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APPENDIX A

ROUGHNESS OF VEGETATED AND COMPOUND CHANNELS:

A REVIEW

Prepared for

Alameda County Department of Public Works

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SUMMARY

In general, little field data has been systematically collected for vegetated and compound channels that would allow confident design of a restored or naturalized channel. Field data on vegetated channels is largely limited to studies of grassed channels or channels planted with a uniform row crop.

Some experimental data suggests that the calculation techniques typically used to estimate the discharge capacity of a compound channel may under- or overestimate discharge capacity. Flows in compound channels have shear stresses and momentum transfer effects at the interface between the main channel and floodplain flows that can result in discharge prediction errors.

On the basis of this review, the following preliminary recommendations for estimate of the discharge capacity of a straight, vegetated compound channel for which calibrated *n*-values are unavailable, with y_r = floodplain flow depth/main channel bankfull flow depth:

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1) For $y_r \leq$ 0.1 .2, neglect flow in floodplain.

2) For $0.3 \geq y_r >$ 0.1 .2, use Method H_i (horizontal division into channel subareas, with the interface included in the wetted perimeter of the main channel).

3) For $y_r > 0.3$, use Method D_e (diagonal interface from the main channel top of bank to the centerline of the channel, with none of the interface included in the estimates of wetted perimeter).

To estimate *n*-values for a channel, use the stiffness method of Kouwen, Li, and Simons, using calibrated or estimated stiffness. Or, if inapplicable or stiffness data is unavailable, any of the generally accepted references, such as USDA (1954) (for slopes > 1%), Chow (1959), Barnes (1967), Hicks and Mason (1991), or McCuen (1989). A table in the latter reference includes estimates for vegetated channels based on data from many sources, including agricultural studies such as are referenced in Section II.

Estimates for floodplain *n*-values should be made based on a verified method, such as one of the following methods, in order of preference:

1) For a wide, wooded floodplain ($5 \geq R \geq 2$), use the vegetation density approach described in Acrement and Schneider (1984). For a grassed floodplain, use the stiffness method of Kouwen, Li, and Simons using calibrated or estimated stiffness.

2) For comparable floodplains, use the photographic-based technique described in Acrement and Schneider (1984).

3) Use a table from a standard reference.

I. INTRODUCTION

A review of the available literature has revealed that little is known about the variation of n *in heavily vegetated channels, except for some grassed waterways. (Petryk and Bosmajian, 1975)*

Various formulas to compute this equivalent or composite roughness can be found in the literature.... To this date, however, very little has been done to evaluate these formulas using natural stream data. (Motayed and Krishnamurthy, 1980)

This report summarizes the status and findings of research regarding the roughness of vegetated and compound channels. Not explicitly included in the discussion is the effect of other factors, such as channel meander, on measured roughness. The document is divided into four sections:

1) description of field-measured roughness values for vegetated floodplains and compound channels;

2) description of theoretical approaches and research, including laboratory studies;

3) recommendations for calculation techniques to apply in estimating roughness values; and

4) list of references.

Most of the references and research cited relate hydraulic resistance to Manning's n-value, primarily because of the prevalence of its use in river engineering. In fact, other measures of roughness or flow resistance may be preferable scientifically because of applicability throughout a full range of flows and dimensional consistency, unlike Manning's equation. Users must be cautious to apply the correct version of Manning's formula for the system of units in which they are working. As Chow (1959) points out, *n*-values vary approximately with the one-sixth power of the roughness height, making a channel *n*-value fairly insensitive to changes in roughness height. However, to the extent that roughness height is affected by flow conditions, Manning's *n*-value for a given channel varies approximately according to this rule for hydraulic reasons alone. Even with these caveats, the use of this formula predominates in practical applications today and is therefore the focus of discussion.

II. FIELD MEASUREMENTS OF ROUGHNESS

Field measurements of channel roughness have concentrated on main channel flow in natural channels, irrigation ditches, channels with vegetation in the main channel, and streams with adjoining agricultural row crops. Studies of the roughness associated with adjoining natural floodplains have been much more limited. Studies of vegetated compound channels, that at high discharges combine the attributes of vegetated channel flow with an unusually deep and width-limited floodplain flow, have been even more limited in number.

Since *n*-values represent a calibration variable that changes with discharge and is highly influenced by a number of variables, it can be quite difficult to apply the results of field measurements directly. Chow (1959) presented the following list of factors influencing Manning's roughness coefficient:

1) surface roughness
2) vegetation
3) channel irregularity
4) channel alignment
5) silting and scouring
6) obstruction
7) size and shape of channel
8) stage and discharge
9) seasonal change (time-dependent vegetative conditions)
10) suspended material and bed load

The studies described below focus on the calibration of roughness in heavily vegetated channels, compound channels, and vegetated floodplains. In some cases the measured *n*-values apply to the entire cross-section; in others, separate *n*-values have been specified for the floodplain or overbank areas alone. They are presented in chronological order.

Manning, R., 1889 and 1895

Various waterways

While Manning's work did not focus on vegetated channels, floodplains, or compound channels, a review of his work provides some interesting insight into the genesis of the uniform flow formula he developed and which has become one of the most widely used methods of estimating discharge. The formula employs a coefficient described

as "roughness," and designated as the "Manning's n" or *n*-value. The formula's genesis was based on 160 observations of flow velocity, of which at least 104 were from artificial channels, 40 from hill streams in Switzerland, and 10 from the Mississippi River. This somewhat narrow range was widened during subsequent years, when Manning tested the formula on 643 observations -- again, dominated by artificial channels (canals and experimental channels), pipes, and large rivers (e.g., the Mississippi, the Rhine, the Seine, the Saône, and the Ohio). None of the notes mentioned vegetation as part of the bed description. Floodplains were not discussed at all.

Ramser, C.E., 1929

Various channels

(Cited in Petryk, Sylvester, and George Bosmajian III, 1975 and Chow, 1959; original work not reviewed.)

In this work carried out for the U.S. Department of Agriculture, various channels were documented photographically and graphs of computed *n*-values relative to water depth are provided. Four of the most heavily vegetated channels have *n*-values computed at approximately 0.05 (low flow depth) - 0.17 (high flow depth in summer). Vegetation ranged from dense willows to heavy stands of trees on the side slopes. *N*-values consistently increased for these four reaches as depth increased.

Pickels, George W., 1931

Drainage ditch, Central Illinois

(Cited in Chow, 1959; original work not reviewed.)

Measurements of changing *n*-values in a vegetated drainage ditches were made. When in good condition in March, 1925, the *n*-value was found to be 0.033. After one year, during which bushy willows and dry weeds had developed in the channel, the *n*-value in April, 1926, was found to be 0.055. During the summers of 1925 and 1926, a dense patch of cattails grew in the channel. (No mention is made as to whether the remains of the cattails were removed prior to the measurement in April, 1926.) At medium summer flows in this condition, the *n*-value was found to be 0.115, and at nearly bankfull discharge, 0.099. The cattails were washed out in September, 1926, and the *n*-value after this event was found to be 0.072.

Scobey, F. C., 1939

Canals

(Cited in Chow, 1959; original work not reviewed.)

Photographs of various canals are presented with accompanying computed *n*-values.

Lane, E. W., 1951

Several large rivers

(Cited in Chow, 1959; original work not reviewed.)

The author presents graphs of water depth versus *n*-value for several large rivers: the Mississippi River (based both on U.S.G.S. and U.S. Army Corps of Engineers data),

the Tennessee River, and the Irrawaddy River in Burma. Bankfull depth is clearly indicated only for the Mississippi River, but the *n*-values of the entire cross section for water flow at depths above bankfull appear to fall in the range 0.026 - 0.034.

