' is a measure of uncertainty at the 100(1-p) percent level, as given in Table 7.6.

Figure 7.15 shows the observed range and dispersion of the W_a/W_i ratio. Analysis of Variance was used to test Hypothesis 7.2 and confirm that the variabilities for each bend type in Figure 7.14 are significantly different at the 95 percent level. The analysis yielded an F-value of 227.25. The critical F-value for 2 and 34 degrees of freedom (between groups and within groups respectively) is 3.28. As the critical F-value is exceeded, the null hypothesis that the three bend types were derived from the same population can be rejected, in favour of Hypothesis 7.2.

	a	$\mathrm{u}_{0.01}$	$\mathrm{u}_{0.05}$	$\mathrm{u}_{0.1}$
Type-e	1.05	0.08 (0.29)	0.05 (0.20)	0.04 (0.16)
Type-b	1.35	0.05 (0.27)	0.04 (0.20)	0.03 (0.16)
Type-c	1.79	0.09 (0.36)	0.06 (0.25)	0.05 (0.20)

Note: Values given refer to mean response confidence limits. Value in parentheses is used to calculate single response confidence limits.

Table 7.6 Constant values used to estimate the mean ratio of bend apex width to inflexion point width, W_a/W_i, within confidence bands for different types of meander bend and for sites with sinuosity of at least 1.2. Coefficients pertaining to the 99, 95 and 90 percent confidence limits are given.

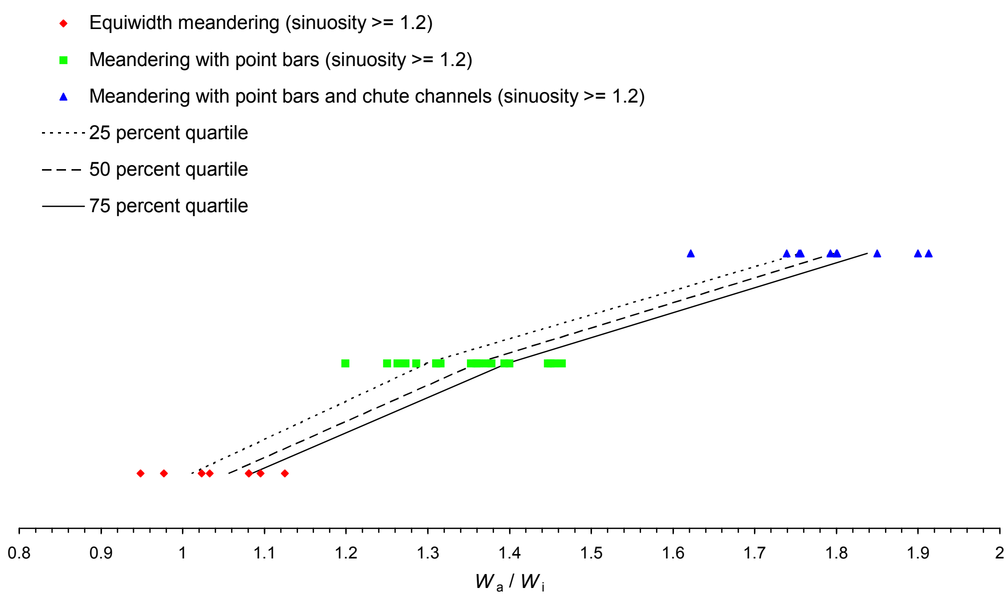


Figure 7.15 Range and distribution of the ratio of bend apex width to inflexion point width, W_a/W_i, for sites with sinuosity of at least 1.2. Source data: 1981 Red River hydrographic survey.

7.4.1.2 Pool Width (at Maximum Scour Location)

Based on the results in Section 7.4.1.1, it is assumed that planform width variability is not significantly a function of sinuosity for bends with sinuosity equal to or greater than 1.2. Therefore only one hypothesis was tested for the ratio of pool width (at maximum scour location) to inflexion point width, W_p/W_i:

Hypothesis 7.3

For bends with sinuosity equal to or greater than 1.2, the ratio of pool width to inflexion point width, W_p/W_i, is a function of bend type, such that the ratio is smallest for 'e-type' (equiwidth) bends (approaching a value of unity) and greatest for 'c-type' (point bars with frequent chutes) bends, according to Equation 7.25.

$$\frac{W_p}{W_i} = \mathrm{f}\left(\text{bend type}\right), \text{ such that } \left(\frac{W_p}{W_i}\right)_e < \left(\frac{W_p}{W_i}\right)_b < \left(\frac{W_p}{W_i}\right)_c \qquad (7.25)$$

where the subscripts 'e', 'b' and 'c' refer to the ratios of 'e-type', 'b-type' and 'c-type' bends respectively.

Similar to Equation 7.24, mean values of the W_p/W_i ratio within 100(1-p) percent confidence limits, $(W_p/W_i)_p$, are given by

$$\left(\frac{W_p}{W_i}\right)_p = a + u_p \tag{7.26}$$

where 'a' is the mean ratio and 'u_p' is a measure of uncertainty at the 100(1-p) percent level, as given in Table 7.7. Confidence bands are applied to the mean ratios in Figure 7.16.

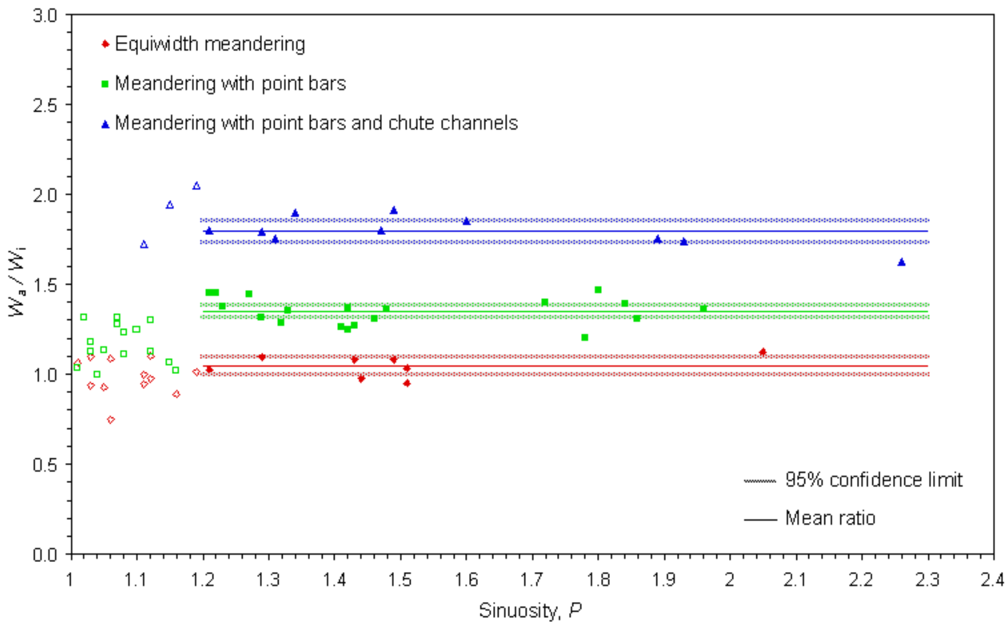
	a	$u_{0.01}$	$u_{0.05}$	$u_{0.1}$
Type-e	0.95	0.15 (0.56)	0.10 (0.38)	0.08 (0.30)
Type-b	1.15	0.12 (0.64)	0.09 (0.47)	0.07 (0.39)
Type-c	1.29	0.26 (1.07)	0.18 (0.74)	0.14 (0.60)

Note: Values given refer to mean response confidence limits. Value in parentheses is used to calculate single response confidence limits.

Table 7.7 Constant values used to estimate the mean ratio of pool width (at maximum scour location) to inflexion point width, W_p/W_i, within confidence bands for different types of meander bend and for sites with sinuosity of at least 1.2. Coefficients pertaining to the 99, 95 and 90 percent confidence limits are given.

Figure 7.16 indicates that the differences between meander bend types in Figure 7.14 are not as clearly defined for the W_p/W_i ratio. Based on the spread of data points alone there is a tendency to reject Hypothesis 7.3 and combine the data into a single data set.

However, Analysis of Variance was used to test Hypothesis 7.3 and confirm that the variabilities for each bend type in Figure 7.16 are significantly different at the 95 percent level. The analysis yielded an F-value of 6.91. The critical F-value for 2 and 34 degrees of freedom (between groups and within groups respectively) is 3.28. As the critical F-value is exceeded, the null hypothesis that the three bend types were derived from the same population can be rejected, in favour of Hypothesis 7.3, despite the observed variability. This is corroborated by the inter-quartile ranges in Figure 7.17 which shows the dispersion of the W_p/W_i ratio for each meander bend type.



Note: filled symbols = sinuosity of at least 1.2; empty symbols = sinuosity less than 1.2

Figure 7.16 Ratio of pool width (at maximum scour location) to inflexion point width, W_p/W_i, as a function of meander bend type only, for sinuosities of at least 1.2. Confidence limits of a mean response are shown at the 95 percent level. Source data: 1981 Red River hydrographic survey.

7.4.2 Location of Bendway Pools

While the location of the meander inflexion points and bend apices are geometrically defined, the location of pools, defined by the position of maximum scour, are not only controlled by the meander configuration but by the complex velocity distribution and large-scale coherent flow structures which pulse sediment along the channel to form alternate zones of scour and fill.

There is a wealth of literature documenting the rhythmic spacing of pools and riffles (e.g. Leopold and Wolman, 1960; Keller and Melhorn, 1978; Hey and Thorne, 1986) but little is known about the location of pools and riffles along the meander path relative to the apices and inflexion points. This is because the mechanisms of scour and fill in curved channels are poorly understood. Using the 1981 Red River Hydrographic Survey, the pool location can be represented empirically by a ëpool-offsetí ratio, defined as the ratio of the channel distance between bend apex and maximum scour location to the channel distance between bend apex and downstream inflexion point, $Z_{a\text{-}p}/Z_{a\text{-}i}$.

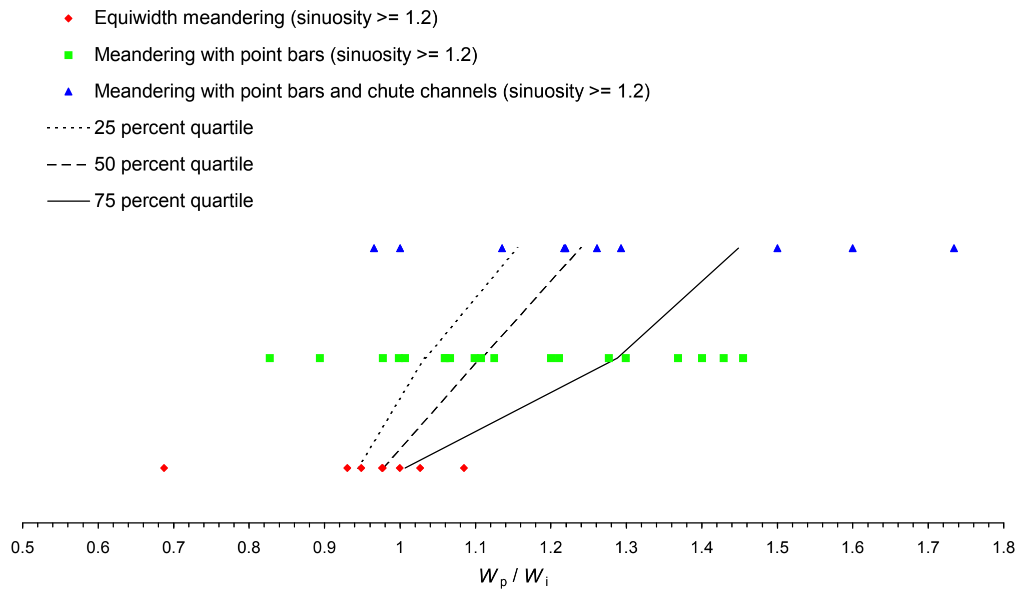


Figure 7.17 Range and distribution of the ratio of pool width (at maximum scour location) to inflexion point width, W_p/W_i for sites with sinuosity of at least 1.2. Source data: 1981 Red River hydrographic survey.

As with the W_p/W_i ratio, only one hypothesis was tested on the pool-offset ratio:

Hypothesis 7.4

The pool-offset ratio, $Z_{a\text{-}p}/Z_{a\text{-}i}$, is a function of bend type such that the ratio is smallest for 'e-type' (equiwidth) bends (approaching a value of unity) and greatest for 'c-type' (point bars with frequent chutes) bends, according to Equation 7.27.

$$\frac{Z_{\text{a-p}}}{Z_{\text{a-i}}} = \mathrm{f}\left(\text{bend type}\right), \text{ such that } \left(\frac{Z_{\text{a-p}}}{Z_{\text{a-i}}}\right)_{\text{e}} < \left(\frac{Z_{\text{a-p}}}{Z_{\text{a-i}}}\right)_{\text{b}} < \left(\frac{Z_{\text{a-p}}}{Z_{\text{a-i}}}\right)_{\text{c}} \tag{7.27}$$

where the subscripts 'e', 'b' and 'c' refer to the ratios of 'e-type', 'b-type' and 'c-type' bends, respectively. Mean values of the pool-offset ratio within 100(1-*p*) percent confidence limits, $(Z_{a\text{-}p}/Z_{a\text{-}i})_p$, are given by

$$\left(\frac{Z_{\text{a-p}}}{Z_{\text{a-i}}}\right)_{p} = \mathrm{a} + \mathrm{u}_{p} \tag{7.28}$$

where 'a' is the mean ratio and 'u_p' is a measure of uncertainty at the 100(1-*p*) percent level, as given in Table 7.8. Confidence bands are applied to the mean ratios in Figure 7.18.

	a	$u_{0.01}$	$u_{0.05}$	$u_{0.1}$
Type-e	0.28	0.21 (1.16)	0.15 (0.85)	0.13 (0.70)
Type-b	0.35	0.15 (1.05)	0.11 (0.78)	0.10 (0.65)
Type-c	0.51	0.29 (1.32)	0.20 (0.94)	0.17 (0.77)

Note: Values given refer to mean response confidence limits. Value in parentheses is used to calculate single response confidence limits.

Table 7.8 Constant values used to estimate the pool-offset ratio, $Z_{a\text{-}p}/Z_{a\text{-}i}$, within confidence bands for different types of meander bend. Coefficients pertaining to the 99, 95 and 90 percent confidence limits are given.

The wide distribution of data points in Figure 7.18 indicates that a regression analysis using sinuosity, P, as the independent parameter is inappropriate as there are no clear visible trends. This suggests that the pool-offset ratio is not a function of sinuosity. Furthermore, the highly variable distributions did not justify dividing the data set at the 1.2 sinuosity threshold as applied previously. Figure 7.19 shows the range and dispersion of the pool-offset ratio for each of the meander bend types. Analysis of Variance was used to test Hypothesis 7.4 and confirm that the variabilities for each bend type in Figure 7.18 are significantly different at the 95 percent level. The analysis yielded an F-value of 2.22. The critical F-value for 2 and 63 degrees of freedom (between groups and within groups respectively) is 3.14. As the critical F-value is *not* exceeded, the null hypothesis that the three bend types were derived from the same population must be accepted and Hypothesis 7.4 rejected.

This analysis suggests that the pool-offset ratio is independent of both sinuosity and bend type, and a single morphological relationship is suitable for all meander bend types studied. The constant values in Equation 7.28 for the full data set are given in Table 7.9. A frequency analysis was undertaken on the full data set and may be used as tentative design guidance for channel restoration design (Figure 7.20).

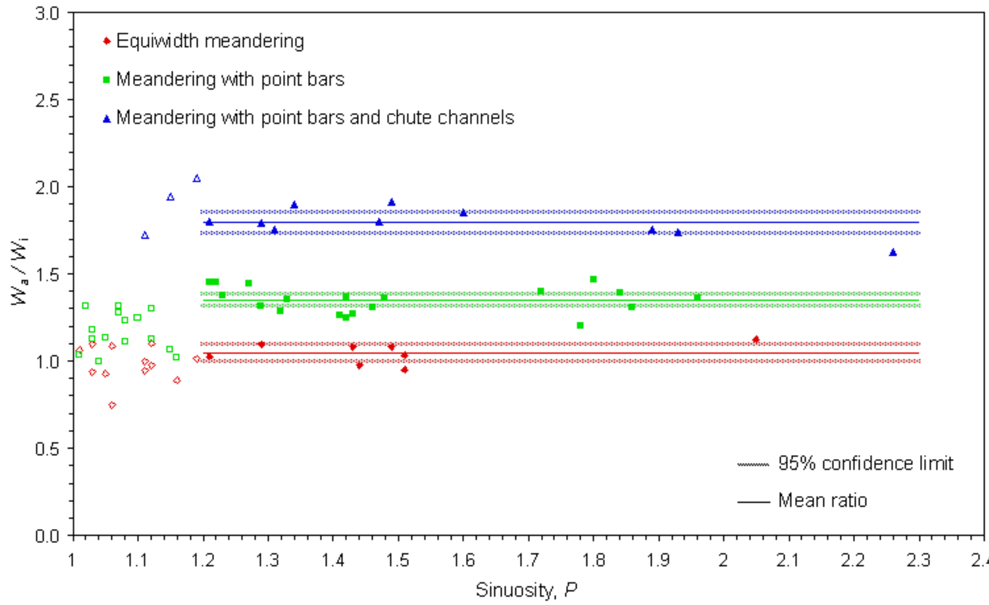


Figure 7.18 Pool-offset ratio, $Z_{a\text{-}p}/Z_{a\text{-}i}$, for different meander bend types. Confidence limits of a mean response are shown at the 95 percent level. Source data: 1981 Red River hydrographic survey.

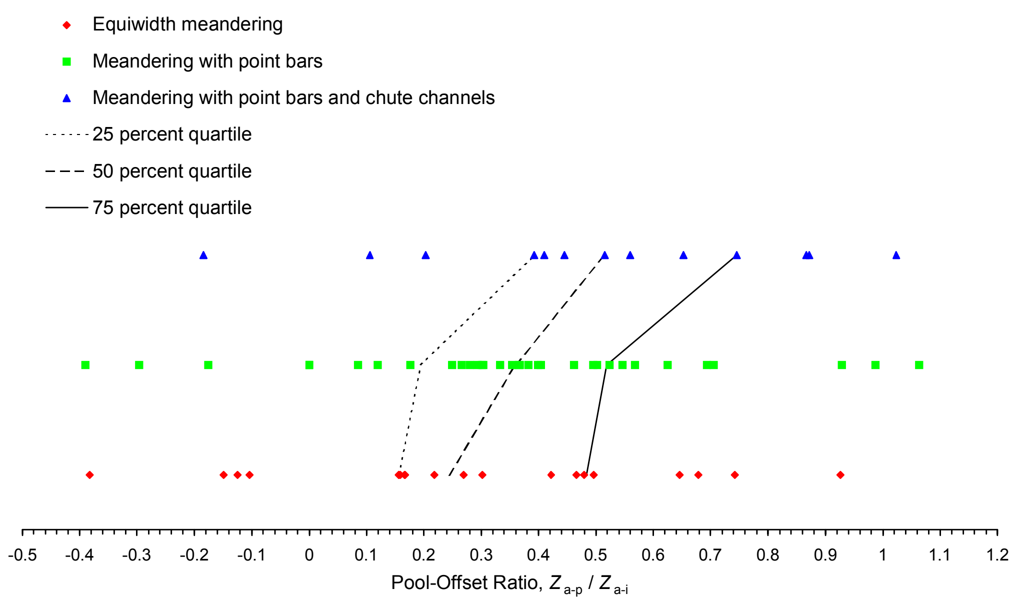


Figure 7.19 Range and distribution of the pool-offset ratio, $Z_{a\text{-}p}/Z_{a\text{-}i}$, for different meander bend types. Source data: 1981 Red River hydrographic survey (unpublished data, USACE District, Vicksburg)

	a	$u_{0.01}$	$u_{0.05}$	$u_{0.1}$
All types	0.36	0.11 (1.00)	0.08 (0.75)	0.07 (0.63)

Note: Values given refer to mean response confidence limits. Value in parentheses is used to calculate single response confidence limits.

Table 7.9 Constant values used to estimate the pool-offset ratio, $Z_{a\text{-}p}/Z_{a\text{-}i}$, within confidence bands for all types of meander bend studied. Coefficients pertaining to the 99, 95 and 90 percent confidence limits are given.

