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Effects of Riparian Tree Management on Flood Conveyance Study of Manning's Roughness in Vegetated Floodplains with an Application on the Embarras River in Illinois

by

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Watershed Science Section
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Abstract

Riparian forests have been proposed by the Technical Advisory Subcommittee of the Upper Embarras River Basin Commission in its alternatives for mitigating flood damages in the Village of Villa Grove and nearby farmlands. In order to evaluate potential reduction in flood stages in Villa Grove, methods for accounting for flow resistances induced by the riparian forests are needed in the hydraulic model for the Upper Embarras River. This project has been designed to better apply the available knowledge in practical field applications, particularly, how to evaluate the vegetal roughness in terms of Manning's *n* coefficient for specified planting scenarios. Approaches presented in this report are literature review on Manning's roughness with emphasis on vegetative roughness, and evaluation and selection of methods for computing vegetative roughness due to riparian forests.

The Petryk and Bosmajian (1975) method was selected for evaluating Manning's *n* for mature trees because parameters could be reasonably obtained with available general field information. Using this approach, effects of riparian forest on floods were evaluated with the scenarios that the two-year floodplain has two densities of trees. The study reach was the channel between Villa Grove and Camargo. Also investigated were the options of having uniform tree density for the whole reach or half of the reach. An interface has been developed for implementing the computed *n* values to a HEC-RAS hydraulic model, and capacity curves were developed to illustrate the effects on flood conveyance among these scenarios. The capacity curves thoroughly included possible boundary conditions and were presented in simple nomographs that relate discharge and downstream elevations to a specified flood elevation in Villa Grove. Therefore it was easier to evaluate the resulting effects of different alternatives.

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Introduction

Repeated flooding in the Village of Villa Grove and nearby farmlands has caused property damages and contributed to the deterioration of quality of life for local residents. The Upper Embarras River Basin Commission, formed in 1988, identified flooding as one of the critical resource concerns in the basin. Its Technical Advisory subcommittee subsequently suggested nine alternatives for mitigating the flooding and drainage problems in this area (Upper Embarras River Basin Planning Commission, 1996). Among the suggestions, Alternatives 7 and 8 appear to be the most effective ones in terms of lowering the flood stages in Villa Grove. Suggested practices in Alternatives 7 and 8 include the combination of channel maintenance downstream from Villa Grove; diverting the Jordan Slough (a tributary to the Upper Embarras River); and tree-planting in the 2-year floodplain above Villa Grove. Trees on the floodplain serve as forest buffer strips. Studies have shown that buffer strips enhance water quality and riverine habitats. The state's Conservation Reserve Enhancement Program (CREP) is also promoting the conversion of crop fields that are inundated by floods with a 2-year return period into buffer strips (Upper Embarras River Basin Planning Commission, 1996).

The Technical Committee devised a hydrologic and a hydraulic model to evaluate the effectiveness of these alternatives. In the modeling approach, the hydrologic model TR-20 with design storms generated flood discharges of 1-, 2-, 5-, 10-, 25-, 50-, and 100-year return periods at selected locations along the main stem and tributaries. The hydraulic model WSP-2 then was applied at channel reaches with these discharges to compute the flood stages. In evaluating their hydraulic effects on flood stages, these alternatives differ primarily in the paths in the channel network, the natural or designed cross sections, and the value of a roughness coefficient. The roughness coefficient is a numerical interpretation of the state of resistance to the flow in the channel, and its magnitudes play an important role in the computed flood stages. Lining materials on the channel boundary, including tree-planting on the floodplain, are the primary causes of the flow resistance. Since riparian forests have benefits for the environment but have the potential to increase flow resistance, hence increase flood stages by reducing the carrying capacity of the channel, proper management practices of these riparian forests are part of the watershed planning. Methods for evaluating the roughness value resulting from designed planning configurations such as spacing, density, or location of trees are necessary. However, information for relating tree maintenance activities to the roughness values is limited.

Study Objectives

The objectives of this study were to:

- Develop or recommend a method for computing the local roughness generated by tree-planting activities on the floodplain.
- Demonstrate the application of the method and evaluate the results using a one-dimensional hydraulic model.

Procedures and Methodology

Mr. Karl K. Visser of the Natural Resources Conservation Services (NRCS), who developed the original TR-20 and WSP-2 models for the Upper Embarras River basin, converted the WSP-2 hydraulic model for the Upper Embarras River to the HEC-RAS model format (HEC, 1997) and the HEC-RAS model was used in this project. The HEC-RAS model has enhanced graphic capabilities and tabulated output that are beneficial to this project's operations. Both WSP-2 and HEC-RAS programs solve the one-dimensional energy equation and use the step method to compute water surface profiles, and both programs use the Manning roughness coefficient to estimate friction losses. The Hydraulic Engineering Center has plans to upgrade the HEC-RAS system to include unsteady flow routing and sediment routing programs that may improve further analysis.

Procedures for this project were as follows:

- Characterize elements contributing to Manning's roughness coefficient in the study reaches.
- Evaluate/develop a method for calculating representative roughness from contributing elements that includes parameters considered in management practices such as diameter of stem (age of trees), spacing (density), and location on the floodplains.
- Evaluate methods for computing composite roughness and their applications in the one-dimensional hydraulic model.
- Collaborate with NRCS on hydraulic model tests.
- Test scenarios on tree-planting specifications.
- Prepare reports and disseminate information.

Acknowledgments

The Conservation 2000 (C-2000) Program of the Illinois Department of Natural Resources provided part of the financial support. Mr. Paul Vehlow is the C-2000 ecosystem projects manager, and Ms. Paula Martel is the regional coordinator. The majority of work was conducted while the authors were affiliated with the Illinois State Water Survey (ISWS).

Many ISWS staff and students from the University of Illinois at Urbana-Champaign contributed to this project. The authors acknowledge the assistance provided by the administrative staff of ISWS, including Misganaw Demissie, head of the Watershed Science

Section, so that the senior author could complete the report after he transferred to the U.S. Geological Survey. Miguel Restrepo analyzed data and developed the BASIC program. Yanqing Lian analyzed data and initially developed the Hydraulic Performance Graph (HPG). Dawn Harrison, Erin Bauer, Michael Meyer, Amy Russell, and Soyoko Umeno helped with data collection, processing, and analysis. Yi Han conducted sediment size and concentration analyses.

The collaborations of Karl Visser and Leon Wendte, Natural Resources Conservation Services, are sincerely appreciated. Karl Visser worked closely with the authors and provided valuable assistance and information. Leon Wendte provided data collected by other projects and was supportive.

Special thanks must be expressed to the late Professor Ben Chie Yen of the Department of Civil Engineering, University of Illinois at Urbana-Champaign, who provided helpful and insightful guidance about flow resistance and held regular research meetings with the authors. Becky Howard formatted the camera-ready copy of the report and Eva Kingston edited the report.

Background

The Upper Embarras River System

The Upper Embarras River Basin in central Illinois (Figure 1) is known for its flat topography. Swamplands used to be the basin's primary land feature, but now the lands have been reclaimed with extensive dredged and dug ditches and subsurface tiles to accelerate drainage. The whole basin is intensely used for agriculture: farming, stock raising, and related industries. Figure 1 also shows the stream network and major townships in the basin.

The natural channels and dug ditches are about 65 miles long (Upper Embarras River Basin Planning Commission, 1996). The main stem starts in the City of Champaign, approximately 15 miles upstream from Villa Grove, flows south, and is joined by two major tributaries, the East Branch and Jordan Slough, before flowing through the Village of Villa Grove. Downstream from Villa Grove the stream meanders within the floodplain and passes by Camargo, the downstream limit of the study basin, the location of a long-term discharge station and the only discharge station for the upper basin. Black Slough is a major tributary to the East Branch, and Long Point Slough is a major tributary to Jordan Slough. A small tributary, West Ditch, enters the Embarras River from the west, upstream of the Chicago and Eastern Illinois Railroad Bridge in Villa Grove.

Channel cross sections and bridges/culverts were surveyed between 1989 and 1990 (USDA, 1992b). These data were used in the WSP-2 hydraulic model, which encompasses a total of 25.1 miles on the Embarras and 39.5 miles on all other tributaries. There are 308 cross sections and approximately 100 bridges and culverts (Figure 2). The West Ditch was not modeled because of its size. Table 1 shows the drainage area, channel length, and slope summarized from the hydrologic and hydraulic models.

The last column in Table 1 shows the average channel slope in the main stems and tributaries. The mild channel slopes in the main stem signify the role of backwater effects. Downstream obstructions, such as debris, contracting sections, confluences, bridges, or culverts slow down flood propagation and add additional stages to the upstream sections. Also shown in the table are that basin areas of the major tributaries are comparable to that on the main stem above the confluence. With relatively steeper slopes, floods from tributaries could reach the main stem before the floods from the main stem upstream. The slope in the main stem also varies from steep to mild as presented in Table 2, which shows the bed slope in six reaches. The cross-sectional identification numbers in column 2 are the same as those used in WSP-2 or HEC-RAS models. With the flat slope downstream, induced backwater effects can add flood stages in Villa Grove. Improving the channel conveyance by reducing roughness values in downstream reaches will help reduce flood stages.

Characteristics of Past Flooding

Climate-related causes for flooding in Villa Grove were examined by reviewing past floods for the timing, discharge, and flood stages. Long-term continuous streamflow data for the

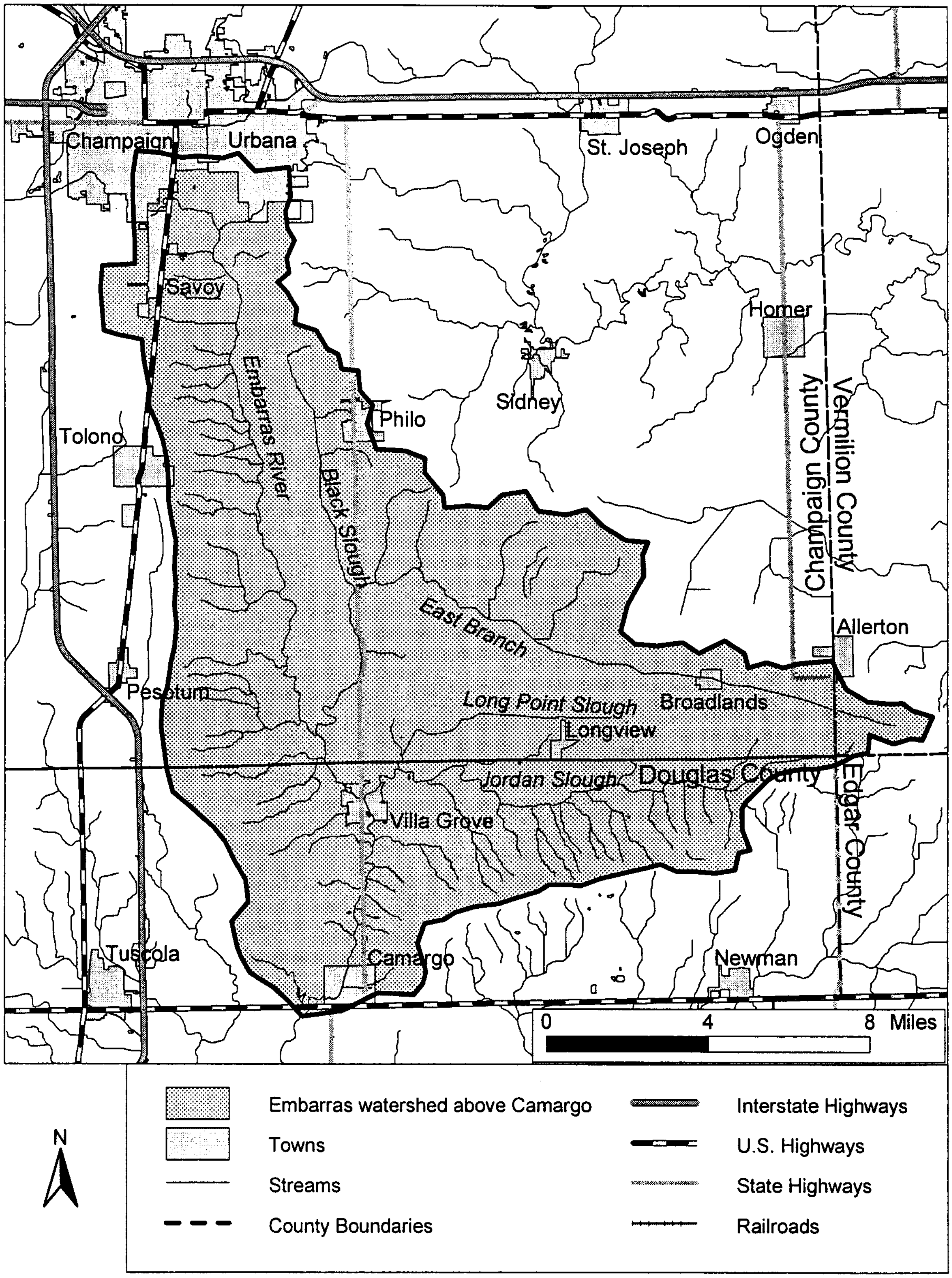

Figure 1. Schematic map of the Upper Embarras River

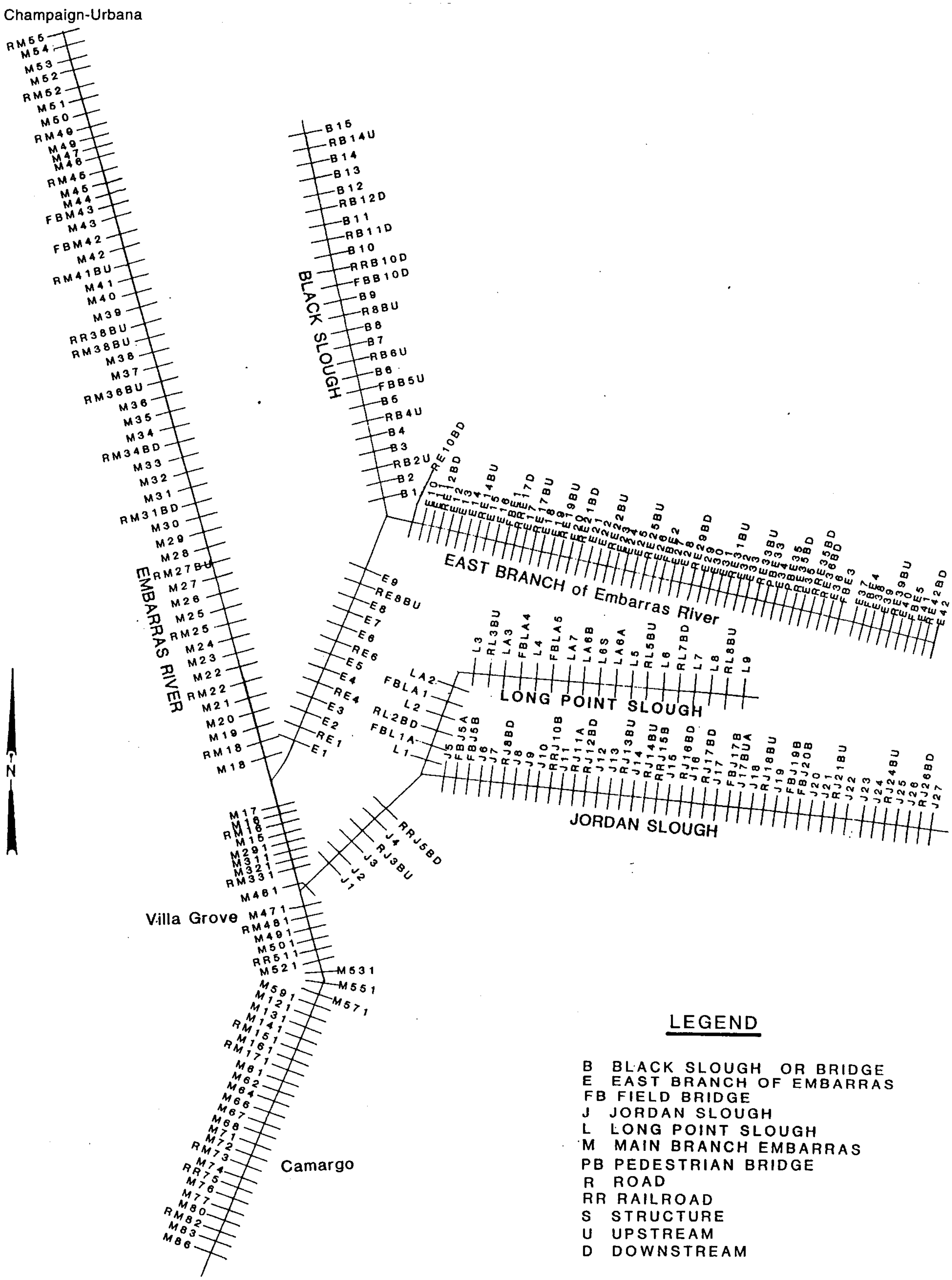

Figure 2. Cross sections used in the WSP-2 model of the Upper Embarras River (USDA, 1992b)

Table 1. Hydraulic Characteristics of the Tributaries and Main Stem (USDA-SCS, 1992a, b)

Sub-basin	*Total area, square miles*	*Channel length, ft*	*Drops in bed elevation, ft*	*Bed slope, ft/mile*
Embarras u/s of East Branch	57	76500	702 – 636	4.6*
Black Slough	12	31000	694 – 645	8.3
East Brach	43	96000	686 – 635	2.8
Embarras from d/s of East Branch to u/s of Jordan Slough	11	12500	636 – 629.5	2.7*
Long Point Slough	11	23000	680 – 632	11.0
Jordan Slough	32	59500	670 – 629	3.6
West Ditch	3			
Headwater Section of the Embarras above Camargo	186	120500	702 – 624	3.4*

Note: * See Table 2

Table 2. Bed Slopes along the Upper Embarras River (adapted from NRCS, 1992)

From – to	*Reach distance, ft*	*Drops, ft*	*Slope, ft/mile*
R1200E – R400N, Champaign	169000 → 228000	645 – 702	5.1
R400N – (section at 153000)	153000 → 169000	641 – 645	1.3
Section at 153000 – Rt. 130 (Villa Grove)	137000 → 153000	631 – 641	3.3
Rt. 130 – (d/s of Villa Grove at 131000)	131000 → 137000	630 – 631	0.9
D/S of Villa Grove – Rock Dam at 112000	112000 → 131000	629 – 630	0.3
Rock Dam – Camargo – US Rt. 36	100000 → 112000	622 – 629	3.1

Note: Reach distance at Camargo is 107460 and distance increases in the upstream direction. East Branch joins at distance 148500, Jordan Slough joins at distance 135500, and Villa Grove is located between 131000 and 140000.

