

River Roughness – the integration of diverse knowledge

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ABSTRACT: In 2000, the Environment Agency of England and Wales identified the need to reduce the uncertainty associated with flood level prediction through using recent research advances on river and flood plain conveyance (Samuels et al, 2002). This has led to the development of the Conveyance Estimation System (CES), involving a partnership between researchers and experts in hydraulics, aquatic vegetation and software development and with close consultation with typical users. A key component of this development was taking knowledge on river resistance from a diverse set of sources, covering different types of vegetation in the river and on the flood plain and, different channel dimensions and bed material type. The calculation methods are documented elsewhere (McGahey & Samuels, 2003) and are based on the Darcy friction factor to represent local river bed stresses coupled with other fluid dynamic processes. Most data sets on flow resistance provide average values of Manning's n for whole river sections and include the influence of bed material, vegetation and larger scale topographic variation. A key requirement of the CES project was to produce a roughness advisor to allow the user to access this information in a structured manner and construct information for the CES conveyance calculation. The field data also needed further interpretation to assess the local resistance properties to include in the calculations. This paper describes the integration of this information on flow resistance into the conveyance calculations in the CES software including an approach for the estimation of friction factors when the flow depth is comparable to the effective roughness length scale.

1 INTRODUCTION

Over the past two decades there has been an improvement in knowledge and understanding related to estimating the conveyance capacity of natural channels. Research into the exact nature of the flow processes such as local friction, turbulence due to lateral shearing and the presence of secondary flows, together with an ever increasing database of observed flow parameters such as the managed programme of research on the Flood Channel Facility (FCF) at HR Wallingford, provides the facility for testing and validating these new approaches. Significant contributions include those of Chang (1983), Irvine & Ellis (1987), Shiono & Knight (1989), Ackers (1993), Bousmar & Zech (1999) and Irvine *et al* (2000). Historically, most one-dimensional hydrodynamic software packages are based on the Manning equation, first published in 1891 (Manning, 1891). This empirically derived equation is not based in rigorous physics and provides meaningless results for sudden changes in area or wetted perimeter with depth, where shape effects are ignored.

In 2000, the Environment Agency commissioned a Scoping Study into reducing the uncertainty re-

lated to flood level predictions. The key outcome was a Targeted Programme of research to be undertaken by a multidisciplinary team of experts and led by HR Wallingford, which would deliver a Conveyance Estimation System (CES). This stand-alone software would comprise three core components: the Roughness Advisor for assistance in the estimation of local roughness, the Conveyance Generator for the improved calculation of conveyance and the Uncertainty Estimator which provides some measure of uncertainty for the predicted water level. The CES would be available as open code for inclusion in any one-dimensional hydrodynamic modelling software.

The conveyance methodology to be incorporated in the CES (McGahey & Samuels, 2003) is based on the depth integration of the Reynolds Averaged Navier-Stokes Equations (HR Wallingford, 2003).

$$gHS_o - \frac{f\beta q^2}{8H^2} + \frac{\partial}{\partial y} \left[\lambda H \left(\frac{f}{8} \right)^{1/2} q \frac{\partial}{\partial y} \left(\frac{q}{H} \right) \right] = \alpha \Gamma + (1 - \alpha) C_{uv} \frac{\partial}{\partial y} \left[\frac{q^2}{H} \right] \quad (1)$$

where:

g	gravitational acceleration (m/s^2)
q	streamwise unit flow rate (m^2/s)
H	local depth normal to the channel bed (m)
S_o	reach-averaged longitudinal bedslope
y	lateral distance across the channel (m)
β	coefficient to account for influence of local bedslope on the bed shear stress
α	function of the reach-averaged sinuosity

The total flow rate Q (m^3/s) is calculated from integrating this lateral unit flow distribution across the channel section, and hence the total cross-section conveyance K (m^3/s) is determined from,

$$K = \frac{Q}{S_f^{1/2}} \approx \frac{\int q dy}{S_o^{1/2}} \quad (2)$$

where the reach-averaged longitudinal friction slope S_f is approximated by the reach-averaged longitudinal bedslope S_o .

Equation (1) has four calibration coefficients: the local friction factor f , the dimensionless eddy viscosity λ , the secondary flow parameter Γ and the coefficient of meandering C_m . The intention of this paper is to describe the approach adopted for determining the lateral variation of f for a given depth of flow. The conversion of a unit roughness 'n' to an equivalent resistance f is described, with particular consideration given to the evaluation of f where the roughness height is large relative to the local water depth.