Ree, W. O., 1958

Row crops grown in a trapezoidal channel

(Cited in Petryk, Sylvester, and George Bosmajian III, 1975; original work not reviewed.)

Data were developed by the Agricultural Research Service based on measurements made in a trapezoidal channel planted with various row crops. Test flows for wheat, including submergence of the wheat, showed *n*-values of approximately 0.10 - 0.125. For Radian Kafir, for unsubmerged flow depths, the *n*-values were computed at 0.04 - 0.11.

Barnes, Harry H., Jr., 1967

Rolling Fork, Kentucky and other U.S. rivers

Almost all of the data presented in Barnes' classic report pertain to roughness values in main channels. Where overbank flow existed, it was generally omitted from the calculation of channel *n*-value. Examples of *n*-values ranging from 0.024 to 0.075 for the main channel are given, including multiple *n*-values for some reaches when more than one flow was analyzed.

In one of the reaches studied, the Rolling Fork, discrete *n*-values were calculated for the overbank area which was flooded to a depth of approximately 5 - 7 feet. Overhanging trees lined the main channel. The identified *n*-value for the overbank was 0.097, while the main channel *n*-value was 0.046. The channel and overbank area is shown photographically and described as having a "fairly dense stand of trees as much as 6 inches in diameter."

Leutheusser, H. J. and W. O. Chisholm, 1972

Unspecified channel, Canada

(Cited in Petryk, Sylvester, and George Bosmajian III, 1975; original work not reviewed.)

An *n*-value of 0.225 was computed for an open channel with steep, heavily vegetated side slopes. No discussion of flow depth was provided.

Wilson, K.V., 1973

Hanging Moss Creek (channelized), Mississippi

A trapezoidal channel was excavated for this creek in 1963. Over the following 8 years, vegetative growth occurred and *n*-values were computed. All measured flow events were contained within the channel. In the first spring after construction, the channel was clean and the *n*-value was computed to be 0.022, with a flow depth of approximately 5 feet. After a summer's growth, the channel was "lined with fairly thick vegetation consisting of small willows, weeds, and grass." The *n*-value at this

time was computed to be 0.045 at a flow depth of 4.5 feet. Six years later, in October 1970, with full summer foliage present, the *n*-value was computed to be 0.07. At this point willow trees 8 - 10 feet tall lined the channel, and the flow depth was 7.4 feet. The following spring, when foliage was barren, the *n*-value was 0.05 at a flow depth of 6.6 feet. A year later, almost 9 years since construction, under similar barren foliage conditions, the *n*-value was computed at 0.07 at a flow depth of 8.1 feet.

Aldridge, B. N., and J. M. Garrett, 1973

Arizona streams

(Cited in Acrement and Schneider, 1984; original work not reviewed.)

The authors presented cross sections and photographs of channels and floodplains in Arizona with known roughness coefficients.

Klaassen, G.J., and J.J. van der Zwaard, 1974

Experimental channel

Values for the Chézy coefficient were calculated for a simulated floodplain planted with hedges and fruit trees. Values of C in the range of 5 - 48 and 13 - 31 $m^{1/2}sec^{-1}$, respectively, were estimated from the simulations.

Ree, W. O. and F. R. Crow, 1977

Earth channel with grasses and row crops

(Cited in Acrement and Schneider, 1984; original work not reviewed.)

Experiments were conducted on an earth channel planted with certain grasses and row crops to determine friction factors. Photographs and descriptions of the vegetation are given.

Powell, K.E.C., 1978

River Bain, U.K.

The author calibrated *n*-values for a trapezoidal, grassed, fairly straight section of the River Bain. The channel was through clays, gravel, and alluvial spoil with a lining of muds and silt. Growing season flow rates were approximately 0.25 m^3/sec and velocities 0.03 m/sec. Flood flow velocities rose to as much as 1.0 m/sec. In August and September, after the growing season, excavators remove growth and muds from the channel. In the winter, little vegetative matter remains to influence channel roughness. In April, the vegetation begins to reappear. By July, grasses reach 1 m in height, with reeds at 0.65m. Recording gages allowed almost daily estimates of *n*-values to be calculated for this reach over a period of ten years. During selected periods, data was collected at 15 minute intervals and *n*-values were calculated at the same interval. Low flows in July produced the largest estimates of *n* (1.91 and 4.48), but the controls produced by the vegetation at very low flows suggest that these values may be invalid. Typical mid-summer values fell in the range 0.3 - 0.5. Typical winter values fell in the range 0.03 - 0.06. The paper discusses aspects of interaction between flow depth, vegetation, and *n*-values.

Motayed, Asok K. and Muthusamy Krishnamurthy, 1980

Various natural channels, U.S.

The authors used streamflow and cross-sectional data collected by the U.S. Geological Survey at 36 stream gage locations in Maryland, Pennsylvania, Georgia, and Oregon. *N*-values for the entire cross section of approximately 0.02 - 0.06 were computed. No information was provided on the vegetation present or the depth of water flow.

Acrement, G. J. and V. R. Schneider, 1984

Densely vegetated floodplains in Louisiana, Alabama, and Mississippi

This report includes photographs and *n*-values for heavily vegetated floodplains that were determined as part of an earlier U.S.G.S. study of backwater and discharge at width constrictions of heavily vegetated floodplains but were not published as part of that earlier work. Fifteen photographs are given, along with information on flow depth, vegetation density, and selected values of roughness representing the effect of different roughness components. Roughness values for the floodplain alone of 0.10 - 0.20 are shown.

Hydraulics Research Limited, 1988

Various rivers, U.K.

This report does not actually report original field data, but rather attempts to synthesize the implications of the field information available. It cites the existence of field data for these two-stage channel reaches: 1) various rivers in England and Wales with overbank flow; 2) River Roding, England (see Sellin, below); and 3) River Main, Northern Ireland. This list probably constitutes the only cases to date in which field data has been collected on vegetated compound channels.

Based on the data from (1), the report suggests that the guidelines for selecting the main channel *n*-value given by Chow (1959) result in *n*-values that vary from observed *n*-values by a considerable range, with the ratio of estimated to observed *n*-values falling in the range of 0.67 - 1.40 for the 10 sites evaluated.

Ramsbottom, D. M., 1989

Various rivers, U.K.

A review of existing measured overbank flow data. This report identifies 15 potential experimental sites where overbank flows have been measured, and estimates overbank and/or main channel *n*-values for six of the sites based on one, or in one case, two storm events. The *n*-value calculations assume a vertical dividing line at the top of the channel bank which was not included in the wetted perimeter. A qualitative description of the channel bed material and overbank topography and vegetation is supplied. The results are summarized in Table 1.

TABLE 1

CALIBRATED *N*-VALUES FOR VARIOUS RIVERS, U.K.

GAUGING STATION	FLOODPLAIN CONDITION	BANKFULL DEPTH (m)	RATIO OF FLOODPLAIN TO TOTAL DEPTH	CHANNEL *n*	OVERBANK *n* (left/right)
Tees - Low Moor	grass; some trees, right bank only	4.27	0.25	0.064	-- / 0.190
Blackwater - Ower	pasture; some trees; bushes on bank	1.81	0.10	0.045	0.094 / 0.099
Severn - Montford	grass	4.6	0.24	0.032	0.027 / 0.042
		4.6	0.10	0.029	0.024 / 0.047
Penk - Penkridge	grass, trees lining right bank	1.6	0.11	0.037	0.137 / 0.061
Trent - N. Muskham	grass, some small trees and bushes	2.62	0.17	0.0366	-- / 0.031

The author also compared estimates of bankfull *n*-value using traditional tabular descriptions to calculated bankfull *n* based on discharge and geometry data at 11 sites. Almost all of the estimated *n*-values were low, with typical differences being 0 - 35%.