Upper Embarras River basin are available at Camargo, approximately 6.3 miles downstream from Villa Grove. The U.S. Geological Survey (USGS) started a discharge and stage recording station at Camargo in October 1960. The Illinois Department of Natural Resources, Office of Water Resources (IDNR-OWR) has maintained crest stage records for the reach from Champaign to Lawrenceville, Illinois since 1971. Three crest-stage stations of interest to the present study in upstream to downstream direction are No. 15 at Route 130 (or Sycamore Street) Bridge, No. 16 at Front (or Harrison) Street Bridge, and No. 17 at Chicago and Eastern Railroad Bridge. The drainage area above the Front Street Bridge is 155 square miles. However, the crest-staff gage records only the highest stage, not the timing of floods between recording periods. Therefore, for more detailed flood information on Villa Grove, data has to be retrieved from various sources, including Villa Grove newspaper files, high water marks, local residents, or publications such as those by the U.S. Weather Bureau or the U.S. Army Corps of Engineers (USACOE).

Tables 3 and 4 were prepared to infer stage and discharge data in Villa Grove. Table 3 from IDNR crest-gage data selects peak stage data above 644 feet-msl, a level related to flood damage

Table 3. Crest Stages above 644.7 feet-msl in Villa Grove (Source: IDNR-OWR, 1996)

Year	*Station 15*	*Station 16*	*Station 17*
1993			644.18
1991		646.68	646.30
1986		644.9	644.6
1985		646.0	645.7
1983	646.25	645.61	645.27
1981		644.41 (u/s)	644.20
1976	644.12 (u/s)*	643.93 (u./s)	
1974	647.89	645.26 (u/s)	New gage installed
1973	644.76	644.59 (u/s)	

Note: *u/s, or upstream, reading was taken at upstream face of bridge.

Table 4. Past Floods in Villa Grove (Sources: USDA, 1992a; USACOE, 1955)

Year	*Referenced date*	*High Stage in Villa Grove, feet*	*Instantaneous peak discharge at Camargo, cfs*
1994	~ Apr 12		8,040
1979	~Mar 4		6,240
1974	June 23		6,230
1950	Approx. Jan 1~6	648.2	6,000
1943	Approx. May 7~20	647.5	
1939	Mar 11-12	647	5,000
1985	~ Feb 24		E5,630*
1961	~ May 9		5,200
1990	Dec 30 - 31	646.5	4,810

Note: *E, estimated discharge.

levels in Villa Grove (see Table 5). Blanks in the table indicate that data were not available. Table 4 was based on published reports such as NRCS (1992b) and USACOE (1955) while instantaneous highest peak discharges at Camargo for the specified period were retrieved from the USGS's automated data processing system ADAPS. Note that there were earlier floods in 1907, 1913, and 1925, but their information was not available. The 1950 flood is referred to as the historical flood in the USACOE report.

If the high water stages recorded at the three stations were from the same event in each recorded year, Table 3 showed relatively minor changes in flood stage from upstream of the confluence with Black Slough to the confluence and downstream at the railroad bridge for most events. However, data inadequacy as demonstrated in Tables 3 and 4 makes an observation difficult. Flood discharges in the Embarras basin above Villa Grove were collected between December 22, 1949 and July 25, 1951 at the Harrison Avenue Bridge in Villa Grove (USACOE, 1955), but the recorded events were not significant. Recently, students and a teacher at Villa Grove High School have started to collect daily stage readings at Route 130 Bridge and nearby rainfalls (data provided by

Table 5. Buildings Flooded and Estimated Damages

Return interval, years	*Stage, feet*	*Buildings flooded*	*Total damages*
100	650.9	569	$4,113,600
50	649.6	371	$1,549,600
25	647.9	42	$509,000
10	646	18	$78,400
5	644.7	N/A	N/A
2	643.2	2	$1,300
1	641.5	0	0

science teacher Linda Shadwick through project collaborator Leon Wendte). These continuous efforts will greatly benefit analyses in the future.

Table 4 showed that floods frequently occurred in late winter or in spring. Such information is useful for later analyses of vegetative resistance, i.e., during dormant or growing seasons. The USACOE (1955, p. 12) has described the timing and sources of floods in the watershed as: "Of the various types of meteorological disturbances which produce precipitation in the Embarras River Watershed, the cyclonic storm is the most frequent cause of excessive runoff. Precipitation normally occurs along the front of the disturbance due to a merging of the warm, moist air from the Gulf region in the period from mid-winter to late spring when ground conditions are conducive to high runoff, have produced major floods in the Embarras basin. Convective storms, which are productive of the greatest rainfall intensities, usually occur during the summer season. Major floods seldom result from these storms since they occur at times when evaporation, transpiration and seepage losses are high." That report also suggested that floods from Jordan Slough and West Ditch cause only minor damage, but backwater flooding from the Embarrass River results in extensive inundation.

Flood Damages in Villa Grove and Vicinity

The Village of Villa Grove occupies both banks of the stream. Approximately two-thirds of the town’s establishments are on the west bank (~ 400 acres). Elevations downtown range from approximately 630 feet in the stream channel to 656 feet at the highest point with an average elevation approximately equal to 645 feet (Federal Insurance Administration, 1978). Table 5 summarizes the estimated damages (Upper Embarras River Basin Planning Commission, 1996) using the number of buildings flooded, and flood characteristics analyzed by the SCS (USDA-SCS, 1992a).

Proposed Tree Planting Activities

The proposed tree planting and maintenance activities involve two segments of the river (Project meeting with Leon Wendte and Karl Visser, 1999). The CREP program will plant trees on 2-year floodplains along the Upper Embarras River from Villa Grove upstream to the Norfolk and Western Central Railroad Bridge, along the East Branch up to the junction with Route 130. Trees will be planted on a 10 foot by 10 foot grid from the edge of the channel up to the 2-year floodplain boundary. Maintenance activities will involve thinning trees in reaches downstream of Villa Grove in the 2-year floodplain.

Field Reconnaissance

Factors that contribute to the magnitude of roughness coefficients need to be evaluated in the field. Field data collection started as soon as the funding was announced in 1998, and three stream walks were conducted in 1999 to assess vegetation conditions upstream and downstream of Villa Grove. The upstream walks were done before young foliage emerges, representing the conditions of late winter or spring months, and the downstream walk was done during the summer when trees were in full bloom and maintenance is targeted. The two stream walks for winter or spring conditions were conducted in late autumn due to the accessibility of field conditions. The observations are as follows.

Embarras River from Bridge on 200N Road South to Bridge on 100N Road

The floodplains were covered in pasture immediately downstream of the 200N Bridge (the reach length from 200N to 100N is approximately one mile). Sparsely distributed young trees appeared about a sixth of a mile downstream from the bridge on 200N road mostly along streambank areas but did not extend onto the floodplain. The average diameter of tree trunks was about 6 inches with tree trunks clustered together and spacing between clusters of about 20 to 30 feet. Debris was observed. Tall grasses remained on the floodplain and were very dense. Occasionally, there were patches of mature trees located mostly on the right bank of the river (looking downstream). Dense trees on floodplains started approximately half a mile south of the bridge on 200N to the bridge on 100N road. The trees zone began to spread out, extending about 150 to 200 feet on the floodplain, and the diameters were in the range of 10 to 15 inches. A density count showed 15 trees in a 30 foot by 30 foot square. Several beaver dams blocked the channel, and there were downed trees in the channel along this reach. Figure 3 presents selected photographs.

East Branch from Bridge on 200N to Confluence of East Branch and Embarras River to Bridge on County Line

Trees along the bank of East Branch between 300N and 200N are mostly taller trees in comparison to the upstream reaches. South of the bridge on 200N, trees are growing on the floodplain on both sides of the river. They are about 8 to 10 inches in diameter and 8 to 10 feet apart in spacing; a field count found 10 trees in a 30 foot by 30 foot square. Tree density varied greatly along the reach; another count at upstream of the County Line Bridge resulted in 15 trees in the 30 by 30 square feet area. Ground covers were sparse. Besides trees there were log jams and downed trees in the river. Due to mild channel slopes and perhaps low discharge at time of reconnaissance, flows were slow in the channel. Figure 3 shows selected photographs.

Embarras River from Villa Grove to Camargo

Vegetation conditions in this reach were observed in July 1999. Overall trees in this segment were mature (diameter as large as 3 feet) and extended on floodplains along the river. Grasses were very dense and also very tall at some open areas. However, in some reaches the floodplain appeared to be free of groundcover under a dense canopy of 40- to 60-foot tall trees.

Figure 3a. Main channel and floodplain downstream from Site 1

Figure 3b. Inundation of floodplain upstream of Site 3

Figure 3c. Floodplain and main channel upstream of Site 2

Figure 3d. Floodplain and main channel north of Site 4

In general, tree diameters varied from 0.5 to 3 feet, but the dominant diameter was approximately 1.5 feet. Trees were very tall, about 50 to 60 feet, with the first 20 feet being trunks with few branches. Tree spacing varied but could be described as an average distance of 30 feet. Figure 4 shows four selected channel or floodplain conditions.

Collection of Stages, Discharges, and Suspended Sediment Data

Although the primary objective of this project was to evaluate methods for computing vegetative roughness and demonstrate the method's applications, this project collected limited data describing the hydraulic characteristics of the stream and trees on the floodplains. These data are presented here for informational purposes and future reference. Stage, discharge, and sediment samples were collected at five locations during the spring of 1998 and 1999 (see Figure 5 and Table 6). However, measurable flooding events did not occur in either 1998 or 1999; therefore, the data were not useful for calibrating flood events. These data revealed the dynamic soil erosion in the watershed, however. During an April storm event in 1999, suspended sediment data were collected at these five sites for analysis of the variations of suspended sediment concentrations between stations and tributaries. The variation between stations was not enough to indicate the effects of buffer strips; perhaps the flood was relatively small and the floodplain was not sufficiently involved. The temporal variations with the three-day period were substantial, on the other hand. Table 7 lists the laboratory results, and Figure 6 shows the variations in suspended sediment (coloration) in sampler bottles.

Figure 4a. Floodplain and main channel downstream of Villa Grove

Figure 4b. Floodplain in Villa Grove to Camargo reach

Figure 4c. Main channel and floodplain in Villa Grove to Camargo reach

Figure 4d. Matthew Hoffman surveys ground cover on floodplains of the reach between Villa Grove and Camargo

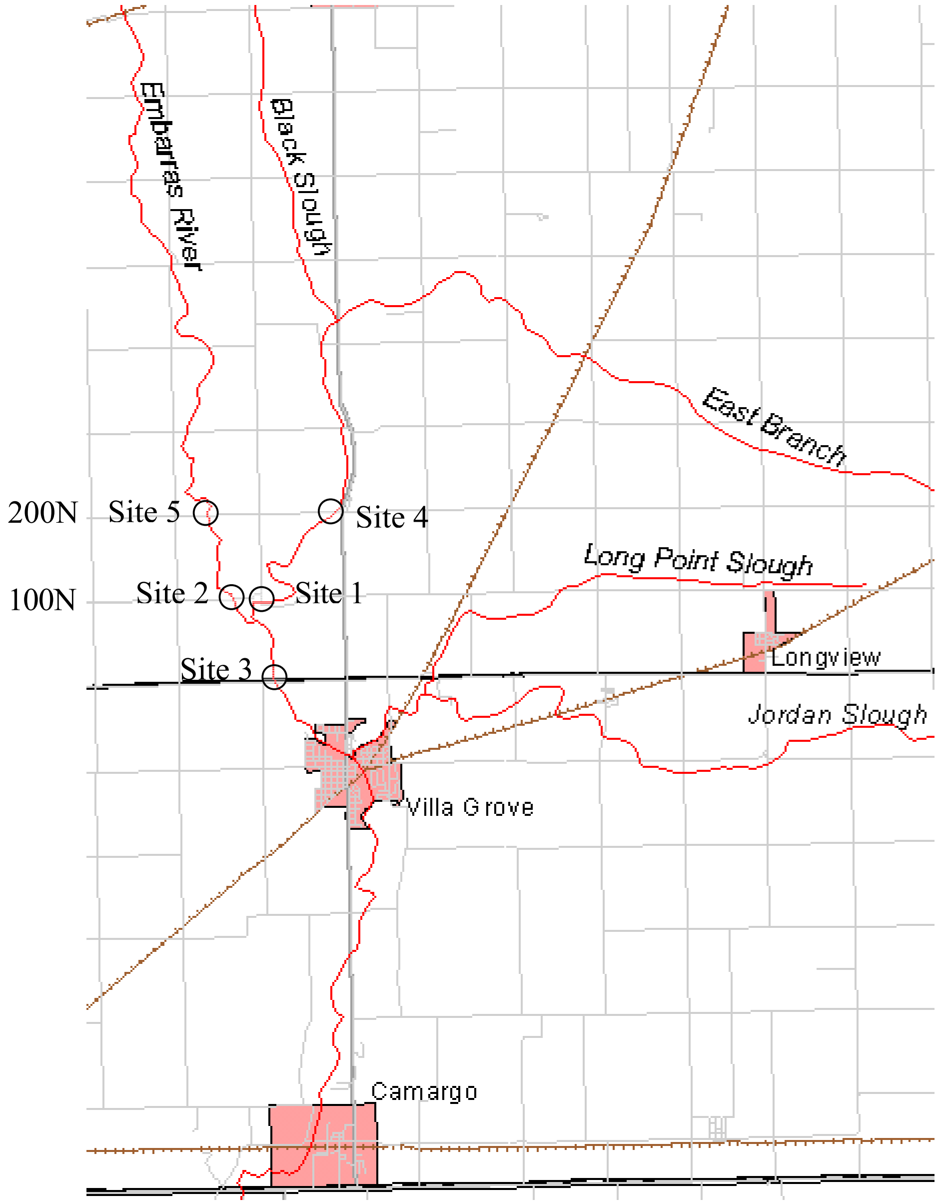

Figure 5. Location of field stations

Table 6. Collected Data on Stage, Discharge, and Suspended Sediment Concentration

	Site 1			Site 2			Site 3		
Date	Stage (ft)	Qw (cfs)	SSC (mg/L)	Stage (ft)	Qw (cfs)	SSC (mg/L)	Stage (ft)	Qw (cfs)	SSC (mg/L)
04/09/1998		260.42			161.44			404.73	
05/11/1998		164.42			115.73			291.88	
05/20/1998	644.85			644.66			642.25		
05/21/1998	641.85			645.86			641.75		
05/24/1998	644.00			644.21			641.94		
05/25/1998	643.60			644.27			641.66		
05/27/1998	641.89			642.08					
06/04/1998									
03/29/1999		33.48	21.552	640.11	21.01	20.535		52.94	18.119
04/20/1999	641.85	160.23	28.352	642.19	135.33	74.952	639.54		45.425
04/21/1999		205.65	219.650		360.36	297.798			325.733
04/22/1999	641.73		21.028	643.02		92.395	639.57		39.677
04/27/1999	640.81		30.838	641.30		69.329	638.11		50.113
05/13/1999	640.51		51.832	640.58		60.391	637.15		48.156
05/18/1999	640.65		82.316	641.61		173.961	637.84		127.104

	Site 4			Site 5		
Date	Stage (ft)	Qw (cfs)	SSC (mg/L)	Stage (ft)	Qw (cfs)	SSC (mg/L)
04/09/1998						
05/11/1998						
05/20/1998	650.37			647.24		
05/21/1998	648.97			645.44		
05/24/1998	649.11			646.86		
05/25/1998	647.91			647.34		
05/27/1998	646.80			644.59		
06/04/1998	645.43			643.86		
03/29/1999	645.25		20.484	643.39		14.667
04/20/1999	646.61	143.64	27.633	644.89	120.96	
04/21/1999		276.17	325.733		315.87	333.559
04/22/1999	647.54		20.470	645.48		107.182
04/27/1999	645.76		26.357	643.97		68.651
05/13/1999	645.58		41.339	643.55		67.615
05/18/1999	645.56		62.105	644.22		303.295

Table 7. Particle Size Distributions of Collected Suspended Sediment and Bank Materials on Floodplain (in percentage finer than sieve sizes specified in column 1)

Size, mm	*Site 1* 5/18/99	*Site 2* 5/18/99	*Site 3* 5/18/99	*Site 4* 5/18/99	*Site 5* 5/18/99
0.063					
0.031	100	100	100	100	100
0.016	93.6	90	87.7	93	82.4
0.008	85	68	68	79.7	64.2
0.004	67.7	55.2	55.6	65.4	53.7
0.002	64.8	48.6	49.9	64.3	45.5

				Floodplain soil samples collected from sites upstream of Camargo	
	Site 2	*Site 3*	*Site 5*		
Size, mm	*4/27/99*	*4/27/99*	*4/27/99*	*Right bank*	*Left bank*
0.063	99.9	99.9	99.8	100	100
0.031	25.4	22.7	22.8	97.6	92.8
0.016	21.8	19.9	20.4	65.2	79.5
0.008	14.2	16	16.2	68.8	62.3
0.004	13.9	14.5	14.6	55	50.2
0.002	11.6	13.2	13.3	45.1	42.4

Figure 6a. Suspended sediment samples collected during an April storm in 1999. Samples were arranged in three groups to show the sites from which they were collected, i.e., from left to right they were collected from sites 5, 2, and 3, respectively, on the Embarras River. Samples to the left in each pair were collected one day later at same site. The printed year on the photo was entered incorrectly.