2 LATERAL DISTRIBUTION OF f FOR A GIVEN DEPTH

Previous approaches for determining the lateral distribution of f for a given depth include:

- 1 Diagrams for pipe resistance such as the Moody Diagram, where the pipe diameter can be replaced by four times the hydraulic radius R (m) for open channel flow.
- 2 The Blasius equation which is applicable for smooth turbulent flows where the flow conditions are dominated by the viscous forces, typically used for experimental flumes ($4000 < Re < 10^5$). This formula is based on pipe flow.
- 3 The Kármán-Prandtl equation which is used at higher Reynolds Numbers, where the Blasius equation under-predicts f .
- 4 The Hazen-Williams equation, which is derived for pipe flow.
- 5 The 'Power Law' approach, which relates the ratio of the Darcy-Weisbach resistance coefficient on the floodplain f_{fp} and main channel f_{mc} respectively, to the relative depth by a power of 3/7

(Shiono & Knight, 1991; Rhodes & Knight, 1994).

- 6 Calibrated relationships that relate the main channel resistance f_{mc} to the floodplain resistance f_{fp} by an empirically derived factor R_f (Knight & Abril, 1996; Abril, 1997).
- 7 The Chezy (1768) equation with an additional conversion to relate the Chezy coefficient C ($\text{m}^{0.5} \cdot \text{s}^{-1}$) to f as a function of the hydraulic radius. The Chezy equation is based on balancing the resistive force due to the wetted perimeter and the square of the average velocity with the driving force due to the longitudinal bedslope and the cross-sectional area. It is therefore applicable where rough turbulent flow dominates as a result of the boundary friction represented by C .
- 8 The Manning equation with an additional conversion to relate Manning's n to f as a ratio of the local depth to the power 1/3. This equation is independent of the Reynolds number and the friction factor depends only on the hydraulic radius. Manning's equation is therefore only recommended for fully rough turbulent flow conditions. The units of n are $\text{s} \cdot \text{m}^{-1/3}$, although some authors believe that the roughness is not related to time and that it includes a gravitational term $g^{0.5}$, giving the actual units of n as $\text{m}^{1/6}$ and thus relating n to the hydraulic radius $R^{1/6}$. Based on the approximations in deriving the Manning equation and the uncertainty in the value of n , it seems unjustifiable to carry the numerical constant to more than three significant figures (Chow, 1959).
- 9 The full Colebrook-White (1937) equation i.e. inclusive of the smooth and rough turbulent laws. White identified two types of surface: those that give a constant f at sufficiently high Reynolds Numbers i.e. where f is dependent on the relative roughness only (rough law) and those for which f continues to decrease as the Reynolds Number increases (smooth law). In general, the values of f appear to be larger for pipes at the same Reynolds Number and relative roughness. Kirshmer (1949) attributed the differences to the secondary flows that are not present in pipe flow. The transition from smooth to fully rough flow is further dependent on the form and spacing of the roughness elements (Henderson, 1966; US Task Force, 1963).

These approaches are not applicable to channels whose beds are in motion. It is assumed throughout that the bed topography is fixed.

The lateral distribution of f in the Conveyance Generator is based on a form of the Colebrook-White equation, with coefficients suitably adapted for open channels.

$$\frac{1}{\sqrt{f}} = -2.03 \log \left[\frac{k_s}{12.27H} + \frac{3.09\nu}{4q\sqrt{f}} \right] \quad (3)$$

This equation was selected over the Chezy and Manning equations, as (i) it covers smooth turbulent, transitional and rough turbulent flows; (ii) it has a strong physical basis as it is derived from the logarithmic velocity profile together with the channel geometry; and (iii) it incorporates the variation of roughness with depth. Data measurements from the Mississippi, Tennessee and Irrawaddy Rivers (Chow, 1959) clearly illustrate this variation of the cross-section roughness n with depth, and in particular, the rapid increase of n at low flows. Ackers (1958) published a thorough study of channel flow and concluded that the Colebrook-White formula is the best available. He noted that k_s should be increased by 20% over the value for the same material in pipes, to correct for the different cross-sectional shape.

In equation (3), the coefficient '2.03' is from taking von Kármán's universal constant as 0.4 (assumes clear water). The coefficients '12.27' and '3.09' are based on this coefficient together with the channel cross-section shape and the nature of the roughness elements. Previous research (Henderson, 1966; Keulegan, 1938; Rouse, 1938; Zegzhda, 1938) has provided advice on the coefficients for a variety of channel types, ranging from smooth rectangular shapes to fully rough, wide, natural channels. The coefficients in equation (3) are based in part on this advice. However, since the application of the Colebrook-White is to local element 'slices' rather than the entire cross-section for which the equation was derived, these predictions of f are further compared to the f -values back-calculated from bed shear stress measurements. The EPSRC FCF managed programme of research provided high quality flow, velocity and shear stress data for straight, skewed and meandering trapezoidal compound channels, situated in a 56m long, 10m wide and 0.4m deep laboratory flume. Velocity measurements were taken with a vane attached to a rotary potentiometer at regular spacing across the sections, for a number of depths within each vertical column. This enables calculation of the local depth averaged velocity profile U_d (m/s), and together with the measured (using a Preston tube) bed shear stresses τ_b (N/m²), the local resistance f can be determined,

$$f = \frac{8\tau_b}{U_d^2 \rho} \quad (4)$$

where ρ (kg/m³) is the fluid density. Comparison of the Conveyance Generator predicted f -values to the FCF Phase A data (Figure 1) confirms the applicability of the selected coefficients in equation (3) for smooth channels over a range of depths, including both inbank and out-of-bank flow.

