method, even when we recognize that if we knew the true effect of urbanization, the model selected might not be the most accurate. What criteria should be used in selecting one design method? First, the conceptual framework of the model should be rational. Second, the model should be flexible so that it can be used for a variety of design problems. Third, the model should be applicable to large regions, not just sites within a single county. Fourth, the input data requirements should be minimal and easily obtainable. Fifth, the method should be highly reproducible; that is, different hydrologists should get the same design at a given location. Sixth, a model should be simple to apply.

In summary, the SCS methods appear to satisfy the six criteria for model selection (i.e., conceptually rational, flexible in design, widely applicable, requiring minimal input, highly reproducible, and computationally simple). Studies have also shown that the methods are reasonably accurate and relatively unbiased when they are applied under the conditions for which they were developed.

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Simple Methods to Evaluate Relative Effects of Longitudinal Encroachments

LEON A. TRAILLE AND DONALD L. CHERY, JR.

To aid highway planners and others who must site structures and fills in natural floodplains, simplified graphical solutions were developed that provide short-cut methods for easy assessment of encroachment impacts. Changes in stage (water-surface elevation) and hydrograph peak discharge due to encroachments were determined. The discussion is limited to encroachments that parallel the channel.

Construction in floodplains of highway fills and other types of built-up areas with alignments generally parallel to the main channel of a river or stream constitutes longitudinal or lateral encroachment. Such encroachments usually reduce storage and conveyance available for passing flood flows and generally alter the characteristics of flooding at the affected site.

The impact of encroachments can be determined by using existing techniques that include an assortment of computer models and other methods. These techniques are complex, however, require costly and time-consuming field data collection and preparation, and are therefore unsuitable at the preliminary design phase for assessing relative impact of encroachment alternatives on flooding. In this paper results are presented from a study that developed simple procedures to evaluate impacts of encroachment options on flood depths and peak-discharge rates. Sample problems are presented to illustrate the procedures developed.

RESEARCH APPROACH

To develop the simplified procedures, representative channel cross sections were selected and a controlled series of tests with existing mathematical models produced a set of predicted changes that were used to develop the graphic plots of relationships among groups of significant variables. The entire range of graphs developed and step-by-step procedures for using them are presented in a user's manual (1).

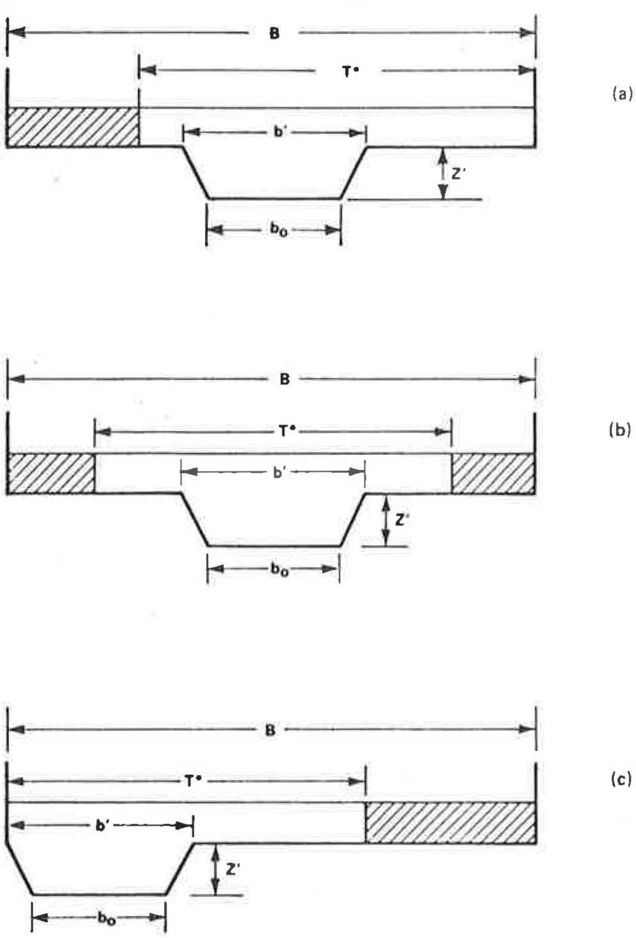
Two aspects of flooding modified by longitudinal encroachments that were addressed in this study are (a) changes in water-surface profile and (b) changes in hydrograph peak-discharge rate. Figure 1 shows schematically the entire range of encroachment conditions with respect to the symmetry of the main channel and overbanks. The simplified procedures can accommodate an asymmetrical channel and overbank cross section.