Figure 6b. Suspended sediment samples collected during an April storm in 1999. Similar to the explanation for Figure 6a, these samples were collected from sites 4,1, and 3 (repeated for comparison purposes). Samples to the left in each pair were collected a day later. The printed year on the photo was entered incorrectly.

Approaches to Determine Manning's Roughness Coefficient

Manning's formula is one of the most widely used formulas in open-channel flow problems. It can be used to compute the flow velocity or discharge in a wide range of channel configurations and produce reasonable results. Although the formula was developed more than a century ago and there are more sophisticated formulas, it is unlikely that the Manning formula will be replaced in the near future. Due to the popularity of the Manning formula, extensive efforts also have been devoted to establishing appropriate Manning's coefficients in various applications. Recently, integrating river and riverine systems has been recognized as the most effective approach to watershed management and the use of restoration/conservation methods is an essential component. It is important to evaluate the performance of those practices in terms of environmental impacts. This investigation on flood mitigation with riparian forest management is an example that prompts us to study the applications of Manning's roughness in these multidisciplinary areas.

There are many published reports concerning Manning's n values. There appears to be confusion on the part of many users who consider the determination of n values to be empirical rather than physically based. Hence the n value becomes an adjusting factor in model calibration. This chapter summarizes relevant background information so the users can develop a rationale to determine Manning's n value in their problems. Many questions raised do not have analytical solutions at present; however, there is sufficient empirical information to proceed. Existing literature was reviewed to find published n values with emphasis on vegetated channels. Discussions also covered the complexities in practical applications when natural channels have distributed roughness and floodplain-main channel geometries. Many materials presented in this chapter were derived from courses taught by and discussions with B.C. Yen of the University of Illinois at Urbana-Champaign, whose publication *Channel Flow Resistance* (Yen, 1992b) is recommended to readers wanting more in-depth knowledge on Manning's formula. Continuous investigations on such topics can be useful for other C-2000 projects as well.

Manning's Equation

Manning's formula has the form (e.g., Chow, 1959)

$$V = \frac{k_n}{n} R^{2/3} S^{1/2} \tag{1}$$

where V is the average velocity, R is the hydraulic radius of the whole channel cross section, S is a slope term, k_n is 1.482 (English unit) or 1 (metric unit), and n is Manning's roughness coefficient. The R is computed as:

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

where A is the cross-sectional area and P is the wetted perimeter; both can be determined by field measurements or from project design. In natural channels, S can be approximated using bed slope or head difference over a distance. Yen (1992b) has defined and discussed various forms in evaluating the slope term. The remaining variable to be determined is the n value. The n value represents the user's interpretation of flow resistance from the channel under consideration and presents a great challenge to users.

It is also worth noting that Manning's formula is based on regression analysis where the exponents were derived from data fitting (Yen, 1992b). In general, uncertainties exist when applying a regression equation that is derived on the basis of fitting data from laboratory flumes and small experimental channels to broader natural conditions. However, Manning's formula has achieved satisfactory results for most cases provided that proper roughness values are used. Manning's equation is one of the uniform flow equations stemming from Chezy's development, i.e., it assumes that flows are steady, uniform, and have a fully developed turbulence regime (e.g., Chow, 1959). Such conditions are rare in natural channels. However, by limiting the analysis to specific channel length and time intervals, one can make reasonable approximations. Some researchers and practitioners prefer to use f, the Darcy-Weisbach friction factor; or C, the Chezy's roughness coefficient, instead of n. As long as basic assumptions about flow are maintained, the velocity can be expressed in any one of the three formulas, and their resistance coefficients are related (e.g., Chow, 1959; Simon s et al., 1982; Yen, 1992b):

$$\sqrt{\frac{8}{f}} = \frac{C}{\sqrt{g}} = \frac{k_n}{\sqrt{g}} \frac{R^{1/6}}{n} \tag{3}$$

where g is the constant for the acceleration of gravity.

Manning's Coefficient

Point, Cross-Sectional, and Reach Values

When referencing n values from a formula, a table, or a photograph, confusion may arise if no distinction is made between the differences among point, cross-sectional, and reach values. When referring to n values from photographs (e.g., Chow, 1959; Barnes, 1967), the reader sees a photograph of a reach of the river that is in compliance with computations carried out. However, the computations of the n values were specified at selected transects in the picture, not for the whole reach. Sometimes the n values may even vary across the cross section. Can the true n values be inferred from photographs? Probably not. Similarly, abundant formulas developed from controlled experiments essentially focus on a point on the cross section. Can these formulas be used directly without field verification? Such concepts, essential for clarifying confusion in flow resistance, have not been extensively examined to date.

Yen (1992b) presented the derivation of these three values and discussed their differences from the momentum and energy approaches. An example where point, cross-sectional, or reach values are similar is the pipe flow with homogenous surficial roughness. For channels, other factors such as the geometric factors (e.g., cross-sectional shapes) and/or flow variations (e.g., the secondary circulation) enter the integrals and these values are no longer equal. Because these factors are fairly complex, analytical developments often have to simplify natural conditions. Without knowing these underlying assumptions, mistakes can be made when a value is taken from the reference and applied to different settings. "Indeed, the point resistance and cross-sectional resistance can be and usually are determined from data obtained from controlled laboratory channels of simplified conditions. Conversely, reachwise resistance is usually deduced from field data. Basically in the past, resistance coefficients in the Weisbach f form were developed on the point (depth) concept, sometimes extended to reaches and cross sections without proper consideration of the geometry effects. On the other hand, Manning's *n* or Chezy's C were developed in the past on the reach concept, and sometimes used as point value without proper adjustment for the geometry effect" (Yen, 1992b, p.33).

Contributing Factors from the Flows

Using the surficial materials to describe *n* values is focused on surface resistance instead of hydraulic resistance. Rouse (1965) analyzed the open-channel resistance using surface resistance, form resistance, wave resistance, and unsteadiness. In addition to the conventional thinking that flow resistance originates from boundary irregularities due to surface textures, Rouse showed that sectional nonuniformity affects flow resistance due to changes in cross-sectional shapes along the channel axis and unsteady free-surface flows like waves or sediment movements. Factors discussed under surface resistance were Reynolds number, relative roughness, and cross-sectional shape; under boundary nonuniformity were nonuniformity of the channel in both profile and plan, and Froude number; and under unsteadiness were various degrees of unsteadiness at given Froude numbers. Given the multiplicity of the factors, an analytical expression for roughness coefficients can be developed only for the grain resistance under "surface resistance", with solutions obtained for uniform flows in pipe or rectangular channel with great width-depth ratios.

Chow (1959) described many interrelated factors in practical flow conditions affecting hydraulic roughness, including surface roughness, vegetation, channel irregularity, channel alignment, silting and scouring, obstructions, and stage and discharge. Other factors such as size and shape of channel, seasonal change, and suspended material and bed load might also affect the Manning *n* values, but the effects were suspected to be less significant.

Yen (1992b) presented a comprehensive analysis on the factors affecting the flow resistance to bridge the gaps between theoretical development and practical applications. Yen pointed out that our current understanding of channel resistance has been achieved only for a special case of steady, uniform, sediment-free flow in channels of impervious rigid boundary, with densely distributed, statistically homogeneous roughness elements. The challenge to engineers is to determine *n* for conditions outside of these restrictions.

Attempts for Adjusting the n Values

The effects of flow factors are to increase the "base" *n* value. Einstein and Banks' experiment (1950) suggested that the total resistance exerted by combined types of roughness equals the sum of the resistance forces exerted by each type individually. Cowan (1956) proposed linearly combining the contributions to the Manning *n* value. In this approach, a base roughness value for a straight, uniform, smooth channel is established; and adjustment factors are added for surface irregularities, variation in shape and size of channel cross section, flow obstructions, vegetation, and meandering of the channel. Note no flow unsteadiness has been considered. Cowan's formula is as follows:

$$n = (n_0 + n_1 + n_2 + n_3 + n_4)m_5 \tag{4}$$

where n_0 is a basic n value for a straight, uniform, smooth channel; n_1 is the adjustment for the effect of surface irregularity; n_2 is the adjustment for the effect of variation in shape and size of the channel cross section; n_3 is the adjustment for obstruction; n_4 is the adjustment for vegetation; and m_5 is a correction factor for meandering channels. Cowan has assigned a range of values to these factors; however, others have attempted to improve the adjustment values (e.g., Simon et al., 1982). Existing knowledge concerning these values is limited. There also is the question if linear superposition is truly representative for reachwise integration. To some degree this formula provides a means to integrate the distributed roughness along the cross section or a reach. Other researchers such as Arcement and Schneider (1989) adopted this method in computing the roughness of the floodplains.

Variation with Completeness of the Routing Equations Used

Since Manning's roughness coefficient has been used to represent the apparent roughness that includes the contributions from flow and geometry, one can reason that different *n* values will be assigned in different routing equations even to the same geometry and flow. For simplified equations, Manning's *n* is the place where users put contributing factors that cannot be described by the routing equation. Therefore, generally the *n* values are not transferable when different models or routing equations are used for the same study.

Variation of n Values with Depth

The simple form of Equation 1 may lead some to consider only one *n* value for the whole cross section without considering the effects of changing flow depth. Stage fluctuations will add or eliminate additional roughness to the cross section, and there are also accompanying changes in channel geometry. Therefore, these effects must be considered in the representative *n* values. Chow (1959, Figure 5-4, p.105) illustrated that the changes in *n* value with stage can be significant. The changes in *n* values with river stages can add another variable in gleaning *n* values from published references. This topic is discussed later under the headings "Composite Distributed Roughness" and in "Variations in Compound Channel".

Dimension of Manning's Coefficient

Manning's formula and *n* values were originally developed in metric units. Not only does the coefficient have a dimension of length to the one-sixth power, when the formula was converted to English units, a different table of *n* in English units was not prepared. Therefore the coefficient carries a mixed unit coefficient of 1.486 [ft$^{//3}$ - m$^{//6}$ /sec] (Yen, 1992a). There is no problem in using the *n* in either system as long as the units are consistent and the proper constant k_n is used. The conversion is straightforward but creates separate tables of *n* values for metric and English units.

References for Selecting Manning's Roughness Coefficients

Values of Manning's roughness coefficients for man-made and natural channels have been presented in enormous references in the forms of tables, photography, formulas, or in combinations thereof. The collection of such knowledge is part of the reason for continuous use of the Manning roughness coefficient approach. The following review includes literature assuming the channel boundaries do not change with time. When using the references, users are reminded of the factors discussed above, i.e., point-, cross-sectional, and reachwise values, adjustments according to flow conditions, variations with flow stages, dimensionality, and geometry of the channels.

- Chow (1959) published the first extensive tables for *n* values. The tables provided base values for the smoothest reach attainable for each bed material. Chow's photo collections covered a wide range of channels from cement lining to riprap to vegetated banks. Brief descriptions of materials on the channel surface, channel alignment, and foliage accompany each black-and-white photo.
- Fasken (1963) showed drainage channels with vegetation in black-and-white photos. Measured data and calculated *n* values at selected sections in one- or two- year span were presented. The description of roughness elements is similar to that of Chow, but Fasken included descriptions of the channel after each photo and the date when the picture was taken.
- Barnes' (1967) photo collections covered natural channels typical of rivers in North America. The color photos are also a supplement to more extensive tables showing *n* values calculated from field measurements (mostly one measurement). Channel reaches were described, including the upstream and downstream cross sections. In addition to focusing on a variety of artificial and natural (single) channels, some floodplains were included. The range of *n* values for natural channels varied from 0.024 to 0.075.
- Benson and Dalrymple (1967) provided base *n* values in average conditions. Their approach was similar to that of Chow.
- Limerinos (1970) developed a formula to calculated refined *n* values for reaches from streams in California having base values that correspond to Benson and Dalrymple's (1967) table. This equation's application is limited. Reach roughness must come primarily from the bed material. Flow must be highly turbulent and the channel made of some type of sand.
- Aldridge and Garrett (1973) arranged *n* values by bed material type and particle size. Bed material varied from soil to concrete for stable channels. Special detail was given to sandy channels. The effects of vegetation and other factors were somewhat neglected. This collection contains natural conditions for channels and floodplains in Arizona.
- Ree and Crow (1977) covered many different kinds of vegetation in crops and grasses, including wheat, cotton, and grass, on waterways of small slope. Descriptions of the channel and tables accompanied photos of the ditches, and individual plants were included to give users a better assessment of the characteristics of vegetation. The experiments were similar to those conducted in Ree and Palmer (1949) where grass was the lining material. The *n* values determined were for grass waterways and specifically for the type of channel setting and vegetation. Most previous investigators claimed that the *n* versus VR relationship was practically independent of channel slope and shape. This claim is probably true in the turbulent flow regime, but not in other flow regimes.

- Schneider et al. (1977) provided color photographs of heavily vegetated floodplains from the Lower Mississippi River. Features significant to the *n* values were provided for the floodplain portion but not for the channels corresponding to each floodplain.
- Simon, Li, and Associates (1982) listed common values for different kinds of bed materials, channels with vegetation, and floodplains with vegetation (table 6-1 in their report). The report also discussed *n* values on sand beds with various bed forms.
- Arcement and Schneider (1989) focused on the roughness values for densely vegetated floodplains. They provided a formula based on the base *n* values and various adjustment factors to determine the *n* values for the floodplain. Color photos for various vegetated floodplains and the computed *n* values are presented. The *n* values were determined for the specified flow conditions described in the text. For other conditions, such as at different flow depths, the *n* values would need to be recomputed.
- Gilley et al. (1991) analyzed roughness for eight different surface residue types, such as corn, cotton, and pine needles. Regression coefficients were given for empirical equations relating roughness coefficient to percent cover and Reynolds number. Calculations were based on surface runoff. Darcy-Weisbach friction factors also were given.
- Hicks and Masson (1991) provided data on a wide range of natural streams typical of New Zealand rivers. They recognized that Manning's *n* can vary with discharges and hence provided multiple parameters measured in the field and a color photograph for each natural reach.
- Coon (1998) evaluated 12 existing formulas for computing *n* values. The type of streams varied from wide, low-gradient to high-gradient channels and narrow, low-gradient channels with streambank vegetation. Coon reported the applicability of these equations and the variation with depth for steep and mildly sloped streams.

Composite Distributed Roughness

The superposition approach for composite roughness proposed by Einstein and Banks (1950) and Cowan (1956) assumes each type of surface roughness is homogenously distributed in the channel. However, surface composition in natural channels is seldom homogeneous in such a manner. Differences may exist between main channel, banks, and floodplains even if a lumped approach is used. It is reasonable to assign a different Manning's *n* value to each representative segment of the cross section, and hence the apparent roughness for the whole cross section is a composite roughness even the flow conditions are ideal. Methods for computing the apparent roughness coefficient on the basis of distributed roughness have been discussed by Chow (1959). Yen (1992b) collected existing methods and derived additional ones. These ten methods all are in a form summarizing individual *n* values with an weighting coefficient. Yen also demonstrated that these methods do not produce consistent results even in a simple trapezoidal channel. Currently, no sufficient work has been done to determine which formula is suitable for composite channels with distributed roughness, or for natural channels of compound shapes, nor with variations in river stages. When river stage fluctuates, part of the boundary roughness added to or removed from the wetted perimeter would change the apparent roughness of the flow in the cross section.