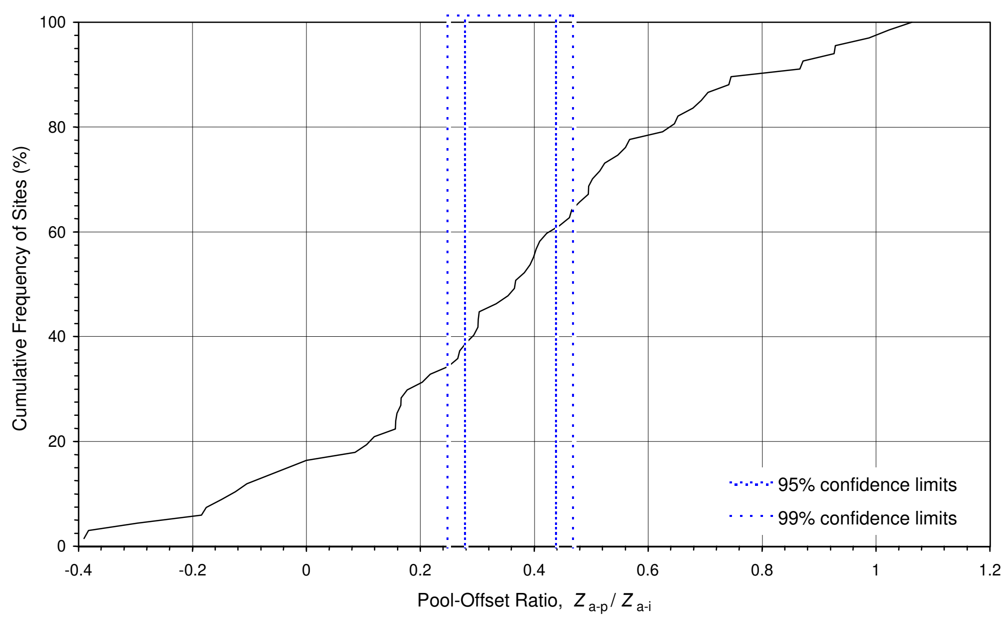


Figure 7.20 Cumulative distribution of the pool-offset ratio, $Z_{a\text{-}p}/Z_{a\text{-}i}$, for all types of meander bend studied. Confidence limits on the mean response are shown. Source data: 1981 Red River hydrographic survey.

Yang (1971, p. 1573) noted that riffles should be located 'in the neighbourhood' of the points of inflexion but may not be centred precisely at the inflexions because of the influence of tributaries, various environmental controls and geological constraints. Although data on the location of riffles were not available from the hydrographic survey, it is expected that a riffle-offset ratio, defined as the ratio of the channel distance between inflexion point and riffle to the channel distance between inflexion point and downstream bend apex, would exhibit a similar frequency distribution to that for pool-offsets in Figure 7.20. Other data sets are required to examine whether this is an acceptable assumption for natural meandering rivers.

The actual design, specification and installation of riffle and pool features are beyond the scope of this study but appropriate guidance for restoration or rehabilitation projects is available in several existing reports (e.g. Hey and Heritage, 1993).

7.4.3 Maximum Scour Depth in Pools

Scour at the toe of the outer bank in meander pools due to high boundary velocities and boundary shear stresses leads to a progressive increase in the bank height (and often bank angle) until mass failure occurs. In rivers where bank retreat cannot be permitted because of infrastructure on the floodplain, methods are required to estimate the minimum bed elevation consistent with the stability of the bank-line, taking into account the safety envelope for any appropriate bank protection. This elevation depends on channel geometry (slope, cross section and planform), hydraulic forces, bed and bank material and the flow and sediment hydrograph (Maynord, 1996, p. 460). However, sophisticated models that account for some or all of these effects are data intensive (e.g. Smith and McLean, 1984) and empirical methods have been demonstrated by Thorne and Abt (1993) to provide better estimates of the observed scour than two analytical bend flow models developed by Bridge (1982) and Odgaard (1989). Scour prediction is also required to ensure bend-flow is fully developed and to prevent a high pulse of sediment discharge being produced when pools are scoured by flows.

Leliavsky (1955, p. 118) noted that the depth at meander bends increases inversely as a function of radius of curvature. Using scour data from the Red River in Louisiana and Arkansas, Thorne (1988) developed a dimensionless equation for the maximum scour depth, D_{max}, scaled on the mean depth at the upstream inflexion point, D_m, as a function of the ratio of radius of curvature, R_c, to channel top width, measured at the upstream inflexion point, W_i. The Red River data revealed that in very long radius bends (R_c/W_i>10), D_{max}/D_m is only between 1.7 and 2. However, when R_c/W_i is less than 5, the relationship with scour depths is non-linear and for bends with R_c/W_i between 2 and 4, D_{max}/D_m may be anywhere between 2 and 4 (Thorne, 1997, p. 196). The best-fit regression equation explains 64 percent of the variance in D_{max}/D_m and is defined as

$$\frac{D_{\mathrm{max}}}{D_{\mathrm{m}}} = 2 \cdot 07 - 0 \cdot 19 \ln\left(\frac{R_{\mathrm{c}}}{W_{\mathrm{i}}} - 2\right) \qquad (7.29)$$

According to Thorne (1998), the lower R_c/W_i limit in Equation 7.29 is 2 which corresponds to the highest observed scour depths and is consistent with Bagnold's (1960) finding that when the radius of curvature-to-width ratio is approximately 2, energy losses at the bend are minimised and flow efficiency maximised.

Based on a data set of 256 river bends on a wide range of rivers and laboratory channels (from various researchers), Thorne and Abt (1993) demonstrated that Equation 7.29 is in good agreement with scour depths found in alluvial channels with both sand and gravel beds. In a later study, Maynord (1996) added 39 observations from the Lower Mississippi River to the Thorne and Abt database (excluding the laboratory channels) and proposed an empirical 'safe' design curve for maximum scour depth which incorporates a factor of safety. The approach is based on a multiple regression analysis and gives an upper-bound estimate of scour. This is considered more appropriate for investigating whether bank stabilisation and protection is necessary rather than a best-fit relationship which may underestimate potential scour and therefore, result in under-designed engineering measures. Maynord (1996) derived a series of equations for different factors of safety, based on the percentage of the computed D_{max} to observed D_{max} ratios less than 0.95. The relationship with only 10 percent of the observed data falling below this threshold is given by a factor of safety of 1.08 and the following expression:

$$\frac{D_{\mathrm{max}}}{D_{\mathrm{m}}} = 1 \cdot 08\left[1 \cdot 8 - 0 \cdot 051\left(\frac{R_{\mathrm{c}}}{W_{\mathrm{i}}}\right) + 0 \cdot 0084\left(\frac{W_{\mathrm{i}}}{D_{\mathrm{m}}}\right)\right] \qquad (7.30)$$

Notably, the equation is a linear function of R_c/W_i. The width-to-depth ratio is included to account for the fact that D_{max}/D_m is partly attributable to the duration of large flow events and is therefore, biased toward large rivers which, as a general rule, exhibit gradual changes in stage during the passage of a flood, compared with the rapid rise and fall of the hydrograph in small streams. However, from a statistical viewpoint, the inclusion of the width-to-depth ratio results in marked collinearity between the two 'independent' dimensionless variables, which misleadingly improves the significance of the equation in terms of explained variance in D_{max}/D_m.

In light of this problem, the data set was reanalysed and divided into two subsets using a width-to-depth threshold value of 60, which is an approximate modal value. The best-fit morphological relationships are given as

$$\frac{W_i}{D_m} < 60 \qquad \frac{D_{max}}{D_m} = 2 \cdot 14 - 0 \cdot 19 \ln\left(\frac{R_c}{W_i}\right) \tag{7.31}$$

$$\frac{W_i}{D_m} \geq 60\,,\ \frac{R_c}{W_i} < 10 \qquad \frac{D_{max}}{D_m} = 2 \cdot 98 - 0 \cdot 54 \ln\left(\frac{R_c}{W_i}\right) \tag{7.32}$$

A practical safe design curve may then be defined as

$$\frac{D_{max}}{D_m} = 1 \cdot 5 + 4 \cdot 5\left(\frac{R_c}{W_i}\right)^{-1} \tag{7.33}$$

Equation 7.33 is an asymptotic relationship with a theoretical minimum D_{max}/D_m of 1.5. From this upper-bound relationship, D_{max}/D_m ranges from 4 to 3 for R_c/W_i between 1.8 and 3. For channels with an R_c/W_i of less than 1.8, it is recommended that the dimensionless scour depth should be fixed at 4. All three relationships are portrayed in Figure 7.21, which shows that Equation 7.33 is a safe curve for both classes of W_i/D_m.

7.4.4 Practical Channel Design Equations

Assuming that confidence is primarily a function of sample size in the analysis of planform width variability, then it is possible to derive a mean band of uncertainty, 'u', suitable for all three types of meander bend to provide a set of practical design equations. In Equations 7.34 and 7.35 the cumulative effects of 'e-type', 'b-type' and 'c-type' bends are represented by the binary parameters, T_e, T_b and T_c, respectively. The value of T_e has a value of '1' for all three types of bend and represents the smallest planform width ratio. If point bars are present but chute channels are rare, then T_b is assigned a value of '1' and T_c is assigned a value of '0'. If point bars are present and chute channels are common, then both T_b and T_c are assigned values of '1'. Obviously T_c can only be given a value of '1' when T_b has a value of '1'.

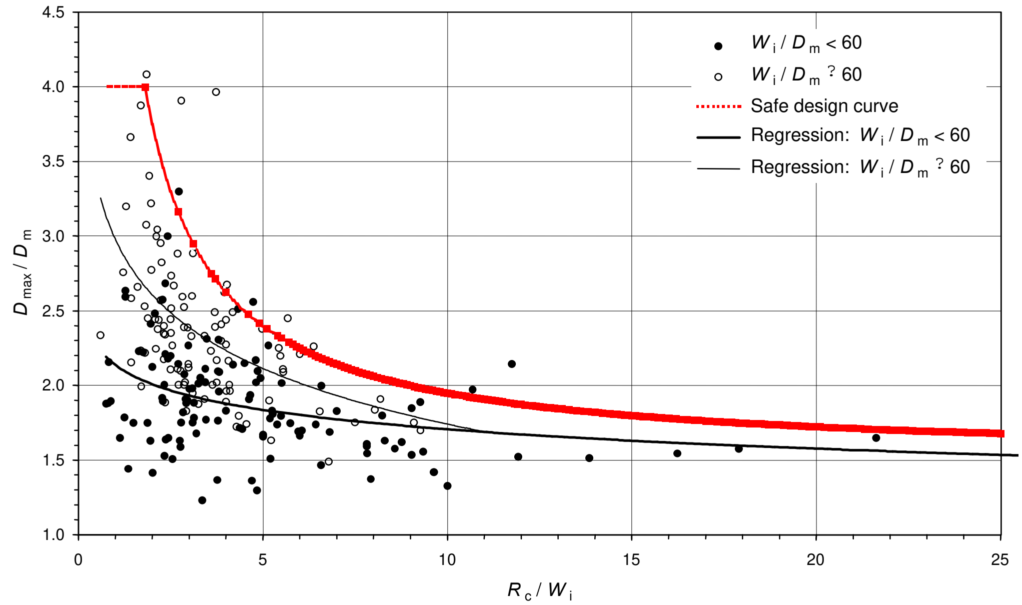


Figure 7.21 Dimensionless maximum scour depth in meander pools as a function of radius of curvature-to-width ratio. Source data: Thorne and Abt (1993); Maynord (1996).

Bend Apex (P>=1.2)

$$\frac{W_a}{W_i} = 1\cdot05 T_e + 0\cdot30 T_b + 0\cdot44 T_c \pm \mathrm{u} \qquad (7.34)$$

Pool Width (P>=1.2):

$$\frac{W_p}{W_i} = 0\cdot95 T_e + 0\cdot20 T_b + 0\cdot14 T_c \pm \mathrm{u} \qquad (7.35)$$

For all three bend types and sinuosities greater than 1, the pool offset ratio is given by

Pool-Offset (P>1.0)

$$\frac{Z_{a\text{-}p}}{Z_{a\text{-}i}} = 0\cdot36 \pm \mathrm{u} \qquad (7.36)$$

Values of 'u' refer to confidence limits on the mean response as given in Table 7.10.

Confidence Limits percent	W_a / W_i	W_p / W_i	$Z_{a\text{-}p} / Z_{a\text{-}i}$
99	0.07	0.17	0.11
95	0.05	0.12	0.08
90	0.04	0.10	0.07

Table 7.10 Uncertainty, 'u', in estimates of width variability around meander bends and location of pools. Values refer to confidence limits on the mean response.

A practical design equation for predicting or constructing maximum scour depths at bends is the upper-bound curve in Figure 7.21, given by Equation 7.33:

$$\frac{D_{\mathrm{max}}}{D_{\mathrm{m}}} = 1.5 + 4.5\left(\frac{R_{\mathrm{c}}}{W_{\mathrm{i}}}\right)^{-1} \tag{7.33}$$

For sites where active meandering is not permitted, bank protection will be required along the outer bank to prevent flow erosion. In addition, this equation should be used together with bank stability charts to establish whether bank stabilisation against mass failure is also necessary.

In this chapter a series of empirical equations have been developed to specify the planform geometry and morphological variability around meander bends that are required for channel restoration design. To determine reach average meander wavelength, a composite data set, consisting of 438 sites, has been used to develop a generic meander wavelength-width relationship with confidence intervals that is suitable for engineering analysis. The equation was demonstrated to be related to the scaling of turbulent flow structures responsible for shaping the forms and features of meandering channels. The estimated meander wavelength, together with sinuosity, derived following guidance in Chapter 6, facilitates the solution of the sine-generated curve equation and specification of the reach-average planform configuration. By manipulating the regular meander path equation, morphological equations have been derived to specify the radius of curvature and meander amplitude (meander belt width) which could assist the design engineer in making recommendations as to whether adjustments to the restored configuration are required to allow the channel to be accommodated within the available floodplain area. Recommendations to modify the sine-generated curve to account for asymmetrical meander bends have also been proposed in this chapter based on existing research.

In Chapter 3, a simple meander planform classification system was developed by identifying the main meander bend types in existing stream classification schemes. Equations expressing morphological variability around meander bends have been developed for each of the three meander bend types in the classification system. The equations describe how width adjusts between the bend apex, location of maximum scour depth and inflexion point and specify the location and depth of the bendway pools. It was found that sinuosity is not a significant parameter in the local variability equations for bends with sinuosity of at least 1.2.

In the following chapter the essential components of channel restoration design framework are applied to a case study where significant post-project channel change has occurred since restoration. The objectives of the application are to investigate whether the channel evolution is tending toward the simulated channel configuration output from the procedure and also to identify any operational difficulties in following the procedure which would highlight areas for further development. Initially, the mechanisms of post-project channel change are identified before applying the enhanced procedure to identify the degree to which the observed adjustments are driving evolution toward the simulated channel configuration, thereby providing success criteria for the design framework.

CHAPTER 8

Case Study

Restoration of Whitemarsh Run, Maryland

8.1 INTRODUCTION

Maryland streams, especially those in the piedmont zone, are particularly challenging to the design engineer because many catchments have been affected by considerable extrinsic influences during the Twentieth Century and, in particular, during the past few decades, which have moulded unnatural channel geometries in many urban streams. Recent catchment history typically includes significant modification of rainfall-runoff processes as a result of flood control projects and rapid, widespread urbanisation. Furthermore, there are high rates of sediment delivery from arable land to streams in the lower piedmont and coastal zones, although catchments are generally experiencing a decline in crop area as a result of urban expansion. Between 1960 and 1980 there was a 100.3 percent growth in urban area in Maryland (U.S. Department of Commerce, 1985). Between 1982 and 1992 there has been a further 24.9 percent increase in urban area and a corresponding decline in the area of crop land by 6.7 percent (U.S. Department of Commerce, 1985, 1996). The rate of channel change in the lower piedmont and coastal zones of Maryland is exacerbated by the characteristic unconsolidated sandy-silt composition of bank material.

Field reconnaissance has revealed that many streams appear to be approaching a new state of equilibrium (currently in a state of quasi-equilibrium) following urbanisation, as indicated by alternate bar formations in incised channels. A number of sites have been targeted for restoration in order to advance this recovery process and recreate stable channel configurations without affecting the designated level of flood protection. In general, the techniques used to design these restored channels are based on a combination of field experience, reference reach observations and basin-wide regime-type curves. While these approaches can yield appropriate target channel dimensions in some cases, they fail to account explicitly for the systematic nature of sediment conveyance from the upstream supply reach(es), through the restored reach and into the existing channel downstream. If this balance is not achieved, it is unlikely that restored channel dimensions will remain stable within the catchment context.

Whitemarsh Run is located in the Gunpowder-Patapsco catchment and flows eastward from a suburban zone north of Baltimore, through the town of White Marsh. Downstream of the town, it becomes the Bird River which flows into Chesapeake Bay (Figure 8.1). Based on hydrological boundaries defined by Dillow (1996), the parent Gunpowder-Patapsco catchment consists of 18 percent urban development, 18 percent forest and 23 percent crop land (U.S. Environmental Protection Agency, 1998). The headwaters are located in the Piedmont zone but the majority of the system is found in the Western Coastal Plain. The drainage area at the White Marsh gauging station, located immediately downstream from the project reach, is 12.25 km^2. There is only one significant tributary upstream from the project reach, North Fork (NF) Whitemarsh Run, which is gauged near the confluence. The Whitemarsh Run catchment is a small but very dynamic system and is characterised by high sediment load of fine gravel material pulsed through the system by a flashy flow regime.

A 1.5 km reach of Whitemarsh Run was restored in September, 1996, but has since undergone significant morphological adjustment in response to the implemented design. As the restored channel configuration was unstable, the United Stated Army Corps of Engineers (USACE) deemed it necessary to investigate whether alternative designs would have been more suitable. The channel restoration design procedure developed in this report was applied as part of a beta-testing programme with the objective to determine if channel evolution in the unstable reach is tending towards the simulated design configuration output from the procedure. The authors carried out fieldwork during November, 1998, assisted by staff from the USACE Baltimore District Office, and data analysis was undertaken at the University of Nottingham.

8.2 THE RESTORATION PROJECT

The pre-restoration channel morphology was characterised by low sinuosity and low depth diversity as a result of previous channelisation works to alleviate flooding and to allow for expansion of commercial development on the floodplain. The main engineering project objectives included:

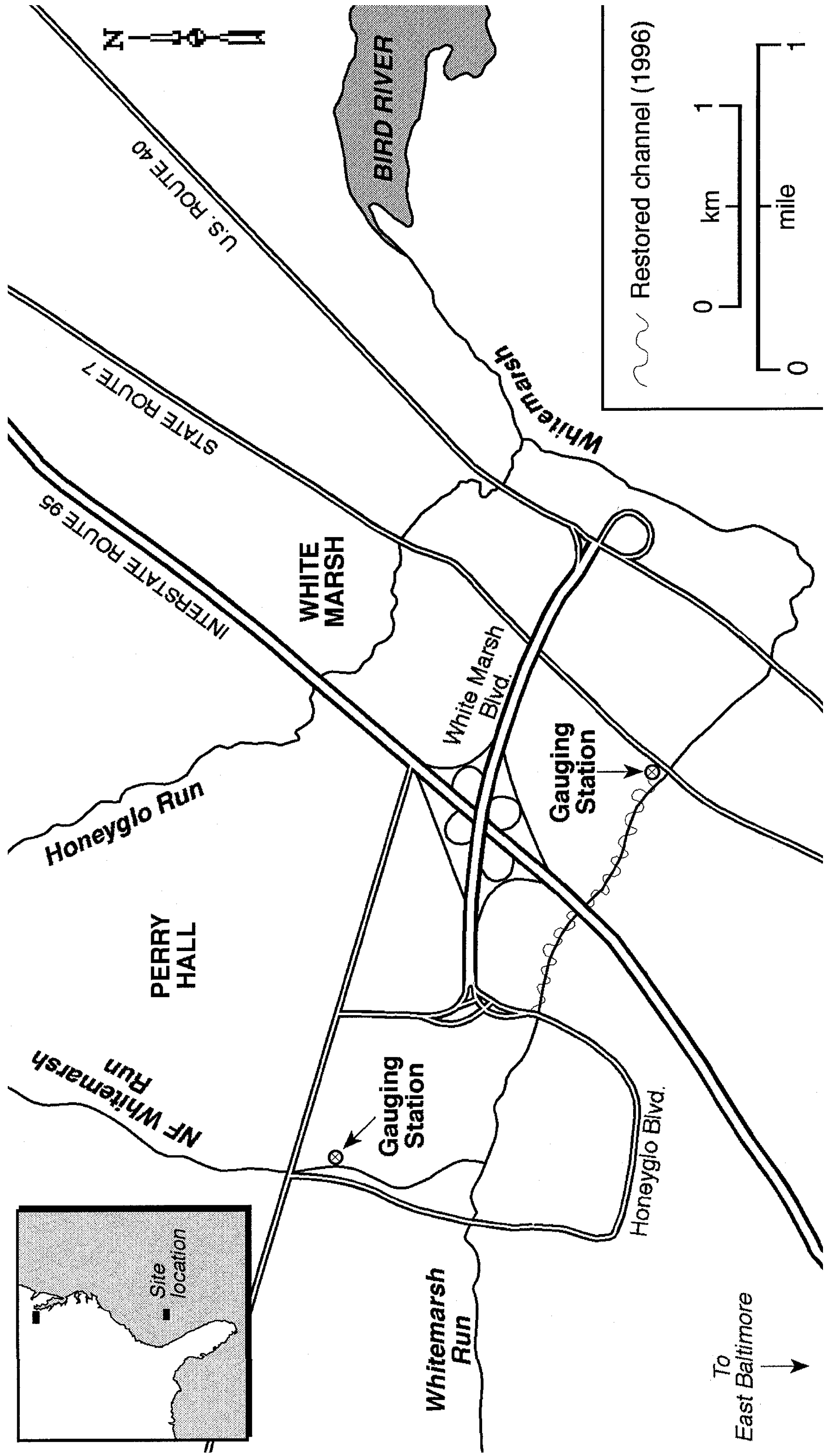


Figure 8.1. Location of the restored reach in Whitemarsh Run, Baltimore County, Maryland.

i) Restoring of the diverse structure and function of a meandering channel to a river system of relatively low sinuosity.
ii) Protecting the bank-lines from erosion.
iii) Improving the aesthetic quality and amenity value of the stream within an urbanised catchment.
iv) Maintaining the present level of flood protection with embankments.