Sellin, R.H.J. et al., 1990
River Roding, U.K.

Measurement of roughness over a 4-year period for a two-stage channel with the upper level channel (the "berm") excavated below the elevation of the natural floodplain. The berm revegetated naturally since the excavation, four to five years before the measurements were taken. Vegetation was particularly thick during high flow measurements because severe storms tended to occur at the height of the growing season. Measurements were taken with the berm vegetation intact and then again after it was cut and cleared except for a narrow interrupted marginal strip of vegetation along the lower channel. No photographic or other evidence of vegetative conditions was provided.

The authors found that the *n*-values for the channel as a whole varied from 0.025 at a depth of 0.15 m above the berm to 0.115 at a depth of 0.65 m above the berm. These extremes both represent uncut vegetative conditions. There is considerable scatter in the data presented, varying both with vegetative and water depth conditions. Most *n*-values were clustered in the range 0.045 - 0.075 for both cut and uncut conditions, with cut conditions clustered toward the lower end of the range.

Hicks, D. M. and P. D. Mason, 1991
Various rivers and canals, New Zealand

Measurements of channel roughness at 78 river and canal reaches were made over a 3-year period at a variety of discharges. A graph showing bed surface material distribution, a graph of calibrated *n*-value versus discharge, cross sections, plan view, and photographs are provided. Calibrated *n*-values are shown for the range of discharges measured. *N*-values of 0.015 - 0.31 were calibrated, with the highest values tending to occur at very low discharges. The highest *n*-values measured during larger flow events were approximately 0.1 or less. No calibrations of floodplain roughness were made.

Higginson, Noel N.J. and Harold T. Johnston, 1992
Various rivers, Northern Ireland

Channel roughness was measured at 68 river sites with catchments ranging in size from 620 km^2 to more than 1000 km^2. No information on channel conditions, including vegetation, was presented. *N*-values at bankfull discharge were found to range from 0.18 to 0.08, with most in the range of 0.025 - 0.055.

Johnston, Harold T., Y. Lam Lau, and Noel N.J. Higginson, 1992
River Main, Ireland

Measurements were made at a constructed reach of vegetated compound channel. At this location, the river has a catchment area of approximately 250 km^2 and an estimated mean annual flood of 85 m^3/s. The bed material of the main channel is coarse gravel and the toe of the banks are protected with large quarried stone. It is 12 meters across. The terraces, each 7.5 meters wide, are formed of gravels and support heavy unmanaged weed growth. *N*-values at a range of water depths were calculated, and were found to range from 0.025 to 0.046, with the maximum *n*-value of 0.055 at a flow depth of 1.9 meters on the berm.

III. THEORETICAL APPROACHES AND RESEARCH

There are two different factors addressed in this review with regard to roughness in a vegetated compound channel: vegetation and channel shape. The research related to each factor will be described below in turn. Again, the issues raised by a meandering channel form are not addressed here, though this channel form may have a substantial effect on the distribution of the shear forces acting on the flow, and thus on the roughness overall.

A. INFLUENCE OF CHANNEL SHAPE ON ROUGHNESS

The papers on compound channel shape address the shear stresses which develop when flow in a main channel is moving much more rapidly than flow on an adjoining floodplain. The difference in flow velocities results in a transfer of momentum from the main channel to the floodplain, and the longitudinal shear stresses cause a loss of energy which reduces the overall compound channel capacity.

Sellin (1964) reported, based on tests in a hydraulicly smooth laboratory flume, that the shape of a compound channel resulted in a reduction in the conveyance capacity because of the interaction of the main channel and overbank areas. He found that the amount of discharge that could be carried in a channel alone added to what could be carried in the floodplains alone exceeded the discharge that could be carried in the compound channel composed of both main channel and floodplains. The calculated value of n shifted from approximately 0.0088 to a maximum of 0.0100 as the flow reached the top of the main channel and spread into the overbanks, up to a point 0.2 feet deep in the overbank region. Then the calculated n value decreased as the overbank flow depth continued to rise. Estimates of n that did not account for this interaction between the channel and overbank resulted in an overprediction of the capacity of the compound channel.

Yen and Overton (1973) hypothesized that an understanding of the shear stresses developed at the interface of the main channel and floodplain flows would provide a means to determine the effect of the interaction on the roughness. They used velocity contours to determine lines of zero shear stress in turbulent flows, and used experimental data to then fit empirical formulas for subarea flow using lines of zero shear as subarea division lines. Measurements of shear stress distributions in compound channels were carried out, including, for example, work by **Myers and Elsawy (1975)**, and **Myers (1978)**.

Rajaratnam and Ahmadi (1979) proposed to predict the effect of compound flow channel interaction on discharge capacity by calculating the velocity profiles with and without the effect of the interaction of the channel and floodplain. Through experimentation with a smooth rectangular compound channel, they concluded that they were able to make relatively accurate estimates of the discharge capacity based on channel geometry and flow

depth. In later work, **Rajaratnam and Ahmadi (1981)** concluded that the reduction and increase of bed shear stress in the main channel and floodplain respectively could be correlated with the ratio of the total depth in the main channel to the depth on the floodplains.

Work reported in **Knight and Demetriou (1983)** was based on measurement of shear force distributions in a smooth-surfaced flume constructed with an adjustable adjoining floodplain on each side of the main channel. In general, the existence of the adjoining floodplains served to slow the flow in the main channel when depths exceeded the elevation of the floodplains. The apparent shear force was found to vary systematically with the ratio of total width to main channel width and the ratio of floodplain depth to main channel flow depth. They found a peak apparent shear force along a vertical interface at low floodplain depths and high floodplain widths relative to the main channel. The vertical shear force between the main channel and the floodplains was found to always be positive, thus indicating a retarding of flow in the main channel, and the horizontal shear force between the lower main channel and the upper main channel with floodplains became negative at greater flow depths, indicating an acceleration of lower main channel flows. Equations were developed to fit their experimental data that predicted the percentage of flow that was carried in the main channel and the lower main channel for the conditions modeled.

Baird and Ervine (1984) performed 136 tests with an experimental flume, measuring point velocities, boundary shear stress, stage, and discharge for an experimental flume at a range of bed slopes. They found that the presence of overbank flow could increase the D'Arcy-Weisbach friction factor in the main channel by 0 - 30%. Analysis of their results showed a complex relationship between the change in friction factor and relative depths of flow in the main channel and floodplain, velocity differences between the main channel and floodplain, and cross sectional geometry. They also developed a correlation between the non-dimensional apparent shear stress and the relative depths of flow, using both their data and the results of earlier research for smooth asymmetric channels. Lastly, they developed a power correlation between the discharge carried in the main channel section only during overbank flow and the ratio of the channel depth to the bankfull depth.