Table 8 lists the ten equations and their assumptions. Some of the methods will be cited later. In application, the user divides the channel cross section into subsections so that hydraulic properties in each subsection can be treated uniformly. Then one hydraulic property is identified so that its value for the whole cross section is equivalent to the sum of contributions from each subsection. Assumptions are imposed to enable or simplify the summation process. Because the assumptions will affect the application of these equations, they are discussed here.

Table 8. Existing Equations for Computing Composite Roughness, Yen (1992b)

Equation	*Reference*	*Basic assumption*	*Eq. no.*
$n_c = \dfrac{PR^{5/3}}{\sum \dfrac{P_i R_i^{5/3}}{n_i}}$	Lotter (1933)	Total discharge is the sum of subarea discharge; slope $S = S_j$	(5)
$n_c = \dfrac{\sum (n_i P_i R_i^{1/3})}{PR^{1/3}}$		Total shear force, $P\sqrt{(\gamma RS)}$, is sum of subarea shear forces; $S = S_j$ and $V_j/V = (R_j/R)^{1/2}$	(6)
$n_c = \dfrac{\sum (n_i P_i / R_i^{1/6})}{P / R^{1/6}}$		Same as above except velocity $V_j/V = 1$	(7)
$n_c = [\dfrac{1}{P} \sum (n_i^2 P_i)]^{1/2}$	Pavlovskii (1931) Einstein and Banks (1950)	Total resistance force, F, is sum of resistance force. Also $S = Sj$, and $V^2/R^{1/3} = V_j^2/R_j^{1/3}$	(8)
$n_c = [\dfrac{1}{P} \sum (n_i^{3/2} P_i)]^{2/3}$	Horton (1933) Einstein (1934)	$V = V_j$, $S = S_j$, and $A = \sum A_j$	(9)
$n_c = \dfrac{P}{\sum (P_i / n_i)}$	Felkel (1960)	Note special case of Eq. 2.1 with $R_j/R = 1$	(10)
$n_c = \dfrac{\sum (n_i P_i)}{P}$		Contribution of component roughness is linearly proportional to wetted perimeter	(11)
$n_c = \exp[\dfrac{\sum P_i h_i^{3/2} \ln n_i}{\sum P_i h_i^{3/2}}]$	Krishnamurthy and Christensen (1972)	Logarithmic velocity distribution over depth h for wide channel, $S = S_j$, $Q = \sum Q_j$, $n = 0.0342k$	(12)
$n_c = \dfrac{\sum n_i A_i}{A}$	USACOE (1968) Cox (1973)		(13)
$n_c = [\dfrac{\sum (n_i^{3/2} A_i)}{A}]^{2/3}$	Colebatch (1941)	Same as Eq. 2.5, but with an error in derivation	(14)

The subscript i in the table represents the parameter in the i^{th} subarea, while parameter without a subscript is the property for the whole cross section. The assumptions in these equations are:

1. *Each subsection has the same mean velocity, which is also equal to the mean velocity of the whole cross section*. Examples are Equations 5 (Lotter, 1933), 9 (Horton, 1933; Einstein, 1934), and 14 (Colebatch, 1941). This fits the premises of Manning's equation on uniform flow but is not suitable to cases of obvious lateral distribution (e.g., depth varies largely between subsections).
2. *The total resisting force equals the sum of the forces developed in each subarea*. An example is Equation 8 (Pavlovskii, 1931; Muhlhofer, 1933; Einstein and Banks, 1950).
3. *The total shear force is the sum of subarea shear forces*. Examples are Equations 6 and 7 (Lotter, 1933). Equation 6 uses $V_j/V = (R_j/R)^{1/2}$, which is based on conveyances, while Equation 7 assumes that $V_j/V = 1$.
4. *The total discharge equals the sum of the discharge of the subdivided areas.* Examples are Equations 5 (Lotter, 1933) and 12 (Krishnamurthy and Christensen, 1972). This assumption should be applicable to rectangular or wide-shallow channels. The methods for dividing compound channels can affect the results of these equations, as appreciable errors have been noted when applying this assumption to shallow floodplain depths in compound channels.
5. *Weighting coefficients were determined according to the ratio of the wetted perimeter or subarea to the whole cross section's wetted perimeter or area.* Examples are Equation 10 (Felkel, 1960) and Equation 13 (Cox, 1973), respectively.
6. *The slope in each subsection is equal to the total slope*. Most of the equations employ this assumption, which may be valid for water surface slopes but not for momentum or energy slopes (Yen, 1992b).

Among these methods, the shear-stress approach should give a more reasonable approximation if the correct shear distribution is predicted. Masterman and Thorne (1992) adopted the iterative shear-stress distribution adjustment by Flintham and Carling (1988) on a trapezoidal channel with vegetated banks. With this approach, Masterman and Thorne demonstrated that the discharge capacity in the bank areas only become significant (>5%) in channels of width-depth ratio less than 9 with the type of grass tested, and there is a 38 percent reduction in discharge capacity for the fully grown vegetation when the width-depth ratio is 5.

Variations in Compound Channels

A channel with floodplains is called a compound channel, which is a common feature of natural channels. A compound channel can have drastically different roughness on the floodplain (like trees) than the main channel (bed materials). The relative width ratio of the main channel and floodplain, relative depth between the main channel and floodplain at a flood stage, and location and orientation of each roughness segment can make differences in flood conveyance. It can be seen that simulations in the floodplain–main channel system really requires two- or three-dimensional models. However, the required supporting data and computational facility have not yet reached a stage to allow such simulations on large scales; one-dimensional models still need to be used. The significance of compound channels in dealing with floods has made determining distributed composite roughness is a very important issue.

When flood stage exceeds bankfull, the inclusion of the floodplain alters flow patterns and hydraulic parameters. Research has shown that the steep velocity gradient between the main channel and floodplain caused momentum to transfer from the main channel to the floodplain, therefore gradually introducing flow movement on the floodplain as stage rises. The mass transfer between channel sections is not linear in such cases. The momentum and mass transfers also interact with relative surface roughness across the channel's wetted perimeter and, in turn, affect the distribution of shear forces. By applying Manning's equation to this situation, complex flow interactions cannot be explained by a whole cross section approach easily. Many researchers have proposed different methods to separate the flow areas, perform computations in each separate area and then sum the individual contributions together. Among the methods, vertical, bisection, horizontal, and zero shear divisions are more commonly seen in the literature (e.g., Posey, 1967; Wormleaton et al. 1982).

Soong and DePue (1996) conducted laboratory experiments and tested these formulas with a straight flume setting but varying roughness in the main channel and on the floodplain. The experimental results were then tested against the 10 equations using vertical and bisection methods. They found that, in general, the weighted formulas using the shear force approach are more appropriate for compound channels and Equations 8 and 5 formed the upper and lower limits for their experimental data. While there is a lack of knowledge in predicting composite distributed roughness, certain percentages of uncertainty also come from our ability to determine the base *n* values.

Methods to Calculate Vegetal *n* Values

Channel with Vegetation

A channel with vegetation is a practical example of distributed roughness. When vegetation emerges, vegetal roughness becomes the major contributor to the overall roughness. Because the flow in and over the vegetation involves complicated interactions between flow, fluid properties, and biophysical properties of the vegetation, new theories are still developing and there is no conclusive and unified approach.

Research on vegetation-induced roughness has been done on grass, row-crops, or shrubs on the channel lining. Ree and Palmer (1949) and later Ree and Crow (1977) developed a series of *n* vs. *VR* relationships for grass waterways. The *n* vs. *VR* plots considered the relative roughness, but many researchers questioned that the relationship has to tie to the type of grass, channel configuration, and slopes. Kouwen et al. (1979) showed that the *n-VR* method did not apply to low slopes (less than 1 percent) because this method applies only when substantial bending of the vegetation occurs. Kouwen and Unny (1973) improved the *n-VR* method by suggesting that vegetation be classified on the basis of flexural rigidity, which is defined as the product of *M* (relative stem density), *E* (modulus of elasticity of the vegetation), and *I* (the stem area's second moment of inertia). Kouwen (1988) suggested *M*, *E*, and *I* be treated as a single quantifiable parameter. Much research has been developed using relative roughness height and flexural rigidity related to the roughness through f, the Darcy-Weisbach friction factor. Several well-known formulas are listed here. It is important to know the type of vegetation and about submergence or emergence issues discussed in the literature. Figure 7 (adapted from Fischenich, 1997) illustrates the type of vegetation on a channel bank.

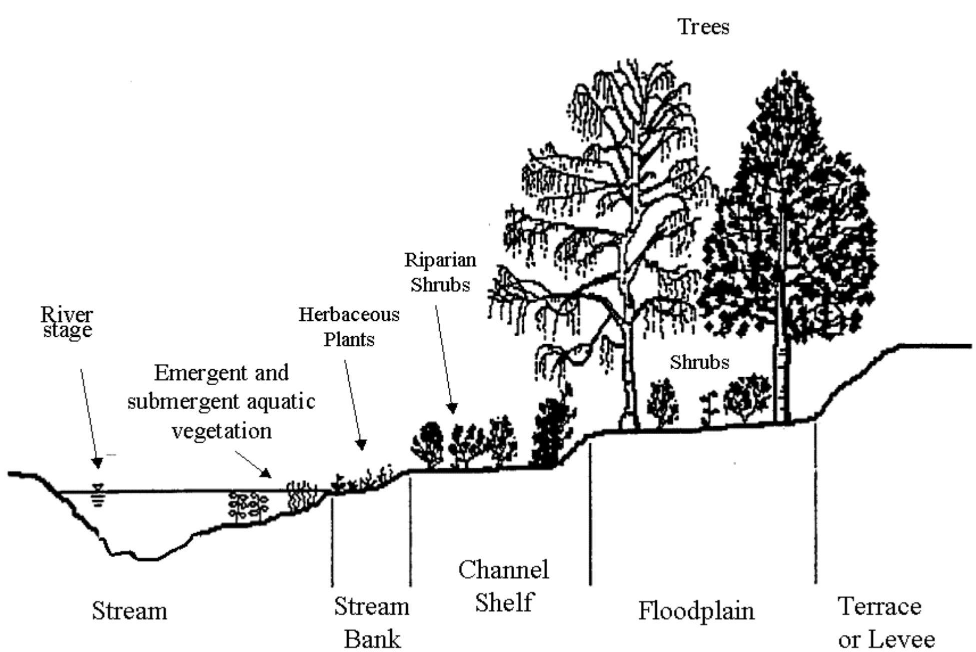

Figure 7. Definition sketch of vegetation in the riparian environment (Adapted from Fischenich, 1997)

Kouwen and Li (1980) showed that the roughness height (k) varies as a function of the amount of drag exerted by the flow

$$k = 0.14h\left(\frac{\left(\frac{MEI}{\tau}\right)^{0.25}}{h}\right)^{1.59} \tag{15}$$

where h is the local height of the strips (in meters) and τ is the local boundary shear stress (N/m^2).

Temple (1987) correlated the *MEI* term with grass vegetation height as:

$$MEI = 319\ h^{2.3} \text{ for growing grass} \tag{16}$$

$$MEI = 25.4\ h^{2.26} \text{ for dormant or dead grass} \tag{17}$$

After the vegetation height is obtained, one can substitute it into a relative roughness equation to obtain the roughness coefficient. The Colebrook – White equation, for example, has the form:

$$\frac{1}{\sqrt{f}} = a + c \log\frac{R}{k} \tag{18}$$

where a is a dimensionless coefficient that is a function of the cross-sectional shape, and c is a dimensionless coefficient equal to the von Karman coefficient.

For nonflexible vegetation, Thompson and Roberson (1976) used vegetation spacing S_v and vegetation diameter d_v , and defined the wake velocity u_w (m/s) behind nonflexible vegetation as:

$$\frac{u_w}{u} = 0.48\left(\frac{s_v}{d_v}\right)^{0.14} \qquad 4 \le \left(\frac{s_v}{d_v}\right) \le 20 \tag{19}$$

$$\frac{u_w}{u} = 0.70\left(\frac{s_v}{d_v}\right)^{0.08} \qquad 20 \le \left(\frac{s_v}{d_v}\right) \le 100 \tag{20}$$

where u is the approaching velocity in m/s. Once the wake velocity is derived, the friction factor can be obtained as:

$$f = \frac{8gRS}{u_w^2} \tag{21}$$

Darby (1999) presented a procedure involving these equations to compute the stage-discharge relationships for compound channels with sand-and-gravel beds, and with flexible and nonflexible vegetation on the floodplains.

The conventional approach to estimate roughness due to wooded vegetation is to treat trees as rigid cylinders and approximate the effect by drag forces (e.g., Li and Shen, 1973). Recent analyses have included vegetation properties such as submerged momentum-absorbing areas (e.g., Fathi-Maghadam and Kouwen, 1997) for trees (partially submerged vegetation) and large shrubs (submerged vegetation). Wooded vegetation is discussed in the following section.

Trees on a single floodplain can have a wide range of shapes and sizes. Unlike grasses or row crops, identifying characteristic features for a particular species can be difficult, and even more so for the wide range of species on a floodplain. The following approaches were considered in this project for evaluating n values due to trees on the floodplain.

Drag Forces Approach

The resistance to flow due to vegetation is generally approached by computing drag forces exerted on individual plants. The effects, or the total resistance, due to the arrangement of plants, diameter of plants, and other factors (for example, density of plants) are reflected in the values of a drag coefficient. Petryk and Bosmajian (1975) approximated the vegetation in a straight reach with drag forces and sought the apparent roughness of the channel using the sum of shear stress approach. The formula has the following form (English unit):

$$n = n_b \sqrt{\left[1 + \left(\frac{C_d \sum A_i}{2gAL}\right)\left(\frac{1.49}{n_b}\right)^2 R^{4/3}\right]} \tag{22}$$

where n_b is the roughness coefficient for the bed, C_d is the drag coefficient for the vegetation, A_i is the projected area of the i^{th} plant in the streamwise direction, A is the total cross-sectional area, L is the length of the channel under consideration, and g is the constant of gravitational acceleration. Equation 22 was derived for a single, uniform channel bed roughness and vegetation spread throughout the channel reach. Once the n_b is determined, the geometric parameters in the equation could be obtained from actual data, only the C_d needs to be determined for the practical situations.

Petryk and Bosmajian considered the vegetation density $C_d \sum A_i /(AL)$ as an effective way of relating the physical plant characteristics as a function of height to flow resistance through the vegetation. They recommended an indirect calculation of the apparent roughness by estimating the vegetation density as a function of height (submerged depth) and then applying the formula to evaluate the hydraulic roughness, i.e., the effective projected area of plants considering the reduction in trunk diameters and increases in branches and foliage or bending (not explicitly counted). They computed correlations for wheat, sorghum, and cotton using data from the Agricultural Research Service (SCS, 1954) and four vegetated channels. For other conditions, they recommended determining the relationship by measurement. With the reasonable range of flow conditions to be expected, Petryk and Bosmajian suggested C_d on the order of 1.0.

Determining the vegetation density is a critical step in applying Petryk and Bosmajian's formula. Flippin-Dudley et al. (1998) designed a field device to measure the vegetation density. The vegetation density is defined as:

$$Veg_d = \sum A_i / AL \tag{23}$$

where A_i is the projected area of an individual tree below water surface, A is the total flow area, and L is a characteristic length, such as unit length of the channel. The flow resistance in channels with partially submerged vegetation is computed as (Fischenich, 1997; Flippin-Dudley et al., 1998):

$$n_{non-submerg} = k_n R^{2/3} \left(\frac{C_d Veg_d}{2g} \right)^{1/2} \tag{24}$$

For estimating the drag coefficient, Flippin-Dudley et al. also found well-fitted relationships between C_d and the product of velocity and hydraulic radius VR, and/or the Reynolds number R_e using their measurements. The equations include:

$$C_d = 2.1(VR)^{-1.1} \quad \text{with leaves absent and debris present} \tag{25}$$

$$C_d = 2.8(VR)^{-1.1} \quad \text{with leaves present and debris absent} \tag{26}$$

or

$$C_d = 9.3\times10^6 (R_e)^{-1.1} \quad \text{with leaves absent and debris present, and based on a kinematic viscosity of } 9.1\times10^{-7}\ \text{m}^2\text{/sec.} \tag{27}$$

$$C_d = 9.1\times10^5 (R_e)^{-1.1} \quad \text{with leaves and debris absent, and based on a kinematic viscosity of } 9.1\times10^{-7}\ \text{m}^2\text{/sec.} \tag{28}$$

These formulas show that C_d decreased as velocity and depth increased, which is correct for flexible roughness. The numerical value of C_d is approximately 1 when $R_e > 2\times10^6$. The values are similar to those reported by Schlichting (1968) that C_d in an idealized two-dimensional flow is about 1.2 for a cylinder Reynolds number range of 8×10^3 to 2×10^5, and decreases as the cylinder Reynolds number of 2×10^5 is approached (Li and Shen, 1973). Klaasen and van Der Zwaard's experiment (1973) showed that the drag coefficients spread around 1 for Reynolds numbers larger than 3×10^3 for sparsely distributed vegetation patterns.

