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CONVERSION OF BRAIDED GRAVEL-BED RIVERS TO SINGLE-THREAD CHANNELS OF EQUIVALENT TRANSPORT CAPACITY

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ABSTRACT

A set of design equations is presented for determining the characteristics of a single-thread gravel-bed channel equivalent in water and bedload transport capacity to a given braided gravel-bed river or stream. The main components of the set are the Meyer-Peter and Muller formula for bedload transport and the Manning formula for flow resistance. In the specification of the design problem, water discharge and sediment size are always assumed known in the single-thread channel along with at least two of width, depth, bedslope, gravel discharge and resistance coefficient. For design cases the set is shown to replicate the behaviour of small-scale braided sand-bed streams with steady flow, as well as producing results and predictions in accord with field observations on braided gravel-bed rivers. A procedure is given and illustrated for a braided to single-thread conversion in natural rivers where water and gravel discharges vary with time. It is founded on the assumption of equivalence of transported load over time, that is, reaches in a braided state, when converted to a single-thread state, discharge similar gravel loads over periods of five to ten years or longer. As a corollary the method provides a starting point for theoretical analysis of changes in channel sediment storage in gravel-bed rivers.

INTRODUCTION

Training of braided gravel-bed rivers into single-thread channels is a long established and continuing activity in New Zealand (Grant, 1948; Nevins, 1969; Williams, 1985). Design methods are based largely on formulae adopted from sand-bed rivers and upon rules of thumb derived from field experience (Nevins, 1969), but varying degrees of success have been achieved. Training works almost invariably require continual and expensive maintenance, a situation which appears to have evolved as the natural outcome of theoretical developments lagging practical requirements. Nevertheless, slow sporadic progress is being made towards more rational design procedures. Theoretically based design guidelines offered by Henderson (1966) are useful if rather inaccurate (Griffiths, 1981); and a design method given in Griffiths (1981) using observations of stable reaches to predict dimensions of design reaches appears logically sound but needs to be tested properly. In contrast the general field of braided gravel-bed river processes and behaviour has received an upsurge in interest over the last decade regarding both laboratory models (Hong and Davies, 1979; Southard and Smith, 1982; Ashmore, 1985, 1988; Davies and Lee, 1988) and natural rivers (Griffiths, 1979; Pickup and Higgins, 1979; Thompson, 1985; Davoren and Mosley, 1986).

Conventional sediment transport and flow resistance formulae are able to model gross characteristics of braided gravel-bed rivers, even though the fundamental mechanisms assumed by the transport formulae do not occur (Davies, 1987; Carson and Griffiths, 1989). Combining this finding with the observation that changes from braided to single-thread channel form occur in stable, natural gravel-bed rivers (Leopold et al, 1964; Henderson, 1966) suggests that a broad but nevertheless improved solution to the problem can now be attempted.

The specific purposes of this analysis are: (1) to select and calibrate suitable transport and resistance formulae for design requirements; (2) to apply these formulae to the problem of designing a single-thread channel of equivalent water and bedload transport capacity to a given braided channel; and (3) to compare design predictions with observations from both laboratory and field channels. The aim is to explore some implications of recent empirical studies, and to outline the basis for a design method for the conversion of braided to single-thread rivers.

THEORY

A major shortcoming in stable channel design theory for gravel-bed rivers is lack of a solution to the following problem: given a stable braided channel and flow regime, determine the characteristics of a stable single-thread channel which will, over a period of time, transport the same gravel load. In the usual specification of the problem (Henderson, 1966) there are seven pertinent variables: water discharge, Q, sediment (gravel) discharge, G_s, mean sediment size, d, channel slope, S, channel width, B, flow depth, D, and resistance coefficient, c. Herein Q and d are always given for the single thread channel together with at least two of the remaining variables. Three variables remain unknown and thus three governing relationships are required for a solution. These take the form of equations for sediment transport, flow resistance and resistance coefficient.