ASSUMPTIONS AND LIMITATIONS

To develop simple procedures with a minimum of input requirements and field data-collection tasks, the following assumptions were made:

1. A single representative cross section and encroachment width can be used to model subreaches.
2. Flow is subcritical before and after encroachment; there is a Froude number of 0.6 or less.
3. Base flow that exists before the arrival of a flood wave is small relative to the amplitude of the flood wave; i.e., the event is single-peaked.
4. Flow does not overtop levees or embankments along its length whenever the water-surface elevation increases; the result is complete blockage of the flow beyond the limits of the constriction (no storage or conveyance).
5. Flow is confined from transverse spreading beyond the outer fringe of the unencroached floodplain when a single floodplain is encroached on.
6. Natural reaches to be constricted are unaffected by downstream controls such as bridge sites, spillways, or stream junctions.
7. Effects of encroachment are independent of valley symmetry or encroachment symmetry.

Figure 1. Longitudinal-encroachment configurations.



CHANGES IN WATER-SURFACE PROFILE

For flood flows with a steady or near-steady discharge rate, the floodplains provide added conveyance for passing the flood. For such flow conditions, flood profiles are governed by the principle of energy and mass conservation and can be solved entirely by using Bernoulli's equation (2). The resulting loss of floodplain conveyance due to longitudinal encroachment causes increased flood stages within the constriction. The relative change in flood stage due to encroachment is a function of the distribution of the total conveyance between the main channel and the floodplains and of the degree of constriction caused by encroachment. This relationship is used to develop the simplified procedures for estimating flood-elevation changes due to a longitudinal encroachment by employing the step-backwater flood-routing program HEC-2 (3). The difference in water-surface elevation is expressed functionally by using the significant variables that govern the flood profile:

$$\Delta H = f(H', K, K_{mc}, \Delta K_0, K_0, F_r) \qquad (1)$$

where

- ΔH = increase in water-surface elevation due to encroachment,
- H' = flow depth above bank-full stage,
- K = total conveyance for design flow at cross section without encroachment,
- K_{mc} = main-channel conveyance,
- K_0 = total overbank conveyance,
- ΔK_0 = conveyance loss due to encroachment, and
- F_r = Froude number at the cross section.

F_r is computed by using the following relationship:

$$F_r = (\alpha Q^2 B / g A^3)^{1/2} \qquad (2)$$

where

$$\alpha = \sum_1^N (K_N^3/A_N^2)/(K^3/A^2) \qquad (3)$$

and where

- subscript N = Nth segment in cross section,
- g = acceleration due to gravity,
- B = unencroached top width of cross section, and
- A = total flow area.

Conveyance is computed as follows:

$$K = (1.49/n)(A^5/P^2)^{1/3} \qquad (4)$$

where n is Manning roughness coefficient and P is the wetted perimeter.

Making the variable grouping in Equation 1 dimensionless gives the following:

$$\Delta H/H' = f[(K_{mc}/K)(\Delta K_0/K_0) F_r] \qquad (5)$$

CHANGES IN HYDROGRAPH PEAK-DISCHARGE RATE

The high frictional resistance and obstruction to the relatively shallow flow depths in floodplains distort a transient (unsteady) flood wave as it travels through natural stream reaches. This additional storage serves to dampen the flood wave in the direction of flow, resulting in hydrograph attenuation (4). The construction of longitudinal fills within the floodplains results in a loss of overbank storage with consequent reduction in the attenuating potential provided by natural conditions, which causes amplification of the outflow hydrograph peak.

DAMBRK, the National Weather Service model of dynamic wave unsteady flow routing (5), was used to generate the data base for simple procedures to assess changes in hydrograph peak-discharge rate. The DAMBRK model is based on the one-dimensional flow continuity and momentum equations, commonly referred to as the Saint Venant equations for unsteady flows (6). A finite difference scheme is used to solve the equations (7, pp. 16-35). This model was chosen because of its capability to simulate stream reaches that have different roughness properties and reach lengths in the main channel and overbanks. Also, the overbanks and the main channel are treated independently in the solution scheme to account for the variation that usually occurs in natural stream reaches. The theoretical basis and capabilities of the model have been described by Fread (5).