Historically, the Manning n -value (Chow, 1959) has been widely used for describing resistance due to vegetation, substrate and other anticipated energy losses in the channel and floodplain system. This n -value is defined here as an engineering n_e -value, as it incorporates energy losses due to local bed friction, turbulence generated from the main channel - floodplain interaction and helical secondary flows that result from the planform channel sinuosity. This Manning n_e is thus all-encompassing.

Due to the widespread use of this Manning n_e -value, most resistance advice, photographs and summation approaches in the literature (Barnes, 1967; Chow, 1959; Cowan, 1956; Hicks & Mason, 1998) are expressed in terms of ' n_e '. The Roughness Advisor (RA) in the CES is thus based on an n rather than an f resistance value to maintain user familiarity and confidence. This was a critical decision from the Project User Consultative Group to receive wider user acceptance. The fundamental difference is that this advice is based on a local n_l value, which is the equivalent 'Manning' n_e -value due to the local bed friction only. This includes boundary losses due to three *components*: surface material (e.g. sand, clay, bedrock), vegetation (e.g. grass, reeds, water lilies) and irregularities (e.g. groynes, urban trash, pools and riffles). All the remaining energy losses are incorporated through the other calibration parameters λ (lateral shear), Γ (secondary flows in straight channels) and C_{uv} (secondary flows in meandering channels) in equation (1).

The roughness advice provided by the RA is based on an extensive literature review of over 700 references covering existing methods and data for estimating roughness (HR Wallingford, 2003). Many of the approaches have been developed for a particular aspect of the river channel e.g. vegetation, boulders or large-scale roughness elements. The measured data is generally given in terms of a combined roughness value for the section i.e. inclusive of the three roughness *components* as well as shape effects. The process of disaggregating the data into unit components due to the different features was a difficult task. Since Chow, which forms the basis of the RA, there have been more studies providing insight into the roughness of different features, especially vegetation.

The RA provides advice through descriptions that are where possible, accompanied by photographs. In the absence of any survey data or channel description, the RA provides advice using the national data set obtained through the national survey of river habitats (River Habitat Survey, Raven et al 1998). This survey was based on a 10 by 10 km square grid and 25 items were assessed at each river cross-section, spaced at 50m intervals, in 500m long river segments. This enables the user to enter the UK grid

reference of the study reach and receive advice on expected in-channel and bank-side aquatic vegetation.

For a channel reach, three roughness *types* have been established in the RA: bed, bank and floodplain. Within each of these types, the roughness can be composed of up to three *components*: surface material, vegetation and irregularities. All roughness components include a minimum, maximum and expected n_l -value and for the vegetation, the seasonal variation of n_l is provided. The surface materials include natural [e.g. bedrock, cobbles, gravel, sand, silt, clay, peat, earth, firm soil, bare ploughed soil] and man-made [e.g. sheet piling, stone block, hazel hurdles, gabion, concrete, rip-rap, wood piling] roughness values. The natural vegetation includes 12 morphotypes (Table 1) based in part on the RHS data as well as categories due to human intervention such as grass, hedges, trees, shrubs and crops e.g. wheat, sorghum, corn, cotton, soybeans and sunflower. Irregularities include urban trash, groynes, exposed boulders, pools and riffles, ridges on ploughed fields, undulations on the floodplain and obstructions such as debris deposits, exposed roots, stumps, logs, piers and isolated boulders.

For solution of equation (1), the lateral variation of f is required, and hence the lateral distribution of this local boundary roughness n_l . Since the local roughness at a point in the section can comprise up to three roughness *components*, these can be combined to get the total unit roughness at a point through,

$$n_l = \sqrt{n_{veg}^2 + n_{sur}^2 + n_{irr}^2} \quad (5)$$

where n_{veg} , n_{sur} and n_{irr} are the local roughness values due to vegetation, surface material and irregularity respectively. These are associated with a depth of 1m, selected as a representative depth of flow for UK rivers. The reason for adopting the “root sum of the squares” approach is twofold: it highlights the contribution of the largest roughness component; and the roughness is squared before being combined since the energy loss is related to the square of the local velocity. The assumption is that the roughness mechanisms are mutually independent and hence the total resistance can be expressed as the sum of the individual resistances.

Table 1. Unit roughness n_l for twelve vegetation morphotypes.