This formulation of the braided to single-thread channel conversion problem contains a number of idealisations and restrictive assumptions including: (1) both braided and single-thread channels are rectangular in cross-section; (2) channel reaches are straight and uniform with a constant and single value of S and d; (3) unless specifically stated to the contrary, G_s and Q are time-independent or steady-state values; (4) for any G_s and Q, corresponding values of B, D and c, along with G_s and Q, apply over an entire cross-section and at all cross-sections of a reach; (5) unless otherwise defined, Q is the bankfull or dominant discharge (Carson and Griffiths, 1989); and (6) a stable channel is one whose gross dimensions and properties remain essentially constant over an extended period of time (say 5 to 10 years or longer): shifts in alignment and local aggradation and degradation may occur without interfering with overall stability provided these variations fluctuate about constant values. In hydraulic calculations for back-water curves or flood routing, equally restrictive and similar 1-dimensional assumptions are made and yet, for reasons which are not entirely clear, good representation is normally obtained. And so it appears, but to a lesser extent, with sediment transport calculations for braided gravel-bed rivers (as indicated by limited evidence noted previously) for reasons which are even less clear, perhaps because the approach is essentially a "blackbox" one (Davies, 1987). In what follows sediment transport and flow resistance equations are selected from the literature, design cases are formulated and solution methods are outlined.

Sediment Transport Equation

The Meyer-Peter and Muller (1948) formula was selected for modelling gravel transport for the following reasons: (1) it was calibrated and tested with a wide range of gravel-sized sediments both uniform and graded, among other materials (Meyer-Peter and Muller, 1948); (2) the equation can accommodate wall roughness, particle roughness and form roughness of bedforms; (3) it can model the outcomes of the experiments of Gilbert (1914), notably the parabolic relation between G_s and B (Carson and Griffiths, 1988); and (4) the formula has enjoyed success as a predictor in natural and laboratory channels (Pickup and Higgins, 1979; Ashmore, 1985). For rectangular channels the equation may be written in general form as

$$G_s = k\, B\, \gamma\, S_s\, [g\,(S_s-1)\, d^3]^{0.5} \left\{ \left(\frac{n_p}{n_c}\right)^{1.5} \left(\frac{Q_r}{Q}\right) \frac{DS}{(S_s-1)d} - \tau_c \right\}^{1.5} \quad (1)$$

in which k is a constant; γ is the specific weight of water; S_s is the specific gravity of sediment; g is the acceleration due to gravity; n_p is the Manning coefficient for particle roughness; n_c is the Manning coefficient of channel roughness and n_p/n_c is an adjustment factor to estimate the shear stress available for transport; Q_r/Q is an adjustment factor to account for variable bed and wall roughness, where Q_r is the portion of Q converted to eddying on the bed; R is the hydraulic radius where $R = BD/(B + 2D)$ and τ_c is the value of the Shields entrainment function = $[RS/(S_s-1)d]$, at initial motion conditions. In their calibration experiments Meyer-Peter and Muller (1948) found $k = 8$ and $\tau_c = 0.047$ where d was the mean size of the bed sediment. Values of n_p were estimated from the Strickler equation (Meyer-Peter and Muller, 1948)

$$n_p = 0.038 d_{90}^{1/6} \quad (2)$$

in which d_{90} is the size than which 90% of the bed material is finer.

Flow Resistance Equation

For ready compatibility with Equation 1 the Manning formula

$$Q = (1/n_c)\, BDR^{0.67} S^{0.5} \quad (3)$$

was adopted. Usually, in design, values of n_c are obtainable from backwater calculations or gauging information. If n_c is unknown then one may use, for instance, the prediction formulae of Griffiths (1989) which accommodate form roughness of gravel beds, and were calibrated with the data of Meyer-Peter and Muller (1948) amongst other information.

Design Cases and Solution Methods

Five design cases are considered (Fig. 1) to illustrate points of theoretical and practical interest. The objective is to begin with a braided gravel channel of known properties and then design a single-thread alternative channel capable of transporting the same gravel load. Cases 1 to 4 (Fig. 1) involve steady or time-dependent conditions for Q and G_s but Case 5 allows both Q and G_s to vary with time, t. Bed and wall roughness are assumed equal, that is, $(Q_r/Q) = R/D$. Generally the solutions involve iterative processes, or further derivative equations; it is thus important to outline how they may be obtained.