Myers (1984) used an experimental compound channel to explore the relationship between the D'Arcy Weisbach friction factor and the Reynolds numbers for the compound channel. Dimensional analysis was used to show that the friction factor in the main channel is a function of the Reynolds number in the main channel and the ratio of that value to the Reynolds number in the floodplain. Myers hypothesized that similar variables, compound channel Reynolds number and the ratio of main channel to floodplain Reynolds number, would control the friction factor of a compound channel. Correlations were developed based on his experimentation with a tiltable, smooth compound channel, and correlation coefficients of 0.82 and 0.90 were found for functions of the main channel friction factor at two different Reynolds number ratios. Correlation coefficients of 0.94 and 0.95 were found for functions of the compound channel friction factor at two different Reynolds number ratios.

Looking at the effects of both channel shape and vegetatively roughened floodplains, **Pasche and Rouvé (1985)** undertook an analysis of the D'Arcy-Weisbach friction factor that would be associated with the components of a compound channel, especially the interface plane between a main channel and a vegetatively roughened floodplain. The existence of four different areas is hypothesized: 1) floodplain not influenced by main channel, floodplain influenced by channel, main channel influenced by floodplain, and 4) main channel not influenced by floodplain. They report on the development of an exact equation for the drag coefficient for cylindrical rods (Linder, K., Der Strömungswiderstand von Pflanzenbeständen, Mitteilungen, Leichtweiss-Institut für Wasserbau, TU Braunschweig, Heft 75, 1982), used in their analysis. Empirical analysis was used to develop an equation for the velocity ratio, and several simplifying assumptions were made to allow the analysis of the friction factor of the interface assuming one-dimensional flow with measurable data. They found that the value of the interface friction factor was primarily dependent on the ratios of cylinder diameter to vegetation spacing and a term referred to as "co-operating width," or the width across the cross section in which flows are affected by the interface between main channel and floodplain flows.

The authors used an experimental compound channel section with variable floodplain widths and roughnesses to investigate the theoretical relationships they had developed. They used regression analysis to determine the empirical constant needed to estimate the friction factor of the main channel boundary, and found good agreement between predicted and experimental results. In comparing their results to the results of others' experiments under uniformly hydraulically smooth conditions, they found that the slopes of the main channel bank appear to make a substantial difference in the apparent shear stresses between the main channel and the floodplain, with sloping banks resulting in lower apparent shear stresses and thus lower resistance. Good agreement was also found between the equation for the floodplain friction factor in the region not influenced by main channel flow. They found that the friction factor for the imaginary wall or interface between the floodplain and main channel was not dependent on main channel depth when the ratio of channel to floodplain depth was less than 3.0. They suggest that this implies that the influence of main channel depth and therefore the main channel bank on the interface friction factor can be ignored for overbank flow with high floodplain roughness. By multiple regression analysis, parameters were determined for their equation for this friction factor which produced good agreement between calculated and observed values. They also used data analysis to develop an equation for dimensionless slip velocity based on a dimensionless vegetation parameter. The cooperating width was calculated with another equation based on the balancing of momentum in the portion of the floodplain influenced by main channel flow, and they found that it gave good agreement with observed values.

Tests were carried out with various widths of floodplain vegetation, and it was found that the friction factor of the interface varies very little in response to the change in width of the floodplain vegetation, even though they found much higher velocities in the partially-vegetated floodplain case. Thus, they concluded, the value of the friction factor for the

interface cannot be sufficiently described by the differences in velocity between the floodplain and main channel subareas, and the idea of a cooperating width is supported.

The authors tested their equation for interface friction factor using data from the River Ihme and found good agreement between the predicted value and the value calculated based on an assumption of proportional distribution of resistance between the boundary and the interface.

Dracos and Hardegger (1987) proposed an empirical approach to adjusting the *n*-value for a compound channel to account for the interaction between subareas using experimental results of their own and results that had been published by other researchers. Assuming that the ratio of the apparent roughness of a compound channel to the roughness of the main channel surface is a function of the increase in wetted perimeter when the bank-full stage is just exceeded and the ratio of the hydraulic radius to the depth of flow, they developed correlations to estimate the apparent *n*-value of the compound channel. Their correlations were based entirely on tests conducted on smooth channels for which the main channel and overbank *n*-values were equal. When they tested their results on a case for which the floodplain was given added roughness, they found a change indicated in a constant used in their correlation. They suggested some means of adjusting the correlation constants based on changes in the hydraulic parameters, but had only two tests on which to test their method. They noted that their work and others' was performed almost entirely on channels flowing in the smooth or transition zone, and would therefore be limited in their usefulness in modeling rough flows such as would be experienced in most natural conditions.

Myers (1987) showed that for a smooth compound channel, the ratios of channel to floodplain Reynolds number, velocity, and discharge is independent of slope and depend only on the ratio of the depth of flow over the floodplain to the depth of flow in the channel. Unlike Knight and Demetriou, Myers showed that at very high ratios of floodplain to channel depth, the velocities observed on the floodplains actually exceeded the velocities in the main channel. Of necessity, at very large depths, the velocities in each subarea become equal. The authors used regression analysis to develop equations of the form:

$$\frac{Q_f}{Q_T} = D[y_r]^d$$

where:

Q_f	=	*floodplain discharge*
Q_T	=	*total cross-section discharge*
y_r	=	*ratio of floodplain to channel flow depth*
D,D	=	*coefficient and exponent derived by regression analysis*

Similar equations were developed for the ratios of channel to floodplain velocity, channel to total discharge and channel to floodplain discharge.

Myers compared the amount of discharge observed in the compound channel to the amount of total discharge that the subareas could carry if isolated from each other, and found that for the experimental channel the carrying capacity of the compound channel was 10% greater at large relative flow depths (e.g., ratio of floodplain flow depth to total flow depth of 0.5) than would be predicted by a divided channel method which assumes that the apparent shear stress along the division lines is equal to the boundary shear stress. This is true because main channel flows are relatively accurately predicted under these conditions, but floodplain flows are underpredicted because of the momentum transfer to the floodplain. However, at low flow depths, overestimation of carrying capacity was up to 10%. The overestimation occurred at relative flow depths for the floodplain to channel of up to 0.4. As Myers points out, this overestimation percentage is probably low for natural channels, since it would increase with floodplain width and roughness. The discharge capacity of the floodplains, on the other hand, was underestimated as a result of non-interaction by up to 26%, peaking at a relative floodplain to channel depth of 0.4 and remaining consistently at 10% or greater for the range of relative depths studied (approximately 0.18 to 0.5).

Recognizing the need for experimental data that includes flows in the rough turbulent zones, the Hydraulics Institute, Ltd. has undertaken a research program with the sponsorship of the U.K.'s Science and Engineering Research Council. **Myers and Brennan (1990)** report on some of the work conducted during the first phase of the research. Their initial report concerns experimentation conducted on a smooth compound channel at fixed slope with variable floodplain widths operating in the smooth turbulent zone.

Myers and Brennan provide a graph of relative depth (floodplain to main channel) versus the ratio of discharge observed in the main channel of a compound channel to the discharge that occurs in the main channel when separated by a wall from the floodplain. Unlike Myers' similar graph in his 1987 paper described above, this graph shows main channel interacting discharges at up to 10% less than non-interacting discharges even at a relative depth of 0.5. Generally, the results obtained from the channel configurations used in this study show greater levels of main channel capacity reduction when acting as part of a compound channel than the results of the experiments conducted earlier at the same relative depths did. The differences in the channel configuration – vertical versus sloping boundaries – used could easily explain these differences, since Myer and Brennan present a graph that illustrates the difference channel configuration makes on these results.