The restored channel was designed to be static-stable by minimising aggradation and degradation while inhibiting the migrating tendency of a natural meandering river by protecting the bank-lines from erosion.

The restored channel design included:

i) A low flow channel with wide, shallow point bars, within rock-lined embankments designed to contain the 50-year recurrence interval flood.
ii) Increased sinuosity from approximately 1.1 to approximately 1.7. This led to a decline in slope from approximately 0.0038 to approximately 0.0025.
iii) Decreased meander wavelength from approximately 350 m to 90 m with a very regular meander path of similar meander bends.
iv) Restructured instream morphology with asymmetrical cross sections around bendways and uniform cross sections at meander inflexion points.
v) Low stage rock vortex weirs at meander crossings to control the grade.
vi) Rootwads and riprap around bendways to prevent bank erosion and lateral shift of planform.
vii) Willow planting to stabilise wide, shallow point bars.

Information defining the engineering analyses adopted to derive the restored channel geometry are unavailable but the design was probably based on empirical, basin-wide regression equations. Observations both upstream and downstream of the restored reach revealed a considerably lower degree of sinuosity than that restored, and which is more suited to expending energy on sediment transport though the system. Therefore, the restoration involved meander creation or ‘enhancement’ rather than ‘reinstatement’.

Since installation there has been piecemeal replacement of rootwads and extension of the rock-lined embankments in response to the post-project channel change described in the following section.

8.3 POST-PROJECT CHANNEL CHANGES

Post-project reconnaissance revealed significant planform and cross-sectional channel changes. In particular, the stream has reduced its sinuosity from that constructed and it has experienced considerable sedimentation in the bendways. During November, 1998, cross-sectional surveys were undertaken at both a representative bend apex and inflexion point within the restored channel. The elevations of the surveys were relative to the same datum used in the original design drawings of March, 1995, thereby enabling the design drawings to be superimposed over the field surveys. Considerable aggradation, at both the bend apex and crossing sections, has occurred during the post-project period, between September 1996 and November 1998 (Figure 8.2). The morphological changes are summarised in Table 8.1.

	Bend Apex	Inflexion Point
Volume change (m^3s^{-1} per unit length)	5.01	2.50
Sedimentation (tonnes per unit length)	13.29	6.63
Sedimentation rate (tonnes yr^{-1} per unit length)	6.05	3.01

Note: specific gravity of bed material is 2.65, density of water is 1000 kg m^{-3}.

Table 8.1 Sedimentation rates in the restored reach of Whitemarsh Run, Maryland.

The direction of channel evolution is summarised in Figure 8.3 in terms of a feedback mechanism operating on a channel designed out of regime which is attempting to recover a stable morphological condition. The imposition of an unstable, high sinuosity channel decreased the slope (1) and, coupled with a high sediment delivery from upstream, led to aggradation in the meander bendways (2). By infilling the designed pools and eroding through the designed point bars, the channel is recovering through straightening (3), which subsequently decreases the sinuosity (4). The restoration of Whitemarsh Run, Maryland, is shown in Figure 8.4.

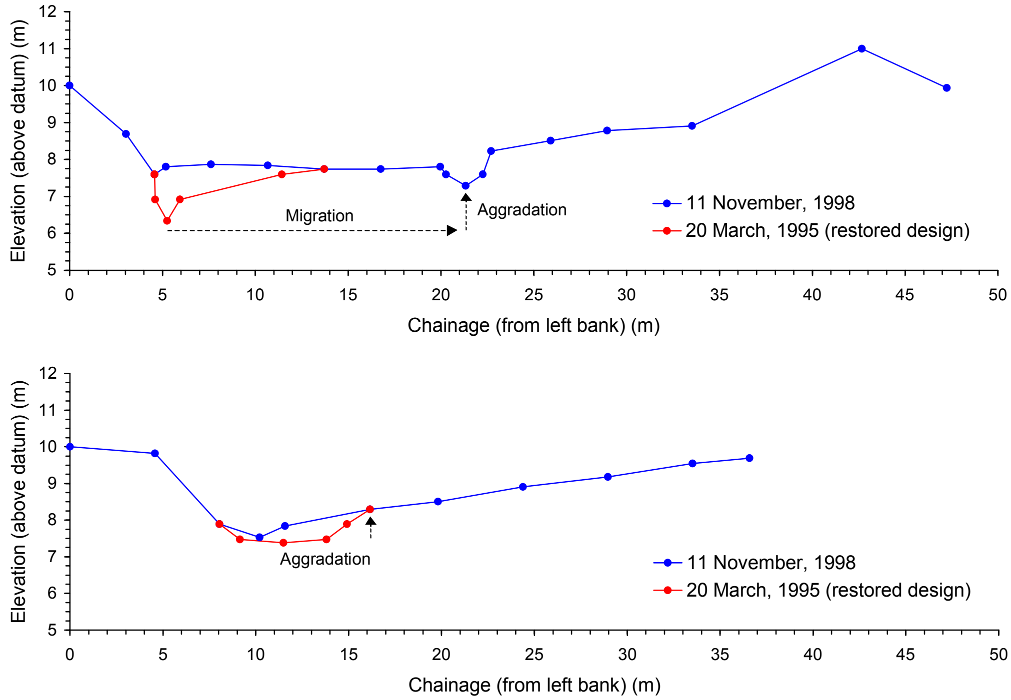


Figure 8.2 Representative design cross sections and post-project channel changes in Whitemarsh Run, Maryland: i) above: bend apex; ii) below: meander inflexion point (200% Vertical exaggeration).

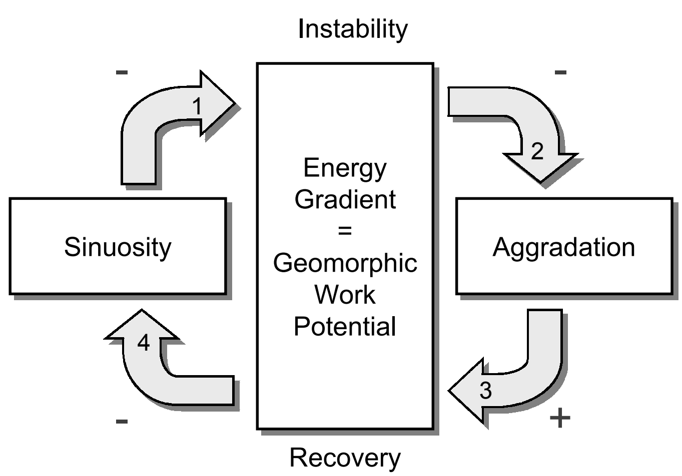


Figure 8.3 Post-project channel change in Whitemarsh Run, Maryland, portrayed by a system of feedback mechanisms comprising stages of morphological adjustment for the recovery of a stable channel alignment.

Figure 8.4 Restoration of Whitemarsh Run, Maryland, September, 1996.

The low flow channel has straightened and widened in places by eroding through the unprotected sandy point bars. Post-project channel changes can be summarised as:

i) Significant aggradation at bends and crossings.
ii) Reduced sinuosity and increased channel slope, via redistribution of sediments.
iii) Isolation of the designed meander bendways as pools are filled and the thalweg adopts a straighter alignment.

These adjustments are shown in Figures 8.5 to 8.7. The degree of post project channel change has probably been amplified by the fact that bankside vegetation was unable to become established prior to widening and straightening.

In summary, the stream power in the restored channel was insufficient to transmit the magnitude of sediment transported from upstream. The restored reach was not designed as a component within the wider fluvial system, and as a result, the reach has acted as a sediment bottleneck which prompted a complex series of channel changes following project implementation. Although the restored channel was intended to be static-stable, the shallow, wide point bars were not stabilised sufficiently to withstand the erosive force of medium-to-high flow events which drive channel evolution.

Figure 8.5 Post-project channel changes in Whitemarsh Run, Maryland, November, 1998. Note the channel straightening and sedimentation in the outer-bank meander pool at the head of restored reach.

Figure 8.6 Post-project channel changes in Whitemarsh Run, Maryland, November, 1998. Note the channel straightening, sedimentation in the outer-bank meander pool and outer-bank protected by riprap in the middle of the restored reach.

Figure 8.7 Post-project channel changes in Whitemarsh Run, Maryland, November, 1998. Sedimentation in the outer bank meander pool with outer- bank protected by rootwads in the middle of the restored reach.

Without considering the potential for post-project sedimentation, the design created desirable river *form* (meander planform with asymmetric cross sections) rather than addressing river *function* (equilibrium sediment conveyance). The inadequacy of the designed channel to transmit sediment through the project reach has led to the river form evolving away from the imposed configuration. Modifications have taken place *within* the designed meander belt width, or 'right-of-way' between the rock-lined embankments. In this respect, the attempt by the designers to guarantee stability by the use of heavy bank protection has also failed.

8.4 SUPPLY REACH ASSESSMENT

A supply reach assessment was undertaken to examine the distribution of sediment transporting flow events and calculate the effective discharge, Q_e, which are input to the restored channel from the upstream reach. The effective discharge was taken to represent the dominant discharge, or channel-forming flow, that would correspond to the bankfull elevation in stable reaches of this gravel-bed river. Although the effective discharge has underestimated the bankfull discharge, Q_b, in sand-bed rivers with highly variable flow distributions (Chapter 5), previous research has shown that effective and bankfull discharges coincide in gravel-bed rivers with mobile beds (see Chapter 2). In Chapter 4 it was hypothesised that the variability in Q_b/Q_e in gravel-bed rivers will not exhibit a strong relationship with flow variability. This is on the basis that high shear stresses are required to mobilise gravel-bed material which renders the very frequent modal flows very ineffective at transporting sediment over the medium- to long-term. Consequently, there is only a small range of in-bank flows capable of moving the bed material, and the bankfull discharge is a highly effective flow. In the absence of data to further examine this correspondence in gravel-bed rivers, it was assumed that flow variability has negligible influence on the effective discharge in this case study.

The Whitemarsh Run stream network can be subdivided into five subsystems (Figure 8.8) which are referenced in this analysis as:

Reach A	Whitemarsh Run main stem upstream from the North Fork tributary confluence;
Reach B	North Fork tributary;
Reach C	Whitemarsh Run main stem downstream from the tributary confluence and upstream from the restored reach, D;
Reach D	Restored reach;
Reach E	Downstream from the restored reach, D.

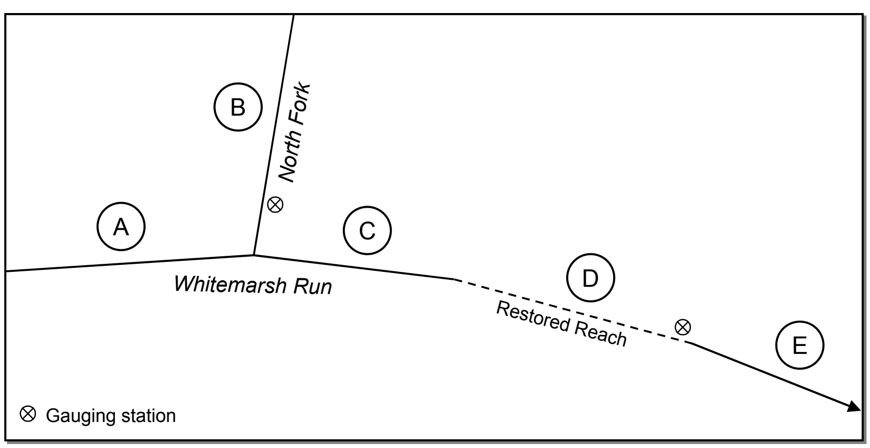


Figure 8.8 Simplification of the Whitemarsh Run stream network in the vicinity of the restored reach showing locations of reaches referenced in the text.

For each reach in the Whitemarsh Run stream network (A to E), magnitude-frequency analysis could be applied to describe the distribution of sediment-transporting flows (and non-sediment-transporting flows) and so calculate the effective discharge. This requires an adequate flow record and a sediment-rating relationship for the reach of interest. If measured sediment load data are unavailable, then a sediment-frequency histogram can be derived using a representative cross section and appropriate sediment transport equation, as described in Chapter 4. The calculated effective discharge is only synonymous with the equilibrium channel-forming discharge if the reach can be considered to be stable. For example, in a degrading reach the effective discharge represents the flow which erodes the greatest quantity of bed material during the period of record. The data available for magnitude-frequency analysis are summarised in Table 8.2.

Reach	Flow Record?	Measured Sediment Load?	Stable Cross Section?
A	✗	✗	✓
B	✓ USGS gauge (01585095)	✗	✓
C	✓ USGS gauge (01585100)	✗	✗
D	✓ USGS gauge (01585100)	✗	✗
E	✓ USGS gauge (01585100)	✗	✗

Table 8.2 Available data for magnitude-frequency analysis in the Whitemarsh Run stream network, Maryland.

In an ideal scenario where a stable cross section can be found which is near a gauging station with negligible flow disparity between the site and the restored reach, an effective discharge suitable for channel design could be derived using the guidance given in Chapter 4. Although Whitemarsh Run is gauged immediately downstream from the restored reach, with 39 years of record (USGS gauge: White Marsh 01585100), field reconnaissance upstream of the restored reach failed to identify a suitable stable site in reach C (the immediate supply reach). Despite this limitation, stable sections in reaches A and B were located and surveyed, the latter being in close proximity to a gauging station with 7 years of record (USGS gauge: NF Whitemarsh Run 01585095). In both cases, the surveys were undertaken at meander inflexion points, bulk samples of bed material were collected and slope was measured between successive riffle crests. All bed material samples were sieved by the authors at the University of Nottingham. Because of the lack of data necessary to directly undertake a magnitude-frequency analysis in reach C, an assumption had to be made in order to synthesise a flow frequency histogram, sediment discharge histogram and effective discharge for the purpose of channel design from the available flow data and stable cross sections in reaches A and B.

To examine the geomorphological significance of the tributary, bed material particle gradations in reaches A, B and C were compared (Figure 8.9; Table 8.3). Samples from reaches A and B were collected at the location of the surveyed cross sections, and the sample in reach C was obtained approximately 50 m downstream of the confluence. Flow distributions from the two gauging stations were also compared by calculating dimensionless flow duration curves using the median flow, Q_{50}, as the discharge denominator (Figure 8.10).

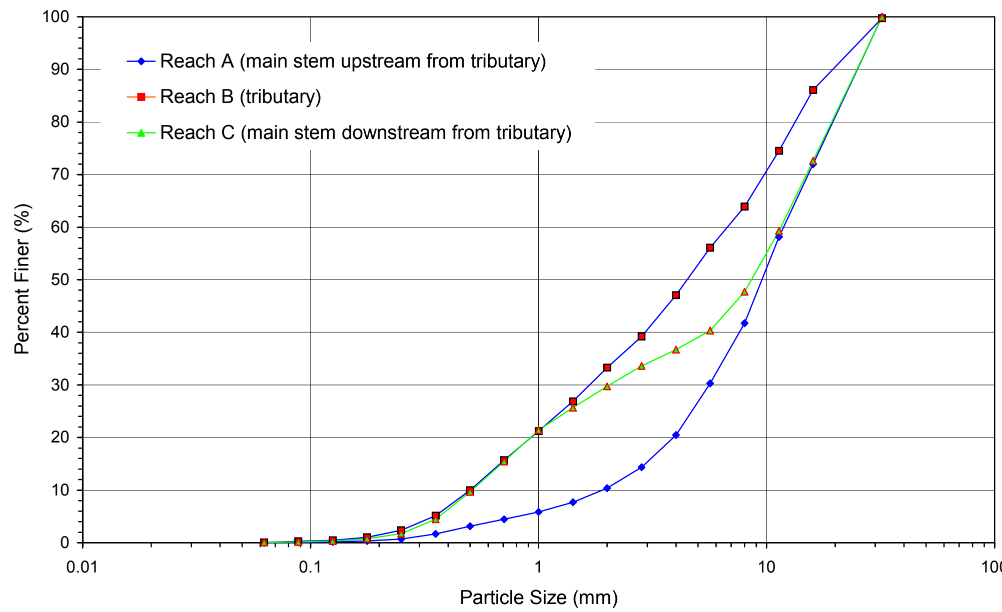


Figure 8.9 Bed material particle size analysis in Whitemarsh Run, Maryland.

	d_{16} (mm)	d_{50} (mm)	d_{84} (mm)	d_{90} (mm)	$\sigma = (d_{84} / d_{16})^{0.5}$	< 2mm (%)
Reach A	3.11	9.53	21.55	25.00	2.63	10.34
Reach B	0.72	4.48	15.03	19.51	4.57	33.28
Reach C	0.73	8.57	21.34	24.85	5.41	29.75

Note: d_x is the particle size for which x-percent is finer; d_{90} is included in the Meyer-Peter and Müller (1948) bed load equation used in magnitude-frequency analysis and analytical channel design; σ = geometric standard deviation of bed particle sizes.

Table 8.3 Particle size statistics of bed material in Whitemarsh Run, Maryland.

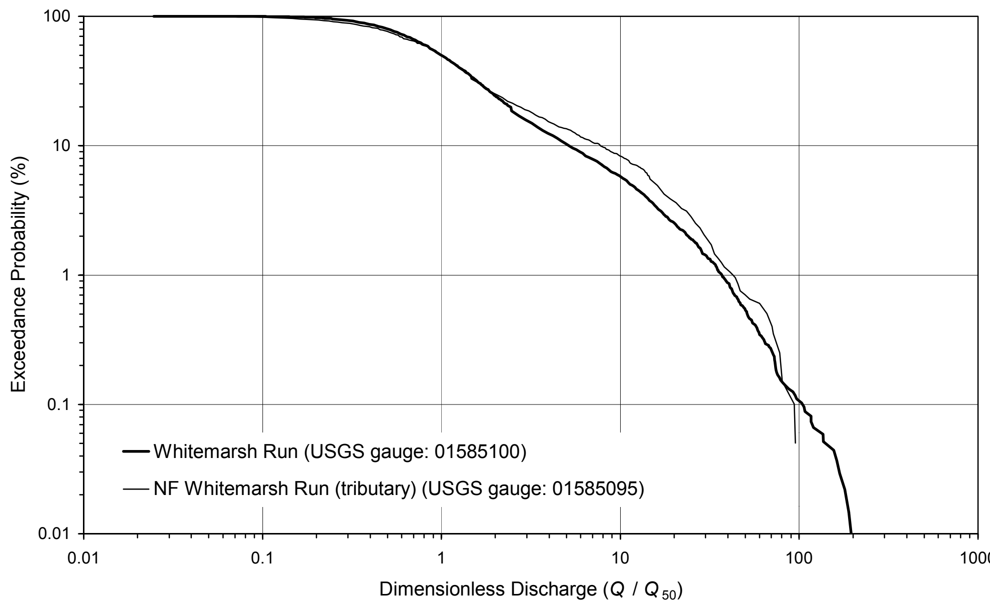


Figure 8.10 Dimensionless flow-duration curves in Whitemarsh Run, Maryland, and tributary upstream from restored reach.

The bed material gradations and summary statistics indicate that the sediment load in the tributary has a significant influence on the gradation of the bed material found below the confluence and, ultimately, in the restored reach. While the percentage of coarse gravel (greater than 10 mm) is constant along the main channel (reaches A and C), the percentage of sand and fine gravel increases below the confluence as a result of the tributary input. This is clarified in the differences between the sand fractions (material finer than 2 mm) in Table 8.3. Therefore, the source of the post-project sedimentation of sandy-gravel material in the restored reach is likely to be the tributary. In light of these findings, it was concluded that the tributary sediment input cannot be neglected from magnitude-frequency calculations.

The 'arithmetic manipulation of flow-duration curves' method (Chapter 4) was used to synthesise a sediment discharge histogram and effective discharge for the restored reach from available flow data and surveyed cross sections upstream of the tributary confluence. This technique is based on the assumption that the entire Whitemarsh Run catchment experiences the same rainfall-runoff events and, therefore, that the shape of the flow-duration curve is identical in all reaches A to E. Figure 8.10 shows that the dimensionless rating curve of the tributary is indeed very similar to the main channel, with intermediate dimensionless discharges only slightly greater in the tributary for the same exceedance probability. The error in the technique increases as the shape of the flow-duration curves differs because of their different dispersions. This is best addressed in terms of a 'flashiness' index. Two suitable indices are given in Table 8.4. Assuming a log-normal flow distribution, I_{F1} describes flow variability for one standard deviation and I_{F2} describes flow variability for two standard deviations. In both cases, the greater the value, the flashier the flow regime. Table 8.4 reveals that the tributary flows are slightly more 'flashy' than those in the main channel. This is probably attributable to the shorter length of the tributary channel and subsequent rapid flow response within an urban area. Flow 'flashiness' often increases as an inverse function of the catchment area. The drainage area recorded at the White Marsh gauge is 12.25 km^2, compared to only 2.15 km^2 at the tributary gauge. Differences between the dimensionless flow-duration curves may also be partly attributable to the different lengths of flow record between the two gauging stations.