The constricted outflow hydrograph peak Q_{m_c} was taken to be functionally related to the following significant physiographic and hydrologic variables:

$$Q_{m_c} = f(Q_{m_0}, Q_{m_n}, t_p, L, B, b', Q_{nB}, n_{c_p}, n_{mc}, T^*) \quad (6)$$

where

Q_{m_c} = outflow peak for the constricted reach,
Q_{m_0} = inflow peak,
Q_{m_n} = outflow peak for the unconstricted reach,
t_p = time to peak of the inflow hydrograph,
L = reach length of constriction measured along the channel,
B = top width of the natural floodplain,
b' = top width of main channel,
Q_{nB} = normal flow bank-full capacity of channel,
n_{c_p} = equivalent Manning roughness coefficient of floodplains (overbanks),
n_{mc} = Manning roughness coefficient of main channel, and
T^* = encroached top width of cross section.

The bank-full depth of the main channel (Z'), the bottom width (b_0), and the bed slope (S_0) were lumped together and incorporated in the variable Q_{nB}. Making the variables in Equation 6 dimensionless gives

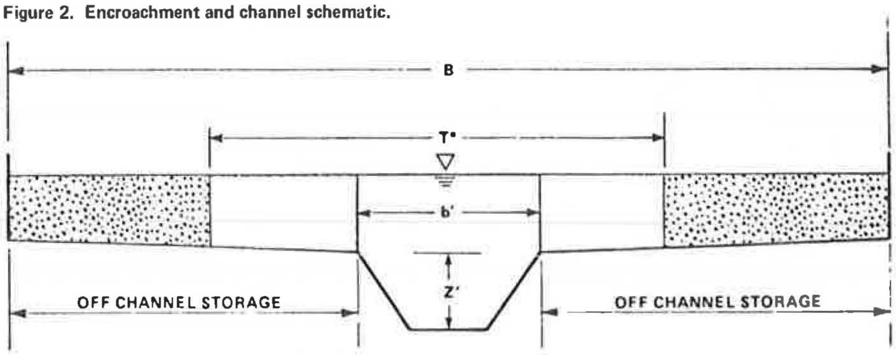
$$Q_{m_n}/Q_{m_c} = f[(L/B)(B/b')(T^*/B)(Q_{m_0}/Q_{nB})(n_{mc}/n_{cp})(Q_{m_0} t_p/B^3)] \quad (7)$$

Figure 2 shows schematically the channel variables used in the analysis. Two distinct situations, case 1 and case 2 in Figure 2, were considered in development of the simplified graphical solution scheme for Equation 7. Case 1 applies to conditions of negligible floodplain velocities, and therefore the overbanks are treated as off-channel storage. Case 2 applies to conditions for which overbank velocities are significant and therefore incorporates conveyance as well as storage effects.

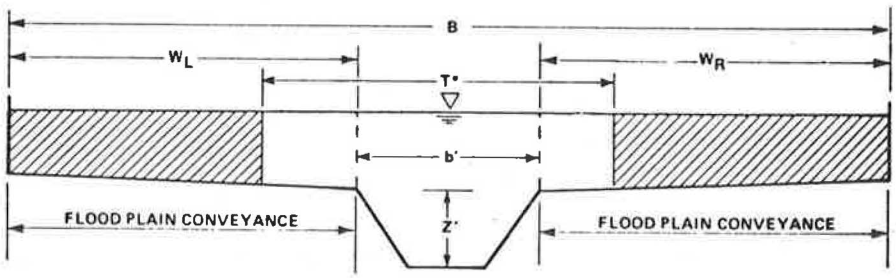
EXAMPLES

To illustrate the application of the simplified estimation procedures, three sample problems are presented. One example estimates the change in

Figure 2. Encroachment and channel schematic.