The authors provide a graph of stage versus *n*-value (noting that Manning's equation is only strictly applicable in the rough turbulent zone, but acknowledging the prevalence of its use). This graph shows a substantial decrease in the total *n*-value for the compound section at flow depths just greater than bankfull. Compound channel *n*-values then rise with depth above bankfull, reaching the expected *n*-value appropriate for a trapezoidal channel case at a relative depth of approximately 0.5. This suggests that for moderate relative depths, the capacity of the total cross section of a compound channel may be substantially underestimated by the assumption that a compound channel's *n*-value remains constant when depths greater than bankfull are reached. Flows in the main channel may be overestimated

using the *n*-value appropriate to the trapezoidal channel, but floodplain flows would be underestimated. The momentum transfer mechanism is responsible for the lower velocities in the main channel and higher velocities in the floodplains (or higher and lower *n*-values, respectively) than would be expected if each of these subareas were non-interacting.

N-values for the main channel and floodplain subareas were calculated based on full velocity traverses, and main channel *n*-values at flow depths above bankfull for the compound channel were shown to be much greater than *n*-values for a trapezoidal shape configuration. Similarly, floodplain *n*-values were generally lower than the *n*-value determined for the trapezoidal shape. At low depths across the floodplain, discharge calculations using separate subareas and the same *n*-value determined for the trapezoidal case lead to overestimation of compound channel carrying capacity since the main channel conveys most of the flow at these depths.

Plots of *n*-value and friction factor versus Reynolds number for each configuration are provided, and the compound channels show increasing *n*-values for increasing Reynolds number. The trapezoidal channel, on the other hand, shows a slightly negative relationship between these variables.

Higginson and Johnston (1992) measured velocities across a constructed, vegetated compound channel section of the River Main in northern Ireland. The terraces are covered with heavy, unmanaged weed growth. Higher velocities were found in the center of the mildly-sloped flood terrace than at either the outside terrace edge or the edge next to the main channel. *N*-values were calculated for the channel as a whole, and they were found to initially drop with increased water depth until flow reached the top of the terrace, and then to rise as the depth of flow on the terrace increased. A maximum measured value of 0.055 was reached at the greatest depth of 1.9 meters on the terrace. Using a vertical division method, they calculated a maximum *n*-value for the terrace alone of 0.16.

Laboratory tests on a physical model of the channel showed similar variations of *n* with depth, except for a fully smooth compound channel. In this case, *n*-values decreased when the water level rose above a minor depth on the terrace. A slight decrease in *n*-value was also observed at large water depths on the fully rough model, but only when a substantial water depth was reached. They were able to calibrate a mathematical turbulence model to fit laboratory measurements of stage versus discharge for both a rough and smooth channel condition. Predicted discharges were slightly low at high depths and slightly high at low depths.

B. INFLUENCE OF VEGETATION ON CHANNEL ROUGHNESS

Studies of grassed channels were reported in **USDA (1954)** for channels with slopes > 1%. This work developed relationships between Manning's *n* and the product of flow velocity and hydraulic radius for different grass varieties and stand characteristics.

Klaassen and Zwaard (1973) examined the roughness contributions of hedges and fruit trees using a rectangular flume at the Delft Hydraulics Laboratory. Tests were first run placing actual hedges, with and without additional organic debris, in five rows ten meters apart. They developed an equation to relate the Chézy C of the configuration to the spacing of the hedgerows, the percentage openings in the hedges, and the surface roughness of the flume. Estimated C values for dense rows (10 meters apart) were found to be up to 6 $m^{1/2}sec^{-1}$ with debris and 13 $m^{1/2}sec^{-1}$ without, or *n*-values of up to 0.16 for hedges with debris and 0.076 without.

Similar experiments were conducted with fruit trees, except using models and a 1:10 scale. While less conclusive because of the difficulty in developing model "trees" that fully reflected the height-area-stiffness characteristics of real fruit trees, they found a maximum roughness in the form of a Chézy C value of approximately 13.7 $m^{1/2}sec^{-1}$ at a density of 0.20 trees per m^2 on a grassy floodplain, equivalent to approximately an *n*-value of 0.067. This occurred at a depth for which their modeled trees had a cross sectional area that was much greater than that of the actual fruit trees they were attempting to model. The authors found no clear distinction between the calculated drag coefficient for a floodplain in which trees were planted parallel to flow and one in which they were staggered to provide maximum blockage, though their results for these simulations on inspection appear to show a tendency for the drag coefficient of staggered-planted orchards to be somewhat higher. Higher drag coefficients indicate greater roughness. Calculated drag coefficients for the trees were found to often be higher than that for a cylinder, ranging from 0.8 to 3.0, with a mean of 1.5. Cylinders fall in the range of approximately 1.0 - 1.1.

Petryk and Bosmajian (1975) re-analyzed data collected earlier by the U.S. Department of Agriculture and reported by Ramser (1929) and Ree (1958). They developed a set of equations based on the fact that forces in the *x* direction must be balanced. By using Manning's equation to express the average boundary shear and assuming that the average cross-sectional velocity was equal to the approach velocity on the vegetation, they were able to express *n*-value in terms of the vegetation density on the floodplain, including the drag coefficient:

$$n = n_b \sqrt{1 + \frac{C_d \sum A_i}{2gAL} \left(\frac{1.49}{n_b}\right)^2 R^{\frac{4}{3}}}$$

where:

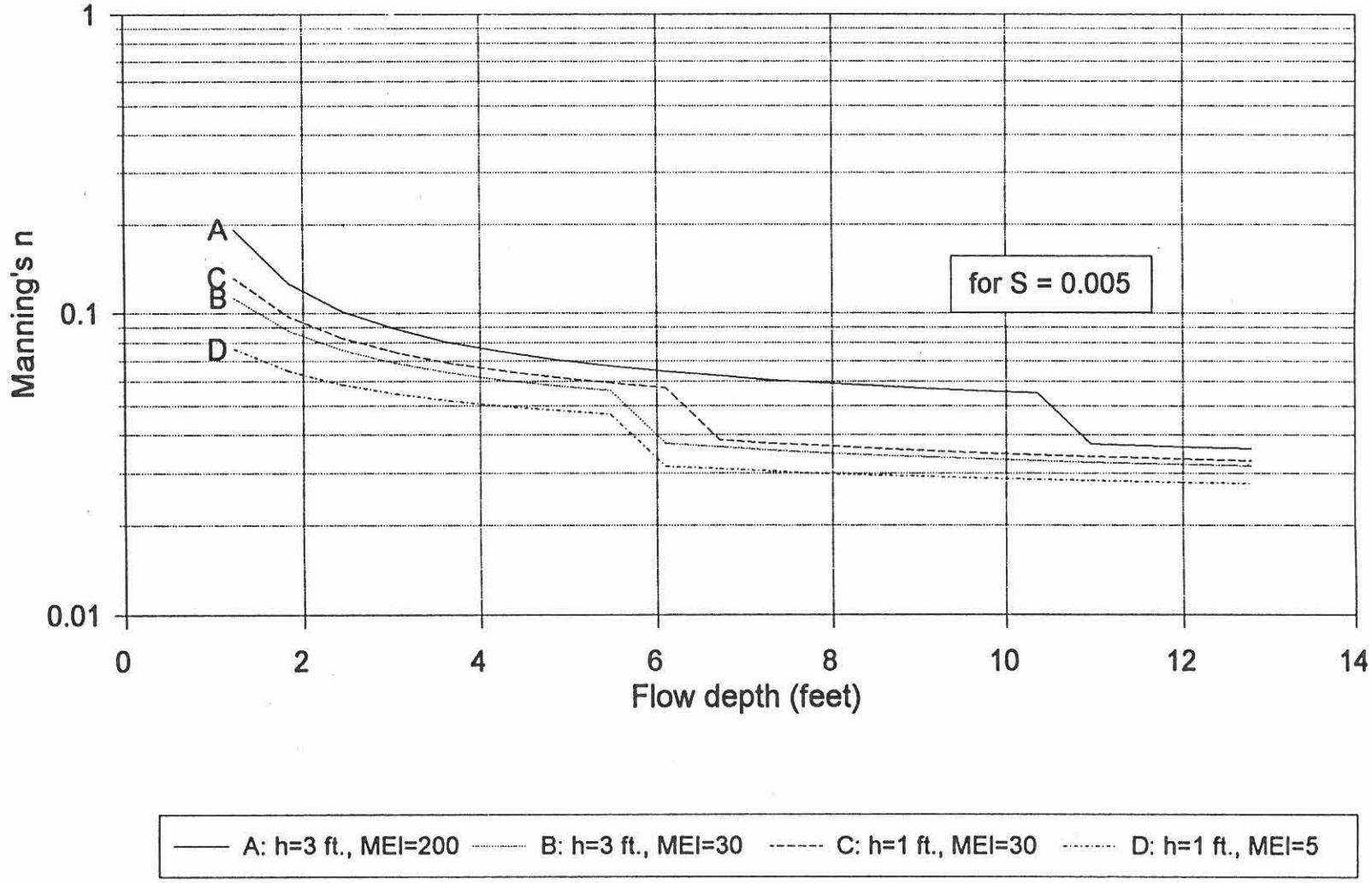
n_b	=	*Manning's boundary roughness exclusive of vegetation*
C_d	=	*drag coefficient of the vegetation*
A_I	=	*projected area of the i-th plant in the streamwise direction*
L	=	*length of channel reach being considered*
A	=	*cross-sectional area of flow*
R	=	*hydraulic radius*
g	=	*gravitational acceleration*