Flow Duration Parameter		White Marsh (USGS: 01585100)	NF Whitemarsh Run (USGS: 01585095)
Minimum Recorded Flow (m^3s^{-1})		0.031	0.001
+ 2 Standard Deviations (m^3s^{-1})	$Q_{97\cdot7}$	0.26	0.025
+ 1 Standard Deviation (m^3s^{-1})	$Q_{84\cdot1}$	0.57	0.08
Geometric Median Flow (m^3s^{-1})	$Q_{50\cdot0}$	1.25	0.21
- 1 Standard Deviation (m^3s^{-1})	$Q_{15\cdot9}$	3.88	0.80
- 2 Standard Deviations (m^3s^{-1})	$Q_{2\cdot3}$	27.97	5.98
Maximum Recorded Flow (m^3s^{-1})		252.89	24.38
$I_{F1} = 0\cdot5\left[\frac{Q_{15\cdot9}}{Q_{50\cdot0}} + \frac{Q_{50\cdot0}}{Q_{84\cdot1}}\right]$		2.65	3.21
$I_{F1} = 0\cdot5\left[\frac{Q_{2\cdot3}}{Q_{50\cdot0}} + \frac{Q_{50\cdot0}}{Q_{97\cdot7}}\right]$		13.59	18.43

Table 8.4 Flow duration parameters and indices of flow flashiness, I_{F1} and I_{F2}, in Whitemarsh Run, Maryland, at White Marsh gauge (USGS 01585100) and in the North Fork tributary (USGS 01585095).

In the absence of measured load, the Meyer-Peter and Müller bed load equation (Meyer-Peter and Müller, 1948) was adopted to derive a sediment frequency histogram and determine the effective discharge. It was assumed that suspended sediment is a minor component of the total load and was not considered in sediment transport calculations. The Limerinos equation (Limerinos, 1970) was used to characterise the roughness of the bed and generally yielded a composite Manning n-value of approximately 0.03 for the bed portion of the channel. The channel banks were assigned a Manning n-value of 0.085, which is suitable for moderately dense scrub with some trees, based on guidance from Chow (1959). Initially, a flow frequency histogram was derived using 25 arithmetic class intervals ranging from the critical discharge for sediment transport to the maximum-recorded discharge. As the effective discharge could not be readily defined, the number of discharge classes was incrementally increased until a definitive peak in the sediment frequency histogram was produced, giving an effective discharge of 31.6 m^3s^{-1} with 30 arithmetic discharge classes (Figure 8.11). Essentially, the same effective discharge was determined using the quasi-event-based magnitude-frequency method with 500 arithmetic classes (Figure 8.12).

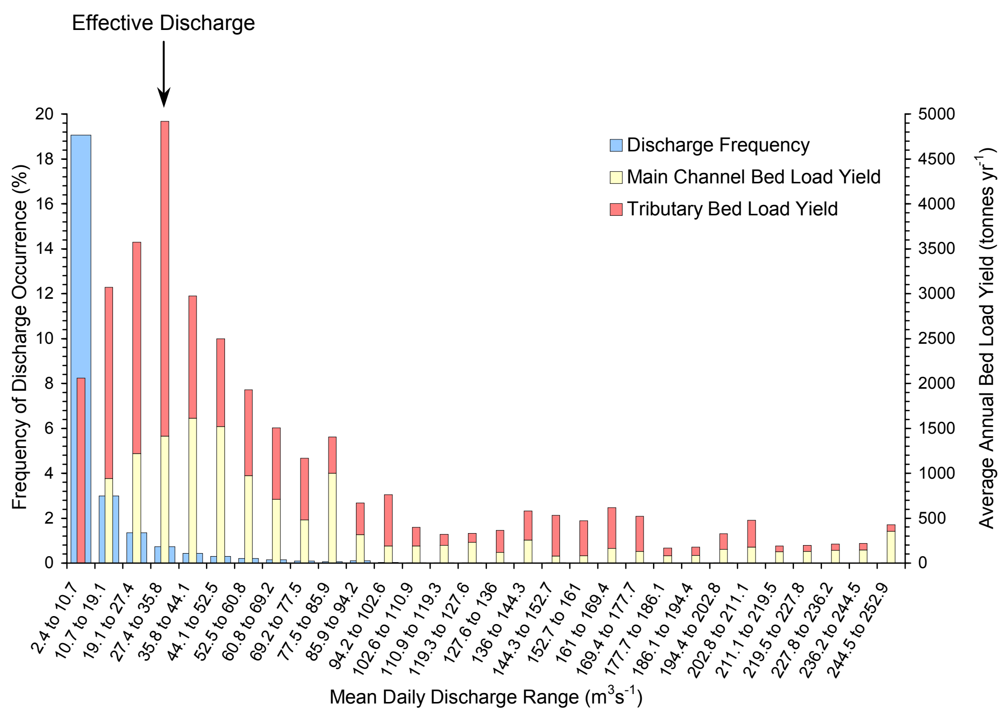


Figure 8.11 Effective discharge calculation for Whitemarsh Run, Maryland, based on the Flow Frequency Histogram method with 30 arithmetic discharge classes.

The sensitivity of the effective discharge to the number of discharge classes was examined using a range of examples up to 1000 classes and very little difference was observed for calculations based on 30 classes or more for this stream. An average value of 29.8 m^3s^{-1}, for calculations based on 30, 50, 100, 250, 500 and 1000, was taken as the effective

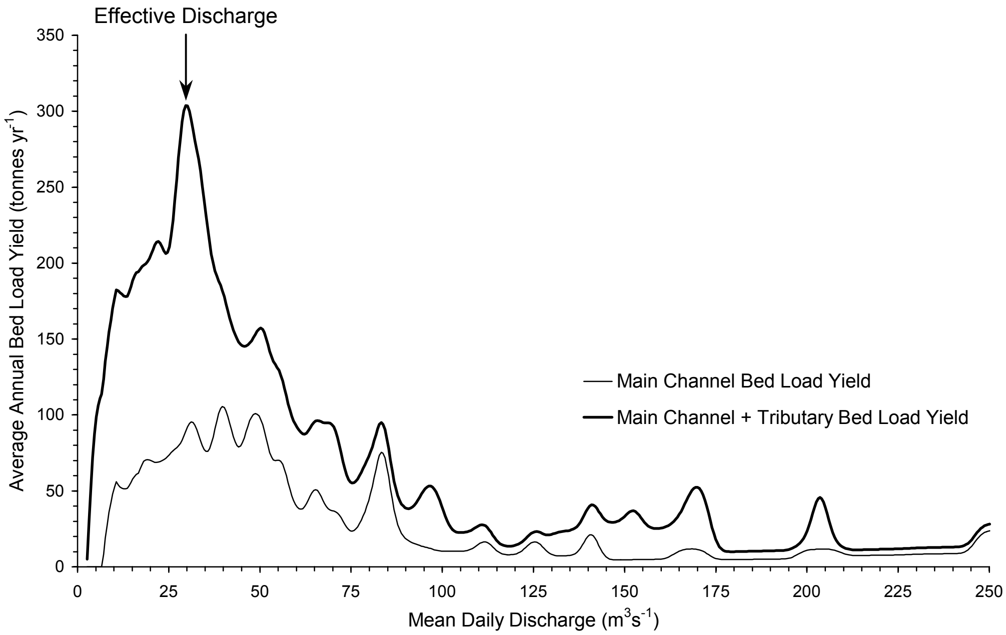


Figure 8.12 Effective discharge calculation for Whitemarsh Run, Maryland, based on the Quasi-Event-Based Magnitude-Frequency method with 500 arithmetic discharge classes.

discharge for channel restoration design. The recurrence interval of the effective discharge, using the annual maximum flow series, was 1.58 years. This is exactly equal to the most probable annual flood, which has been used as the bankfull discharge by Dury (1973, 1976) and approximates the 1.5-year flood that Hey (1975) found to be representative of bankfull discharge in U.K. gravel-bed rivers. These comparisons support the assumption that the calculated effective discharge is a reasonable channel-forming discharge in this case study.

8.5 SIMULATED CHANNEL RESTORATION DESIGN

The enhanced channel restoration design procedure was applied to calculate depth, slope and sinuosity at the effective discharge using the Meyer-Peter and Müller (1948) bed load equation and the Limerinos (1970) flow resistance equation, both suitable for gravel-bed rivers. In the absence of measured bed material load, these equations were used in both the Supply Reach Assessment and the analytical design stages of the procedure following the recommendation in Chapter 6 that using the same sediment transport equation will cancel any systematic errors produced in its application. Confidence bands for stable bankfull width were derived from modified Hey and Thorne (1986) hydraulic geometry equations suitable for gravel-bed rivers and banks with less than 50 percent tree/shrub cover (Chapter 5). Based on reconnaissance surveys of upstream reaches, the simulated design assumed moderately rough banks with a Manning n-value of 0.085 and side slopes of 1:1.

Figure 8.13 presents the results of the analytical design in terms of a stable width-slope curve (primary axis) and stable width-depth curve (secondary axis). Suitable design solutions are located within the 95 percent confidence band. The chart shows that the restored design slope of 0.0025 is much lower than the simulated range of slopes within the confidence band. This indicates that aggradation should be expected in the restored channel. The valley slope through the restored reach is estimated as 0.0043, giving the constructed restoration design a sinuosity of approximately 1.7, which is significantly greater than that calculated using the enhanced design procedure. This may explain the observed channel response through sedimentation in the meander bendways and channel straightening. Meander wavelength was estimated from the composite equation given in Chapter 7 (Equation 7.7) as a function of the range of stable widths within 95 percent confidence limits. Table 8.5 presents a summary of the restored and simulated design parameters.

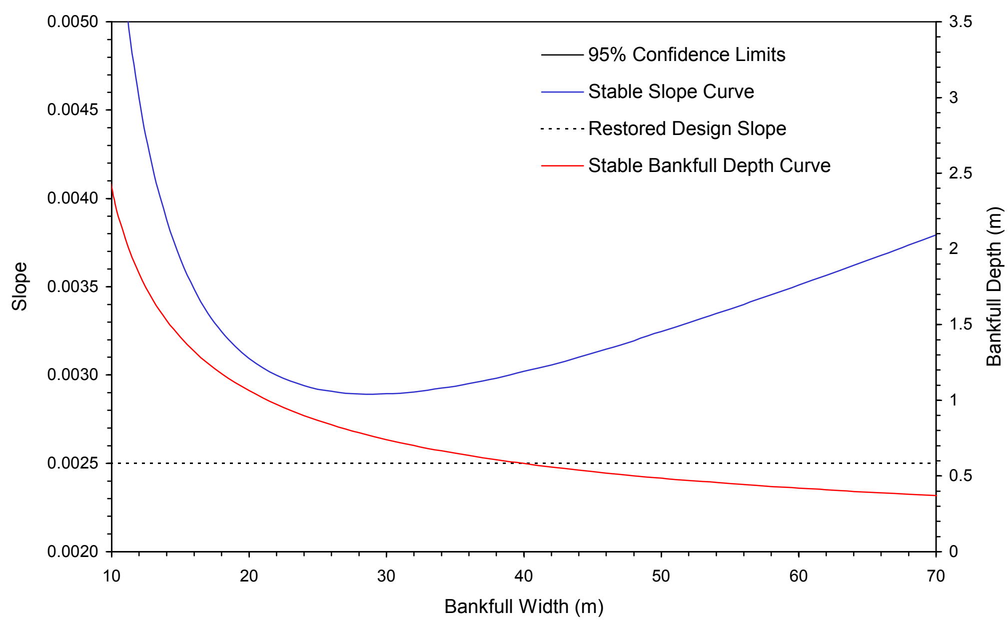


Figure 8.13 Stable width-depth-slope diagram for Whitemarsh Run, Maryland. The range of stable slope (primary axis) and depth (secondary axis) are shown between 95 percent confidence intervals of the mean response applied to the width equation, based on hydraulic geometry.

	Width (m)	Depth (m)	Slope	Sinuosity	Meander wavelength (m)
Actual Restoration	24.51	0.90*	0.0025	1.7	90
Enhanced Design	20.45 (19.19 to 21.80)	1.04 (0.98 to 1.11)	0.0031 (0.0030 to 0.0032)	1.40 (1.37 to 1.43)	242 (216 to 272)

Note: width and depth refer to the effective discharge elevation (*mean depth). Cross section parameters refer to a representative meander inflexion point. Values in parentheses are within 95 percent confidence limits of the mean response.

Table 8.5 Comparison of the restored channel geometry of Whitemarsh Run, Maryland, with the simulated design geometry calculated from the enhanced design procedure.

Interestingly, the stable slopes within the confidence band in Figure 8.13 have values very close to the minimum slope, corresponding to the minimum stream power extremal hypothesis. Assuming a sine-generated curve meander path, the simulated design has a planform geometry with significantly larger meander bends than those that were constructed, both in terms of wavelength and amplitude, despite having a lower sinuosity. The observed pattern of post-project channel change, with several pools choked with sediment and others subjected to impinging flow, is consistent with the notion that the restored channel is evolving toward a more stable configuration that corresponds to the design parameters produced in this analysis.

8.6 SEDIMENT IMPACT ASSESSMENT

To attain geomorphic stability, the mean annual sediment load for the restored channel (capacity) must match the mean annual sediment load input from the supply reach (supply). In Chapter 3, the Capacity-Supply Ratio (CSR), was defined as the bed material load transported *through* the restored reach by the natural sequence of flow events over an extended time period divided by the bed-material load transported *into* the restored reach by the same flow events over the same time period. The ratio was used in this case study to evaluate the success of the actual restored channel and that simulated from the enhanced design procedure. Comparison of sediment supply and capacity for the actual restored design revealed a CSR of only 0.64 (Figure 8.14), indicating that the restored

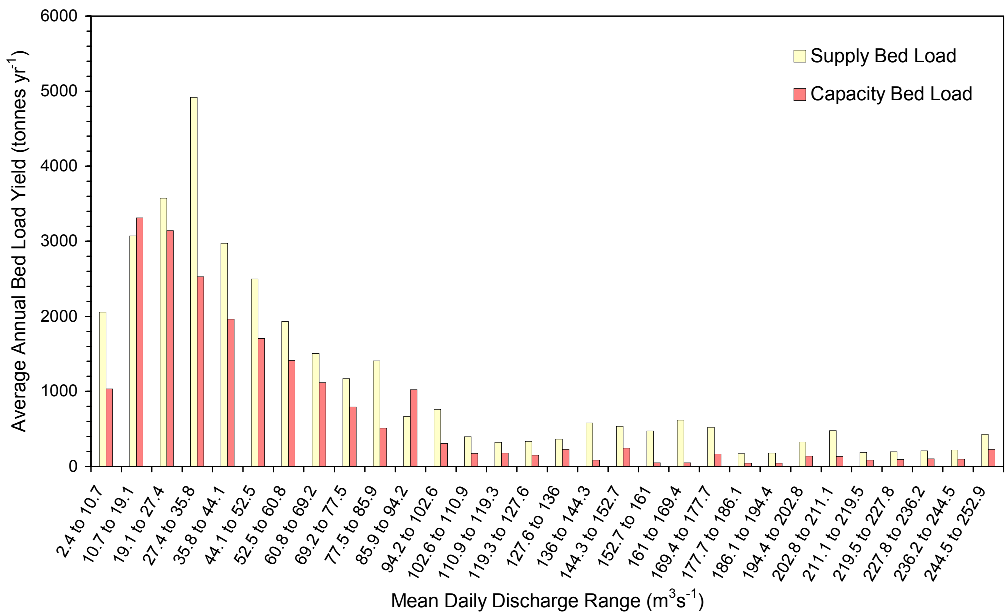


Figure 8.14 Comparison of sediment supply and capacity for the actual restored channel design based on 30 arithmetic discharge classes. Total capacity yield / total supply yield (CSR) is 0.64. The minimum discharge is the critical discharge for sediment transport in the supply reach.

channel has the potential for approximately 36 percent of the input load to be deposited in the restored reach over the medium- to long-term. This result is consistent with the observed aggradation in the restored meander bends. The CSR for the initial simulated design is 0.90, using mean values of width, depth and slope within the 95 percent confidence band (Table 8.5). This is within the recommended 10 percent deviation from unity, considered sufficient to prompt the channel to attain a stable configuration. Increasing the slope from 0.00307 to 0.00324 is all that is required to yield a CSR value of exactly 1.0, giving a stable sinuosity of 1.33 (Figure 8.15).

It is important to note that the CSR parameter is derived from a simple one-dimensional technique and is based on the *total* bed material load transported. Disparities between supply and capacity within the sediment frequency histograms show potential for some minor channel change and event-driven fluctuations in the short-term which is not unexpected in a meandering river. These minor adjustments reflect the natural, dynamic nature of stable river channels that is encouraged in the geomorphic engineering approach and are, therefore, desirable outcomes of the enhanced design framework.

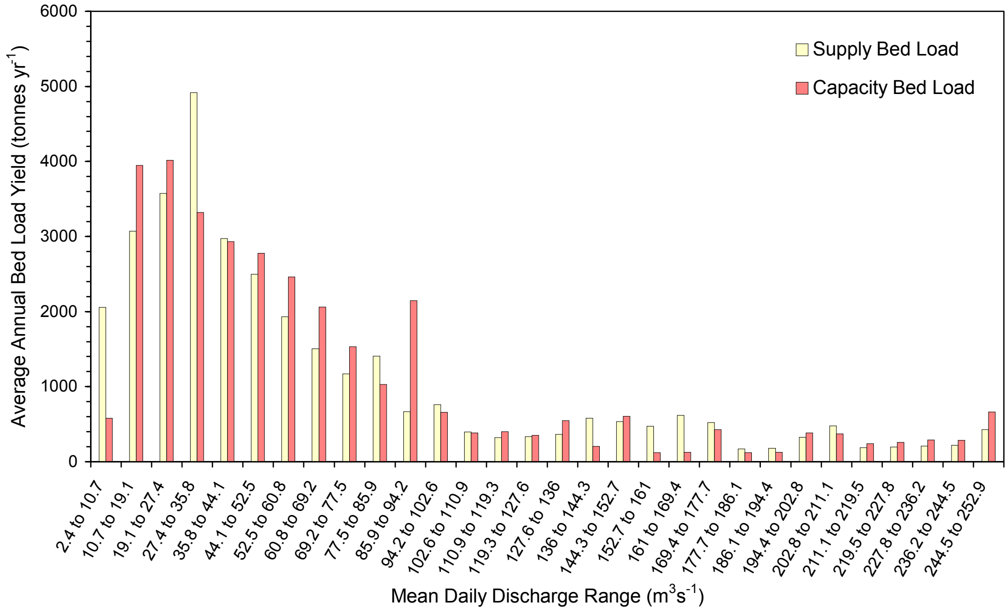


Figure 8.15 Comparison of sediment supply and capacity for the enhanced design based on 30 arithmetic discharge classes and increased slope of 0.00324. Total capacity yield / total supply yield (CSR) is 1.0. The minimum discharge is the critical discharge for sediment transport in the supply reach.

The sediment capacity-supply analysis confirms that the alignment of the constructed channel was too sinuous for the prevailing flow regime and sediment supply to be a stable configuration. However, despite the calculated sinuosity of 1.33, observed sinuosities at stable sites further upstream are generally in the range 1.1 to 1.2. This suggests that the design procedure may have underestimated the total input sediment load. This could be partially a function of the design equations currently available in the analytical method for calculating depth and slope. The Meyer-Peter and Müller bed load equation by definition does not account for suspended load, however the particle gradations indicated that approximately a third of the bed material is composed of sand or finer sediment. Furthermore the bed load equation assumes a critical Shields parameter of 0.047, which is a conservative estimate for gravel-bed rivers, with values reported as low as 0.02 by Andrews (1983). Consequently, sediment transport during low-flow events may have been underestimated. In light of these considerations, it is recommended that the analytical method should be developed to account for total sediment load in gravel-bed rivers. The analytical method is in development by the authors at the University of Nottingham with the objective of incorporating a range of flow resistance and sediment transport equations to improve applicability.

8.7 SUMMARY: PERFORMANCE AND OPERATION TESTING

In this case study the enhanced design procedure was used to assess the reasons for instability in an existing restoration scheme and identify the degree to which post-project adjustments in the restored reach were driving evolution toward the stable form predicted by the design procedure. While this is a useful exercise, only a real application of the design procedure will fully test its performance. However, based on the observed channel change in Whitemarsh Run since restoration in 1996, it has been shown that the simulated restored channel would have been more stable than the actual restored channel as the direction of recovery appears to favour the output dimensions from the enhanced procedure. In this respect the results of the beta-testing are encouraging.

As this case study is essentially a stability assessment only the basic elements of the enhanced design procedure were applied to test this objective. In an actual project application, the planform relationships derived in Chapter 7, including equations for natural variability around meander bendways, would be applied to specify the full range of design parameters required in the Design Brief.

In terms of performance, the effective discharge has a recurrence interval of 1.58 years which has been shown by several researchers to correspond to the bankfull discharge. This is an encouraging result and suggests that the variability of the flow regime in this case study does not significantly affect the ratio of bankfull to effective discharges as found in Chapter 5 for sand-bed rivers. As the channel-forming discharge is the main process-driver in channel restoration design, further research into the difference between bankfull and effective discharges in different types of river is required to substantiate the initial findings in this report.

The main operational difficulties whilst applying the procedure were encountered during the Supply Reach Assessment. Identifying stable reaches in a small, generally urbanised catchment was not a straightforward task and required an extensive reconnaissance study to assess the geomorphological status of the catchment. The effective discharge was considered to be a more objective measure of the channel-forming flow as bankfull discharge was not readily identifiable from field indicators in the reference reaches. This was not unexpected as many reaches exhibit some evidence of previous incision.