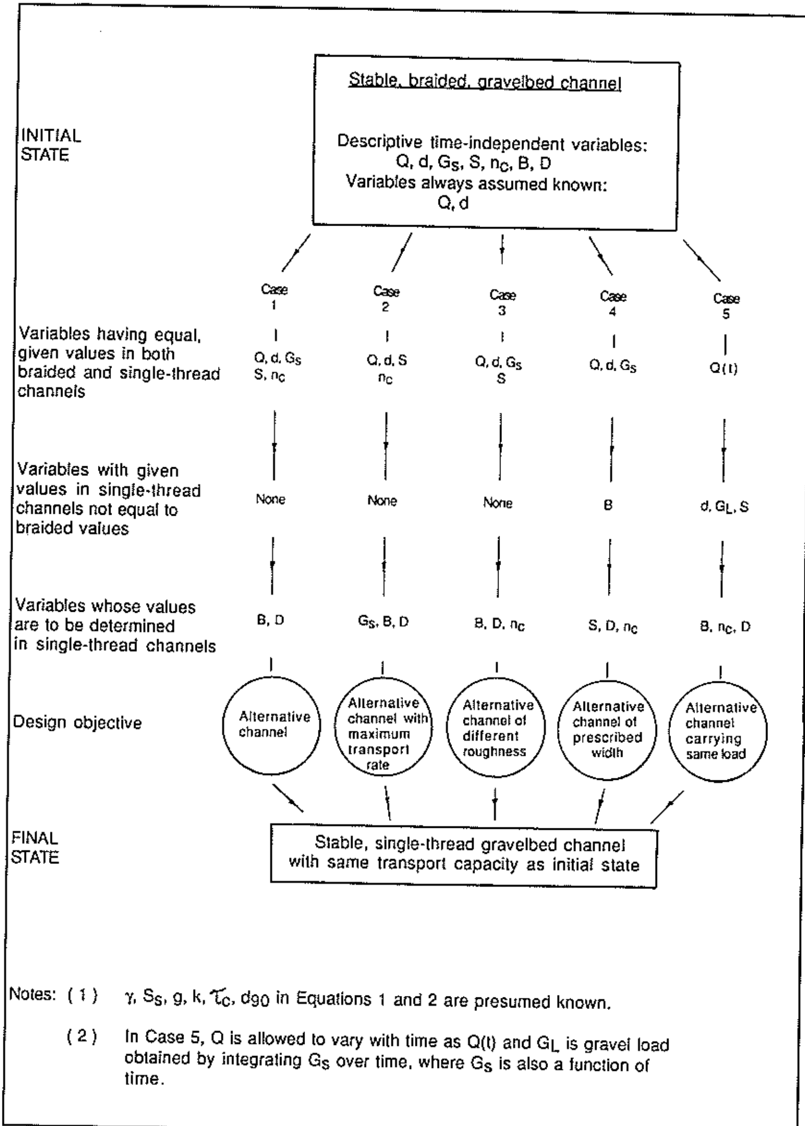

FIG. 1—Flow chart illustrating design cases involving conversion of a stable, braided, gravel-bed channel to a stable, single-thread channel of equivalent transport capacity.

Case 1 For given Q, d, G_s, S and n_c (Fig. 1), Equations 1, 2, and 3 yield a set of (G_s,B) pairs which define a parabolic relation supplying two values of B for all G_s, except the maximum value. An explicit expression for $G_s(B)$ is not available as B and D are implicitly related in Equation 3. A method of solution is to nominate B and from Equation 3 calculate D. Then Equation 1 furnishes G_s with n_p defined by Equation 2.

Case 2 Here we seek an alternative single-thread channel with the maximum possible G_s for given Q, d, S, and n_c. (The reason for treating this problem is noted later.) Carson and Griffiths (1987) have shown that this occurs when

$$1 - \left[\frac{9\,(m-2)}{2\,(5\,m+6)}\right] = \frac{\tau_c(S_s-1)dn_c^{1.5}}{RSn_p^{1.5}} \tag{4}$$

$$\frac{(m+2)^2}{m}\left\{\frac{\tau_c}{\dfrac{S}{(S_s-1)d}\left[\dfrac{m+30}{2\,(5\,m+6)}\right]}\right\}^{2.67} = \frac{n_p^4\,Q}{n_c^3 S^{0.5}} \tag{5}$$

in which m = B/D and where R=mD/(m+2). A method of solution, for known Q, d, S and n_c (Table 1) is to solve Equation 5 for m and then substitute this

TABLE 1—Measured and calculated bedload transport rates ($g.s^{-1}$) for flume experiments of Ashmore (1985).