However, these values are for partial submergence conditions. Wu et al. (1999) investigated the variation of roughness coefficients with depth for partially submerged and submerged vegetation represented by a horsehair mattress. Test results revealed that the roughness coefficient reduces with increasing depth under the partially submerged condition. However, when the mattress is fully submerged, the vegetative roughness coefficient tends to increase at low depths but then decreases to an asymptotic constant as the water level continues to rise. The results show a consistent trend of variation for the drag coefficient versus the Reynolds number. This trend can be represented by a vegetative characteristic number k. Given information such as bed slope, vegetation height, and k, one can apply the proposed model to predict the roughness coefficient corresponding to different flow depths.

Linear Superposition

Arcement and Schneider (1989) modified Cowan's method (1956) for densely vegetated floodplains. The "Modified Channel Method" has a form similar to Cowan's for channels and is as follows:

$$n = (n_b + n_1 + n_2 + n_3 + n_4)m_5 \tag{29}$$

where:

n_b = a base value of n for the floodplain's natural bare soil surface,
n_1 = a correction factor for the effect of surface irregularities on the floodplain,
n_2 = a value for variations in shape and size of the floodplain cross section, assumed to be 0,
n_3 = a value for obstructions on the floodplain,
n_4 = a value for vegetation on the floodplain, and
m = a correction factor for sinuosity of the floodplain, equal to 1.0.

In order to evaluate the floodplain roughness, Arcement and Schneider provided a table containing the range of values for these adjustment factors. The table divides the correction for vegetation into five different categories from small to extreme. This modified channel method is also applied in conjunction with the method developed by Petryk and Bosmajian (Equation 22) as the "Vegetation-Density Method" for the floodplain's density. Essentially all the n values reported in their report were computed with the vegetation density method. However, the estimated C_d values seem too high, and most of the cases discussed were in the low Reynolds number regime. Up to present time, the knowledge of reasonable n values for each component has not been thoroughly investigated.

Equations Based on Laboratory and Field Data

Freeman et al. (1998) developed a regression equation based on large flume tests with live plants. For submerged vegetation, their equation takes the form:

$$n_{veg} = 0.0257\left(\frac{EI}{\rho V^2 A^* H^2}\right)^{0.166}\left(\frac{H}{Y_0}\right)^{0.227}\left(MW_p^2\right)^{1.25}\left(\frac{H'}{D_s}\right)^{0.508} \tag{30}$$

The equation for partially submerged vegetation is:

$$n_{veg} = 0.0393\left(\frac{EI}{\rho V^2 A^* H^2}\right)^{0.216} \left(MW_p^2\right)^{0.246} \left(\frac{H'}{D_s}\right)^{0.547} \tag{31}$$

In both cases the following variables are used:

n_{veg} = Manning's n value for vegetation
ρ = Density of water (slugs/cu. ft.)
D_s = Diameter of stem (ft.)
M = Plant density (plants/sq. ft.)
V = Average velocity (ft./sec.)
H' = Height of leaf mass (ft.)
E = Modulus of elasticity (lb/sq. ft.)
I = Area moment of inertia of plant stem (ft.^{4})
A^* = Effective plant area (sq. ft.)
H = Plant height = Y_0 if partially submerged (ft.)
W_p = Width of leaf mass (ft.)
Y_0 = Depth of flow (ft.)

Equations derived from laboratory data are valuable because they were based on physically measured phenomena. However, care also needs to be taken in applying these equations to different types of vegetation. The experiments used plants such as dogwood, elderberry, and willow. All plant types were broadleaf, deciduous vegetation with diameters of an inch or less. Vegetation that varies too much from these types may yield poor results, for example, mature trees without submerged branches or leaves under given flows. Tests on these equations sometimes significantly underestimated values, so the calculated n value should be checked for reasonableness before being used.

Integration of Drag Forces in Composite *n* Values

In channels with multiple surface roughness and vegetation, a generalized formula can be developed using summation of shear forces. The balance of force for the study reach and using the same nomenclature is:

$$\gamma ALS = \sum C_d \frac{\gamma V_j^2 A_j}{2g} + \sum \tau_i P_i L \tag{32}$$

where subscript i represents the i^{th} roughness segment and j stands for j^{th} tree with V_j representing the approaching velocity for the j^{th} tree. Substituting τ_i with $\gamma R_i S_i$ and relating RS to channel properties through Manning's formula, the apparent roughness for the cross section has the form:

$$n = k_n R^{2/3} \left[\frac{C_d \sum A_j}{2AgL} + \frac{\sum P_i n_i^2 / R_i^{1/3}}{A k_n^2}\right]^{1/2} \tag{33}$$

Tests on this equation were not performed because of a lack of information in existing data.

It is worth mentioning that Rahmeyer et al. (1999) derived a set of equations for computing vegetative resistance in floodplains and compound channels. The basic equation used for composite distributed roughness is Equation 5, and the composite roughness is computed as:

$$n = k_n \left(\frac{V_*}{V}\right)\left(\frac{1}{V_*}\right)(R_h)^{\frac{2}{3}}(S)^{\frac{1}{2}} \tag{34}$$

where $\left(\frac{V_*}{V}\right)$ is called the resistance coefficient, or the C_d. For submerged vegetation ($y_0 > 0.8H$), the resistance coefficient has the form:

$$\frac{V_*}{V} = 0.183\left(\frac{E_s A_s}{\rho A_i V_*^2}\right)^{0.183}\left(\frac{H}{Y_0}\right)^{0.243}(MA_i)^{0.273}\left(\frac{\nu}{V_* R_h}\right)^{0.115} \tag{35}$$

For partially submerged vegetation ($y_0 < 0.8H$), the resistance coefficient has the form:

$$\frac{V_*}{V} = 9.159E(-05)\left(\frac{E_s A_s}{\rho A_i V_*^2}\right)^{0.207}(MA_i)^{0.0547}\left(\frac{\nu}{V_* R_h}\right)^{-0.490} \tag{36}$$

Assessing Applicability to Upper Embarras River

After evaluating existing field information and data that could be retrieved from the literature, Equations 22 (drag force approach) and 30 and 31 (regression based on laboratory data) were compared for their applicability in mature and young tree scenarios. When applying Equation 22, the drag coefficient was assigned a constant value (a value of 1.13 was used) or computed by Equations 25 or 26, with the base *n* value to be 0.045, the same as the main channel, and other parameters retrieved from the HEC-RAS model. Mature trees were assumed to be rigid for the range of flow velocity encountered in the Upper Embarras River. For managed conditions, the spacing tested was 40 feet by 40 feet in each direction for mature trees and 10 feet by 10 feet for young trees on the 2-year floodplain.

The analysis has been done for all the reaches planned for tree-planting activities. It was found that with the constant C_d value, the Petryk and Bosmajian equation generally gives a lower but reasonable estimate of *n*, as compared to published values. When C_d is computed by Equation 25, the estimated *n* values appeared to be overestimated for part of cross sections but for most of the cross sections when Equation 26 is used. The partially submerged vegetation equation by Freeman et al. (1998) can be used for young trees but not for mature trees. In general, the parameters used in the regression equations for describing vegetal characteristics are not readily available to trees observed in the field. Equations 34 and 35 were not evaluated together with the above-mentioned equations due to their published timing. However, they have been included in a computer program (see appendix). The conclusion is that Equation 36 has a tendency to underestimate *n* values for young trees. Mature trees were not tested.

Clearly, the computed *n* values will vary from cross section to cross section. Table 9 contains the computed Manning's *n* values for several selected representative cross sections (cross sections are numbered according to their river miles), and such information has been communicated to the collaborator at the NRCS. The selected cross sections are:

- Embarras Downstream of Villa Grove: 1187+20, 1235
- Embarras Upstream of Villa Grove: 1422, 1564, 1638, 1786
- East Branch: E22 and E38.

Table 9. Calculated Manning's *n* Values for Tree-Planting at 100-year Flood

Location	*Depth on the floodplain, feet*	*Equation 22 for mature trees*	*Equation 22 for young trees*	*Equation 31 for young trees*
E22	11.6	0.052	0.10	0.097
E38	10.9	0.051	0.099	0.096
1187.2	12.2	0.052	0.10	0.093
1235	14.1	0.053	0.102	0.092
1422	13.3	0.053	0.101	0.091
1564	9.9	0.05	0.098	0.106
1638	7.7	0.049	0.095	0.102
1786	5.1	0.047	0.089	0.099

Effects of Tree Maintenance on Flood Conveyance Using HPG and Capacity Curves

By applying the *n* values discussed in the previous chapter to the HEC-RAS model, Visser (2000) analyzed the effects on flood stages of tree planting upstream and tree thinning activities downstream of Villa Grove. Among the analyses, Visser showed a 1-foot decrease at the Front Street Bridge in Villa Grove for a 100-year flood. This chapter presents work developed afterward with a focus on incorporating the Petryk and Bosmajian formula to the HEC-RAS model and developing capacity curves for the evaluation of the effects.

Incorporating the calculated vegetative roughness to the HEC-RAS model involves an iterative procedure. This is because the formula requires hydraulic parameters calculated by the HEC-RAS program for the given discharge, which in turn are the function of *n* values. After the iterations, the resulting *n* values need to be applied, for example, at locations representing the 2-year floodplain on each cross section. Then the same procedure is repeated for the next discharge. Doing these computations manually, especially for a large network, can introduce errors easily. A simplified approach uses one or a few representative *n* values and codes them at the designated segments of each cross section for the study reaches. Resulting differences are investigated in this chapter, along with how the HEC-RAS program composite distributed roughness.

Capacity curves (Yen and Gonzalez, 1994) were selected to analyze the tree thinning activities in the project because they present comprehensive information as simple graphics. Since the HEC-RAS program performs backwater computation from downstream to upstream stations, and the capacity curve is analyzed in separate river reaches, the tree planting effects on reaches upstream of Villa Grove were not evaluated here. Visser has evaluated tree planting effects using the full network model. When taking the reachwise analysis, storage provided by trees in the planting areas, and hence reduction in flood stages, in Villa Grove unsteady flow modeling should be used to evaluate, or to re-evaluate, the incoming discharges.

Composite Roughness Calculated by the HEC-RAS Program

Composite Roughness

The HEC-RAS program subdivides a cross section into overbank areas and the main channel as the basis for subdivision (Figure 8). Further division in each subarea is obtained with user-specified *n* value breakpoints. As shown in Figure 8, the left overbank is further divided into two areas with two distinctive *n* values. In this project, the subsegment represented by n_2 could be the 2-year floodplain, and an *n* value is calculated and substituted for the original *n* value for each cross section.

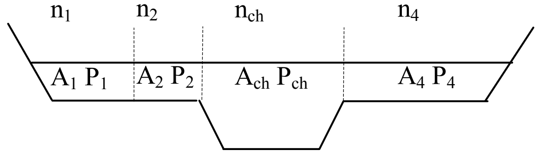

Figure 8. Subdivisions in HEC-RAS

The program computes the total conveyance by summing the conveyances in the left and right overbank, and the main channel. The incremental conveyance K_i for each i[th] subarea on the floodplain is computed as:

$$K_i = \frac{k_n}{n_i} A_i R_i^{2/3} \tag{37}$$

If there is no further subdivision on a floodplain, the whole floodplain area is used in Equation 37. The main channel conveyance is computed as a single conveyance element using the composite formula (9), which is

$$n_{ch} = [\frac{1}{\mathrm{P}} \sum (n_i^{\ 3/2} P_i)]^{2/3} \tag{38}$$

for the i^{th} subareas in the main channel. This equation is only applied if (1) there are multiple roughness values within the main channel and (2) the main channel side wall slope is steeper than 0.2 (i.e., 1V:5H). If the side slope is not steep enough, the HEC-RAS program will use the same procedure as for the overbank areas (HEC, 1997). In that case, the composite roughness is computed as Equation 5. As shown earlier, the composite methods could produce different results. Equations 5 and 6, when compared to experimental results (Soong and DePue, 1996), were identified as upper and lower limits of the ten equations tested. Figure 9 shows the results of this test. For a prismatic trapezoidal channel without floodplains, the HEC-RAS results converge to those of Equation 9. For more natural channel configurations, the HEC-RAS results also fall within a reasonable range.

Sensitivity of n Values to Computed Stages

A sensitivity analysis was conducted to evaluate how and in what manner the floodplain *n* values would affect the flood stages. To simplify the analysis, each channel cross section in the model was divided into three segments only: left overbank, main channel, and right overbank. The *n* value for the floodplains was then increased incrementally, and the corresponding change in stage was analyzed. Figure 10 shows one of the results at one selected cross section in the lower reach of the Upper Embarras River. The top figure shows the computed stages corresponding to different flood discharges when different *n* values were assigned to the floodplain. The middle figure is a sketch of the cross-sectional shape at this site with locations of the floodplains and main channel. The *n* values on the floodplain are replaced in each run. The lower plot is based on the same data but shows the deviation from a selected water surface elevation vs. the incremental *n* values tested. The results show the range of errors that can result from specifying a different n value in the lower reach of the Upper Embarras HEC-RAS model. For example, the water surface elevation for the 100-year flood would increase approximately 1 foot if the *n* value is 0.05 higher than the assessed value. Such a difference in *n* value is quite large in general.

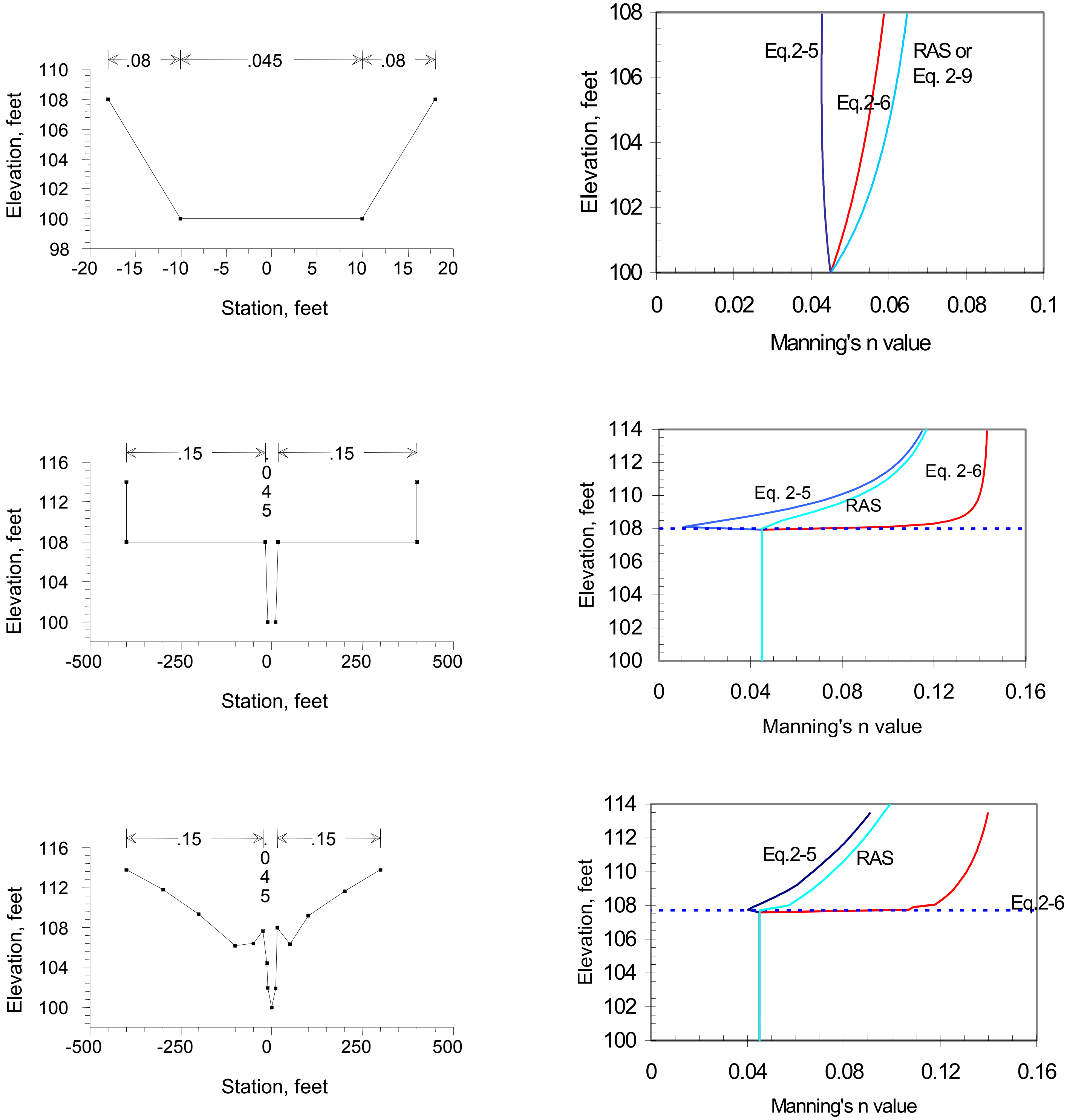

Figure 9. Comparisons of the method for composite distributed roughness used in HEC-RAS program and those defined in Equations 5 and 6