Aquatic vegetation by RHS vegetation type	Unit roughness n_l		
	Min	mean	max
1. None If none visible check predictions from RHS data set			
2. Free-floating plants typically algae or duckweeds, medium deep drainage channel [depth = 1.1 to 2.5m, velocity = 0.1 to 0.6m/s]	0.010	0.030	0.040
3. Filamentous algae attached shallow nutrient rich waters [depth	0.000	0.015	0.050

= 0.05 to 0.5m, low velocities]			
4. Mosses attached to bed or banks [depth \approx 1m]	0.000		0.030
5. Trailing bank-side plants [depth = 0.5 to 3+m]	0.000	0.050	0.100
6. Emergent reeds, rushes, flag and large grasses	0.020	0.150	0.200
7. Floating-leaved typically water lillies in deeper slower waters	0.030	0.100	0.140
8. Emergent broad-leaved rooted plants, water cress or water parsnip	0.050	0.150	0.50+
9. Submerged broad-leaved, pondweeds [depth = 0.06 to 1.2m, velocity 0.2 to 0.9m/s]	0.020	0.100	0.200
10. Submerged fine-leaved, shallow rivers, chalk streams, water crowfoot, pondweeds [depth = 0.2 to 0.6m, velocity = 0.2 to 0.7m/s]	0.020	0.300	0.45+
11. Submerged fine-leaved, medium depth rivers, regular management [depth = 0.6 to 1.2m, velocity = 0.3 to 0.8m/s]	0.021	0.100	0.249
12. Submerged fine-leaved, medium to deep rivers, some management [depth > 1.2m, velocity = 0.3 to 1.0m/s]	0.010	0.080	0.120

Previous work (Einstein & Banks, 1950) employed a similar approach whereby the measured bed shear stress relative to the square of the velocity (τ/v^2) was compared to the addition of this term for the independent resistance components. This is similar to equation (5) with both sides squared. The local roughness contributions from flow over (i) flat blocks, (ii) an offset and (iii) pegs were measured. These were combined and compared to the total measured roughness due to all three components, and the measured total resistance values were found to correlate well with the combined data. It was concluded that the separate resistances should be expected to have an additive property provided the roughness types do not exhibit excessive mutual interference.

4 CONVERSION OF LOCAL MANNING n_l TO AN EQUIVALENT ROUGHNESS k_s

Solution of equation (3) requires the conversion of the local roughness ‘ n_l ’ to an equivalent length scale ‘ k_s ’. Although the roughness height k_s is traditionally defined in terms of the sand grain dimension (Nikuradse, 1933), the vegetation k_s can be interpreted as an equivalent turbulence length scale i.e. a measure of the turbulent eddy size. This length scale may be greater than the water depth, e.g. willow trees, boulders etc., as it represents the horizontal mixing action, whereby eddies may be larger in plan than the local water depth. The equivalent k_s value can be determined from (variation of Strickler, 1923; Ackers, 1958),

$$n_l = 0.038k_s^{1/6} \quad (6)$$

This equation is only applicable where the depth H is in the range of 7-140 times the roughness size. For the flow conditions under which the equation was derived, this was found to be suitable, as it was applied to the whole cross-section or three regions: i.e. the main channel and two floodplains. For application within the Conveyance Generator, two issues arise:

- 1 In equation (1), the solution is required for each lateral division across the section and hence for a given section the local roughness height may be equivalent to a large fraction of the depth or in some cases, exceed it e.g. tall grass on the floodplain. This typically occurs at low inbank and low overbank flows.
- 2 Large typically-used Manning n -values result in highly inflated k_s values. E.g. should the bed material be characterised by large boulders, with an equivalent Manning n -value of 0.5, this would produce a k_s value over 5,000 km, which is meaningless in terms of the channel depth anywhere in the cross-section or entire river reach (Table 2).

An alternative approach is to use the rough turbulent law, i.e. the first term in equation (3),

$$\frac{1}{\sqrt{f}} = -2.03 \log \left[\frac{k_s}{12.27H} \right] \quad (7)$$

where f is given by,

$$f = \frac{8gn_l^2}{H^{1/3}} \quad (8)$$

Hence at a depth of 1m, k_s is given by,

$$k_s = 12.27H10^{\left[\frac{H^{1/6}}{\sqrt{8gn_l(-2.03)}} \right]} = 12.27 \times 10^{\left[\frac{1}{\sqrt{8gn_l(-2.03)}} \right]} \quad (9)$$

This relationship and equation (6) can be viewed in Figure 2 and Table 2 for a typical range of n_l values. As n_l approaches infinity, the bracketed term approaches zero, and k_s asymptotes to 12.27, the coefficient in the Colebrook-White law. In the Acker's approach, k_s approaches infinity, and hence equation (9) is preferable. For the region $0.01 < n_l < 0.025$, both laws predict a similar relationship. These approaches are not suitable for experimental flumes where the smooth turbulent law dominates e.g. in Table 2 the values of k_s for $n_l \leq 0.01$ decrease rapidly. The assumption is that the unit roughness k_s is constant with depth, although this has been shown otherwise for whole cross-section k_s -values in tidal reaches. Knight (1981) considered the variation of n , f and k_s with stage in the 1.2 km long tidal reach of the Conwy Estuary and found that all three param-

eters varied significantly with depth. In particular, the Nikuradse roughness height k_s was found to vary significantly with stage, increasing from 0.2 m at high water to a value (2.5m) comparable with the water depth at low water.