However, calculation of the effective discharge was more complicated than the practical guidance flowcharts (Figures 4.10 to 4.12) derived in Chapter 4 suggest, because of the significant tributary input immediately upstream from the restored reach, even though two gauging stations are near the restored reach. In light of this, and because all design parameters are ultimately related to the 'channel-forming flow', the Supply Reach Assessment is considered to be the critical stage in the design procedure and where further research should be directed if the procedure is to become widely applicable.

In the next chapter, the salient findings from the case study application and the development of the design stages in preceding chapters are discussed in the context of the channel restoration design framework presented in Chapter 3. In this re-examination, the most important stages in the procedure are identified and the geomorphic engineering approach that was proposed in the Introduction is presented as a central platform for restoring river channels and accounting for natural systems variability, while meeting multifunctional goals of channel stability and low maintenance commitments. The most important element of the design framework is considered to be the selection of the channel-forming discharge during the Supply Reach Assessment, although considerable uncertainty remains as to the morphological significance of the effective discharge. Despite this uncertainty, the design framework presented in this report is considered to be appropriate technology for river restoration.

CHAPTER 9

Conclusions and Recommendations

9.1 INTRODUCTION

In this closing chapter, the channel restoration design framework developed in Chapter 3 is re-examined in the context of the discussion and developmental research on the sequence of design stages reported in Chapters 4 to 7 and in light of experience gained through practical application outlined in Chapter 8. This re-examination deals specifically with four fundamental issues identified at the outset of this study: i) the argument that the adoption of a geomorphic engineering approach is central to meeting modern, multifunctional objectives in river management and addressing the causative problems inherent to unstable and degraded rivers; ii) assessment and rankings of the importance of the various components that constitute the design framework, iii) a critical review and re-evaluation of the conceptual basis for the main geomorphological process driver: the channel-forming discharge, its calculation and its representation in the design framework, and; iv) concluding remarks on the validity of the design framework as an appropriate platform for generating realistic restoration design solutions that mimic the natural channel morphologies and environmental attributes in undisturbed systems, while meeting multifunctional goals of channel stability and low maintenance commitments.

9.2 THE CASE FOR GEOMORPHIC ENGINEERING

The geomorphic engineering approach to river restoration is an environmentally aligned approach with objectives usually coupling fluvial geomorphology with habitat biodiversity, on the basis that the morphological diversity produced by a dynamically stable river provides a sustained and diverse range of physical habitats (Environment Agency, 1998). If channel restoration design is performed without reference to issues of fluvial geomorphology, then it is unlikely that channel dimensions will be stable in the medium- to long-term. In such cases any environmental or aesthetic benefits will be temporary unless a programme of channel maintenance is invoked to defend them and this renders the results, in essence, unsustainable.

In Chapter 1 a case was presented for river restoration as an appropriate management solution to the various problems of channelisation, evident through: i) channel instability; ii) low ecological diversity, iii) downstream flooding; iv) poor aesthetics and recreation; v) impeded recovery, and; vi) unsustainable maintenance. As these attributes may be used to define project objectives and success criteria in a restoration scheme, it is important that they are re-evaluated in the context of the geomorphic engineering approach to channel restoration design adopted here and the associated design framework.

i) ***Channel Instability***

Re-establishing equilibrium between the sediment supply and available transport capacity in the restored reach is the primary objective of the design framework. Removing sediment imbalance is accomplished through consideration of the fluvial system around the restored reach as consisting of three, interlinked units: the upstream supply reach (or reaches) which defines the sediment input; the restored reach, where channel dimensions and slope must be designed to transport the input sediment load with negligible erosion or sedimentation in the medium- to long-term, and; the downstream reach, which has a specific sediment load demand that must be met by output from the restored reach. On this basis, design variables representing the character of the flow and sediment regimes are defined for the upstream supply reach(es). The initial restoration design is based on a single discharge: the geomorphologically important channel-forming flow (Section 9.4). This design is then examined against the complete range of sediment-transporting flows through a sediment impact assessment and calculation of the CSR. The CSR is essentially a measure of net deviation from medium- to long-term stability and is calculated at the end of the design procedure to indicate whether any small adjustments to the designed channel morphology or slope are necessary to match capacity to supply and so ensure sustainability of the restored morphology. By this means, a quality assurance loop is closed. Specifically, the actual distribution of flows is used to define a representative design discharge, which becomes the primary independent variable for calculating stable channel dimensions. The efficacy of the restored channel configuration in transmitting the actual sediment supplied from upstream is then checked against the same flow distribution and the design is adjusted to remove any imbalance. Hence, sediment continuity forms both a basis for the analytical channel design method and, through this closure loop, the primary criterion to evaluate the potential morphological sustainability of the restored channel.

Calculation of channel width, based on typed hydraulic geometry equations does not explicitly account for sediment discharge. However, the simultaneous solution of sediment transport and flow resistance equations to derive stable values of depth, slope and sinuosity (given the valley slope) is, fundamentally, a stability assessment based on sediment continuity principles. Indeed, the Copeland analytical method for sand-bed streams (Copeland, 1994) was developed as an objective measure of channel instability that could provide the design engineer with an insight into how the depth, slope and width of a disturbed stream deviate from a stable, regime condition, in terms of aggradation or degradation.

The design framework is therefore centred on sediment continuity, and this potentially limits its applicability to disturbed catchments with system-wide instability. This is the case because the method relies on the assumption that the supply reach(es), from which input parameters are derived, is quasi-stable in terms of equilibrium sediment conveyance. If the input reach cannot meet this requirement, the restored reach might not be stable without an artificial control of sediment transport at the head of the reach, such as a sediment trap or weir. As river restoration is most relevant in disturbed catchments, the applicability of the design procedure is restricted in some cases without additional practical guidance to attenuate upstream and/or downstream channel instability, where necessary. Channel rehabilitation methods could be employed to tackle the instability based on a combination of grade control structures, designed and sited according to regional stability relationships that predict stable slope, and bank stabilisation. Appropriate guidance for employing these channel rehabilitation methods has been developed by Watson et al. (1999) through the Demonstration Erosion Control (DEC) Project, which addresses channel instability in the Yazoo River basin of the Lower Mississippi Valley.

The main conceptual limitation of the design approach developed in this report is that channel stability is assessed using a simple, trapezoidal channel geometry and uniform bed sediment. The Copeland analytical method was developed for straight channels but it has been applied here to meander inflexion points. This may be acceptable because inflexion points exhibit the most uniform cross-sectional topography found in meandering rivers. In a one-dimensional approach that does not account for secondary flows and sediment transport around bendways, this was an essential bridging-principle in the design procedure. This simplicity, though, is also a strength in the approach as it can be used in low-cost schemes where hydrodynamic and morphological modelling are unfeasible. Given the assumption that reach-averaged channel geometry could represent the morphology at the inflexion point, it was imperative to include guidance on the location and maximum scour depth of meander pools and width variability around bendways to ensure that bend flow is fully developed in the restored channels and large quantities of sediment are not mobilised from the pools during potentially destabilising flood flows that could occur during the initial adjustment phase post-construction. Using a geomorphic engineering approach to designing local morphological variability into the reach-average channel mould is also essential for providing a range of physical habitats and addressing the eco-hydraulic objectives of a restoration project.

ii) ***Low Ecological Diversity***

In ecologically impoverished streams, the objective of restoration is usually to restore a diverse range of aquatic fauna, or target species assemblage, through the provision of physical habitats. In the geomorphic engineering approach, the types and levels of habitat diversity that are sustainable in the restoration reach are defined by the type of river and the prevailing catchment context. The approach recognises that constructed habitats often constitute form without function, geomorphologically, and they will not be permanent without maintenance, which makes them unsustainable. Since ecological recovery usually follows morphological recovery, the objective, therefore, is to provide a range of habitats that is appropriate to the morphological type of river and catchment setting, defined by the nature of the sediment supply. Approaching the problem though sediment transport considerations is vital if the restored channel is to support the characteristic assemblage of bars, pools, riffles, and eroding banks in a meandering channel that have important ecological value and can eliminate the ‘bottlenecks’ in the life-stages of the aquatic species that are apparent in channelised or moribund rivers. Imitating natural systems variability, through confidence bands and morphological equations for width variability around bendways, appropriate siting of pools and specification of pool depths are essential components of the geomorphic engineering approach to river restoration developed here and vital to support a diverse range of physical habitats.

iii) ***Downstream Flooding***

Simple channel enlargement, through channelisation, is unsustainable morphologically and transmits the flood problem downstream through rapid transition of unattenuated flood peaks. The geomorphic engineering approach to channel restoration design involves designing a stable reach-average sinuosity based on sediment transport considerations. Increasing the sinuosity in this way attenuates the flood peak in the same way as occurs in nature. To imitate the hydrological characteristics of natural rivers, though, requires a coupled meandering channel-floodplain system. Therefore, the design framework is most applicable to streams with wide floodplains that experience over-bank flows at stages above the bankfull elevation. This is desirable from an ecological perspective, as frequent flooding is important to sustain marginal and wetland habitats. However, in most restoration schemes, there is only a narrow, or often absent, floodway with infrastructure on the floodplain, and flood protection is imperative. In these cases, a compound channel with set-back embankments is the best way to retain the function of the reach in providing downstream flood protection.

This compound channel design involves a smaller, inner channel having the attributes of a natural, regime channel, and a larger channel surrounding it and designed to convey flood flows without causing property damage on the surrounding floodplain. This type of configuration is difficult to design analytically because of the significant shear stresses at the interface between over-bank flow and main channel flow resulting from lateral momentum exchange. To account for these energy losses in over-bank flow calculations using an empirical approach, further research should be directed at the development of an empirical roughness multiplier applied to the composite Manning *n*-value of the main channel. These advances should facilitate in the restoration of more natural cross sections and better integrate the objectives of flood control into river restoration projects.

iv) ***Poor Aesthetics and Recreation***

It is generally perceived that engineered channels have poor aesthetic appeal to the public and recreational amenity value is poor. Recreation functions are growing throughout the western world and 'users' of the river landscape demand greater access to the river and high quality resources for hiking, biking, fisheries, boating, etc. In environmental agencies, these functions now match flood defence and land drainage in importance. Through a geomorphic engineering approach, restoration of the environmental attributes that are characteristic of meandering rivers can significantly improve the visual quality of the riverine landscape. This is particularly relevant in urbanised catchments where the natural river is an important recreational resource. With an objective to imitate natural systems, the channel design framework developed here provides a starting point for addressing these wider issues.

v) ***Impeded Natural Recovery***

Theoretically, cessation of maintenance coupled with removal or redesign of engineering structures in a degraded river should allow the river to recover provided that there is sufficient energy in the system to drive channel change. The design framework recognises that the river is ultimately the best restorer of its natural morphology and should be allowed to participate in its own recovery. This is accomplished through designing an approximate channel mould based on the broad dimensions of the river (width, mean depth, slope, sinuosity, meander wavelength, etc) and then allowing the river itself to develop the intricate cross-sectional detail and intra-reach morphological features during the later stages of the recovery process. The channel design should be close enough to a stable geometric configuration in terms of sediment conveyance as to limit the extent of post-project channel change, but should not be so detailed as to generate manufactured precision, given that only a one-dimensional approach is used and that every stable river is to some degree unique. The wide confidence limits on a single response applied to downstream hydraulic geometry equations exemplify the wide ranging channel dimensions actually found in natural channels. Undertaking a sediment impact assessment based on magnitude-frequency analysis and calculation of the CSR are critical in indicating the likely degree of post-project adjustments, both in the short-term through a comparison of sediment supply and transporting capacity driven by individual flows in the flow-frequency distribution and in the medium- to long-term if the CSR deviates markedly from unity. However, from a geomorphic engineering perspective, short-term and local morphological adjustments are not only acceptable, they are welcome. Provided that the CSR is close to unity, such small-scale or short-term imbalances are unlikely to trigger channel evolution to irreversible change.

vi) ***Unsustainable Maintenance***

A restored channel is economic if it meets multifunctional objectives and requires only a minimum maintenance commitment. As the design framework is based on sediment continuity principles, it follows that the restored channel should be self-sustaining in terms of aggradation and degradation and post-project control of sediment yields should be minimised. This is a major requirement of engineered channels and allows emphasis to shift from resource-demanding maintenance commitments to inexpensive monitoring programmes. Thorne et al. (1997) presented a case for geomorphologically sound river engineering on the basis of four categories that must be addressed if schemes, such as restoration projects, are to be successful:

i) Engineering
ii) Economy
iii) Efficiency
iv) Environment

The channel restoration design framework is based on bringing together geomorphological principles of river management and conventional engineering methods. It is accepted that 'designing with natural processes' is the preferred engineering management strategy rather than attempting to 'tame' rivers. Incorporation of natural variability, a catchment-based approach to sediment continuity and allowing the river to fine-tune the restored channel design are all principles embodied in the designing with nature approach and central to the design framework. The geomorphic engineering approach is essentially an environmentally aligned approach and, thus, cost-effective with the intangible benefits of biodiversity, aesthetics etc. as described above. Based on the requirement for sediment continuity, theoretically the design procedure produces efficient channels, whereby efficiency over the entire range of sediment-transporting flows is examined through the sediment impact assessment at the end of the procedure and used to indicate appropriate maintenance if deemed necessary. Finally, the environmental benefits of the design procedure naturally follow from the matching of the supply load with the sediment-transporting capacity which promotes stable physical habitats and, therefore, a high ecological carrying capacity.

9.3 ASSESSMENT AND RANKING OF COMPONENTS IN THE APPROACH

The best practice design procedure requires the application of a range of different techniques including: field reconnaissance; stream classification; detailed site survey; magnitude-frequency analysis; analytical solution of non-linear equations; hydraulic geometry analysis, and; sound professional judgement throughout. Detailed reviews of existing approaches have shown that all the components of the design approach have different types of limitations. In bringing together the various techniques to form a coherent design framework, it has been an underlying principle to exploit the strengths of each component, thereby overcoming, to some degree, their individual limitations.

The fundamental problem in river engineering is indeterminacy in the estimation of stable channel dimensions. Therefore, the challenge to the restoration designer is to simulate 'real' systems in the absence of advanced equations accurately describing complex relationships between river form and process. Geomorphic engineering, as it is defined in this report, provides a practical solution by striking a balance between empirical-statistical and analytical (process-based) methods. In the design framework, the geomorphological method of downstream hydraulic geometry and associated confidence bands provided the basis from which to derive a series of enhanced width equations, with varying predictive capabilities, that facilitate a realistic solution to the indeterminacy problem that is unachievable using analytical methods alone.

New width equations have been derived for different 'types' of sand-bed and gravel-bed rivers. Stream classification, at this very broad level, is essential to guide the design engineer toward the most appropriate morphological equations. Given that hydraulic geometry was devised almost half a century ago, it was envisaged at the outset of this study that there would be a very extensive database of existing regime-type data and an associated wide-range of different stream types, each with their own set of hydraulic geometry relationships. However, two of the immediate findings were that there is a paucity of data from sand-bed rivers, despite analytical approaches for sand-bed channels being well-developed, and that the most appropriate stream typing system was a broad level categorisation based on bed and bank sediment and vegetation characteristics. The collection of new data from U.S. streams with sand-beds has provided a useful starting point for extending the database of stable river data and through the derivation of new width equations (even though based on calculated rather than measured bankfull discharge) has increased the range of applicability of the design method. However, as an empirical width equation provides important input data to the analytical derivation of depth, slope and sinuosity, it is recommended that further research builds on the regime database and the enhanced width equations that have been compiled here. This will require a carefully planned fieldwork programme targeted at specific stream types. For example, further data are required from U.S. gravel-bed streams with different types of bank characteristics from which width equations could be derived that would complement those available for U.K. streams based on the Hey and Thorne (1986) and Charlton et al. (1978) data sets. As new and improved equations become available, the applicability of the design procedure will improve.

The Copeland analytical method is the only component of the design procedure that must be based on a computer program, as it incorporates complicated iteration routines. However, the method is not data intensive, is simple to operate and can be applied routinely following reconnaissance investigations. The method for sand-bed streams has provided a methodological template that is capable of incorporating other sediment transport and flow resistance functions, thereby giving the method greater applicability. In this report, the method has been extended to gravel-bed rivers that are bed load dominated and has been applied effectively to a river restoration case study in Maryland. Further research should be directed toward giving end-users greater flexibility in choosing suitable design equations for particular types of stream. In particular, the method should be enhanced to account for mixed-bed streams where both bed load and suspended load are important components of the bed material load. Also, as the analytical method has not been widely tested, further case studies should be used to evaluate the method and support in its continued refinement.

To the geomorphologist who is interested in spatial and temporal scales of river channel change, the analytical component of the design framework facilitates examination of the sensitivity of the design variables to future trends in catchment runoff and sediment transport patterns which control channel stability/instability. This flexibility can give the designer useful insights into the potential useful life span of the project and can assist in recommending appropriate and economic levels of post-project maintenance and monitoring, and assess the need to rehabilitate the system or isolate the restored reach using appropriate technology (e.g. Watson et al., 1999).

Statistical confidence bands have provided a mechanism through which natural rivers can be used as realistic analogues for channel restoration design. Hydraulic geometry equations are limited in that they give deterministic solutions that are statistically improbable. However, hydraulic geometry concepts have been applied here as a basis to develop enhanced width equations that account for the natural variability portrayed by the scatter of data points about the best-fit equations. The advantage of using confidence bands to account for uncertainty in estimates is that natural variability can be described objectively without the need to understand fully the complex networks of causal mechanisms controlling that variability, which are poorly understood. Furthermore, they provide a mechanism of incorporating natural variability into engineering design drawings.

Through its development and application, the underpinning geomorphological method in the design framework was magnitude-frequency analysis (MFA). Even though considerable advances have been made during this study, the relationship between 'bankfull', 'effective' and 'channel-forming' discharge remains equivocal. In the approach adopted here, the channel-forming discharge is the main fluvial driver that is used to assess channel stability and derive stable channel dimensions. The salient findings and remaining uncertainties concerning the channel-forming discharge and remaining uncertainties are discussed in Section 9.4.

From the outset of this study, the primary objective has remained to derive a practical procedure for deriving the stable dimensions appropriate in restoring meandering rivers. During the development of the individual design stages, it became increasingly clear that a definitive, generic, procedure with step-by-step guidance could not be produced, as there remain several areas where further research is necessary and professional experience and judgement cannot be completely substituted by a 'cookbook' method. Consequently, a design *framework* more suitably describes the outcome of this research, which should be used to guide end-users working on a specific project (with its unique objectives, catchment context and system conditions) toward a specific procedure (or procedures) appropriate to that project.

9.4 CHANNEL-FORMING DISCHARGE OR DISCHARGES?

The channel-forming discharge is the overriding design parameter and important, either directly or indirectly, though its relationship with bankfull width, in all of the preceding chapters. Throughout this report and especially in the case study presented in Chapter 8, it became apparent that the most critical channel design phase of the framework is the Supply Reach Assessment, during which the main input design variables are determined. This section summarises the salient findings concerning the channel-forming discharge and how they have been incorporated into the design framework.

Initially it was assumed that, while bankfull discharge is the desired design discharge, its measurement is generally problematic and subjective. The preferred alternative measure of the channel-forming flow is the effective discharge that is considered to be an objective flow with morphological significance as it accounts for sediment transport. The flow frequency method was considered to be best practice in this regard and so detailed practical guidance for its computation was presented. A significant limitation of the procedure, however, is the assumption that flow data are available to use in the calculations. In practice, it is more likely that data are unreliable because the period of record is too short or unavailable, because the project site is not close to a gauging station. In these circumstances, alternative methods are required to simulate flow frequency data. While several techniques are available, they have not been widely applied or tested. Further research is necessary to develop these methods, together with guidance on their application to restoration projects.

The best practice procedure presented in Chapter 4 is a ‘class-based’ approach, whereby discharge is represented by a series of arithmetic class intervals. The selected class interval should be small enough to accurately represent the frequency distribution of flows, but large enough to produce a continuous smooth distribution, with no classes having a frequency of zero. This usually demands several attempts at calculating the effective discharge until a satisfactory result is produced. However, the effective discharge calculated this way varies with class size and is therefore less objective than was initially presumed. To alleviate the subjectivity introduced by class size selection, a ‘quasi-event-based’ approach has been developed based on very small class sizes and extracting the general trend of sediment frequency distribution by using moving average smoothing.

This recognises the overall variability and episodic nature of individual sediment transporting events. The technique requires further research, development and testing but is considered to have potential to be applied in many geomorphological studies that require an examination of the distribution of sediment transporting flows.

Since this study is concerned with channel restoration design rather than magnitude-frequency analysis, *per se*, the discussion of the channel-forming discharge was initially limited to a single chapter. However, research did not progress sequentially with the design stages. The final piece of research undertaken was the analysis of the sand-bed data discussed in Chapter 5, as the fieldwork programme extended from 1998 to 1999. While the primary objective of the data collection was to develop enhanced width equations for sand-bed streams, the results questioned the assumption that the effective discharge can be equated with the bankfull discharge.