and

$$\frac{C_d \sum A_i}{(AL)} = \textit{Density of vegetation in the channel}$$

This equation assumes that the mean velocity is independent of the depth of flow. Making assumptions about the boundary roughness exclusive of vegetation, and assuming a drag coefficient value of 1.0 and a hydraulic radius value equal to the depth of flow, the authors plotted estimated relationships for *n*-value versus depth for the most heavily vegetated of Ramser and Ree's cases. They found very good agreement between their calculated values and those measured by Ramser and Ree. The authors suggest either developing information on the relationship of vegetation area over a representative length of floodplain or calibrating a vegetation density value with a representative prior flow event.

Research reported in **Kouwen, *et al.*, 1981** suggests that accurate estimates for *n* can be estimated by considering the relative "stiffness" of the vegetation, slope, flow depth, and vegetation height. Stiffness values have been developed for stands of numerous grasses based on published flow data and similarities to experimental data using plastic strips. Implicit in the relationships presented by these authors is a variation in *n*-value with relative flow depth which depends on the effective roughness height of the vegetation. The theoretical relationship between *n* and relative depth changes abruptly when flow depth is sufficient to make the vegetation prone. This implies that observation of the flow depth at which a stand of vegetation tends to become prone could supply useful data from which to estimate vegetative stiffness. Figure 1 shows a sample graph using these theoretical relationships.



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Note: MEI = stiffness index; h = height of vegetation

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Flow Depth vs. Roughness for Selected Vegetation Classes
using the method proposed by Kouwen, Li, & Simons, 1981

Figure 1

C. TECHNIQUES FOR ESTIMATING FLOODPLAIN ROUGHNESS FROM FIELD DATA

Several techniques have been developed to estimate the roughness of floodplains from field data under the assumption that this information can be used to predict subarea or total discharge capacity. **Acrement and Schneider (1984)** built on the earlier work of Aldridge and Garrett (1973) and others, including Petryk and Bosmajian (1975). In one approach, the authors present a table of factors for adjusting the base roughness assigned to the floodplain based on irregularity, variation in cross-section, obstructions, vegetation, and meander, similar to the technique sometimes applied to channels.

In a second approach, the authors suggest use of the vegetation density approach of Petryk and Bosmajian that relies on an effective drag coefficient. They supply a graph that relates effective drag coefficient to hydraulic radius for a wide, wooded floodplain that they say is based on verified *n*-values, though no source is cited. A linear estimate of the relationship is shown as approximately:

$$C* \approx 21.58 - 3.64R \quad \textit{for } 2 \leq R \leq 5$$

where:

$C*$	=	*effective drag coefficient*
R	=	*hydraulic radius in feet*

It is interesting to note that the drag coefficients suggested by this data are much higher than the value of approximately 1.0 that Petryk and Bosmajian assumed in their analysis.

Photographic identification of floodplains with verified *n*-values is also recommended by the authors, and photographs with calibrated roughness values are supplied in the report.

D. TECHNIQUES FOR ESTIMATING ROUGHNESS THROUGH ALLOCATION OF DISCHARGE INTO SUBAREAS

Although the available experimental data suggests that calculation of discharge through application of a technique relying on division into subareas is a crude tool at best, it is a widely recognized and easily applied approach that requires no new data collection. It also offers a technique for indirectly calculating values of channel and overbank roughness if stage, discharge, and geometry data are available. Lastly, it is the approach for estimating discharge which is most likely to be employed using estimates for channel and overbank roughnesses that are developed by any of the available methods. Therefore, assessments of this method's accuracy are presented here so that its applicability and limitations may be better understood. For those interested in other approaches, some based on the approaches

described in Section III.A., a good summary of the techniques that have been developed may be found in **Samuels, 1989**.

James and Brown (1977) (summary quoted from Williams and Julien, 1992) found that "[a]s the ratio of the floodplain to the channel widths (aspect ratio) increased, the apparent roughness decreased, leading to the conclusion that total discharge is over or underestimated, depending on the calculation method used.... This roughness decrease was most evident for low ratios of total flow depth to channel depth (total depth less than floodplain depth), Y/D, but was negligible for Y/D greater than 1.4. In the range 1 < Y/D < 1.4 , traditional single or separate channels methods should not be used, but for Y/D greater than 1.4, no corrections to the methods are required. They found that the longitudinal slope does not have much influence on the interaction."

Motayed and Krishnamurthy (1980) compared 5 different approaches for calculating the composite roughness of a channel using USGS cross section data from 36 different streams in Maryland, Georgia, Pennsylvania, and Oregon. They compared the results from 4 of the equations to the results of an analysis that developed a calculated *n*-value using detailed velocity data from each cross section, an assumption of logarithmic velocity distributions in each subarea, and application of the Manning-Strickler formula. The results of this equation were assumed to be the "true" *n*-value to which the results of the other methods were compared.

The equations they evaluated included estimates of composite *n*-value assuming that: 1) mean velocity is uniform across the cross section, relating subarea wetted perimeters and *n*-values to the total wetter perimeter and *n*-value (Horton's formula); 2) total resistance equal to the sum of resistances in each subarea and a hydraulic radius for each subarea equal to the hydraulic radius for the entire cross section, again relating subarea *n*-values and wetted perimeter values for each subarea to a composite *n*-value (Einstein and Banks' formula); 3) total discharge is equal to the sum of the subarea discharges, relating total and subarea wetted perimeter and hydraulic radius and subarea *n*-values to a composite *n*-value (Lotter's formula); and 4) a logarithmic velocity distribution, relating subarea and total wetted perimeter, depth, and subarea *n*-value to a composite *n*-value.