Run Number	Measured transport rate	Sampling standard error +	Calculated transport rate (Equation 1)*	Calculated maximum transport rate (Equations 1, 4, and 5)++
1	9.08	±.283	9.04	10.9
6	2.52	±.108	2.13	3.61
7	0.90	±.040	1.51	2.11
8	7.93	±.238	12.7	13.6
9	21.0	±.482	18.2	20.2
10	4.76	±.166	4.58	6.49
11	1.13	±0.084	1.63	2.65

\+ Due to fluctuations in the transport rate with time. Precision error of measurement is negligible by comparison.

++ Insufficient information is given in Ashmore (1985) to calculate sampling standard error for either this maximum rate or the ordinary rate. Sampling error will be dominated by sampling errors in B and D (based on 80 observations per run where B, D were not too variable). Error analysis of Equation 1 suggests that the sampling standard error of a calculated rate is unlikely to exceed that of the corresponding measured transport rate.

* Using a best fit value of k = 4

value into Equation 4 to obtain R and thus D and B. Equation 1 then yields G_s(maximum).

Case 3 This case is similar to Case 1 except that n_c is an additional unknown. Griffiths (1989) gives prediction formulae for Darcy-Weisbach friction factor, f, as a function of Shields Entrainment Function and relative roughness, D/d. With $n_c = (f/8g)^{0.5}R^{0.167}$ a solution method is to nominate R, put R = D and then use Griffiths (1989) to determine n_c. As Q, d, G_s, and S are known (Fig. 1), Equation 1 then supplies B, and with R specified D can be determined. Equation 3 then supplies a value of Q and the whole process is repeated until predicted and prescribed values of Q match.

Case 4 Here the width of the single-thread channel is given, along with Q, d and G_s, but S, D and n_c are unknown. A method of solution is to nominate a value of S along with a value of R; Equation 3 then furnishes (with B and R giving D) an estimate for n_c. Substitution of this into Equation 1 yields a value of G_s to be compared with the known value (Fig. 1). This process is repeated until an estimate of R is found which gives a match for G_s. For this R, Equation 3 yields another figure for n_c which is compared with the value given by Griffiths (1989) or a similar predictor. The entire process is repeated until a match with n_c is achieved.

Case 5 In this case both Q and G_s are functions of time (Fig. 1). It is presumed that conditions in, say, an upstream braided reach are completely known (from backwater curves, gauging information and field survey) or calculable (from Equations 1, 2, 3 for example) and the task is to design a downstream reach with different but known d and S to pass the same gravel load over time. One solution method begins by calculating from Equations 1, 2, and 3 a gravel transport rating for the upper reach in the form of G_s v Q, that is, the function $G_s(Q)$. With Q(t) expressed as a flow duration curve, total load for the period of Q(t) denoted as G_L, is determined in the usual manner from

$$G_L = \sum_{i=1}^{i=j} G_s\,[(Q_i + Q_{i+1})/2]\,\Delta\,\tau) \qquad (6)$$

where Δt is the percentage of time the flow lies between Q_i and Q_{i+1} and there are j discrete values of Q defining the flow duration relation. Now, to match G_L between the upper and lower reaches the two ratings, which will differ owing to different S and d (Fig. 1), must intersect so that the relatively higher values, for a given large Q, of one rating are compensated by relatively lower values of the same rating for given small Q. The intersection point yields the set (G_s', Q'). With Q' guessed, G' read from the upper rating and n_c eliminated between Equations 1 and 3, D is obtainable for given G_s', Q', d and a nominated value of B. In turn B (with D) gives R and from Equation 3, n_c. The process is repeated for different B until n_c agrees with predictions of Griffiths (1989) or other estimates. Once B is determined, then for a specified value of Q, R may be found from Equation 3 and Griffiths (1989) and so, with Equation 1, a rating $G_s(Q)$ can be calculated for this value of B. Then, using Equation 6 and the flow duration curve, G_L for the lower reach can be found. The process is repeated for different values of Q′ until the upstream and downstream G_L values equate (Fig. 4).