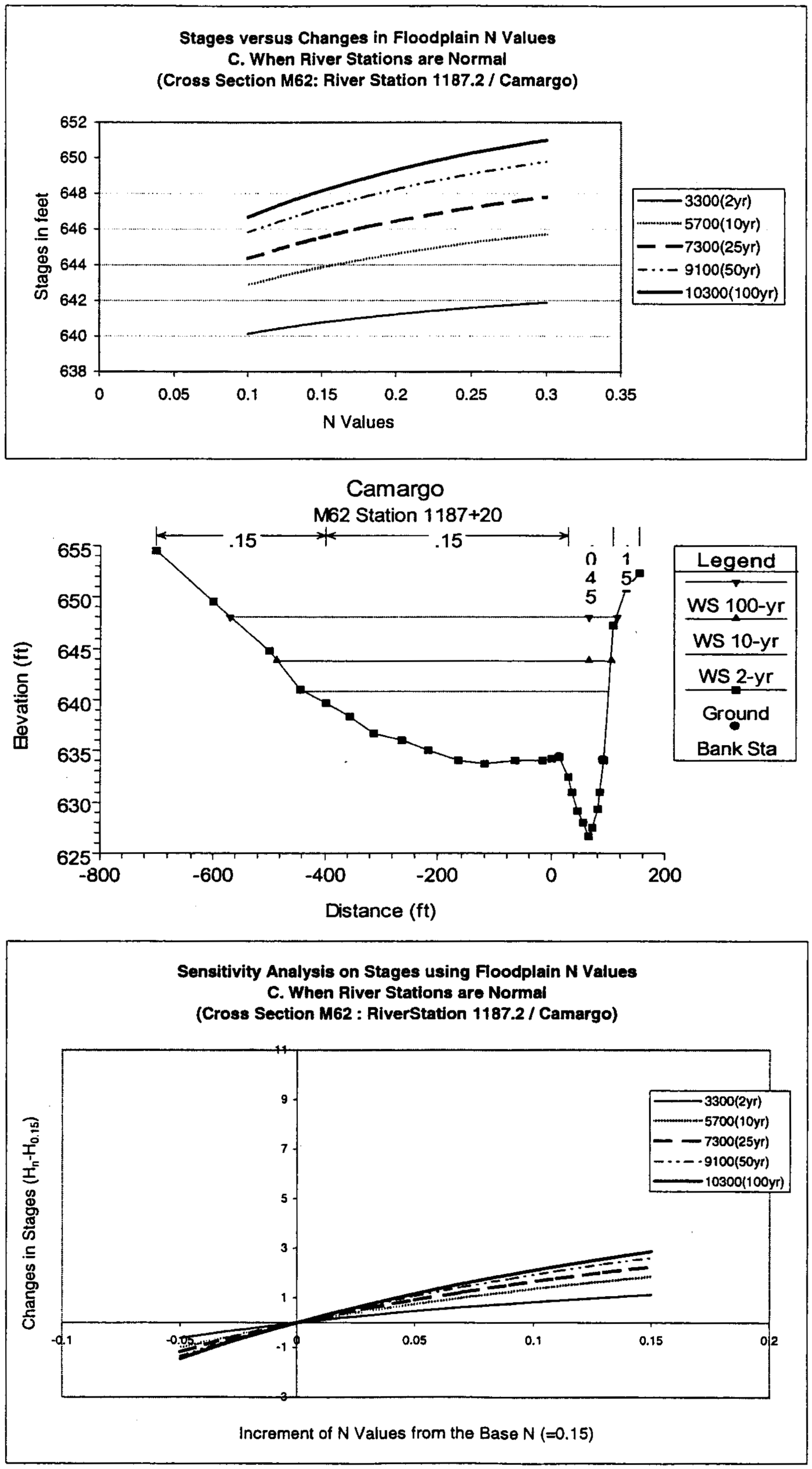

Figure 10. Sensitivity analysis for section 1187.2 in the lower reach of the Upper Embarras River

HPG and Capacity Curves

HPG

The Hydraulic Performance Graphics (HPG) and capacity curves were developed by Professor Ben C. Yen of the University of Illinois at Urbana-Champaign. The methods have been applied successfully to study the channel capacity of the Boneyard Creek in Champaign-Urbana (Yen and Gonzalez, 1994; Gonzalez et al., 1997). The present study applied capacity curves to demonstrate the effect of tree-thinning activities. Since the HPG is the basis for developing capacity curves, a brief description about the HPG follows.

The HPG was developed for a river segment that has cross sections with similar hydraulic properties. Figure 11 is an HPG developed for one reach of the lower Upper Embarras River. Each curve is an equal discharge line where every point represents the solution of a pair of up- and downstream depths. Therefore, each point of the curve contains three pieces of basic data sought by river engineers. A complete HPG is obtained by gathering representative discharge curves together. An immediate advantage of the HPG is that it provides complete information about the stage-discharge information for the river reach to users, making it useful for decision-making.

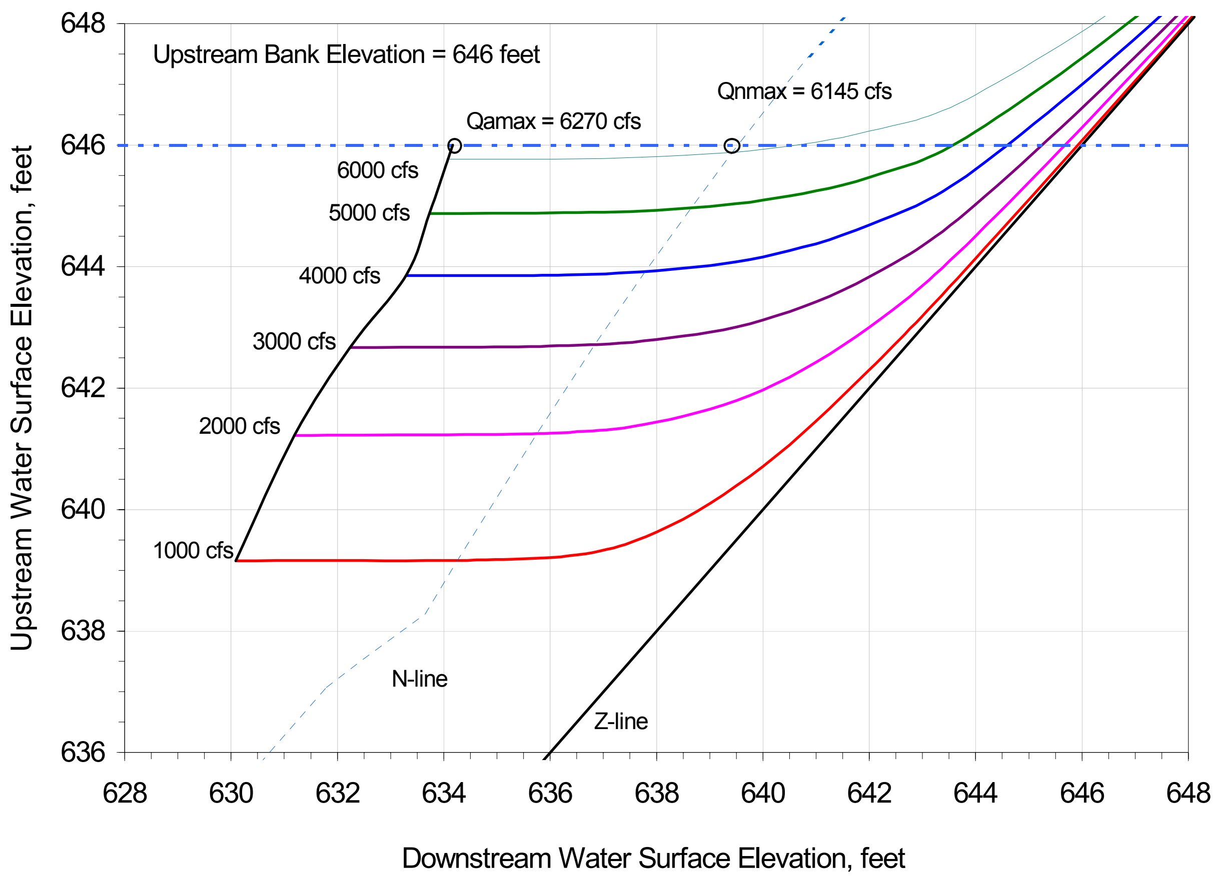

Figure 11. The HPG for reach 2.72 miles downstream of Villa Grove (existing condition: Floodplain *n* = 0.08)

In practice, the depth is converted to elevations in Figure 11. The usage of HPG is further expanded by adding three auxiliary lines and bank elevations. On a discharge curve, three points are defined. Point C corresponds to a situation when the downstream depth equals the critical depth. The maximum discharge a channel can carry for a specific downstream depth is when the downstream depth becomes the critical depth. Any higher upstream depth will require a higher downstream depth for the same discharge. This will lead to the situation where the depths at up- and downstream sections become equal (normal flow), as represented by point N. Such a discharge curve, expressed as a function of up- and downstream depths, will never intersect a depth combination corresponding to the horizontal water surface; at that point, the discharge becomes zero, as denoted by point Z. Therefore, for a given discharge, the possible up- and downstream depths are bounded by point C to the left, any feasible depth combinations between points C and N represent the water surface under the M2 backwater curve (e.g., Chow, 1959), and any feasible combination of depths between points N and Z represent the water surface under the M1 backwater curve. Adding the bank elevations at the entrance and exit of the study segment provides another use of HPG.

Capacity of a Channel Segment

Because flooding starts when flood stage exceeds the bank elevation, this condition is used as a criterion of the upper limit to the allowable capacity of this river segment in the following analysis with tree management practices. In Figure 11, the downstream bank elevation is 648 feet-msl, and the upstream bank elevation is tentatively set to 646 feet-msl. The HPG shows that the *capacity of a channel segment* (the maximum steady discharge the channel can carry without spilling water overbank anywhere within the segment) is 6270 cubic feet per second or cfs (the absolute maximum capacity because of the intersection of the C-curve and the upstream bank elevation), or 6145 cfs (the maximum uniform flow capacity, which is at the intersection of the N-curve and the upstream bank elevation). The downstream elevation becomes less significant in this case because of high valley sections. If this reach is the most flood-prone reach of the river system, the carrying capacity of the channel reach is adequately presented in the figure.

Carrying Capacity of a River Reach and Applications to the Embarras River

The capacity for the overall river reach is different than the capacity of a river segment because each river segment has its own limiting bank elevation and capacity. Therefore, depending on the criterion used, each discharge and downstream elevation will form a pair with a solution for that criterion. Because a river reach has multiple segments, a suitable solution from each segment for this criterion forms the capacity curve for the whole channel reach. Yen and Gonzalez (1994) defined the carrying capacity as the maximum discharge that the channel can deliver without spilling overbank. However, some segments of a river or some rivers do not have clearly defined bank elevations, such as the Village of Villa Grove in this study. Certain artificially set criteria have to be used. This superimposed criterion defines the maximum carrying capacity of a channel segment differently than those from the natural channel banks.

Figure 12 is a capacity curve determined for the lower reach of the Upper Embarras River using the existing *n* value for the floodplain. The limiting criterion is set to the flood stage in

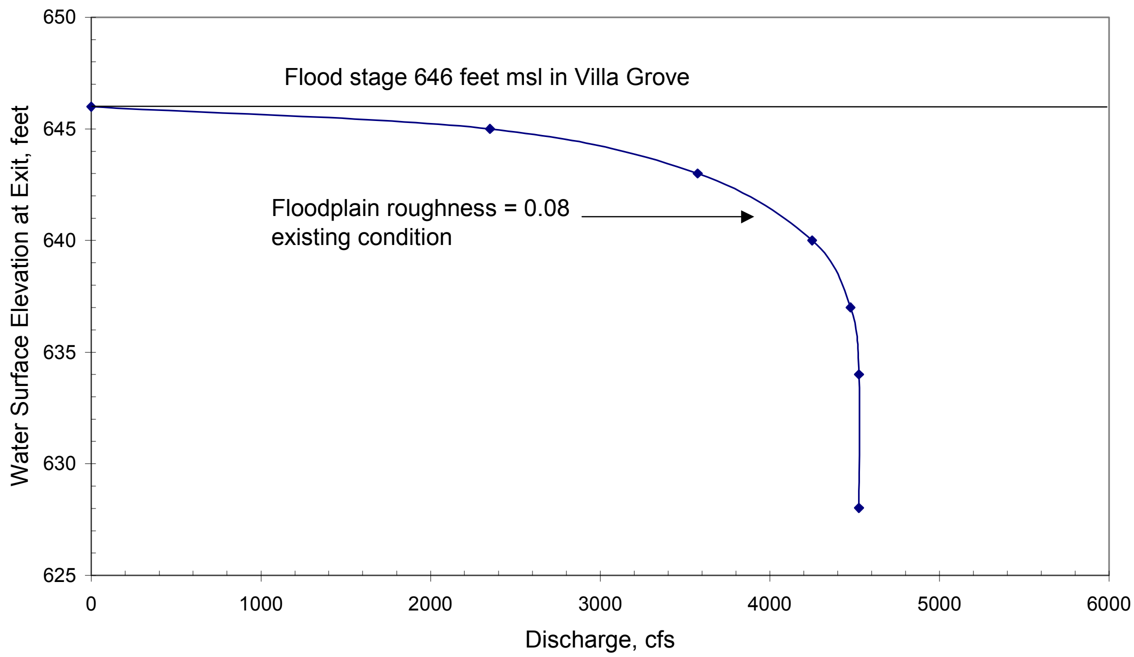

Figure 12. Capacity of Upper Embarras River for the existing condition

Villa Grove because it is the primary concern in the study. For presentation purposes, the elevation of 646 feet-msl is retained. For any point on the curve, the possible pair of discharge and downstream elevation (at Camargo) that would not cause the flood elevation to exceed 646 feet in Villa Grove can be determined.

Analysis of Tree Maintenance Using the Capacity Curves

The effects of riparian forest management on channel carrying capacity were investigated using a hypothetical example in which the mature trees in the lower reach of the Upper Embarras River were thinned to a density of a 40-foot single span in each direction uniformly on the 2-year floodplain. The following three scenarios were tested:

1) Thinning the whole reach or only half and determining which half in order to achieve a level of acceptable effect.
2) The differences in model results between using a constant representative *n* value or the computed *n* values for tree maintenance activities.
3) Comparing the differences in selecting different flood elevations in Villa Grove.

Case A. Reach Length to Perform Tree Thinning

The case was investigated using the limiting elevation in Villa Grove to be 644 feet-msl, i.e., how much more discharge can pass through the study reach without overtopping 644 feet-msl in Villa Grove. Figure 13 shows the existing conditions and all three thinning options. The capacity curves clearly showed the improvements achieved with thinning trees, but thinning trees

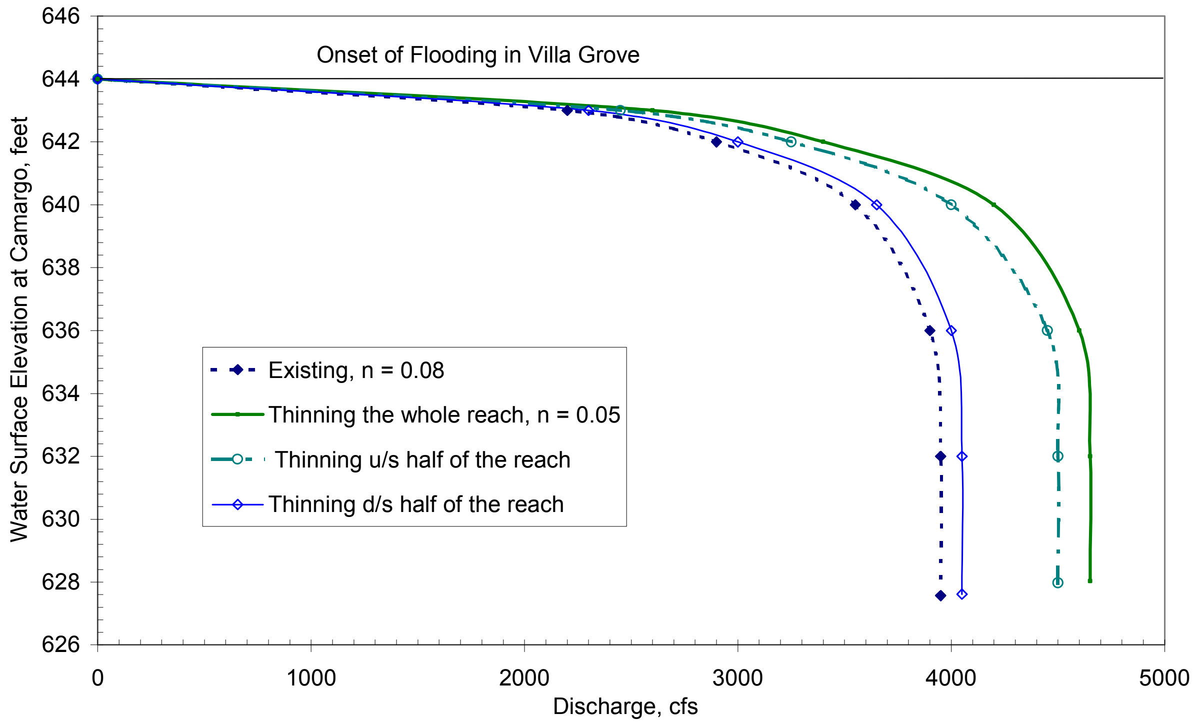

Figure 13. Comparison of three tree-thinning options in the study reach

for the upper half of the reach can attain effects similar to thinning the whole reach. On the other hand, thinning the lower half of the reach did not improve the existing conditions much.