Equation 3 was shown to have the least scatter in *n*-values compared to the assumed "true" *n*-value:

$$n_c = \frac{PR^{5/3}}{\sum_{i=1}^{N} \frac{P_i R_i^{5/3}}{n_i}}$$

The authors acknowledge that the assumption of logarithmic velocity distributions in high flow stages may not be valid.

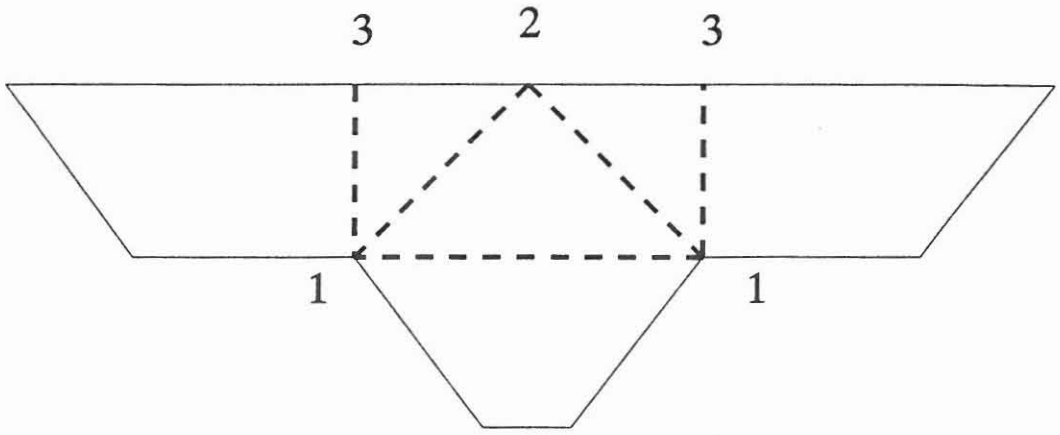
Wormleaton, Allen, and Hadjipanos (1982) used a symmetrical compound section to examine the relative discharge capacity of the main channel and floodplains. Their experimental channel had floodplains with 4 different *n*-values varying from 0.11 - 0.21 on the floodplains and an *n*-value of 0.10 in the main channel. The size of their floodplains was fixed. They developed expressions for the horizontal and vertical apparent shear stresses at the boundaries between the main channel and floodplain flow. They suggest that if the apparent shear stresses could be calculated, they could be used to suggest the most appropriate technique for dividing the channel into subareas for discharge estimation. Where these apparent shear stresses are small, they might be ignored, or if they are similar to boundary shear stresses, it may be appropriate to use them to locate a subarea boundary. Alternatively, some other means of accounting for their presence could be identified.

Using data from part of their own work and three other studies of both symmetrical and asymmetrical floodplains, they used regression analysis to develop an equation for apparent shear stress and compared it to a limited number of their own results for validation. The equation appeared to represent observed values quite well, though it was based on only 55 sets of results. Their analysis showed a strong correlation between apparent shear stress acting on vertical interface planes and the differences in velocity between subsections, the ratio of depths of flow in the floodplain and main channel, and the ratio of the width of flows on the floodplain and in the main channel.

They found that apparent shear stresses across horizontal and diagonal planes were much smaller than those across vertical interfaces. For deeper flow depths in the case of horizontal, and to some extent diagonal, interfaces, the momentum transfer across these interfaces was found to be from the floodplain to the main channel. At lower depths, the transfer was from the main channel to the floodplain.

The authors examined two different forms of three different methods of dividing the compound channel into subareas. They analyzed vertical division methods, in which vertical boundaries are envisioned extending from the edge of the main channel bank to define the main channel and floodplain edges. They also examined the case of horizontal subdivision, in which horizontal boundaries are assumed to extend across the main channel, at the elevation of the floodplain. Lastly, they looked at diagonal division planes, extending from the edge of the main channel bank to the center of the main channel at the water surface. Two cases were examined for each method, one in which the subarea boundaries were included in the wetted perimeter of the main channel only, and one in which they were excluded (V_i, V_e, H_i, H_e, D_i, and D_e, defined as shown in Figure 2). Each method of analysis was applied to each of the four floodplain *n*-value conditions at several different depths of flow.

They found that the ratio of calculated discharges to observed discharges tended to increase with increases in roughness for all calculation methods. Method V_e was found to consistently overestimate discharges, even at higher depths. V_i and D_e yielded similar results, though the latter was more accurate. For low flow depths and higher roughness, D_e



H: 1-1
D: 1-2
V: 1-3

H_i, D_i, V_i: the division line is included in the wetted perimeter of the main channel

H_e, D_e, V_e: the division line is excluded from the wetted perimeter

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Definition of Subarea Division Methods
(after Wormleaton and Merrett, 1990)

Figure 2

still substantially overestimates discharge. The ratio of calculated to observed discharge generated by H_e tended to be lower, and method H_e was found to underestimate discharge at higher flow depths. H_i and D_i yielded similar results, though flows tended to be underestimated except for lower flow depths on channels with rougher floodplains. All of the methods underestimated the total discharge in some cases, but never more than about 20%. Overestimates of discharge of up to 80%, however, also occurred, particularly with rougher floodplains. To develop a conservatively low estimate of the discharge capacity, the authors concluded, use of method H_i or D_i should be considered. In general, the H and D methods yield better results than the V methods overall.

Comparison of the ratio of apparent shear stress to theoretical average shear stress around the boundary of the main channel subdivision (including vertical or other interfaces) were carried out for each case. The results suggest that calculation of this value is in fact a good indicator of whether the subarea interface should be included in the wetted perimeter for the most accurate estimate of channel discharge.

In **Prinos and Townsend (1984)**, the authors reviewed a variety of calculation methods and proposed a new approach. Four of the seven methods presented relied on division into subareas. Some of the methods had been proposed by earlier researchers, and experimental results from earlier work and Prinos and Townsend's own flume studies were reviewed. One method reviewed involved calculation of a composite *n*-value using each of the common methods of calculating a composite *n*. They showed that these methods resulted in consistent underestimation of the discharge by up to 37% for their experimental data due to faulty assumptions in the composite *n*-value equations. An approach using a vertical plane to subdivide the main channel and floodplain, with and without inclusion in the main channel wetted perimeter (V_i and V_e, respectively) was also tested. The former overestimates discharge for low floodplain depths, presumably because the method assumes equal apparent shear stress and boundary shear stress in the main channel. The latter overestimates total discharge because it ignores the effect of the apparent shear force on the imaginary boundary interface plane. The method of using a horizontal interface with no inclusion of the interface in the wetted perimeter (method H_e) was also tested and found to give somewhat better results, though discharge was usually still overestimated at low floodplain depths.

The authors also tested two other methods which did not rely on subdivision of areas, work reported in Le Van Kiyen (1968) and Karasev (1969), methods proposed in work not reviewed in this document. These methods were based on correction factors. Le Van Kiyen relied on empirical relationships that were a function of the following: degree of asymmetry, shape of cross section, relative depth of flow, and relative roughness. Karasev relied on the momentum equation to obtain expressions used to develop the correction factor. The author notes that both of these methods provided superior results to any of the others tested. The authors' proposed method recommends using the apparent shear stress to calculate channel and floodplain flow velocities and therefore total compound channel discharge. They found good agreement between their method and experimental results,

although they note that their method assumes that the friction factor is constant under both isolated and interacting conditions, an assumption which is not generally true. The best method overall, based on correlation of results with measured discharges, was found to be the method relying on the momentum equation. Method H_e was found to give the second best results for the experimental data examined. The authors' method, which also relied on the momentum equation, also gave good results for the range of relative depths in which the assumption of equal friction factors under both interacting and isolated conditions was true.