APPLICATION

In order to give worked examples of the various design cases (Fig. 1) and to compare results with observations, two suitable data sets were found in the literature. For cases 1 to 4 (Fig. 1) only laboratory data could be used because of absence of values of certain parameters, notably G_s which is notoriously difficult to measure, in field data sets for braided rivers. In what follows comments are made about data selection, design formulae are calibrated with this data, and design solutions are obtained and discussed.

Data Selection

The laboratory flume data of Ashmore (1985) was chosen for illustrating cases 1 to 4 (Fig. 1) because it was the only set found with the requisite parameters. Ashmore (1985) conducted 10 flume runs involving braided channels at 7 different combinations of bed slope and water discharge. Non-uniform sand bed material was used (d = 1.16 mm) and all runs were steady-state Froude models of typical prototype braided streams (Ashmore, 1985). Of the runs, numbers 2, 3, and 4 were replications of runs 1, 7 and 6 respectively. With the former set 2 nozzles were used to introduce sediment to the flume; with the latter (and all others) 4 nozzles were employed. These different experimental arrangements produced differences between replicate runs which are too large for the purposes of this analysis. For example, run 4 had supercritical flow whereas its replicate, run 6, had subcritical flow; and runs 3 and 7 differed markedly in width and roughness. Accordingly, the results of runs 2, 3 and 4 are not considered here leaving runs, 1, 6, 7, 8, 9, 10, 11 (there was no run 5) (Ashmore, 1985, p. 64).

In addition to laboratory data, data from the Waimakiriri River (Griffiths, 1979, 1981; NCCB, 1986; Carson & Griffiths, 1987), supplemented by information from the Ohau River (Davoren & Mosley, 1984), were chosen to illustrate case 5 largely because data were available from two reaches as is required in this example.

Calibration of Design Formula

Because all variables in Equations 1, 2, and 3 are either known from data or are to be calculated, the only calibration required is determination of τ_c and k in Equation 1. Ashmore (1985) adopted $\tau_c = 0.03$ based on the performance of the Parker (1978) relation in modelling his laboratory data. Field work by Andrews (1983) with gravel also indicates a value of 0.03. However, the particle Reynolds Number $Re_\star = u_\star d/\upsilon$ (where $u_\star$ is shear velocity = $(gRS)^{0.5}$ and υ is kinematic viscosity of water) in runs 1, 6, 7, 8, 9, 10 and 11 (calculated using d_{90} = 2.67 mm and $\upsilon = 1.25 \times 10^{-6}\ m^2s^{-1}$) of Ashmore (1985) averages 73. If τ_c for $Re_\star > 600$ is taken as 0.03, as with Parker (1978) and Andrews (1973), then, presuming a decrease from $\tau_c = 0.03$ with decreasing $Re_\star$ at the same rate as that from $\tau_c = 0.056$, $Re_\star = 600$ in Sheilds' curve (Henderson, 1966), it follows that if $Re_\star = 73$ then τ_c is about 0.02. With $\tau_c = 0.02$ the data of Ashmore (1985) were fitted to Equation 1 (with n_c determined by Equation 2) to yield a regression result of k = 4. Figure 2 shows an excellent performance