Case B. The Effects of Using a Representative n Value versus the Computed n Values

Figure 14 presents the differences between the two approaches. The Embarras River model runs using the computed *n* values showed higher capacity than those using a representative constant value. The increases may not be significant in this case, but the time saved in implementing the computation is tremendous. Also because of the difficulty in recalculating Manning's *n* values for each test stage, only one Manning's *n* corresponding to the 100-year flood was used. Considering these factors, it is recommended to use the computed *n* values for all simulations. A BASIC computer program was developed for this purpose (see Appendix).

Case C. The Capacity Curves According to Stage Criteria in Villa Grove

Using the capacity curves with a specific stage criterion in the area of interest allows the managers to assess if the designed practice can meet the requirements and evaluate the effectiveness of the practice. This use of the capacity curves is presented using three flooding levels in Villa Grove selected arbitrarily for comparison purposes. As shown in Table 5, a flood level of 644 feet-msl corresponds to minimum damage to local residents; a flood level of 646 feet-msl corresponds to medium damage at a 10-year flood frequency; and a flood level of 651 feet-msl corresponds to high flood damage and an elevation slightly higher than the 100-year flood. Figures 15, 16, and 17, respectively, show the capacity curves developed according to these three criteria.

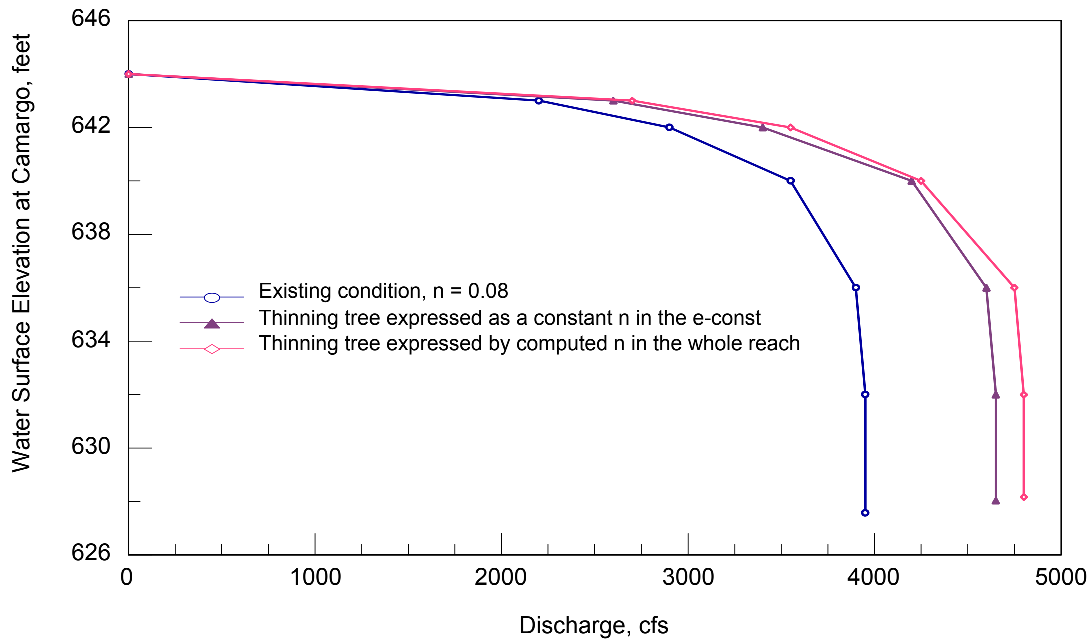

Figure 14. Effects of using either a constant *n* or the computed *n* to represent tree-thinning activities

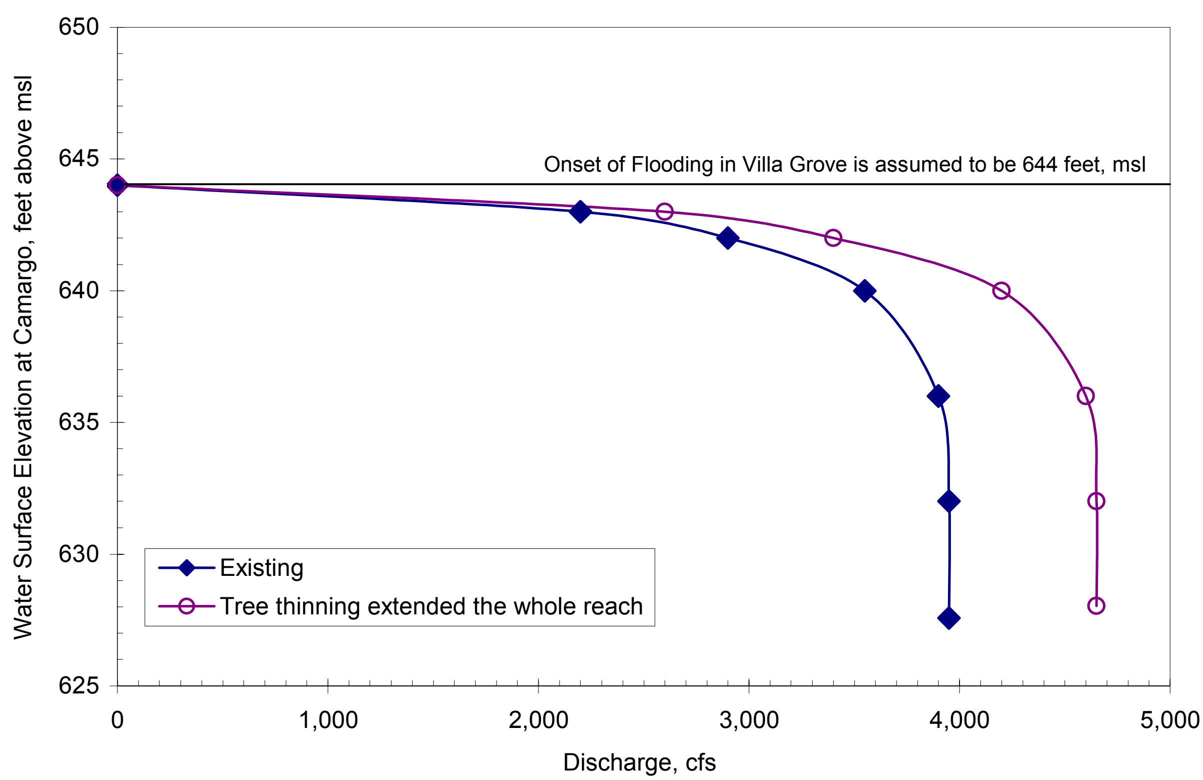

Figure 15. Capacity curves for the existing and tree-thinning conditions; the onset of flooding stage in Villa Grove is set at 644 feet

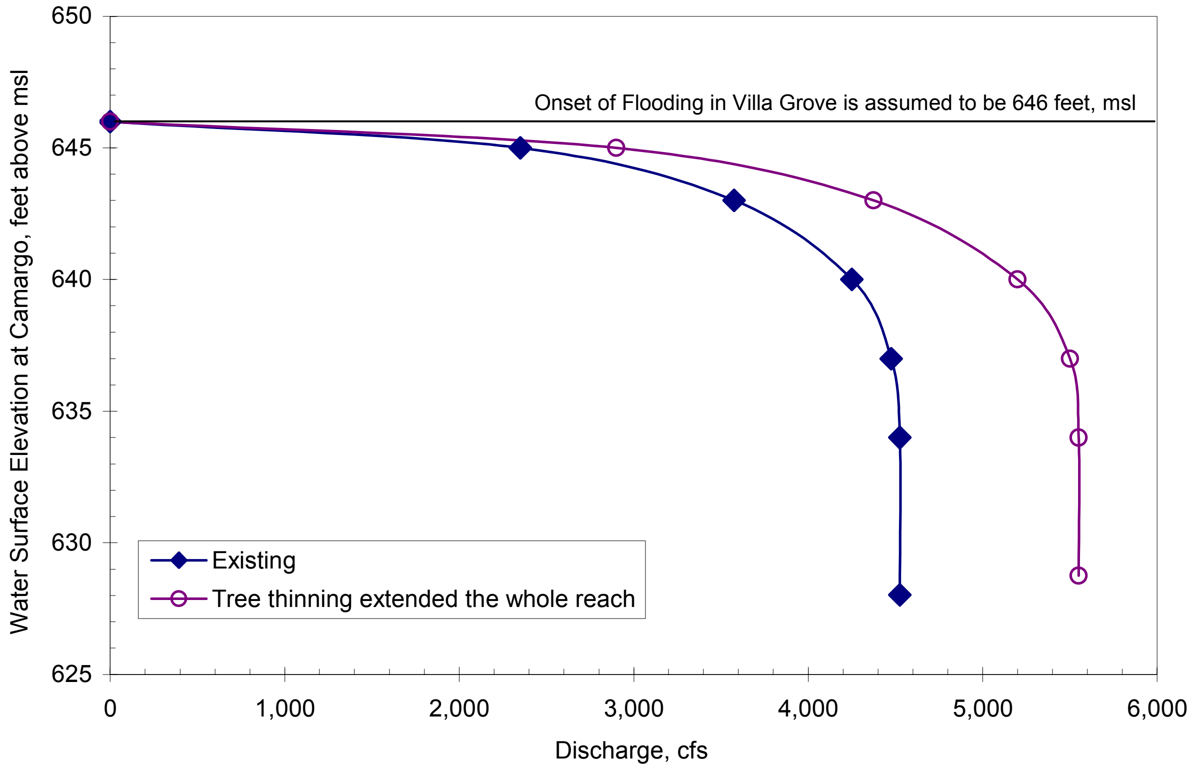

Figure 16. Capacity curves for the existing and tree-thinning conditions; the onset of flooding stage in Villa Grove is set at 646 feet

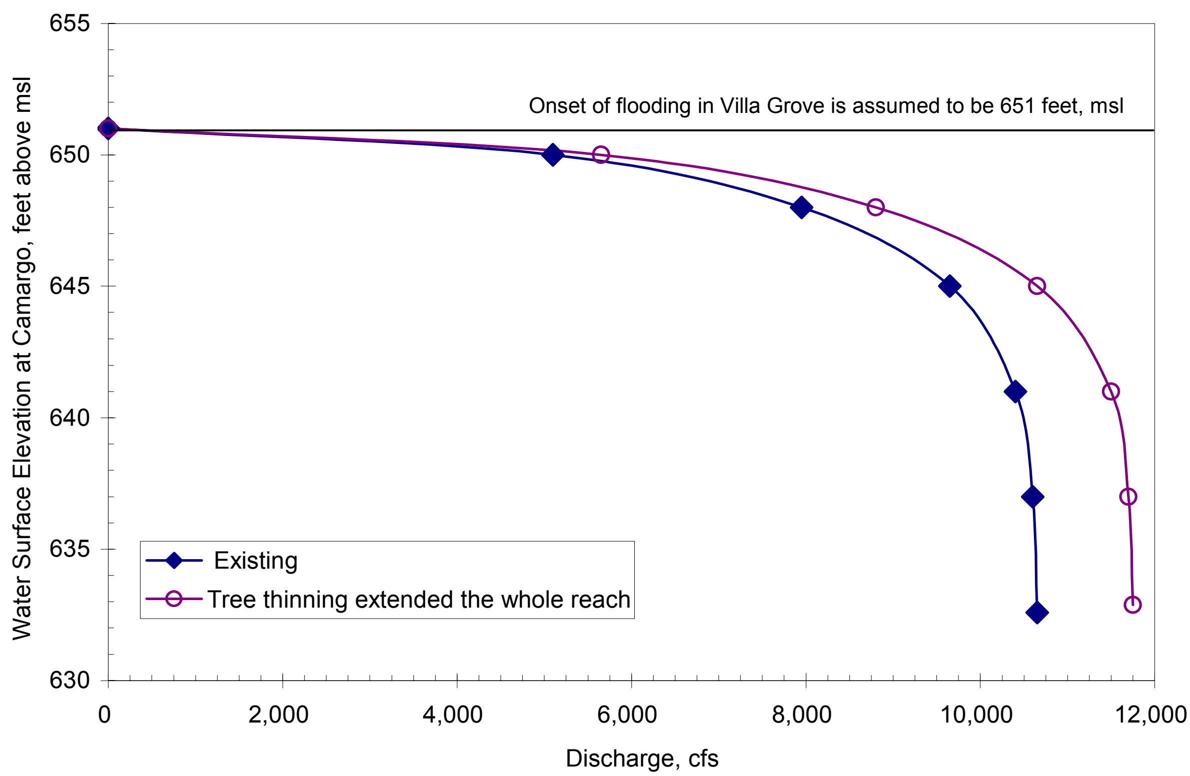

Figure 17. Capacity curves for the existing and tree-thinning conditions; the onset of flooding stage in Villa Grove is set at 651 feet

With the limiting stage at 644 feet-msl, the channel can carry a maximum discharge of 3950 cfs when the downstream reach is at critical depth. Note that it is uncommon to have critical stages at the exit in natural channels, however; therefore the carrying capacity is more likely to be less in the upper portion of the curve. Thinning trees improves the carrying capacity of the reach overall with an increase at the maximum value around 700 cfs. If the limiting stage were raised to 646 feet-msl, the maximum carrying capacity of the channel increased to around 4500 cfs. Referring to Table 4, this is in the range of reported flooding in Villa Grove. Thinning trees can improve the carrying capacity around 1000 cfs at the maximum value. When the limiting stage is at the 651 feet-msl, the improvement in carrying capacity is around 1100 cfs, not much higher than the previous case. Therefore the effectiveness of the tree-thinning can be observed from these comparisons.

Summary

A method for computing Manning's roughness coefficient to represent riparian forest has been determined. The formula was applied to evaluate the effects of tree maintenance activities and the results were implemented in an HEC-RAS model to evaluate the effect on flooding in the Upper Embarras River. The riparian forest management practice examined here was the tree-thinning activities. With this selected method, the authors demonstrated that specific tree-thinning practices could improve the carrying capacity in the study reach.

Several products developed in this project may be useful to other users:

1. *A review on Manning's roughness coefficient*. Current status on vegetative roughness was discussed. Potential errors in using Manning's coefficient were also presented so that readers would be better informed when determining the roughness coefficient.
2. *A collection of existing formulas for computing the vegetative roughness represented in terms of Manning's roughness coefficient*. Management practices could be translated by way of parameter values in these formulas for calculating the *n* values, although all the formulas involve different degrees of approximation. For wooded vegetation, the approach by Petryk and Bosmajian (1975) was selected for mature trees.
3. *Techniques in implementing the formula to the HEC-RAS model*. Results were compared, and the appendix of this report contains a computer code for those wish to use it.
4. *Field data.* Limited field data have been collected and presented.
5. *The application of capacity curves*. The capacity curves appear to be useful for evaluating and comparing different management scenarios. In addition to presenting basic information in a comprehensive way, this project demonstrated that using capacity curves with a specific limiting criterion can let managers conduct objective-oriented design and evaluate whether the management practices can meet the requirements directly.

The present scope of study included theoretical analysis of the subject. Field verifications are needed for either the roughness formulas or for the river model. With heterogeneous field conditions and current knowledge about vegetative roughness, the computations involve many approximations, and results discussed in the chapters should be viewed for the relative effectiveness between different scenarios, *not* for their absolute values.

For demonstration purposes this project examined the lower reach of the Upper Embarras River. The effects of tree-planting on the reaches upstream of Villa Grove have been evaluated for their *n* values but are not implemented to the HEC-RAS model because of its limitations. Storage effects have to be evaluated by re-running the hydrologic model and by using a unsteady flow model.