Wormleaton and Hadjipanos (1985) carried out further research using the same experimental channel and *n*-values as in their 1982 work. Their goal in this effort was to determine the method that most accurately estimated the discharge in each of the channel subareas, rather than simply the total channel discharge. They used four different methods of dividing the channel cross section into subareas, and measured the percentage error in discharge estimates that resulted. They analyzed V_i, V_e, H_i, and H_e, as defined above. Each method of analysis was applied to each of the four floodplain *n*-value conditions at three different depths of flow.

The authors found that even the method that most successfully estimated total discharge in these 12 situations had as much as a 22% error, and that none of them satisfactorily calculated flow between the main channel and the floodplains. The vertical methods tended to overestimate main channel discharge and underestimate floodplain discharge. Method H_e tended to give reasonable main channel discharge estimates when the floodplains were smooth, but overestimated discharge for rougher floodplains. Method H_i gave better main channel estimates for shallow flow across rough floodplains, but was less accurate when floodplains were smoother or flow was deeper. At higher floodplain depths, the vertical methods give better total discharge estimates, but the horizontal methods do a better job of allocating discharge between subareas.

The Hydraulic Research Institute (1988) conducted research on the accuracy of discharge calculations using data from Water Authorities in England and Wales for rivers with overbank flow. Their work suggested that for straight main channels with parallel berms, method D_e gave the best results, though V_i was also fairly accurate. Improvement was achieved when floodplain flow was ignored altogether for low floodplain to main channel depth ratios, $y_r \leq 0.1$, where main channel depth is measured only to the bankfull or floodplain elevation. They also recommend the use of H_i, with the division line added to both subareas, for cases of meandering main channel provided $y_r \geq 0.3$.

Ramsbottom, D. M. (1989) reviewed data that had been collected at 15 different channel rivers in the United Kingdom for which overbank flow data was being collected. The investigator used the division into subareas method to estimate discharge at 9 sites, basing the *n*-values for the main channel on the calculated bankfull discharge *n* and the overbank *n* on estimates based on traditional estimation techniques. The V_i, V_e, D_e, H_e, and single channel methods were used to estimate discharge based on these *n*-value assumptions. (It is unclear what *n*-value was used for the single channel method.) The author concluded that

most methods produced predictions in the ±10% range, except H_e and the single channel method, which underpredicted the discharge. Methods V_i and D_e were identified as providing reasonable results if reasonable roughness estimates could be made. The results indicated that the predictions using method D_e were somewhat more conservative than those produced using method V_i.

More recently, **Wormleaton and Merrett (1990)** have used another experimental compound channel to explore this issue further. The channel used in this study was a symmetrical channel with sloping main channel and floodplain sides, except for the case of the widest floodplain, which had vertical sides. One of the 5 configurations uses a roughened floodplain with a floodplain to main channel bottom width ratio of three. The other cases use a smooth channel with floodplain to main channel bottom width ratios of 5.47, 3, 1, and 0 (the latter representing a trapezoidal channel without floodplains).

They found that the discharge for the roughened floodplain case was even less than the trapezoidal channel case at up to a floodplain to main channel depth ratio of 0.5, clearly indicating the retarding effect of the roughened floodplain on the main channel flow. They found a Manning's *n*-value for the main channel of 0.01, based on the trapezoidal channel. To estimate the *n*-value that would be represented by the roughened floodplain case, they used an assumption of two dimensional flow and measured flow characteristics outside the region assumed affected by the main channel - floodplain interface or side shear. This effort yielded an estimate of *n*-values for the floodplain that ranged from 0.0156 to 0.451 for relative floodplain to total flow depths of 0.04 to 0.5.

Three methods were examined for estimating discharge using the subarea technique: V_i, D_e, and H_e. For the total channel discharge, the authors found that method V_i gave the highest value over most of the depth range and method H_e the lowest. Except for the widest floodplain case using method H_e, the error in estimated total discharge decreases for all of the approaches with increased floodplain depths. Method D_e was found to perform best overall, especially for narrower floodplains.

Errors in estimates for main channel discharge were much higher than those for the total discharge. These errors did not decrease to the same extent at higher floodplain depths as the total discharge estimates did. The same order of discharge estimates existed for main channel flow estimates, with V_i yielding the highest values and H_e the lowest. V_i and D_e appear to overestimate main channel discharge under most conditions, V_i to a greater extent. Method H_e yielded underestimates for main channel discharge in most cases, especially at higher floodplain depths.

The authors used an index of the degree of interaction of the main channel and floodplains, the ratio of boundary shear force to streamwise weight component of the fluid, to evaluate two of the methods. V_i was chosen for ease and prevalence of use, and D_e for its overall accuracy. Using an equation to modify the total and subarea discharge estimates of each method by the force ratio, the authors were able to greatly improve the accuracy of the

discharge estimates by both methods, especially for the cases with wider floodplains. Both methods performed least well for the case with roughened floodplains, though the modified discharge estimate was still much more accurate than the unmodified discharge estimate.

In practical use, the authors argue, a method must be developed to estimate the force ratio from readily obtained channel information. Since floodplain force ratios can be derived from main channel force ratios, it is sufficient to derive a value for the main channel force ratio. In addition, a value for the vertical interface apparent shear stress can be related to the main channel force ratio. The authors therefore used regression analysis to develop an equation relating the force ratio for the main channel to the difference in average velocities between the subareas, the depth of flow on the floodplain, and the floodplain width using their experimental results. There was excellent agreement in the experimental results and the equation thus developed, though it is strictly only applicable for the range of experimental conditions on which it is based.

IV. RECOMMENDATIONS FOR CALCULATION TECHNIQUES

Based on the work reviewed in Sections I - III, recommendations have been developed for the calculation of carrying capacity of a rough compound channel. Because channel design is often required to meet certain flood control design criteria, it is recommended that a conservative estimate be used initially in the design process and a refinement of the stage-discharge relationship be carried out as part of the monitoring process. Modification of the channel management plan can then be used to achieve the desired stage-discharge relationship.

The following preliminary recommendations apply to calculation of total compound channel discharge for a straight channel for which calibrated *n*-values are unavailable, with y_r = floodplain flow depth/main channel bankfull flow depth:

1) For $y_r \leq 0.2$, neglect flow in floodplain.

2) For $0.3 \geq y_r > 0.2$, use Method H_i.

3) For $y_r > 0.3$, use Method D_e.

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To estimate *n*-values for a channel, use the stiffness method of Kouwen, Li, and Simons, or, if inapplicable or stiffness data is unavailable, any of the generally accepted references, such as USDA (1954) (for slopes > 1%), Chow (1959), Barnes (1967), Hicks and Mason (1991), or McCuen (1989). A table in the latter reference includes estimates for vegetated channels based on data from many sources, including agricultural studies such as are referenced in Section II. Cowan's method, described in Chow (1959), is particularly suited for accounting for the specific effects of vegetation in small to moderate unlined channels. This method uses an incremental approach to calculating *n*, and estimates the effect of vegetation on *n*-value to be in the range 0.005-0.100.

Estimates for floodplain *n*-values should be made based on a verified method, such as one of the following methods, in order of preference:

1) For a wide, wooded floodplain (5 > R > 2), use the vegetation density approach described in Acrement and Schneider (1984).

2) For comparable floodplains, use the photographic-based technique described in Acrement and Schneider (1984).

3) Use a table from a standard reference.

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