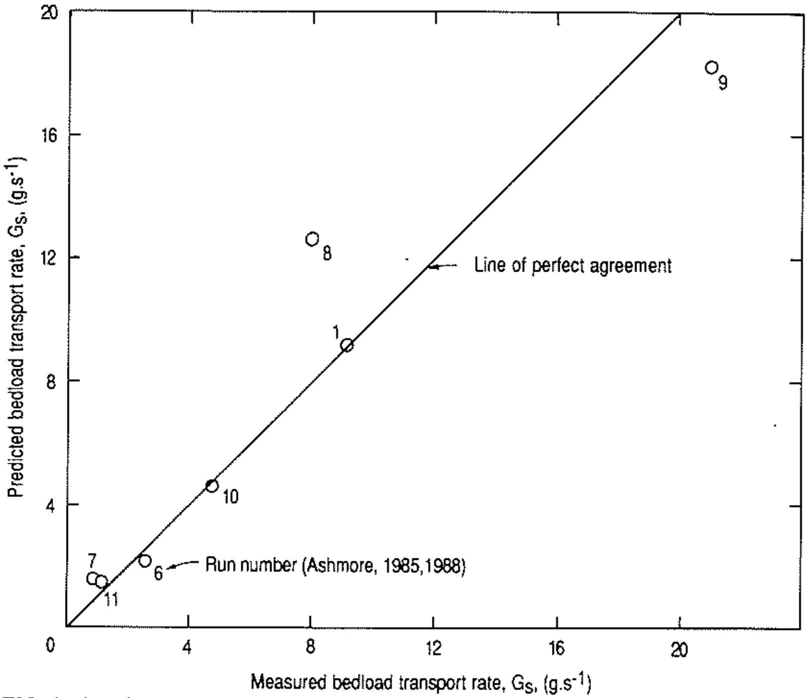

FIG. 2—Predicted versus measured bedload transport rates for selected flow experiments of Ashmore (1985, 1988).

of the calibrated Equation 1 in modelling the laboratory data — a necessary outcome for making inferences from the design cases. The only point of difficulty in this calibration arose over the value of n_p. With a data value of d_{90} = 2.6 mm, Equation 3 yields n_p = 0.014, and yet run 8 yields a measured value of n_c = 0.012. Since n_c must be greater than or equal to n_p the measured lower limit of n_p = 0.012 was adopted for all runs.

For the natural river example (Case 5), k = 8 and τ_c = 0.059 were selected following the success of Equation 1 in modelling field data from the Ohau River using these values (Griffiths, 1985).

Design Solutions for Laboratory and Natural Channels.

Case 1 Data from run 1 of Ashmore (1985) were used to calculate $G_s(B)$ shown in Figure 3. Relevant values are Q = 0.003 $m^3.s^{-1}$, S = 0.015, d = 1.16 mm, S_s = 2.65, B = 0.975 m, R = 0.085 m, G_s = 9.08 $g.s^{-1}$, n_c = 0.014 and n_p = 0.012. The width of the required alternative channel, which according to the results from runs 4a and 4b of Ashmore (1985) would be single thread is, from Figure 3 (with G_s = 9.08), about 0.13 m. It is open to question whether a laboratory channel would reproduce this behaviour with just width and depth changing between braided and single-thread forms; the main point of the example is, however, to demonstrate that two real values for B are possible.

Case 2 Table 1 compares G_s values for all runs calculated by Equation 1

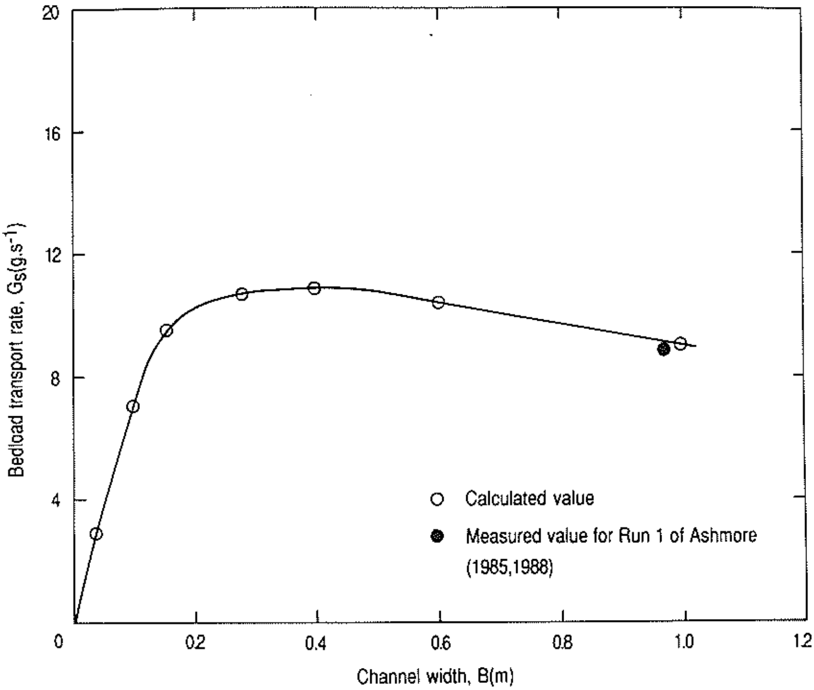

FIG. 3—Bedload transport rate versus channel width calculated from Equations 1, 2, and 3, and as measured for Run 1 of Ashmore (1985, 1988).