As a general rule in sand-bed rivers, it was found that the mean annual discharge and the bankfull discharge form lower and upper bounds respectively to the range of effective discharge, while the 2-year recurrence interval flow is an upper bound to the range of bankfull discharge. These findings present a potential dilemma to geomorphologists and river engineers trying to define a channel-forming flow, as the effective discharge in sand-bed streams only corresponds to the bankfull discharge in certain cases. In fact, from a numerical basis, it was shown that the peak in the sediment histogram (effective discharge) cannot correspond to the upper break point in the cumulative sediment curve (bankfull discharge). Analysis of the U.S. data showed that both sediment rating and flow variability are important influences on the magnitude and variability of the ratio between bankfull and effective discharges. Tentative design equations were derived for calculating bankfull discharge as a function of the effective discharge and either: i) the ratio of 2-year flow to the mean annual discharge, or; ii) the percentage of the long-term sediment load transported by discharges not exceeding the effective discharge. The latter relationship was able to account for 80 percent of the variance in the ratio of bankfull to effective discharges in the sand-bed data set. However, in gravel-bed rivers, existing research has shown that in many cases the bankfull discharge and effective discharge are equivalent flows. This is further corroborated by the effective discharge calculated in the Maryland case study which corresponded to the most probable flood, as this has been shown by Dury (1973, 1976) to correspond to the bankfull discharge.

Despite the progress made in this study, considerable uncertainty remains concerning the morphological significance of the effective discharge, how it relates to the bankfull discharge in different types of streams and its utility as a design discharge for river restoration. To resolve these problems, further research should be directed at answering these questions. This should be possible now that a standardised procedure for calculating the effective discharge is available and with the continued development of event-based approaches to magnitude-frequency analysis. The effective discharge involves time-event compression, which represents a time series by a unique flow event with a unique frequency. However, this assumption does not adequately account for the channel-forming capabilities of other flows experienced by the river. It is recommended that further investigations should focus on identifying a *range* of effective flows that is causally linked to channel morphology and instream sedimentary features. It is envisaged that using cumulative sediment curves, derived from magnitude-frequency analyses, will provide an appropriate starting point to meet this objective.

9.5 APPROPRIATE TECHNOLOGY

Channel restoration design methods should be practical and feasible to apply and auditable if they are to be useful. Complex analytical approaches based on hydrodynamic and morphological models are too costly, data intensive and require advanced modelling skills to be routine tools for river management. Even if detailed channel geometries could be derived using advanced, process-based methods, it is unlikely that they could be constructed by earth moving machinery to the specifications in the design drawings. Channel typing and use of hydraulic geometry methods alone are too simplistic, since they do not account for sediment discharge continuity and give an illusion of applicability that leads to misuse. In light of these considerations, this report has presented a framework for channel restoration design that attempts to bridge the divide between reconnaissance level geomorphological designs at one extreme and numerical modelling of hydrodynamics, sediment transport and morphological change at the other.

The river is a complex system but must be represented in a simplified form in order to approach river restoration from a physical basis using the existing knowledge base of river mechanics. This may be initially represented by a set of broad channel dimensions at the reach scale that are relative to a specific design discharge, although natural river morphology is very detailed and shaped by a wide-range of interrelated process drivers, site specific issues and environmental controls, that are not fully understood by river engineers and fluvial geomorphologists alike. Confidence bands provide an objective measure of natural variability in channel dimensions but do not provide casual explanations for the observed ranges of dimensions, forms and features found in natural, stable rivers with similar boundary conditions. On the basis of these considerations, the simplified approach provides the appropriate technology for river restoration provided that it can be accepted that some degree of post-project channel change is inevitable as the river adjusts to accommodate the design. This further highlights the value of the sediment impact assessment which can indicate the potential for short-term channel changes.

The approach presented in this report is not a 'cookbook' procedure for all river restoration schemes but is a framework within which the sound judgement of practitioners with experience in applied river science may be applied and should be considered as a prototype that will become increasingly more applicable to solving the problem of channel restoration design with continued research, development and testing.

By coupling geomorphological principles of river management with river engineering methods, the geomorphic engineering approach provides an essential framework for channel restoration design in meandering rivers and, through a methodology based on sediment continuity and natural systems variability, can provide a vital platform to meet the multifunctional objectives of river restoration projects within the context of the catchment.

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Appendix A
Arithmetic-Based and Logarithmic-Based Effective Discharges for 55 Reported Sites Using 25 Discharge Class Intervals

Reference	River	USGS Gauge	Flow Record	P	Skew	Kurt	α (m^3s^{-1})	β (m^3s^{-1})	a	b	$Q_{e,a}$ (m^3s^{-1})	F_a (%)	$Q_{e,l}$ (m^3s^{-1})	Rep Q_e (m^3s^{-1})	$Q_{e,l}/Q_{e,a}$	Pred $Q_{e,a}$ (m^3s^{-1})	Pred $Q_{e,a}/Q_{e,a}$
Biedenharn and Thorne (1994)	Mississippi at Vicksburg, MS	07289000	10/49 to 09/82	12053	-0.01	-0.81	9.551	0.584	0.00000513	2.420	29534.48	11.710	30344.94	33000	1.03	22829.74	0.77
Nash (1994)	Eel nr Dos Rios, CA	11472150	10/68 to 09/98	9519	0.43	-0.80	0.906	2.199	0.310	1.760	35.62*	15.852	766.86	192.33	21.53	97.54	2.74
Nash (1994)	Eel at Fort Seward, CA	11475000	10/68 to 09/98	10957	0.04	-1.00	2.896	2.251	0.070	2.020	135.39*	22.885	3254.54	473.54	24.04	3174.82	23.45
Nash (1994)	Eel at Scotia, CA	11477000	10/68 to 09/98	10957	0.21	-0.99	3.698	1.986	0.030	2.140	2018.98	1.980	3559.01	177.35	1.76	3627.59	1.80
Nash (1994)	Klamath at Orleans, CA	11523000	10/68 to 09/98	10957	0.43	-0.52	5.012	0.967	-	2.360	696.22	5.977	4767.58	0.21	6.85	535.62	0.77
Nash (1994)	Mad nr Arcata, CA	11481000	10/68 to 09/98	10957	0.12	-1.01	2.084	1.943	0.300	1.990	126.58	8.110	344.61	21.45	2.72	338.08	2.67
Nash (1994)	Paria at Lees Ferry, AZ	09382000	10/68 to 09/98	10957	0.64	0.86	-0.914	0.992	453.000	3.020	74.93**	0.005	65.72**	57.7	0.88	2.93	0.04
Nash (1994)	Redwood Creek at Orick, CA	11482500	10/68 to 09/98	10957	-0.09	-0.93	2.025	1.818	0.180	2.020	113.60	5.454	285.67	32.76	2.51	220.90	1.94
Nash (1994)	San Pedro at Charleston, AZ	09471000	10/68 to 09/98	10957	0.67	2.66	-1.056	1.211	23.830	2.510	10.42*	1.634	403.22**	55.6	38.69	3.19	0.31
Nash (1994)	San Luis Rey at Oceanside, CA	11042000	10/68 to 09/98	10245	0.48	0.29	-0.954	1.573	4.400	1.510	5.14*	7.003	195.42**	-	38.00	1.36	0.26
Nash (1994)	Sespe Creek nr Fillmore, CA	11113000	10/68 to 09/98	8872	0.03	-0.45	-0.862	2.293	1.690	1.430	16.48*	5.131	637.85**	-	38.70	4.05	0.25
Nash (1994)	Thomes Creek at Paskenta, CA	11382000	10/68 to 09/96	10139	-0.39	-0.24	0.555	2.182	0.430	2.170	14.44*	18.710	165.17	133.17	11.44	458.08	31.72
Nash (1994)	Trinity at Hoopa, CA	11530000	10/68 to 09/98	10957	0.45	-0.65	4.218	1.172	0.010	2.480	1007.25	1.438	1800.85	1.87	1.79	518.45	0.51
Nash (1994)	Brandywine Creek at Wilmington, DE	01481500	10/68 to 09/98	10957	0.55	0.49	2.351	0.745	0.170	1.820	25.68	10.643	19.50	98.5	0.76	16.54	0.64
Nash (1994)	East Nishnabotna at Red Oak, IA	06809500	10/68 to 09/98	10957	0.31	-0.08	2.070	1.135	1.930	2.280	25.87*	15.578	403.39	232	15.59	41.25	1.59
Nash (1994)	Big Raccoon Creek nr Fincastle, IN	03340800	10/68 to 09/98	10957	-0.04	-0.12	0.408	1.453	4.190	1.660	6.94*	13.426	63.51	76.2	9.16	6.06	0.87
Nash (1994)	East Fork White at Seymour, IN	03365500	10/68 to 09/98	10957	0.32	-0.27	3.777	1.058	0.780	1.490	105.73	19.610	283.68	252	2.68	75.60	0.72
Nash (1994)	Arkansas at Arkansas City, KS	07146500	10/68 to 09/98	10957	0.72	0.46	3.469	0.987	1.060	1.850	45.16*	31.655	1423.36	666	31.52	73.48	1.63
Nash (1994)	Arkansas nr Kinsley, KS	08313000	10/68 to 09/98	10326	-0.09	-0.83	-1.268	2.247	2.940	1.870	3.37*	16.248	44.58	-	13.23	22.76	6.75
Nash (1994)	Tygarts Creek nr Greenup, KY	03217000	10/68 to 09/98	10957	-0.43	-0.10	0.889	1.849	0.870	1.390	14.61*	14.593	112.29	105	7.68	9.23	0.63
Nash (1994)	Conococheague Creek at Fairview, MD	01614500	10/68 to 09/98	10950	0.46	-0.24	2.453	0.959	0.190	1.750	16.79*	34.267	197.66	-	11.78	23.18	1.38
Nash (1994)	NW Branch Anacostia at Colesville, MD	01650500	10/68 to 09/98	5786	0.61	1.67	-0.905	0.954	4.000	2.020	1.36*	8.799	56.87**	5.1	41.85	1.02	0.75
Nash (1994)	Minnesota at Mankato, MN	05325000	10/68 to 09/98	10957	-0.18	-0.70	4.104	1.424	2.240	1.390	312.61	13.154	607.09	532	1.94	133.63	0.43
Nash (1994)	Mississippi at St Louis, MO	07010000	10/68 to 09/98	10957	0.16	-0.46	8.575	0.569	0.310	1.560	29534.48	0.003	30344.94	9073	1.03	6348.27	0.21
Nash (1994)	Muddy Creek at Vaughn, MT	06088300	10/69 to 09/98	7518	0.35	-0.89	0.761	0.856	6.280	1.770	6.58	12.587	7.47	41.7	1.14	3.76	0.57
Nash (1994)	Pembina at Wallhala, ND	05099600	10/68 to 09/95	8717	0.06	-0.28	0.088	2.128	5.680	1.550	7.53*	18.916	161.61	197	21.45	13.18	1.75
Nash (1994)	Delaware at Trenton, NJ	01463500	10/68 to 09/98	10957	0.49	-0.34	5.549	0.744	0.010	1.800	416.54	26.187	875.03	2900	2.10	400.15	0.96
Nash (1994)	Stony Brook at Princeton, NJ	01401000	10/68 to 09/98	10957	-0.01	-0.26	-0.429	1.550	0.770	1.410	1.94*	23.783	24.68	44.7	12.74	1.74	0.90

Table A1 Arithmetic-based and logarithmic-based effective discharges for 55 reported sites using 25 discharge class intervals. Note: P = discharge measurements; Skew = skewness; Kurt = kurtosis; α = mean of natural logarithm of discharge; β = standard deviation of natural logarithm of discharge; a =sediment rating coefficient; b = sediment rating exponent; $Q_{e,a}$ arithmetic-based effective discharge; F_a = Exceedance probability of $Q_{e,a}$; $Q_{e,l}$ = logarithmic-based effective discharge; Rep Q_e = Reported effective discharge; Pred $Q_{e,a}$ = Predicted $Q_{e,a}$ based on a log-normal flow distribution; * = effective discharge in first class; ** effective discharge in last class.

Reference	River	USGS Gauge	Flow Record	P	Skew	Kurt	α (m^3s^{-1})	β (m^3s^{-1})	a	b	$Q_{e,a}$ (m^3s^{-1})	F_a (%)	$Q_{e,l}$ (m^3s^{-1})	Rep Q_e (m^3s^{-1})	$Q_{e,l}/Q_{e,a}$	Pred $Q_{e,a}$ (m^3s^{-1})	Pred $Q_{e,a}/Q_{e,a}$
Nash (1994)	Animas at Farmington, NM	00936500	10/68 to 09/97	10579	0.08	1.80	2.715	1.004	2.100	1.720	95.55	1.27	129.52	67.4	1.36	31.18	0.33
Nash (1994)	Pecos at Santa Rosa, NM	08383000	10/68 to 09/92	8766	1.23	0.74	-0.510	1.500	11.170	2.180	27.60	2.59	32.60	37.1	1.18	8.54	0.31
Nash (1994)	Rio Grande at Otowi Bridge, NM	08313000	10/68 to 09/97	10592	0.67	0.12	3.468	0.749	5.450	1.660	25.58	36.63	190.87	102	7.46	46.44	1.82
Nash (1994)	San Juan at Shiprock, NM	09368000	10/68 to 09/97	10592	-0.06	0.30	3.746	0.821	6.200	1.650	86.89	5.96	185.33	88.8	2.13	65.67	0.76
Nash (1994)	Maumee at Waterville, OH	04193500	10/68 to 09/98	10957	0.14	-0.60	4.197	1.347	0.290	1.630	576.36	2.42	933.16	818	1.62	208.57	0.36
Nash (1994)	Stillwater R at Pleasant Hill, OH	03265000	10/68 to 09/98	10957	0.48	-0.04	1.681	1.277	0.960	1.540	9.95*	84.43	164.55	105	16.54	12.96	1.30
Nash (1994)	Bixler Run nr Loysville, PA	01567500	10/68 to 09/98	10957	0.76	0.55	-1.057	0.950	1.300	1.330	1.26*	96.38	3.22	6.2	2.56	0.47	0.37
Nash (1994)	Juniata at Newport, PA	01567000	10/68 to 09/98	10957	0.42	-0.34	4.397	0.946	0.020	1.870	102.49*	81.94	721.52	-	7.04	176.92	1.73
Nash (1994)	Susquehanna at Harrisburg, PA	01570500	10/68 to 09/98	10957	0.19	-0.37	6.481	0.935	0.010	1.840	1698.32	16.99	4752.22	-	2.80	1358.97	0.80
Nash (1994)	Colorado nr Cisco, UT	09180500	10/68 to 09/98	10957	1.00	0.65	5.054	0.697	0.120	2.100	147.36	38.15	1101.40	613	7.47	267.35	1.81
Nash (1994)	Fremont nr Caineville, UT	09330230	10/68 to 09/98	10226	-0.10	0.56	0.615	0.549	10.350	2.310	2.36	49.81	2.83	18.4	1.20	2.74	1.16
Nash (1994)	Green at Green River, UT	09315000	10/68 to 09/98	10957	0.73	0.18	4.855	0.689	0.380	2.030	575.79	1.19	556.41	420	0.97	209.49	0.36
Nash (1994)	Green nr Jensen, UT	09261000	10/68 to 09/98	10957	0.71	0.01	4.573	0.693	0.320	1.970	343.47	2.48	344.57	501	1.00	154.39	0.45
Nash (1994)	San Juan nr Bluff, UT	09379500	10/68 to 09/98	10957	0.03	0.25	3.857	0.794	12.930	1.760	100.04	11.97	195.71	124	1.96	76.47	0.76
Nash (1994)	Dan at Paces, VA	02075500	10/68 to 09/98	10957	0.81	1.21	4.141	0.703	0.270	1.780	114.88	22.69	539.78	435	4.70	92.45	0.80
Nash (1994)	Levisa Fork at Big Rock, VA	03207800	10/68 to 09/98	10957	0.09	-0.27	1.650	1.212	2.330	1.620	14.19*	92.33	108.49	147	7.65	12.95	0.91
Nash (1994)	Rappahannock at Remington, VA	01664000	10/68 to 09/98	10957	-0.32	0.32	2.449	1.172	0.240	1.700	26.10*	92.48	368.31	162	14.11	30.26	1.16
Watson et al. (1997)	Abiaca Creek at Cruger, MS	07287160	10/91 to 10/93	69262	1.86	4.90	0.603	0.671	2.249	2.170	39.67	0.15	42.31	45.08	1.07	3.10	0.08
Watson et al. (1997)	Abiaca Creek nr Seven Pines, MS	07287150	10/91 to 10/93	69086	2.09	6.30	0.556	0.666	3.625	2.172	2.66*	92.85	76.24	91.83	28.65	2.93	1.10
Watson et al. (1997)	Batupan Bogue at Grenada, MS	07285400	06/85 to 10/93	286469	0.70	-0.66	1.657	1.660	1.686	1.832	258.57	0.96	299.15	332.10	1.16	51.91	0.20
Watson et al. (1997)	Fannegusha Creek nr Howard, MS	07287355	03/87 to 10/93	224989	1.31	1.92	0.309	1.244	5.218	2.138	11.87	96.42	173.05	234.86	14.58	7.93	0.67
Watson et al. (1997)	Harland Creek nr Howard	07287404	11/86 to 10/93	236100	1.60	3.18	-0.180	1.150	7.251	2.139	172.78	0.11	182.25**	146.26	1.05	3.77	0.02
Watson et al. (1997)	Hickahala Creek nr Senatobia, MS	07277700	02/86 to 10/93	267709	2.41	6.77	0.540	0.991	2.522	2.031	10.94*	96.19	233.52	227.72	21.35	4.72	0.43
Watson et al. (1997)	Hotopha Creek nr Batesville, MS	07273100	10/87 to 10/93	175296	1.08	0.98	-0.580	1.310	7.645	1.828	5.91*	96.99	246.37**	264.22	41.67	2.32	0.39
Watson et al. (1997)	Long Creek nr Pope, MS	07275530	12/86 to 10/93	236808	1.54	3.07	-0.205	1.245	3.768	2.016	15.05*	97.97	617.89**	605.75	41.05	3.93	0.26
Watson et al. (1997)	Otoucalofa Creek nr Water Valley, MS	07274252	07/85 to 10/93	270846	1.45	1.61	1.140	1.567	4.773	1.691	571.67**	0.52	500.67**	527.34	0.88	17.08	0.03
Watson et al. (1997)	Senatobia Creek nr Senatobia, MS	07277730	02/86 to 10/93	265045	2.32	6.19	-0.518	1.129	5.455	1.956	7.04*	96.80	218.49	196.04	31.04	2.01	0.29

Table A1 Concluded.