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Appendix
The Computer Program

```
Dim Stations(39) As Double
Dim HydRadii(3, 39) As Double
Dim ManTable(3, 39) As Double
Dim Vels(3, 39) As Double
Dim Depths(3, 39) As Double
Dim Shears(3, 39) As Double
Dim Slopes(2, 39) As Double

Private Function IsStation(Value As Double) As Boolean
'This function returns a false or true value based on whether or not the station identification is presented'
    TempBool = False
     For i = 0 To 38
       If Stations(i) = Value Then
          TempBool = True
          i = 38 + 1
       End If
    Next i
    IsStation = TempBool
End Function
Private Function SplitClean(liner As String) As String
'The split function returns quite a few empty elements. To fix this problem and leave only values, then use the filter function with a False argument to clean the array'
    Tempo = Split(liner)
      ReDim nTempo(UBound(Tempo))
       k = 0
         For i = 0 To UBound(Tempo)
        If Len(Tempo(i)) <> 0 Then
       nTempo(k) = Tempo(i)
      k = k + 1
     End If
    Next i

'The array has a long tail of blanks and need to be removed. To do so, use the ReDim statement with the Preserve option.'
    ReDim Preserve nTempo(k - 1)
     FinalResult = Join(nTempo, " ")
    SplitClean = FinalResult
End Function
Private Function Manning(radius As Double, depth As Double, vel As Double, shear As Double, slope As Double) As Double
'This is a wrapper that calls the functions wanted for Manning's computation'
    Manning = Petryk(radius)
    'Manning = PetrykCd(radius, vel)
    'Manning = Freeuns(radius, depth, vel)
    'Manning = Freesub(radius, depth, vel)
```

```
    'Manning = Freesubshear(radius, depth, shear, slope)
    'Manning = Freeunshear(radius, depth, shear, slope)
End Function
Private Function Petryk(radius As Double) As Double
'This function computes the value of the Manning's n given the value of the hydraulic radius, using
    'Petryk's formula (English unit)'
    nb = 0.045
    Cd = 1.13
    g = 32.2
    spacing = 30
    treedia = 1.5
    vegden = treedia / (spacing ^ 2)
    n = nb * Sqr(1 + (Cd * vegden / (2 * g)) * (1.49 / nb) ^ 2 * radius ^ (4 / 3))
    Petryk = n
End Function
Private Function PetrykCd(radius As Double, vel As Double) As Double
'This function computes the value of the Manning's n given the value of the hydraulic radius, using
Petryk's formula (English unit)'
'This function also calculates the Cd value based on the V and R.'
    nb = 0.045
    Cd = 2.1 * (vel * radius) ^ (-1.1)
    If (Cd >= 12) Then Cd = 12#
    g = 32.2
    spacing = 40
    treedia = 1.5
    vegden = treedia / (spacing ^ 2)
    n = nb * Sqr(1 + (Cd * vegden / (2 * g)) * (1.49 / nb) ^ 2 * radius ^ (4 / 3))
    Petryk = n
End Function
Private Function Freeuns(radius As Double, depth As Double, vel As Double) As Double
'This function computes the value of the Manning's n given the value of the hydraulic radius, using Freeman's unsubmerged formula (English unit)'

    PI = 3.1415926535
    e = 5000000#
    h = 5#
    rho = 62.428
    nu = 0.0000105
    stemdia = 3#
    spacing = 5#
    ars = (PI * (stemdia ^ 2)) / 4#
    A = stemdia * depth
    density = 1# / (spacing ^ 2)
```

```
    n1 = (e * ars / (A * rho * (vel ^ 2))) ^ 0.242
    n2 = (spacing * A) ^ 0.0623
    n3 = (nu / (vel * radius)) ^ 0.662
    n = 0.0000022 * 1.486 * n1 * n2 * n3
    Freeuns = n
End Function
Private Function Freesub(radius As Double, depth As Double, vel As Double) As Double
'This function computes the value of the Manning's n given the value of the hydraulic radius,
using Freeman's Unsubmerged formula (English unit)

    PI = 3.1415926535
    e = 5000#
    h = 5#
    rho = 62.428
    nu = 0.0000105
    stemdia = 0.2
    spacing = 2.5
    ars = (PI * (stemdia ^ 2)) / 4#
    A = stemdia * depth
    density = 1# / (spacing ^ 2)

    n1 = (e * ars / (A * rho * (vel ^ 2))) ^ 0.141
    n2 = (h / depth) ^ 0.175
    n3 = (density * A) ^ 0.191
    n4 = (nu / (vel * radius)) ^ 0.0155
    n = 0.039 * n1 * n2 * n3 * n4
    Freesub = n
End Function
Private Function Freesubshear(radius As Double, depth As Double, shear As Double, slope As
Double) As Double
'This function computes the value of the Manning's n given the value of the hydraulic radius,
using Freeman's Submerged formula (English unit), in its
    'version using shear velocity

    PI = 3.1415926535
    e = 50#
    h = 5#
    rho = 62.428
    nu = 0.0000105
    stemdia = 0.2
    spacing = 2.5
    ars = (PI * (stemdia ^ 2)) / 4#
    A = stemdia * depth
    density = 1# / (spacing ^ 2)
    svel = Sqr(shear / rho)
```

```
   n1 = (e * ars / (A * rho * (svel ^ 2))) ^ 0.183
   n2 = (h / depth) ^ 0.243
   n3 = (density * A) ^ 0.273
   n4 = (nu / (svel * radius)) ^ 0.115
   n5 = (1# / svel) * (radius ^ (2# / 3#)) * Sqr(slope)
   n = 1.486 * 0.183 * n1 * n2 * n3 * n4 * n5
   Freesubshear = n
End Function
Private Function Freeunshear(radius As Double, depth As Double, shear As Double, slope As
Double) As Double
'This function computes the value of the Manning's n given the value of the hydraulic radius,
using Freeman's Unsubmerged formula (English unit), in its version using shear velocity'

   PI = 3.1415926535
   e = 5000000#
   h = 5#
   rho = 62.428
   nu = 0.0000105
   stemdia = 3#
   spacing = 5#
   ars = (PI * (stemdia ^ 2)) / 4#
   A = stemdia * depth
   density = 1# / (spacing ^ 2)
   svel = Sqr(shear / rho)

   n1 = (e * ars / (A * rho * (svel ^ 2))) ^ 0.207
   n2 = (density * A) ^ 0.0547
   n3 = ((svel * radius) / nu) ^ 0.49
   n4 = (1# / svel) * (radius ^ (2# / 3#)) * Sqr(slope)
   n = 1.486 * 0.00009159 * n1 * n2 * n3 * n4
   Freeunshear = n
End Function
Private Sub DelayIt(HowMuch As Long)
'This is a simple routine to delay (some sort of a timer) the execution of the application'

   For j = 0 To HowMuch
   Next j
End Sub
Private Sub RunRAS()
'This subroutine starts the HEC-RAS visual interface, waits for the program to finish (that's the
for loop) and then closes the MSDOS windows it created so the focus is back on the main HEC-
RAS visual window. It is advisable to adjust the initial timer to wait for HEC-RAS to run the
first time, or simply open HEC-RAS manually then close it so that the system caches it, so it
starts faster later on'

   RAS = Shell("c:\Hec\Ras\ras.exe", vbNormalFocus)
```

```
    'RAS = Shell("d:\programs\ras\ras.exe", vbNormalFocus)
    Call DelayIt(100000000)
    SendKeys "%ss{ENTER}", True
    Call DelayIt(200000000)
    'Close two windows to come back to RAS
    SendKeys "%{ }C", True
    Call DelayIt(100000000)
    SendKeys "%{F4}", True
End Sub
Private Sub WriteValues(OutFile As String)
'This subroutine selects the desired table output then writes it to a text file for further processing
and exits HEC-RAS for another iteration.'

    'Select table output format
    SendKeys "%vp", True
    'Select the user table and pick first choice
    SendKeys "%u{ENTER}", True
    'Select dump to text file
    SendKeys "%f{DOWN 2}{ENTER}", True
    'Save the filename c:\miguel\david\embarras\karl\<OutFile>
    SendKeys "d:\projects\C-2000\headwaters\embarras\matt\visualB\" & OutFile & "{ENTER}",
True
    'Close the table output interface
    SendKeys "%fx", True
    'Get out of HEC-RAS
    SendKeys "%fx", True
End Sub
Private Sub ChangeManning(InFile As String, OutFile As String)
'This subroutine reads a template geometry file for HEC-RAS and looks for the occurrences of
the word HERE (it is case sensitive). The program replaces the word by the appropriate value of
Manning's n for that cross section. The template file is specified in the InFile part, and the output
file name is specified in the OutFile part.'

    Open "d:\projects\C-2000\headwaters\embarras\matt\visualB\" & InFile For Input As #4
    Open "d:\projects\C-2000\headwaters\embarras\matt\visualB\" & OutFile For Output As #5
    Line Input #4, liner
    i_left = 0
    i_right = 0
    While Not EOF(4)
        Line Input #4, liner
        If InStr(liner, "HERA") <> 0 Then
            TempStr = Replace(liner, "HERA", Format(ManTable(1, i_left), "#.000"), , ,
vbTextCompare)
            liner = TempStr
            'For river mile=1351.5, we only have left bank manning's change
            If ManTable(0, i_left) = 1351.5 Then
```

```
            i_right = i_right + 1
          End If
          i_left = i_left + 1
        End If
        If InStr(liner, "HERB") <> 0 Then
          TempStr = Replace(liner, "HERB", Format(ManTable(2, i_right), "#.000"), , ,
vbTextCompare)
          liner = TempStr
          'For river mile=1073.6, we only have right bank manning's change
          If ManTable(0, i_right) = 1073.6 Then
            i_left = i_left + 1
          End If
          i_right = i_right + 1
        End If
        Print #5, liner
    Wend
    Close #4
    Close #5
End Sub
Private Sub ComputeManning()
'This subroutine takes care of reading the Manning's table and computing the Manning's n values
using the formula specified in the Manning function for all the stations'

    For i = 0 To 38
        ManTable(0, i) = HydRadii(0, i)
        ManTable(1, i) = Manning(HydRadii(1, i), Depths(1, i), Vels(1, i), Shears(1, i), Slopes(1,
i))
        ManTable(2, i) = Manning(HydRadii(2, i), Depths(2, i), Vels(2, i), Shears(2, i), Slopes(1,
i))
    Next i
End Sub
Private Sub ExtractManning(InFile As String, OutFile As String)
'This is a simple utility to scan the HEC-RAS geometry file specified in InFile, pick the cross
sections descriptions, and then print out the descriptions with the manning's n values together
with the location of change and a third column that is present in the HEC-RAS file (which I don't
know exactly what it is for at this moment. The results are printed into the OutFile name.'

    Dim liner As String
    Open "d:\projects\C-2000\headwaters\embarras\matt\visualB\" & InFile For Input As #2
    Open "d:\projects\C-2000\headwaters\embarras\matt\visualB\" & OutFile For Output As #3
    Line Input #2, liner
    While Not EOF(2)
        Line Input #2, liner

        'Write out the description of the station
        If InStr(liner, "BEGIN DESCRIPTION") <> 0 Then
```

```
            Print #3, " "
            Line Input #2, liner
            While InStr(liner, "END DESCRIPTION") = 0
               Print #3, liner
               Line Input #2, liner
            Wend
         End If

         'Write out the Manning's values.
         If InStr(liner, "Mann") <> 0 Then
            ValMan = Split(liner)
            Print #3, "Number of Mannings: " & ValMan(1)
            Line Input #2, liner
            While InStr(liner, "Bank Sta") = 0 And InStr(liner, "#") = 0
               Manns = Split(SplitClean(liner))
               'Print nicely the results into the text file
               For i = 0 To UBound(Manns) Step 3
                  Print #3, Format(Val(Manns(i)), "@@@@@ ") & Format(Val(Manns(i + 1)),
"0.000 ") & Format(Val(Manns(i + 2)), "@@")
               Next i
               Line Input #2, liner
            Wend
         End If
      Wend
      Close #2
      Close #3
End Sub
Private Sub LoadRadius(InFile As String)
'This subroutine loads the hydraulic radii from the output table defined in HEC-RAS by the user.
It assumes the data for the lower embarras part of the river only, so if another part of the project
is to be analyzed, changes should be made.'

      Dim liner As String
      Dim miles As Double
      Open "d:\projects\C-2000\headwaters\embarras\matt\visualB\" & InFile For Input As #6
      Line Input #6, liner
      While Not EOF(6) And InStr(liner, "Hydr Radius") = 0
         Line Input #6, liner
      Wend
      k = 0
      While Not EOF(6)
         Line Input #6, liner
         If InStr(liner, "Bridge") = 0 And InStr(liner, "Lower Embarras") <> 0 Then
           'If there are changes in the ouput table defined
           'in HEC-RAS, for example: output the value of
           'velocities on the left and right banks, then
```

```
        'the following lines should be changed to
        'make sure that the new information is incorporated
        'correct

        miles = Val(Mid(liner, 18, 9))
        If IsStation(miles) = True Then
          HydRadii(0, k) = miles
          HydRadii(1, k) = Val(Mid(liner, 38, 10))
          HydRadii(2, k) = Val(Mid(liner, 76, 10))
          Vels(0, k) = HydRadii(0, k)
          Vels(1, k) = Val(Mid(liner, 64, 8))
          Vels(2, k) = Val(Mid(liner, 102, 8))
          Depths(0, k) = HydRadii(0, k)
          Depths(1, k) = Val(Mid(liner, 52, 9))
          Depths(2, k) = Val(Mid(liner, 90, 9))
          Shears(0, k) = HydRadii(0, k)
          Shears(1, k) = Val(Mid(liner, 112, 10))
          Shears(2, k) = Val(Mid(liner, 123, 10))
          Slopes(0, k) = HydRadii(0, k)
          Slopes(1, k) = Val(Mid(liner, 135, 10))
          k = k + 1
        End If
      End If
    Wend
    Close #6

'Clean up possible non-sense zeros in the vels and depths tables. I think they exist because of
interpolated cross sections and so it was a way to save space. They are senseless in my
computations and so it is just a matter of copying the previous values (from downstream to
upstream. For i = 37 To 0 Step –1’
      If Vels(1, i) = 0 Or Vels(2, i) = 0 Then
        Vels(1, i) = Vels(1, i + 1)
        Vels(2, i) = Vels(2, i + 1)
      End If
      If Depths(1, i) = 0 Or Depths(2, i) = 0 Then
        Depths(1, i) = Depths(1, i + 1)
        Depths(2, i) = Depths(2, i + 1)
      End If
      If Shears(1, i) = 0 Or Shears(2, i) = 0 Then
        Shears(1, i) = Shears(1, i + 1)
        Shears(2, i) = Shears(2, i + 1)
      End If
    Next i
End Sub
Private Sub TableOut(Filenam As String, table() As Double)
```

```
'This subroutine is used to print to a text file the values of a table of values that will change 'from iteration to iteration. Only about 3 iterations are needed to attain convergence. Also useful for debugging 'purposes'

    Open "d:\projects\C-2000\headwaters\embarras\matt\visualB\" & Filenam For Output As #10
    For i = 0 To 38
      Print #10, Format(table(0, i), "   0.00") & " " & Format(table(1, i), " 0.0000") & " " &
Format(table(2, i), " 0.0000")
    Next i
    Close #10
End Sub
Private Sub cmdExecute_Click()
'The main routine to execute when the "Proceed" button is clicked'

    Stations(0) = 1352.2
    Stations(1) = 1351.5
    Stations(2) = 1350.05
    Stations(3) = 1349.45
    Stations(4) = 1334.3
    Stations(5) = 1318.4
    Stations(6) = 1308.9
    Stations(7) = 1289.1
    Stations(8) = 1249.3
    Stations(9) = 1238#
    Stations(10) = 1235#
    Stations(11) = 1234.85
    Stations(12) = 1234.5
    Stations(13) = 1234.2
    Stations(14) = 1229.1
    Stations(15) = 1218.1
    Stations(16) = 1211.1
    Stations(17) = 1206.7
    Stations(18) = 1206.35
    Stations(19) = 1205.95
    Stations(20) = 1205.6
    Stations(21) = 1199.6
    Stations(22) = 1187.2
    Stations(23) = 1163.2
    Stations(24) = 1130.7
    Stations(25) = 1124.6
    Stations(26) = 1117.9
    Stations(27) = 1091.15
    Stations(28) = 1076.2
    Stations(29) = 1075.1
    Stations(30) = 1074.1
    Stations(31) = 1073.6
```

```
Stations(32) = 1072#
Stations(33) = 1071#
Stations(34) = 1069.6
Stations(35) = 1054.8
Stations(36) = 1022.65
Stations(37) = 1002.8
Stations(38) = 997.3

Call LoadRadius("iter0.txt")
Call ComputeManning
Call TableOut("Mann0.txt", ManTable)
Call ChangeManning("template.gxs", "tree.g03")

Call RunRAS
Call WriteValues("iter1")
Call DelayIt(200000000)

Call LoadRadius("iter1.txt")
Call ComputeManning
Call TableOut("Mann1.txt", ManTable)
Call ChangeManning("template.gxs", "tree.g03")

Call RunRAS
Call WriteValues("iter2")
Call DelayIt(200000000)

Call LoadRadius("iter2.txt")
Call ComputeManning
Call TableOut("Mann2.txt", ManTable)
Call ChangeManning("template.gxs", "tree.g03")

Call RunRAS
Call WriteValues("iter3")
Call DelayIt(200000000)

Call LoadRadius("iter3.txt")
Call ComputeManning
Call TableOut("Mann3.txt", ManTable)
Call ChangeManning("template.gxs", "tree.g03")

Call RunRAS
Call WriteValues("iter4")
Call DelayIt(200000000)

Call LoadRadius("iter4.txt")
Call ComputeManning
```

```
    Call TableOut("Mann4.txt", ManTable)
    'Call TableOut("shear4.txt", Shears)
    Call ChangeManning("template.gxs", "tree.g03")

    Call RunRAS
    Call WriteValues("iter5")
    Call DelayIt(200000000)

    Call LoadRadius("iter5.txt")
    Call ComputeManning
    Call TableOut("Mann5.txt", ManTable)
    Call ChangeManning("template.gxs", "tree.g03")

    MsgBox "Finished!"
End Sub
```

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