(see also Figure 2) with maximum values of G_s calculated by using given values of Q, d, S and n_c (Fig. 1). In all instances the maxima are the larger values. This suggests that stable braided channels do not necessarily adjust themselves to attain a maximum transport rate. One school of thought believes this adjustment occurs in single-thread stable channels under similar external conditions to those imposed here and provides a basis for stable channel design because, with Q, d, and S given, then B, D, and c can be determined subject to G_s being a maximum. Further details are given in Carson and Griffiths (1987, pp. 103–104).

Case 3 Data from run 1 of Ashmore (1985) were again used in this example. The design single-thread channel is much narrower (B = 0.314 m versus B (braided) = 0.975 m) and somewhat rougher (n_c = 0.017 versus n_c = 0.014). To some extent this matches the behaviour of runs 4a and 4b of Ashmore (1985) (rejected here for replication reasons): run 4a was braided (B = 0.728 m) and, for unknown reasons, switched to single-thread (B = 0.378 m) at the same Q and S to become run 4b. Roughness decreased from n_c = 0.021 to n_c = 0.019, but then G_s was allowed to vary from 3.28 $g.s^{-1}$ in the braided channel to 5.47 $g.s^{-1}$ in the single-thread, whereas here, G_s is held fixed. Nevertheless, despite these different controls the laboratory observation suggests the design example is realistic and probably reproducible in a flume. Unfortunately we cannot construct a design example from run 4a to predict run 4b because there are 4 unknowns — B, G_s, n_c and D.

Case 4 Here again run 1 data were employed to yield a design channel with S = 0.013, n_c = 0.014 and R = 0.02 m for a prescribed single-thread width of B = 0.2 m. The prediction of a flatter slope and increased depth is in accord with field examples (Leopold et al, 1964; Henderson, 1966) of braided channels changing to single-thread channels confined by rock-walled gorges or banks composed of material more cohesive than the bed material.

Case 5 A stable reach just downstream of Halkett Groyne in the Waimakiriri River (Griffiths, 1979) was chosen as the upstream braided reach for this example. Flow gaugings, flood-level observations and backwater curve calculations furnished the required data. A typical set is: Q = 2250 m^3s^{-1}, d_{50} = 0.025 m, S = 0.0052, R = 0.896 m, B = 850 m, n_c = 0.032, n_p = 0.025. For this set Equation 1 yields G_s = 750 kgs^{-1}. A bedload rating or $G_s(Q)$ relation was calculated and combined with the measured flow duration curve according to Equation 6 to yield the function $G_L(Q)$ shown in Figure 4. (The units of G_L are kg per unit period where the total period is the one for which the flood duration curve applies.) The water discharge responsible for transporting most of the load is about 2250 m^3s^{-1} or approximately 1.5 times the mean annual flood. Bankfull discharge in this reach is of the order of 2000 m^3s^{-1} in rough agreement with the classical theory that the maximum value of $G_L(Q)$ (one definition of the dominant discharge) should be the bankfull discharge (Carson & Griffiths, 1987).

Prescribed characteristics of the lower reach, located some 2 km above Old Highway Bridge (Griffiths, 1979), include: S = 0.0015, d = 0.023, n_p = 0.023 and G_L = 51 units. After lengthy numerical trials the intersection point of the $G_s(Q)$ ratings for the upper and lower reaches was found to be G_s' = 400 kgs^{-1}, Q′ = 1900 m^3s^{-1} for which B = 215 m, n_c = 0.028, and G_L = 50 units. The actual width of the single-thread gravel-bed channel in this artificially confined reach is around 350 m, which may be too wide as persistent aggradation