Table 2. Comparison of k_s predictions based on the calculation approach of Ackers and the simplified Colebrook-White Law.

n_l	Equivalent k_s (Colebrook-White Rough Law)	Equivalent k_s (Ackers, 1958)
s.m ^{-1/3}	m	m
0.0010	3.0373E-55	3.3212E-10
0.0050	9.2805E-11	5.1894E-06
0.0100	3.3745E-05	3.3212E-04
0.0500	9.4780E-01	5.1894E+00
0.1000	3.4102E+00	3.3212E+02
0.5000	9.4980E+00	5.1894E+06
0.8000	1.0455E+01	8.7064E+07
1.0000	1.0795E+01	3.3212E+08
100000	1.2270E+01	3.3212E+38

5 CONVERSION OF EQUIVALENT ROUGHNESS k_s TO LOCAL FRICTION FACTOR f

Once k_s has been determined, the full Colebrook-White equation (3) can be used for the solution of f at a given depth. This approach is not universally applicable, and it can provide unrealistically large values of f . Consider the term in brackets in equation (3). Should the flow be characteristic of a natural river, with a large k_s/H ratio, the viscous forces and hence the smooth law contribution become negligible. As k_s/H approaches 12.27, the bracketed term approaches one, and the logarithm tends to zero. The result is that,

$$\frac{1}{\sqrt{f}} = \text{very small positive number} \quad (10)$$

and the inverse of this small number squared provides an unrealistically large f -value (Table 3). This f is meaningless and thus at high k_s/H ratios, a Power Law is introduced.

$$\frac{8}{f} \propto a_2 \left(\frac{k_s}{H} \right)^{a_3} \text{ for } H < a_4 k_s \quad (11)$$

The mathematical formulation of the velocity distribution in the overlap region of the boundary layer (i.e. between regions dominated by either viscous or inertia forces) must be either of the logarithmic or the Power Law form. The logarithmic form (i.e. basis for the Colebrook-White Law) is generally favoured as it covers a wider range of Reynolds Numbers, whereas the Power Law must change with Reynolds Number and also with roughness. In the literature, the Power Law generally relates f to $R^{1/3}$.

For simplicity, it is assumed in equation (11) that $a_3 = 1$ (Samuels, 1985). The coefficients a_2 and a_4 can hence be derived through ensuring equations (6) and (10) are continuous in f and that the derivative of f with respect to depth, df/dH , is continuous at the cross-over point between these two laws i.e. at $H = a_4 k_s$. Expressing equation (11) at the upper limit of its range of applicability,

$$f = \frac{8 k_s}{a_2 H} = \frac{8}{a_2} \frac{1}{a_4} \quad (12)$$

and taking the derivative with respect to depth gives,

$$\frac{\partial f}{\partial H} = -\frac{8 k_s}{a_2 H^2} = -\frac{8}{a_2 a_4 H} \quad (13)$$

Rearranging equation (7) in terms of f and setting $a_1 = 8 \times 2.03^2 [\log_{10} e]^2 = 6.218$ yields,

$$f = \frac{8}{a_1} \left[\ln \left(\frac{12.27 H}{k_s} \right) \right]^{-2} \quad (14)$$

and df/dH is given by,

$$\frac{\partial f}{\partial H} = \frac{-16}{a_1 H} \left[\ln \left(\frac{12.27 H}{k_s} \right) \right]^{-3} \quad (15)$$

Equating the expressions for f and df/dH from equations (12) to (15) gives $a_2 = 41.3015$ and $a_4 = 1/1.6606$. Thus for a k_s/H ratio greater than 1.66, the Power Law is used to solve for f , where

$$f = \frac{8}{41.3015} \frac{k_s}{H} \quad (16)$$

To prevent unrealistically large predictions of f , the upper limit is set at a ratio of $k_s/H = 10$, which corresponds to an f -value of 1.937. This restricts f from approaching infinity as H approaches zero for shallow flows. The maximum measured resistance in Nikuradse's (1933) experiments is approximately 0.125 in the laminar region, and the entire turbulent region falls well below an f -value of 0.1. Essentially this large f -value would occur in regions with extremely shallow water relative to the roughness elements that are expected to have a virtually negligible contribution to the overall cross-section conveyance. Thus the exact value of f is of little significance.

For $k_s/H < 1.66$, equation (7) is employed, and the smooth law component is only introduced for experimental channels.

Figure 3 illustrates how, for a given n_l value ($n_l = 0.05$), these friction laws are selected depending on the k_s/H ratio. For this case, the Colebrook-White equation increases rapidly below a stage of 0.5m, i.e. the region where the k_s/H ratio approaches 12.27. The Power Law replaces the Colebrook-White Law in this region and is restricted at $k_s/H = 10$.