Appendix B
U.S. Sand-Bed River Data

Site	River	USGS Gauge	Guage Location	Flow Days	Site Location (USGS Quadrangle)	Survey Date	S	p	Average Bed Material d_{16}	d_{50}	d_{84}	σ	% silt-clay	% sand	% gravel	Average Bank Material % silt-clay	% sand	% gravel
1	Black Warrior	02465000	Northport, AL	10957	Tuscaloosa 15', AL	15 Sep 1998	0.00022	2.3	0.18	0.26	0.37	0.70	0.58	99.02	0.40	4.89	95.11	0.00
2	Cahaba	02424500	nr. Sprott, AL	3653	Summerfield SW 7.5', AL	21 Apr 1999	0.00041	1.4	0.22	0.30	0.41	1.36	0.06	99.35	0.59	4.59	95.41	0.00
3	Cossatot	07340500	nr. Dequeen, AR	4392	Geneva 7.5', AR	12 Dec 1998	0.00079	1.7	0.08	0.12	0.19	0.66	0.03	99.98	0.00	0.06	99.95	0.00
4	Red (site A)	07337000	Index, AR	10592	Daniels Chapel 7.5', TX-AR-OK	13 Dec 1998	0.00014	2.2	0.16	0.26	0.55	1.87	0.15	95.97	3.88	0.04	99.96	0.00
5	Red (site B)	07337000	Index, AR	10592	Daniels Chapel 7.5', TX-AR-OK	13 Dec 1998	0.00014	2.2	0.16	0.26	0.55	1.87	0.15	95.97	3.88	0.04	99.96	0.00
6	Big Sioux	06485500	Akron, IA	10957	Akron 7.5', IA-SD	02 Nov 1998	0.00025	1.7	0.32	0.59	1.11	1.85	0.30	93.70	6.00	70.30	28.05	1.65
7	Cedar	05465000	nr. Conesville, IA	10957	Nichols 7.5', IA	29 Oct 1998	0.00033	1.2	0.32	0.64	1.65	2.28	0.00	88.70	11.30	82.45	17.55	0.00
8	East Nishnabotna	06809500	Red Oak, IA	10957	Coburg 7.5', IA	04 Nov 1998	0.00060	1.4	0.28	0.43	0.90	1.79	0.10	95.50	4.40	53.40	46.45	0.15
9	Iowa	05454500	Iowa City, IA	10957	Hills 7.5', IA	30 Oct 1998	0.00023	1.9	0.32	0.54	0.91	1.69	0.09	98.91	1.00	72.80	27.00	0.20
10	Iowa (site A)	05452500	nr. Belle Plaine, IA	3652	Ladora 7.5', IA	31 Oct 1998	0.00033	1.9	0.26	0.36	0.49	1.37	0.00	96.30	3.70	68.35	29.85	1.80
11	Iowa (site B)	05452500	nr. Belle Plaine, IA	3652	Ladora 7.5', IA	31 Oct 1998	0.00033	1.9	0.29	0.47	0.96	1.81	0.09	95.81	4.10	85.90	14.00	0.10
12	Iowa	05451500	Marshalltown, IA	10957	Marshalltown 7.5', IA	31 Oct 1998	0.00053	1.4	0.44	0.84	1.70	1.96	0.10	91.30	8.60	37.70	61.85	0.45
13	Nodaway	06817000	Clarinda, IA	10957	Clarinda South 7.5', IA	03 Nov 1998	0.00042	1.1	0.26	0.35	0.48	1.36	0.00	99.90	0.10	66.90	33.10	0.00
14	Rock	06483500	nr. Rock Valley, IA	10932	Rock Valley 7.5', IA	02 Nov 1998	0.00051	1.8	0.29	0.50	1.09	1.94	0.10	94.70	5.20	76.00	23.75	0.25
15	Wapsinicon	05422000	nr. De Witt, IA	10957	De Witt 7.5', IA	29 Oct 1998	0.00049	1.3	0.27	0.44	0.76	0.60	0.10	96.41	3.50	50.05	48.70	1.25
16	Big Raccoon Creek	03341300	Coxville, IN	9934	Mecca 7.5', IN	31 Jul 1998	0.00054	1.2	0.28	0.50	0.81	1.70	0.05	98.01	1.94	8.78	91.23	0.00
17	Brouillets Creek	03341420	nr. Universal, IN	1918	Clinton 7.5', IN	31 Jul 1998	0.00088	1.2	0.38	1.61	8.88	0.21	0.10	53.01	46.90	4.10	95.91	0.00
18	Fawn	04098500	Orland, IA	1277	Shipshewana 7.5', IN	02 Aug 1998	0.00016	2.3	0.25	0.54	2.11	0.34	0.84	82.92	16.25	15.65	84.36	0.00
19	St. Joseph	04178000	nr. Newville, IN	10957	Hicksville 7.5', OH-IN	01 Aug 1998	0.00019	2.0	0.30	0.61	1.40	2.16	0.88	93.96	5.16	3.13	96.88	0.00
20	Sugar Creek	03362500	nr. Edinburgh, IN	10957	Marietta 7.5', IN	29 Jul 1998	0.00040	1.2	0.48	1.34	3.86	0.35	0.18	63.82	36.00	0.67	80.11	19.23
21	Wabash	03340500	Montezuma, IN	10957	Dana 7.5', IN	03 Aug 1998	0.00007	1.8	0.27	0.50	1.14	0.49	0.09	90.46	9.45	3.71	96.30	0.00
22	Wabash	03342000	Riverton, IN	10957	Merom 7.5', IN	04 Aug 1998	0.00013	1.5	0.38	0.91	2.45	0.39	0.06	70.32	29.63	8.27	91.73	0.00
23	White, East Fork	03365500	Seymour, IN	10957	Seymour 7.5', IN	29 Jul 1998	0.00028	1.7	0.62	1.63	4.40	0.37	0.14	58.15	41.71	4.23	95.78	0.00
24	White, West Fork	03360500	Newberry, IN	10957	Lyons 7.5', IN	30 Jul 1998	0.00019	1.7	0.39	1.05	3.53	0.33	0.07	69.52	30.42	12.57	87.00	0.43
25	White	03374000	Petersburg, IN	10957	Iona 7.5', IN	30 Jul 1998	0.00014	1.9	0.36	0.59	1.54	0.48	0.22	89.23	10.55	5.99	94.01	0.00
26	Licking	03249500	Farmers, KY	9496	Farmers 7.5', KY	05 Aug 1998	0.00025	2.9	0.33	1.38	3.29	0.32	0.96	63.12	35.92	3.04	96.97	0.00
27	Mud	03316000	nr. Lewsburg, KY	3653	Dunmore 7.5', KY	06 Aug 1998	0.00028	2.1	0.08	0.14	0.29	0.53	4.11	95.89	0.00	10.44	89.57	0.00
28	Red	03283500	Clay City, KY	10957	Clay City 7.5', KY	05 Aug 1998	0.00040	1.7	0.64	1.60	4.20	0.39	0.66	56.82	42.52	7.05	92.96	0.00
29	Rough	03319000	nr. Dundee, KY	866	Dundee 7.5', KY	06 Aug 1998	0.00011	2.1	0.09	0.15	0.24	0.59	3.32	96.68	0.00	5.17	94.83	0.00

Table B1 U.S. sand-bed river data: i) site, gauge, slope and sediment data. Note: S = slope; p = sinuosity; d_{16}, d_{50}, d_{84} = bed material particle sizes for which 16%, 50%, 84% finer; σ = bed material geometric standard deviation, $(d_{84} / d_{16})^{0.5}$.

Site	River	USGS Gauge	Guage Location	Flow Days	Site Location (USGS Quadrangle)	Survey Date	S	p	Average Bed Material d_{16}	d_{50}	d_{84}	σ	% silt-clay	% sand	% gravel	Average Bank Material % silt-clay	% sand	% gravel
30	Nodaway	06817500	nr. Burlington Junction, MO	5498	Burlington Junction 7.5', MO	03 Nov 1998	0.00040	1.3	0.17	0.35	2.91	4.09	1.40	79.90	18.70	74.45	25.55	0.00
31	Big Black	07290000	nr. Bovina, MS	10957	Big Black 7.5', MS	21 Jul 1998	0.00015	2.0	0.24	0.38	0.63	0.61	1.66	98.13	0.21	5.64	94.37	0.00
32	Big Black	07289500	Pickins, MS	3652	Cameron 7.5', MS	13 Aug 1998	0.00021	1.8	0.20	0.32	0.47	0.65	5.06	94.40	0.54	3.44	96.56	0.00
33	Buttahatchee	02439000	nr. Sulligent, AL	3652	Sulligent 7.5', AL-MS	21 Apr 1999	0.00044	1.7	0.21	0.28	0.37	1.33	0.31	99.69	0.00	0.00	100.00	0.00
34	Chickasawhay	02478500	Leakesville, MS	10592	Clark, 7.5', MS	24 Jul 1998	0.00019	1.5	0.23	0.39	0.62	0.60	0.61	97.97	1.42	2.47	97.53	0.00
35	Leaf	02472000	nr. Collins, MS	10957	Hot Coffee 7.5', MS	25 Jul 1998	0.00043	1.3	0.32	0.62	2.09	0.39	0.04	83.16	16.80	13.33	86.68	0.00
36	Leaf	02473000	Hattiesburg, MS	10957	New Augusta 7.5', MS	23 Jul 1998	0.00036	1.4	0.28	0.42	0.93	0.55	0.03	93.59	6.38	4.07	95.94	0.00
37	Pearl	02489500	nr. Bogalusa, LA	9858	Bogalusa 15', LA-MS	17 Sep 1998	0.00013	1.7	0.34	0.65	1.93	0.42	0.17	84.86	14.97	17.29	82.72	0.00
38	Pearl	02489000	nr. Columbia, MS	3652	Columbia South 7.5', MS	04 Aug 1999	0.00010	2.4	0.12	0.51	3.95	5.65	6.34	72.59	21.07	9.40	90.61	0.00
39	Tallahala Creek	02474500	nr. Runnelstown, MS	10785	Ovett SE 7.5', MS	22 Jul 1998	0.00058	1.4	0.22	0.33	0.63	0.59	0.33	96.83	2.85	4.76	95.25	0.00
40	Tombigbee	02437000	nr. Amory, MS	6445	Wren 7.5', MS	24 Apr 1999	0.00021	1.4	0.17	0.27	0.39	1.49	0.29	98.59	1.12	7.37	92.64	0.00
41	Fishing Creek	2083000	nr. Enfield, NC	10957	Enfield 7.5', NC	22 Jun 1999	0.00017	2.0	0.51	1.07	1.99	1.97	1.87	82.37	15.76	51.84	48.16	0.00
42	Lumber	02134500	Boardman, NC	10957	Fair Bluff 7.5', NC-SC	23 Jun 1999	0.00022	2.0	0.10	0.17	0.27	1.69	3.71	95.13	1.16	24.46	75.55	0.00
43	Neuse	02089500	Kinston, NC	10957	Kinston 7.5', NC	22 Jun 1999	0.00015	1.7	0.34	0.57	0.85	1.59	0.01	99.57	0.42	41.55	58.46	0.00
44	South	02107000	nr. Parkersburg, NC	6574	Garland 7.5', NC	23 Jun 1999	0.00027	1.5	0.26	0.53	1.11	2.06	0.42	98.44	1.14	15.82	84.19	0.00
45	Cimarron (site A)	07158000	nr. Waynoka, OK	9668	Fairview 7.5', OK	09 Dec 1998	0.00083	1.4	0.30	0.63	1.56	2.29	0.11	90.09	9.80	0.03	99.97	0.00
46	Cimarron (site B)	07158000	nr. Waynoka, OK	9668	Fairview SE 7.5', OK	09 Dec 1998	0.00083	1.4	0.24	0.82	2.38	3.13	0.04	79.51	20.45	0.02	99.98	0.00
47	Washita	07326500	Anadarko, OK	10957	Anadarko East 7.5', OK	10 Dec 1998	0.00038	2.0	0.20	0.29	0.40	1.39	0.01	99.99	0.00	0.04	99.97	0.00
48	Washita	07331000	nr. Dickson, OK	10957	Joy 7.5', OK	11 Dec 1998	0.00043	1.4	0.10	0.16	0.21	1.47	0.01	99.99	0.00	0.04	99.96	0.00
49	Congaree	02169500	Columbia, SC	10957	Wateree 7.5', SC	25 Jun 1999	0.00014	2.6	0.45	0.66	1.25	1.67	0.06	99.03	0.91	64.96	35.05	0.00
50	Pee Dee	02131000	Peedee, SC	10926	Florence East 7.5', SC	24 Jun 1999	0.00012	1.6	0.33	0.62	1.37	2.04	0.06	94.93	5.01	51.02	48.98	0.00
51	Hatchie	07030000	nr. Stanton, TN	3652	Brownsville 7.5', TN	12 Aug 1998	0.00019	1.7	0.12	0.19	0.32	0.61	2.73	97.04	0.23	3.96	96.04	0.00
52	Wolf	07030500	Rossville, TN	3775	Rossville 7.5', TN	12 Aug 1998	0.00045	1.6	0.22	0.35	0.55	0.63	0.35	99.65	0.00	2.97	97.04	0.00
53	Brazos (site A)	08114000	nr. Humble Camp, TX	10957	Missouri City 7.5', TX	14 Aug 1999	0.00012	2.0	0.14	0.22	0.34	1.56	4.33	95.67	0.00	59.47	40.54	0.00
54	Brazos (site B)	08114000	nr. Humble Camp, TX	10957	Thompsons 7.5', TX	14 Aug 1999	0.00012	2.0	0.09	0.24	0.47	2.31	7.33	83.80	8.87	68.07	31.94	0.00
55	Neches	08041000	Evadale, TX	10957	Silsbee 15', TX	18 Aug 1999	0.00018	2.7	0.07	0.23	0.57	2.88	12.92	87.08	0.00	61.88	38.12	0.00
56	Sabine	08028500	nr. Bon Wier, TX	10957	Merryville 15', TX-LA	18 Aug 1999	0.00014	1.5	0.06	0.14	0.30	2.19	16.00	84.00	0.00	33.19	66.81	0.00
57	Trinity	08066500	Romayor, TX	10592	Davis Hill 7.5', TX	19 Aug 1999	0.00013	1.7	0.21	0.28	0.38	1.36	0.02	99.98	0.00	50.39	49.61	0.00
58	Nottoway	02047000	nr. Sebrell, VA	10957	Sebrell 7.5', VA	21 Jun 1999	0.00016	1.3	0.69	1.28	2.20	1.78	0.01	81.24	18.75	54.98	45.03	0.00

Table B1 Continued.

Site	Effective Discharge	Bankfull Discharge	Q_b / Q_e	Y_b / Y_e	Q_2 / Q_m	Bank

	Site	Effective Discharge							Bankfull Discharge									Q_b / Q_e	Y_b / Y_e	Q_2 / Q_m	Bank
		Q_e (m^3s^{-1})	W (m)	D (m)	A (m^2)	P (m)	R (m)	Y_e (%)	Q_b (m^3s^{-1})	W (m)	D (m)	A (m^2)	P (m)	R (m)	RI (yrs)	Y_b (%)					
1	Black Warrior	350.2	79.3	3.86	305.9	80.9	3.78	18.6	1324.2	109.1	7.09	773.7	113.1	6.84	1.04	69.5		3.8	3.7	12.7	T2
2	Cahaba	33.9	42.6	1.06	45.3	43.5	1.04	6.7	816.5	61.0	6.58	401.4	67.1	5.98	2.25	87.6		24.1	13.0	13.7	T2
3	Cossatot	18.0	42.5	0.72	30.4	44.1	0.69	12.2	246.7	49.5	3.55	175.6	55.0	3.19	1.36	81.1		13.7	6.6	30.2	T2
4	Red (site A)	1347.9	218.0	5.26	1146.1	221.4	5.18	56.8	1351.5	218.0	5.27	1148.2	221.4	5.19	1.21	57.0		1.0	1.0	5.3	T1
5	Red (site B)	1347.9	224.9	5.07	1140.3	228.1	5.00	55.5	1367.8	225.0	5.12	1151.4	228.3	5.04	1.21	56.8		1.0	1.0	5.3	T1
6	Big Sioux	55.0	54.6	1.58	86.5	55.6	1.55	11.5	197.2	58.3	3.55	206.7	62.2	3.32	1.49	44.8		3.6	3.9	7.6	T1
7	Cedar	408.4	210.1	1.99	418.4	211.4	1.98	42.3	615.1	210.5	2.59	545.2	212.1	2.57	1.50	64.7		1.5	1.5	4.5	T1
8	East Nishnabotna	19.5	40.4	0.69	27.8	40.6	0.68	13.5	307.1	58.6	3.17	185.7	61.2	3.04	1.83	82.4		15.8	6.1	20.3	T1
9	Iowa	173.9	94.8	2.08	197.2	95.7	2.06	42.2	362.1	98.6	3.24	319.2	100.6	3.17	3.49	85.4		2.1	2.0	3.6	T1
10	Iowa (site A)	119.8	64.3	1.86	119.8	66.1	1.81	46.5	287.9	65.4	3.24	212.1	68.1	3.11	1.77	89.9		2.4	1.9	11.9	T1
11	Iowa (site B)	119.8	66.7	1.97	131.2	68.3	1.92	46.9	302.7	71.5	3.37	241.3	74.3	3.25	1.77	90.2		2.5	1.9	11.9	T1
12	Iowa	73.9	57.0	1.37	78.1	57.9	1.35	27.9	281.9	61.1	3.15	192.7	64.0	3.01	2.29	89.0		3.8	3.2	8.2	T1
13	Nodaway	12.5	65.0	0.40	26.0	65.1	0.40	4.3	444.6	87.0	3.39	294.8	89.2	3.30	2.56	84.0		35.5	19.5	27.5	T1
14	Rock	15.0	41.9	0.62	26.0	42.9	0.61	5.2	181.4	54.3	2.51	136.1	57.6	2.36	1.69	54.5		12.1	10.5	12.6	T1
15	Wapsinicon	162.8	120.7	1.38	166.3	123.0	1.35	47.3	267.4	121.8	1.95	236.8	124.8	1.90	1.75	77.1		1.6	1.6	5.3	T1
16	Big Raccoon Creek	35.3	34.0	1.15	39.0	34.7	1.12	42.4	145.9	39.4	2.61	102.7	41.5	2.47	1.68	92.2		4.1	2.2	16.5	T2
17	Brouillets Creek	7.5	23.3	0.55	12.8	23.8	0.54	4.0	117.6	28.1	2.95	82.9	30.7	2.69	1.09	57.5		15.6	14.4	31.1	T2
18	Fawn	inadequate flow data							17.8	32.8	1.15	37.8	34.6	1.09	-	-		-	-	-	T1
19	St. Joseph	58.3	40.1	1.88	75.4	42.1	1.79	48.8	98.2	58.4	2.04	119.1	60.5	1.97	1.45	77.2		1.7	1.6	7.6	T1
20	Sugar Creek	28.5	33.9	1.17	39.6	34.9	1.14	12.2	69.1	35.1	2.03	71.1	37.1	1.92	1.06	35.2		2.4	2.9	15.5	T1
21	Wabash	629.4	155.1	4.82	748.2	157.7	4.74	35.8	957.9	178.0	5.58	992.9	181.3	5.48	1.16	62.6		1.5	1.7	5.2	T1
22	Wabash	931.0	176.4	5.00	882.5	180.3	4.90	48.4	1019.0	204.2	4.61	941.4	208.2	4.52	1.18	55.2		1.1	1.1	4.5	T1
23	White, East Fork	107.4	77.8	1.77	137.4	78.6	1.75	10.7	313.0	87.2	3.27	284.9	88.9	3.20	1.05	45.9		2.9	4.3	10.9	T1
24	White, West Fork	227.9	93.9	2.86	268.4	95.9	2.80	19.3	561.6	123.9	4.07	503.7	127.1	3.96	1.15	59.5		2.5	3.1	6.6	T1
25	White	604.4	164.7	3.87	637.8	166.9	3.82	26.7	1030.4	171.4	5.25	899.8	174.3	5.16	1.05	51.4		1.7	1.9	5.0	T1
26	Licking	99.7	36.5	2.97	108.3	38.8	2.79	52.5	200.7	43.2	4.19	180.9	46.7	3.87	1.69	93.8		2.0	1.8	7.5	T2
27	Mud	1.6	9.5	0.45	4.3	9.7	0.44	2.4	37.7	16.3	2.69	43.8	19.0	2.31	1.00	54.7		23.3	22.5	36.1	T2
28	Red	30.3	26.4	1.55	41.0	29.2	1.40	11.8	172.7	35.2	3.83	134.8	40.7	3.31	1.34	71.4		5.7	6.1	16.2	T2
29	Rough	64.3	25.6	3.16	80.8	28.6	2.83	30.8	165.7	37.5	4.60	172.1	42.3	4.07	1.13	78.5		2.6	2.6	7.9	T2

Table B2 U.S. sand-bed river data: ii) effective discharge and bankfull measurements. Note: Q_e = effective discharge; Q_b = bankfull discharge; W = width; D = average depth; A = cross-sectional area; P = wetted perimeter; R = hydraulic radius; Y_e = bed material transported by discharges not exceeding Q_e over period of flow record; Y_b = bed material transported by discharges not exceeding Q_b over period of flow record; RI = recurrence interval of Q_b; Q_2 = 2-year recurrence interval flow; Q_m = mean annual (time-averaged) discharge; T1 = less than 50% tree cover; T2 = at least 50% tree cover. Values in parentheses are in-channel / total cross section ratios.