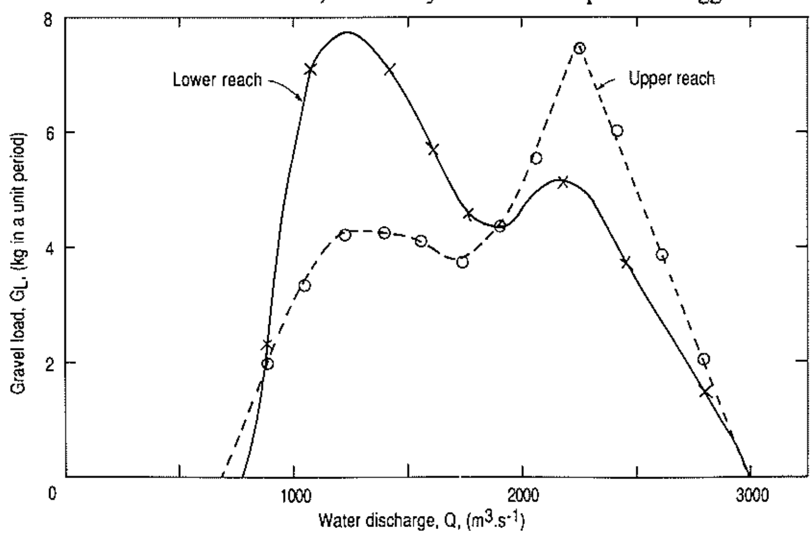

FIG. 4—Gravel load versus water discharge for two reaches of the lower Waimakariri River, New Zealand.

has occurred since circa 1930. Re-design aimed at halting this deposition and achieving a stable channel might be approached in the above manner, but would need to account for a compound section consisting of a central gravel-bed channel flanked by berms: calculations would be lengthy but straightforward.

In contrast to the upper reach the largest value of $G_L(Q)$ occurs at $1250m^3s^{-1}$ (Fig. 4) which is about 0.85 times mean annual flood. Bankfull discharge is approximately $1500m^3s^{-1}$ but cannot strictly be compared with the former figure as this applies to a stable reach of narrower width. A final, important point concerning Figure 4 is this: if the reaches in a section of river are not all identical then the gravel outflow ratings, $G_s(Q)$, will differ from reach to reach so that, between floods, inflows of gravel will not balance outflows and storage changes will occur. Over periods of, say, 5 to 10 years however, G_L values must be the same for a channel to be stable. That storage occurs is well known to river-control specialists; but Figure 4 offers a theoretical starting point for an analysis of this problem and perhaps also that of the evolution and transport of sediment waves (Griffiths, 1979; Meade, 1985; Ashmore, 1985).

FUTURE WORK

For various design cases results and predictions have been obtained which are in accord with observations from braided and single-thread (sand-bed) channels in the laboratory and (gravel-bed) channels in the field. The design equations require further performance testing and appraisal. Experimenters are therefore encouraged to model conversions of braided to single thread channels for the design cases presented. A promising start in this direction has already been made by Davies and Lee (1988). Equally, field data and observations on actual conversions, such as in the Tukituki, Wairau (Marlborough) and Ashburton Rivers of New Zealand, to name just three, are required to test the ideas put foward here and to develop further design procedures. There is an increasing demand for knowledge of the subject as a number of actual conversion schemes in the braided rivers of New Zealand near the end of their design lives. Important decisions about future river control and floodplain management (Bewick, 1988) will soon have to be made. These decisions will depend to a large degree on our understanding of sediment transport in braided and single-thread gravel-bed rivers: the less known the more conservative and uneconomic river control measures are likely to be.

CONCLUSIONS

The main conclusions of this study are:

(1) The Meyer-Peter and Muller formula for bedload transport, the Manning formula for flow resistance and the predictive equations of Griffiths (1989) for resistance coefficient can replicate very well the behaviour of small-scale, braided, sand-bed streams with steady flow. Examples from the literature demonstrate that, in natural channels where the resistance coefficient is usually known from measurements, the first two formulae also perform well.

(2) The design equations, calibrated with laboratory data, suggest that stable

braided channels do not necessarily adjust themselves to attain a maximum transport rate for imposed water discharge, sediment size and bed slope.

(3) Conversion of a braided to a single-thread channel of prescribed width is shown, using laboratory data, to be accompanied by a reduction in slope and increase in depth — both features known to occur in the field.

(4) The basis for a design method is presented for conversion of a braided river to single-thread form. It accommodates naturally varying flow and gravel discharges and is based on the assumption of equivalence of transported load over time. One important inference arising from the procedure is that, in natural rivers with channel characteristics changing downstream, gravel storage must vary from reach to reach. The procedure may offer a starting point for a much needed theoretical analysis of this frequently observed phenomenon.

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