For smooth turbulent flow e.g. experimental flow conditions, a lower limit on k_s is set i.e. $k_s = 0.1$ mm. This corresponds to an n_l value of 0.0109. The distribution for f is determined from the full Colebrook-White equation (3). Since the rough term will have a virtually insignificant contribution, the distribution of f will be based on the viscous effects and can be solved iteratively. The Blasius Law would be a suitable alternative.

The complete approach for estimating the lateral distribution of f for a given depth of flow, from the local n_l values provided by the RA, is illustrated in Figure 4.

Table 3. Calculated resistance f values for an increasing absolute roughness over depth relationship.

Roughness ratio k_s/H	Equivalent resistance f
0.5	0.13
1	0.20
1.66	0.32
2	0.39
8	7.03
10	30.74
12	2598.70
12.26	1935416.14
12.27	∞

6 CONCLUSIONS

An extensive literature review of roughness methods and data has provided a source of interdisciplinary knowledge and advice on river roughness. This roughness information is presented within the RA, a core component of the CES. The RA provides unit roughness values for the three component roughnesses: surface material, vegetation and irregularities. This roughness is based on skin friction only, and does not incorporate form losses or shape effects, the latter of which is considered in the conveyance calculation. The unit roughness values are combined to form the total unit roughness at a point within the channel section, and hence converted to an equivalent k_s value that is constant with depth. The resistance f for a given depth of flow is determined from this k_s value. Consideration has been given to the extrapolation of the existing friction laws for flow conditions that they were not specifically derived for e.g. large roughness values relative to depth.

7 RECOMMENDATIONS

The RA may be extended to provide more advice on representation of urban floodplains, changes in roughness with velocity and / or relative depth [e.g. grass bending at high flows] and form losses between consecutive cross-sections. The roughness

calculation within the Conveyance Generator may be updated to include a function that accounts for the additional resistance due to bedforms.

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ALL FIGURES:

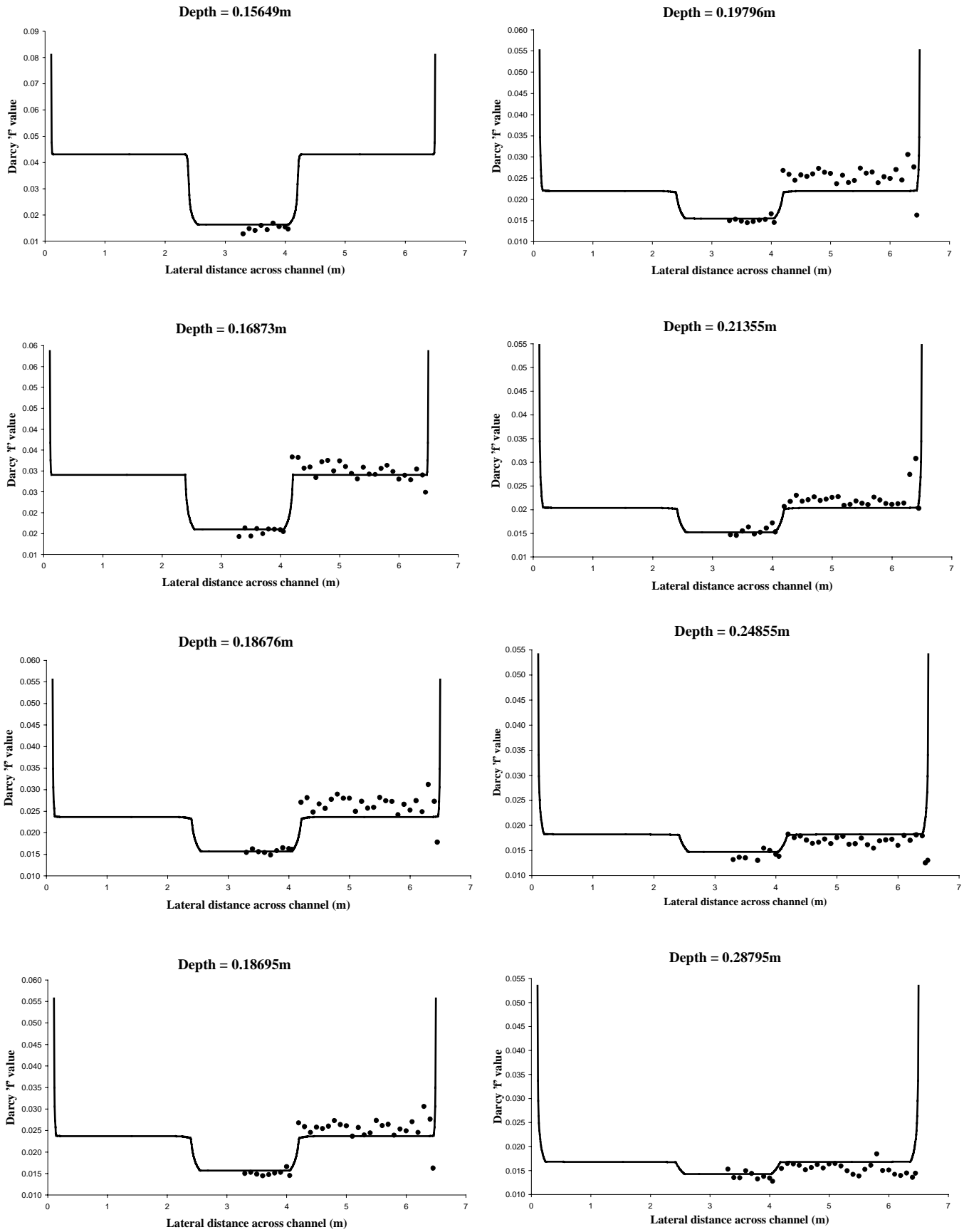


Figure 1. Lateral distribution of resistance f with depth

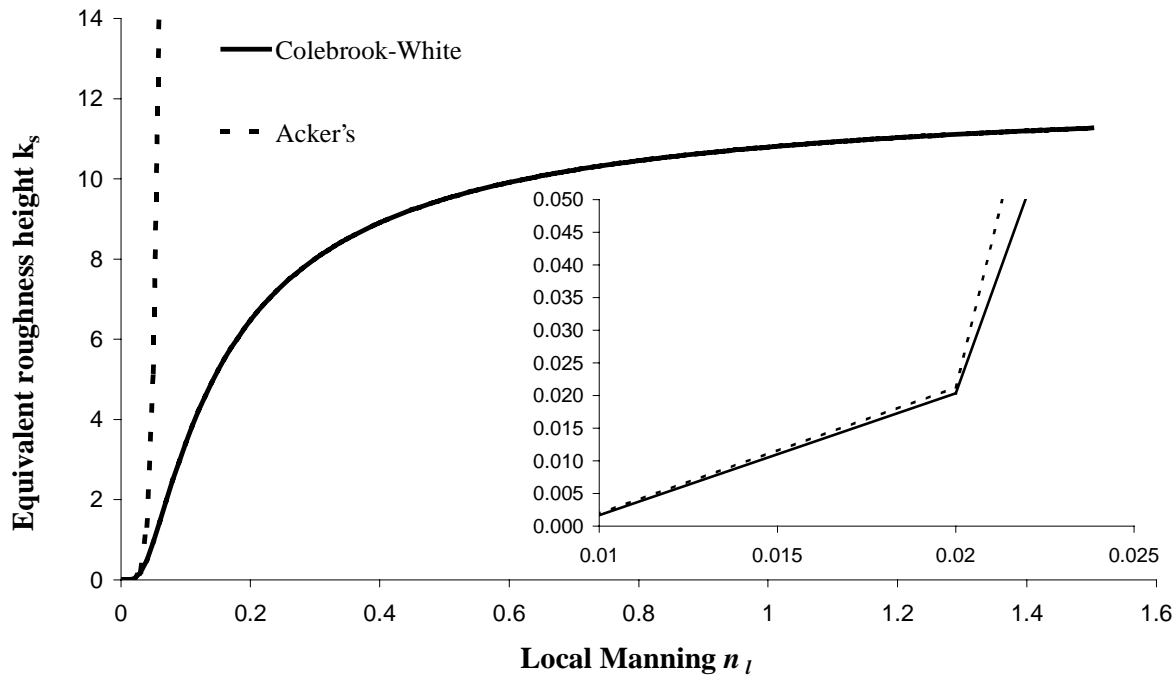


Figure 2. Conversion of n_l to k_s for a typical range of n_l values