		Q_e (m^3s^{-1})	W (m)	D (m)	A (m^2)	P (m)	R (m)	Y_e (%)	Q_b (m^3s^{-1})	W (m)	D (m)	A (m^2)	P (m)	R (m)	RI (yrs)	Y_b (%)				
30	Nodaway	36.3	96.6	0.70	68.0	97.2	0.70	10.1	559.8	119.4	3.59	429.0	122.0	3.52	2.97	89.7	15.4	8.9	20.9	T1
31	Big Black	256.2	53.9	5.04	271.8	60.0	4.53	23.4	425.0	64.9	6.41	415.6	72.5	5.74	1.23	50.0	1.7	2.1	6.3	T2
32	Big Black	193.4	61.6	3.06	188.5	63.0	2.99	36.7	279.3	67.6	3.70	249.8	69.3	3.61	1.13	57.6	1.4	1.6	11.7	T2
33	Buttahatchee	30.2	17.6	1.90	33.4	19.6	1.70	22.8	92.6	21.7	3.49	75.7	26.5	2.86	1.02	58.2	3.1	2.6	20.9	T2
34	Chickasawhay	406.9	71.6	4.50	322.1	75.0	4.30	51.6	750.4	78.7	6.34	498.7	84.0	5.93	2.50	84.4	1.8	1.6	5.5	T2
35	Leaf	21.0	52.0	0.68	35.5	52.1	0.68	4.6	510.5	82.7	4.38	362.0	84.7	4.27	2.40	84.9	24.4	18.5	12.9	T2
36	Leaf	62.6	62.7	1.39	86.9	63.9	1.36	9.8	482.6	78.3	4.50	352.2	81.8	4.31	1.44	72.3	7.7	7.4	8.6	T2
37	Pearl	982.6 (0.97)	899.9 (0.17)	1.35 (4.21)	1212.8 (0.70)	902.2 (0.17)	1.34 (4.16)	48.2	657.3	148.6	4.43	658.7	150.8	4.37	1.11	24.8	0.7	0.5	4.0	T1
38	Pearl	876.3 (0.93)	1131.1 (0.09)	1.27 (5.79)	1440.0 (0.55)	1134.3 (0.10)	1.27 (5.64)	68.1	591.3	107.4	5.91	634.5	110.6	5.74	1.18	38.3	0.7	0.6	4.4	T1
39	Tallahala Creek	79.5	38.2	1.77	67.5	39.3	1.72	33.9	173.9	42.6	2.76	117.5	44.4	2.65	1.44	72.6	2.2	2.1	8.3	T2
40	Tombigbee	124.7	69.9	2.08	145.7	71.2	2.05	12.1	489.4	80.5	4.52	364.0	84.0	4.34	1.22	55.5	3.9	4.6	11.4	T2
41	Fishing Creek	63.5	37.5	2.30	86.3	39.6	2.18	38.7	114.6	43.3	3.09	133.9	46.1	2.91	1.72	72.2	1.8	1.9	9.1	T2
42	Lumber	60.0 (0.98)	219.9 (0.18)	0.44 (4.08)	97.0 (0.73)	220.6 (0.18)	0.44 (4.02)	43.7	44.6	39.2	1.46	57.2	39.9	1.43	1.02	26.4	0.7	0.6	3.5	T1
43	Neuse	224.6 (0.90)	721.2 (0.08)	0.63 (5.68)	457.1 (0.45)	723.3 (0.08)	0.63 (5.51)	63.0	135.4	56.7	2.70	152.8	58.6	2.61	1.02	30.6	0.6	0.5	4.1	T1
44	South	16.2	18.3	1.24	22.7	19.1	1.19	24.7	18.0	19.8	1.25	24.7	20.6	1.20	1.04	30.4	1.1	1.2	5.0	T1
45	Cimarron (site A)	10.8	120.4	0.23	27.6	120.8	0.23	6.4	383.6	125.6	2.15	269.4	127.4	2.11	1.72	87.4	35.7	13.7	76.3	T1
46	Cimarron (site B)	10.8	115.3	0.25	28.7	115.5	0.25	5.6	415.3	128.5	2.32	297.7	130.0	2.29	1.81	87.5	38.6	15.5	76.3	T1
47	Washita	13.9	50.5	0.52	26.5	50.6	0.52	7.5	130.7	55.1	2.09	115.1	56.5	2.04	1.80	63.7	9.4	8.6	10.8	T1
48	Washita	51.0	81.5	0.86	70.1	81.8	0.86	7.9	463.6	142.9	2.77	395.6	145.0	2.73	1.29	64.9	9.1	8.2	14.4	T1
49	Congaree	260.2	98.9	3.03	299.2	102.3	2.92	20.0	555.4	101.2	4.93	498.8	107.2	4.65	1.00	52.6	2.1	2.6	7.3	T2
50	Pee Dee	504.9	99.1	4.82	477.9	102.6	4.66	41.3	690.5	105.2	5.67	596.3	109.3	5.46	1.28	64.2	1.4	1.6	3.9	T2
51	Hatchie	234.6 (0.99)	193.3 (0.23)	1.26 (3.62)	243.2 (0.83)	196.3 (0.24)	1.24 (3.45)	49.7	179.3	44.4	3.81	169.0	47.4	3.57	1.02	33.3	0.8	0.7	7.0	T2
52	Wolf	7.6	15.4	0.83	12.8	15.9	0.80	7.6	63.0	29.3	2.02	59.1	31.1	1.90	1.03	50.6	8.3	6.6	16.3	T2
53	Brazos (site A)	462.9	96.7	4.46	431.6	99.0	4.36	24.8	1451.5	121.5	8.22	998.3	126.1	7.92	2.17	79.2	3.1	3.2	6.3	T2
54	Brazos (site B)	462.9	99.7	4.56	454.9	101.9	4.47	25.2	1308.8	126.1	7.80	982.9	130.0	7.56	1.86	73.8	3.1	2.9	6.3	T2
55	Neches	523.5 (0.82)	826.0 (0.09)	1.19 (4.35)	983.5 (0.38)	829.6 (0.09)	1.19 (4.18)	64.4	220.1	71.7	3.37	241.6	74.9	3.22	1.13	19.3	0.4	0.3	3.4	T2
56	Sabine	457.3	132.5	3.59	476.1	135.4	3.52	37.2	1199.3	145.6	6.27	912.6	150.3	6.07	3.20	92.0	2.6	2.5	4.3	T1
57	Trinity	659.3	183.5	3.76	689.5	185.2	3.72	36.1	1306.6	188.6	5.89	1110.0	192.0	5.78	2.31	80.0	2.0	2.2	4.9	T1
58	Nottoway	84.9 (>0.99)	145.0 (0.31)	0.86 (2.97)	124.1 (0.92)	146.7 (0.32)	0.85 (2.90)	34.0	75.4	44.2	2.36	104.5	45.9	2.28	1.05	27.5	0.9	0.8	5.9	T2

Table B2 Continued.

Appendix C
Red River Data from the 1981 Hydrographic Survey

River Mile*	Q_2 ($m^3 s^{-1}$)	S (10^4)	P	R_c (m)	L_m (m)	W_i (m)	D_m (m)	D_{max} (m)	W_i / D_m	R_c / W_i	V ($m s^{-1}$)	W_a (m)	W_p (m)	W_a / W_i	W_p / W_i	Z_{a-p} / Z_{a-i}	D_{max}/D_m	Bend Type
423.6	2495	1.440	1.60	1219.2	5140	304.8	4.90	9.27	62.2	4.00	1.67	563.85	487.66	1.85	1.60	0.20	1.89	c
421.2	2495	1.436	1.86	1066.8	3460	360.6	4.65	11.10	77.5	2.96	1.49	472.42	396.22	1.31	1.10	0.28	2.39	b
417.3	2495	1.779	1.89	1066.8	3660	312.4	4.69	8.95	66.6	3.41	1.70	548.61	380.98	1.76	1.22	1.02	1.91	c
414.4	2495	1.728	1.96	762.0	3660	302.7	4.01	10.09	75.5	2.52	2.06	411.46	304.79	1.36	1.01	0.40	2.52	b
412.1	2495	1.790	1.51	1524.0	6280	295.0	4.18	7.94	70.6	5.17	2.02	304.79	320.02	1.03	1.08	0.68	1.90	e
407.4	2495	2.334	1.10	762.0	4040	330.1	4.00	9.39	82.5	2.31	1.89	411.46	335.26	1.25	1.02	0.33	2.35	b
406.8	2495	2.327	1.16	1143.0	4940	449.0	3.71	8.41	121.0	2.55	1.50	457.18	457.18	1.02	1.02	0.00	2.27	b
405.4	2495	2.057	1.78	457.2	2120	305.0	3.73	9.64	81.8	1.50	2.19	365.74	396.22	1.20	1.30	0.09	2.58	b
402.6	2495	2.318	1.46	457.2	1780	221.0	4.85	12.05	45.6	2.07	2.33	289.55	182.87	1.31	0.83	0.69	2.48	b
401.5	2832	1.845	1.12	2133.6	5520	304.8	5.56	10.17	54.8	7.00	1.67	335.26	335.26	1.10	1.10	-0.10	1.83	e
399.1	2832	1.770	1.27	838.2	4140	358.1	4.45	10.42	80.5	2.34	1.78	518.13	320.02	1.45	0.89	0.37	2.34	b
397.4	2832	1.509	1.31	914.4	4140	330.1	4.20	8.81	78.6	2.77	2.04	579.09	426.70	1.75	1.29	0.65	2.10	c
395.4	2832	1.440	1.44	762.0	3220	312.0	4.21	10.29	74.1	2.44	2.16	304.79	304.79	0.98	0.98	-0.12	2.44	e
393.2	2832	1.442	1.32	914.4	3840	320.0	4.09	9.78	78.2	2.86	2.16	411.46	457.18	1.29	1.43	0.50	2.39	b
391.9	2832	2.012	2.26	914.4	2840	281.9	4.90	9.87	57.5	3.24	2.05	457.18	320.02	1.62	1.14	0.87	2.01	c
389.9	2832	1.845	1.19	914.4	4220	240.2	5.10	10.00	47.1	3.81	2.31	243.83	182.87	1.02	0.76	0.27	1.96	e
388.4	2832	1.128	1.29	1752.6	3120	289.6	4.94	8.39	58.6	6.05	1.98	380.98	396.22	1.32	1.37	0.30	1.70	b
386.6	2832	2.455	1.49	1219.2	4656	282.0	4.76	9.98	59.2	4.32	2.11	304.79	289.55	1.08	1.03	0.17	2.10	e
386.5	2832	1.568	1.02	1066.8	5420	325.5	4.82	9.74	67.5	3.28	1.81	426.70	380.98	1.31	1.17	0.52	2.02	b
385.2	2832	1.563	1.12	609.6	3460	297.2	4.72	10.53	63.0	2.05	2.02	335.26	281.93	1.13	0.95	1.06	2.23	b
383.6	2832	1.393	1.15	609.6	3260	266.7	5.00	9.48	53.3	2.29	2.12	518.13	380.98	1.94	1.43	0.52	1.90	c
382.7	2832	1.052	1.84	3048.0	1728	251.5	5.26	11.47	47.8	12.12	2.14	350.50	266.69	1.39	1.06	0.55	2.18	b
376.3	2832	0.758	1.08	2743.2	3264	335.3	5.04	8.82	66.5	8.18	1.68	373.36	312.40	1.11	0.93	0.37	1.75	b
375.2	2832	1.248	1.07	609.6	4320	335.3	5.07	9.68	66.1	1.82	1.67	441.94	312.40	1.32	0.93	-0.39	1.91	b
373.8	2832	1.313	1.07	914.4	2736	358.1	4.87	9.71	73.5	2.55	1.62	457.18	426.70	1.28	1.19	-0.18	1.99	b
372.4	2832	0.647	1.12	762.0	3456	320.0	5.49	12.69	58.3	2.38	1.61	312.40	213.35	0.98	0.67	0.17	2.31	e

Table C1 Red River Data from the 1981 Hydrographic Survey. Note: * = river mile at inflexion point upstream of meander bend; Q_2 = 2-year recurrence interval flow; S = water surface slope; P = sinuosity; R_c = radius of curvature; L_m = meander wavelength; W_i = width at upstream inflexion point; D_m = mean depth at upstream inflexion point; D_{max} = maximum scour depth in bend pool; V = mean velocity at upstream inflexion point; W_a = width at bend apex; W_p = width at maximum scour location; Z_{a-p} = channel distance between bend apex and pool (at maximum scour location); Z_{a-i} = channel distance between bend apex and downstream inflexion point; bend type-e = equiwidth meandering; bend type-b = meandering with point bars; bend type-c = meandering with point bars and chute channels.

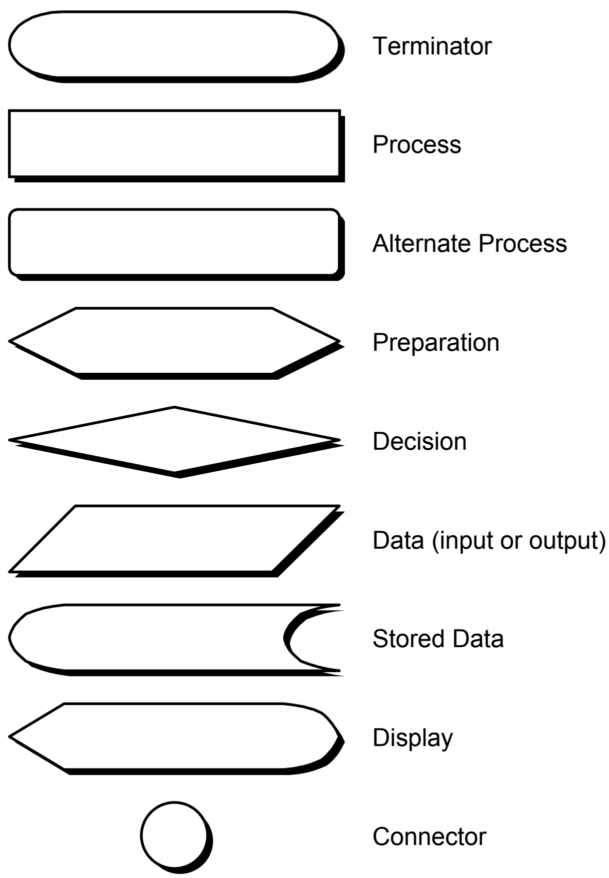
River Mile*	Q_2 ($m^3 s^{-1}$)	S (10^4)	P	R_c (m)	L_m (m)	W_i (m)	D_m (m)	D_{max} (m)	W_i / D_m	R_c / W_i	V ($m s^{-1}$)	W_a (m)	W_p (m)	W_a / W_i	W_p / W_i	Z_{a-p} / Z_{a-i}	D_{max}/D_m	Bend Type
371.2	2832	1.323	1.43	1066.8	2832	282.0	5.13	11.00	55.0	3.78	1.96	304.79	281.93	1.08	1.00	0.16	2.14	e
366.4	2832	2.095	1.48	1066.8	4320	358.1	4.22	9.04	84.9	2.98	1.87	487.66	457.18	1.36	1.28	0.38	2.14	b
362.2	2832	1.586	1.29	1371.6	4176	320.0	4.74	8.17	67.5	4.29	1.87	350.50	312.40	1.10	0.98	-0.38	1.72	e
360.6	2832	1.894	1.42	609.6	3312	289.0	4.92	10.87	58.7	2.11	1.99	396.22	320.02	1.37	1.11	0.30	2.21	b
359.2	2832	1.377	1.11	1219.2	5760	274.3	5.06	8.65	54.2	4.44	2.04	259.07	228.59	0.94	0.83	0.30	1.71	e
357.5	2832	1.323	1.51	1219.2	5088	297.2	4.95	9.73	60.0	4.10	1.93	281.93	281.93	0.95	0.95	0.22	1.97	e
355.1	2832	1.326	1.03	1371.6	4704	335.3	4.60	9.22	72.9	4.09	1.84	396.22	304.79	1.18	0.91	0.27	2.00	b
353.6	2832	1.326	1.04	2743.2	2448	365.8	4.52	7.92	80.9	7.50	1.71	365.74	350.50	1.00	0.96	0.18	1.75	b
353.0	2832	1.852	1.08	914.4	3984	396.2	4.20	9.14	94.3	2.31	1.70	487.66	502.90	1.23	1.27	0.99	2.18	b
351.7	2832	1.845	1.49	762.0	1680	350.5	4.38	10.68	80.0	2.17	1.84	670.53	426.70	1.91	1.22	0.75	2.44	c
349.8	3312	1.851	1.72	990.6	3312	304.8	4.66	8.74	65.4	3.25	2.33	426.70	365.74	1.40	1.20	0.29	1.88	b
348.1	2832	2.119	1.15	762.0	2832	342.9	4.63	11.90	74.1	2.22	1.78	365.74	320.02	1.07	0.93	-0.30	2.57	b
346.3	5712	1.513	1.21	1524.0	5712	335.3	4.76	8.29	70.4	4.55	3.58	487.66	487.66	1.45	1.45	0.00	1.74	b
342.8	3936	1.237	1.23	990.6	3936	342.9	4.46	9.02	76.9	2.89	2.57	472.42	365.74	1.38	1.07	0.62	2.02	b
340.2	4512	1.336	1.41	990.6	4512	289.6	4.47	10.43	64.8	3.42	3.49	365.74	350.50	1.26	1.21	0.35	2.33	b
338.2	4944	0.925	1.11	2971.8	4944	320.0	4.53	7.70	70.6	9.29	3.41	320.02	396.22	1.00	1.24	0.93	1.70	e
335.1	4080	0.902	1.06	914.4	4080	365.8	5.14	12.09	71.2	2.50	2.17	274.31	236.21	0.75	0.65	-0.15	2.35	e
330.9	3552	1.126	1.01	914.4	3552	441.9	5.25	10.53	84.2	2.07	1.53	457.18	441.94	1.03	1.00	0.50	2.01	b
329.8	5904	1.134	1.47	1371.6	5904	304.8	5.38	11.56	56.7	4.50	3.60	548.61	457.18	1.80	1.50	0.44	2.15	c
327.1	4032	1.134	1.21	1219.2	4032	457.0	5.18	9.48	88.2	2.67	1.70	822.92	792.44	1.80	1.73	0.11	1.83	c
325.6	4416	1.134	1.22	1676.4	4416	304.8	5.09	10.26	59.9	5.50	2.85	441.94	426.70	1.45	1.40	0.12	2.02	b
323.9	5280	1.132	1.19	1219.2	5280	304.8	5.08	11.55	60.0	4.00	3.41	624.81	502.90	2.05	1.65	0.41	2.27	c
319.6	4896	1.573	1.12	1828.8	4896	304.8	5.79	9.63	52.6	6.00	2.77	396.22	304.79	1.30	1.00	0.40	1.66	b
315.6	7440	1.357	1.16	1676.4	7440	274.0	5.48	9.84	50.0	6.12	4.95	243.83	228.59	0.89	0.83	0.16	1.80	e
310.7	3774	1.369	1.33	1676.4	3774	518.1	5.06	10.96	102.4	3.24	1.44	701.01	548.61	1.35	1.06	0.46	2.17	b
309.2	3408	1.820	1.21	304.8	3408	327.6	5.78	13.50	56.7	0.93	1.80	335.26	304.79	1.02	0.93	0.16	2.34	e

Table C1 Continued.

River Mile*	Q_2 ($m^3 s^{-1}$)	S (10^4)	P	R_c (m)	L_m (m)	W_i (m)	D_m (m)	D_{max} (m)	W_i / D_m	R_c / W_i	V ($m s^{-1}$)	W_a (m)	W_p (m)	W_a / W_i	W_p / W_i	Z_{a-p} / Z_{a-i}	D_{max}/D_m	Bend Type
307.9	2784	1.894	1.03	2438.4	2784	312.4	6.31	10.15	49.5	7.81	1.41	350.50	350.50	1.12	1.12	0.00	1.61	b
307.0	3600	1.844	1.01	2133.6	3600	243.8	6.09	9.87	40.0	8.75	2.42	259.07	228.59	1.06	0.94	0.50	1.62	e
305.9	8496	1.265	1.11	2286.0	8496	335.3	5.63	9.51	59.6	6.82	4.50	579.09	457.18	1.73	1.36	-0.18	1.69	c
303.0	5232	1.763	1.43	1219.2	5232	335.3	5.25	10.53	63.9	3.64	2.97	426.70	327.64	1.27	0.98	0.71	2.01	b
300.7	4704	0.947	1.93	1828.8	4704	350.5	5.74	9.37	61.1	5.22	2.34	609.57	441.94	1.74	1.26	0.56	1.63	c
295.8	9744	1.338	1.06	2286.0	9744	266.7	5.81	9.16	45.9	8.57	6.29	289.55	228.59	1.09	0.86	0.48	1.58	e
292.4	3888	1.466	1.03	2590.8	3888	312.4	5.34	8.71	58.5	8.29	2.33	342.88	304.79	1.10	0.98	0.47	1.63	e
289.1	4896	0.919	1.34	1219.2	4896	304.8	4.95	9.71	61.6	4.00	3.25	579.09	304.79	1.90	1.00	0.39	1.96	c
287.1	3456	0.906	1.29	762.0	3456	221.0	6.22	11.10	35.5	3.45	2.51	396.22	213.35	1.79	0.97	0.87	1.78	c
285.5	4224	2.072	1.05	2133.6	4224	236.2	6.09	9.36	38.8	9.03	2.94	266.69	251.45	1.13	1.06	0.93	1.54	b
284.2	3840	2.942	1.80	914.4	3840	229.0	6.22	12.30	36.8	3.99	2.70	335.26	228.59	1.46	1.00	0.57	1.98	b
281.7	4080	2.681	2.05	1066.8	4080	243.8	6.36	10.92	38.3	4.38	2.63	274.31	167.63	1.13	0.69	0.65	1.72	e
279.1	3456	2.649	1.42	838.2	3456	243.8	5.90	12.46	41.3	3.44	2.40	304.79	274.31	1.25	1.13	0.25	2.11	b
278.0	3408	1.954	1.03	1905.0	3408	243.8	5.92	9.44	41.2	7.81	2.36	228.59	182.87	0.94	0.75	0.42	1.59	e
277.1	3455	2.655	1.05	838.2	2976	206.0	6.02	12.17	34.2	4.07	2.79	190.49	121.91	0.92	0.59	0.74	2.02	e

Table C1 Continued.

Appendix D
Definition of Flowchart Symbols



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14. ABSTRACT

A framework for channel restoration design is presented that attempts to bridge the divide between reconnaissance level geomorphological designs at one extreme and numerical modelling of hydrodynamics, sediment transport and morphological change at the other. Reestablishing equilibrium between the sediment supply and available transport capacity in the restored reach is the primary objective of the design framework. A geomorphic engineering approach is presented, which recognises that the river is ultimately the best restorer of its natural morphology and should be allowed to participate in its own recovery. This is accomplished through designing an approximate channel mould, based on the broad dimensions of the river, and then allowing the river itself to develop the intricate cross-sectional detail and intra-reach morphological features to complete the recovery process.

Geomorphic engineering provides a practical solution by striking a balance between empirical-statistical and analytical (process-based) methods. The range of techniques that comprise the design approach facilitate a realistic solution to the indeterminacy problem and confidence bands applied to 'typed' morphological equations provide a mechanism through which natural rivers can be used as realistic

(Continued)

15. SUBJECT TERMS

Channel-forming discharge	Hydraulic geometry	River management
Channel restoration design	Hydraulic geometry	River restoration
Effective discharge	River engineering	Stable channel design

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analogues for channel restoration design. By accounting for natural systems variability, the design framework is an appropriate platform for generating restoration design solutions that mimic the natural channel morphologies and environmental attributes in undisturbed systems, while meeting multifunctional goals of channel stability and low maintenance commitments. Rather than constructing physical habitats that constitute form without function, geomorphologically, the types and levels of physical habitat diversity that are sustainable in the restored reach are defined by the type of river, the nature of the sediment and flow regimes and the catchment context. The approach presented is not a 'cookbook' procedure for river restoration but is a framework within which the sound judgement of practitioners with experience in applied river science may be applied.