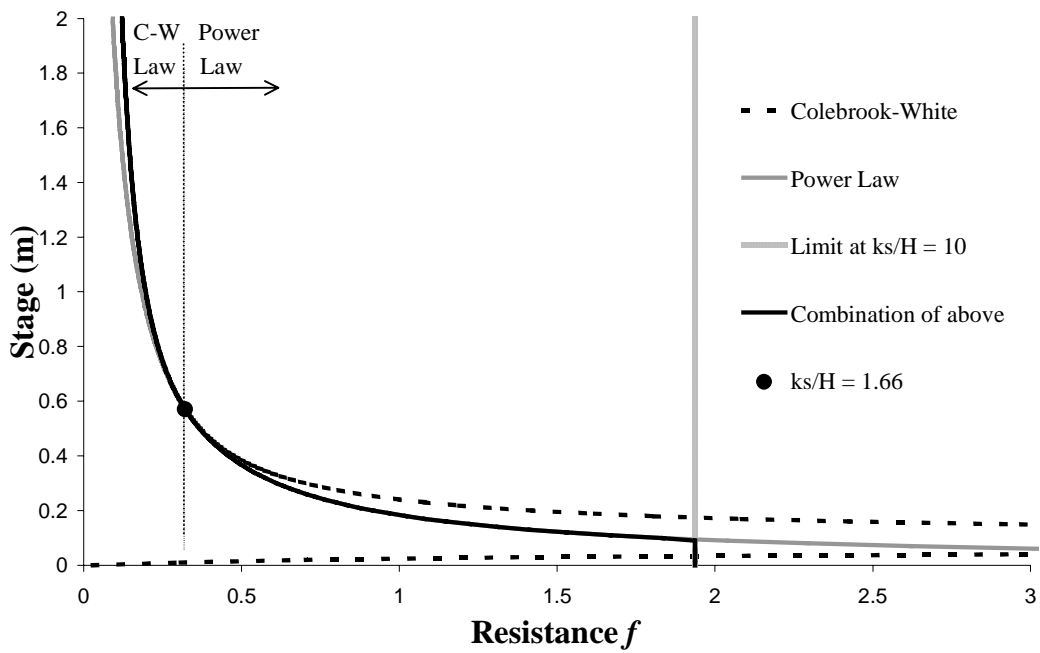


Figure 3. Graphical representation of the combined Friction Laws

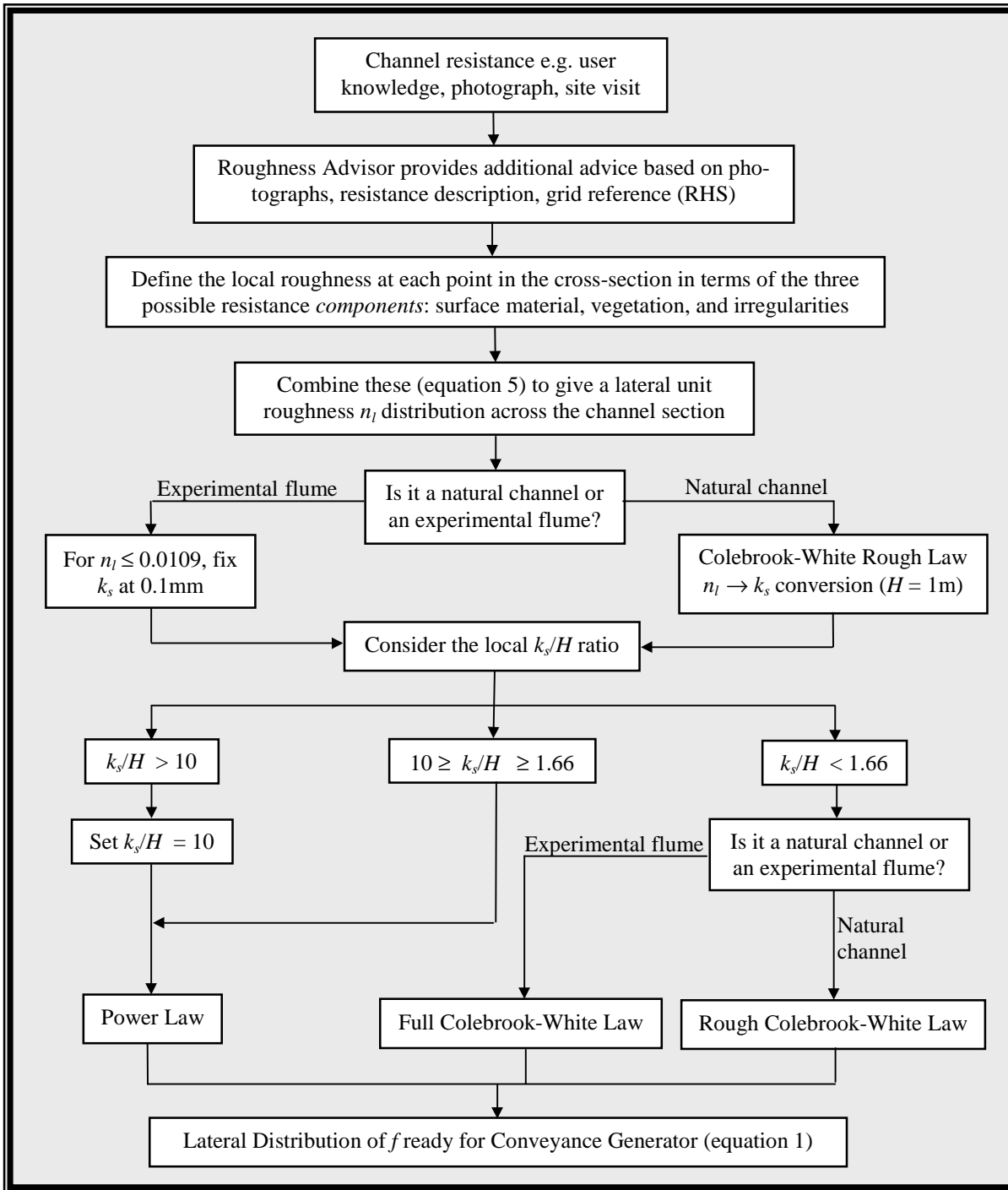


Figure 4. Complete calculation process for the resistance f from the Roughness Advisor n_l -values