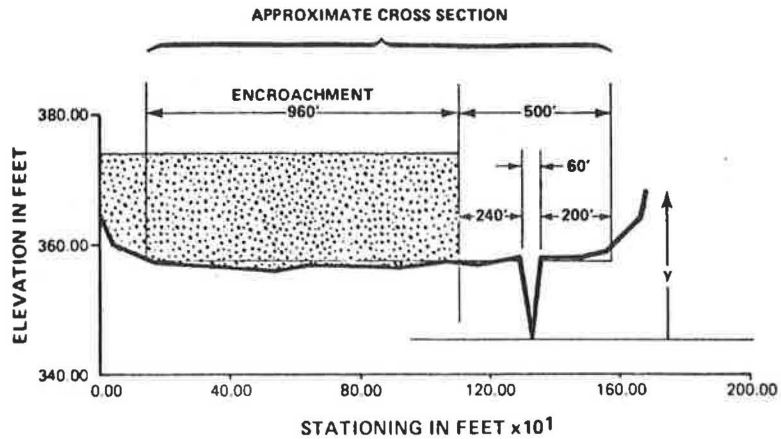


CASE 1



CASE 2

Figure 3. Sample cross section.



water-surface elevation given the width of the encroachment. The next estimates an allowable encroachment given a limit to the increase in water level. The third example estimates the change in peak discharge for a given width of encroachment.

Example 1: Change in Water-Surface Profile for a Given Width of Encroachment

A proposed 960-ft encroachment is placed in a representative stream cross section as shown in Figure 3. The resulting change in water-surface elevation is estimated by a simple procedure given the following information:

Parameter	Amount
Design flow peak (Q)	12,000 ft^3/sec
Main channel slope (S_0)	0.0014
Left overbank roughness (n_L)	0.12
Right overbank roughness (n_R)	0.09
Main channel roughness (n_{mc})	0.06

The rectangular approximation to the overbanks uses as datum the bank-full elevation of the channel shoulders (which have the same elevation) and ends where the overbanks begin to depart significantly from the horizontal plane. For the rectangular approximation of the overbank and an approximate triangular shape of the main channel (assuming an isosceles triangle), shown in Figure 3, the following parameters were determined:

Parameter	Amount (ft)
Left overbank top width (W_L)	1,200
Right overbank top width (W_R)	200
Top width of main channel (b')	60
Bank-full depth (Z')	12
Top width of section (B)	1,200 + 200 + 60 = 1,460

With these parameters the following steps give the desired estimate of the increase in water-surface elevation.

Step 1

Compute the normal flow depth (y_n) from Manning's equation, expressed as follows:

$$Q = 1.486 S_0^{1/2}(1/n_{mc})\,A_{mc}(y)\,R_{mc}(y)^{2/3} + \Sigma(1/n_{ob})\,A_{ob}(y)R_{ob}(y)^{2/3} \quad (8)$$

where

Q = design discharge rate,

A_{ob} = flow area of one overbank,
R_{ob} = hydraulic radius of one overbank, and
R_{mc} = hydraulic radius of main channel determined at A_{mc}/P_{mc}.

$$12{,}000 = 1.486 \times (0.0014)^{1/2}\Big[(1/0.06)\Big([1/2\,(60 \times 12) + 60\,(y_n - 12)]^{5/3}/\big\{2[(30^2 + 12^2)^{1/2} + (y_n - 12)]\big\}^{2/3}\Big) + [(1{,}200/0.12) + (200/0.09)]\,(y_n - 12)^{5/3}\Big] \quad (9)$$

The hydraulic radius of each overbank is approximated by the depth of flow in the rectangular overbanks; i.e., $R_{ob} = y_n - 12$. Successive estimates of y_n by trial and error eventually give $y_n \simeq 17$ ft.

Alternatively, the conveyance in the subsections can be computed independently by using Manning's equation and plotted graphically. From this procedure, the normal flow depth (y_n) can be determined. The main channel conveyance as a function of depth (K_{mc}) can be determined from Equation 4 as follows:

$$K_{mc} = (1.486/0.06)\big\{[1/2(60 \times 12) + 60(y - 12)]^5/2[(30^2 + 12^2)^{1/2} + (y - 12)]^2\big\}^{1/3} \quad (10)$$

where y is total flow depth. For three trial solutions (y = 12, 15, and 18 ft) K_{mc} = 28,018, 51,907, and 82,680 ft^3/sec, respectively.

Assuming that the hydraulic radius of the overbanks can be estimated by the depth of flow y - z', the overbank conveyances (K_L') are determined as follows:

$$K = (1.486/n_{ob})\,W_{ob}\,(y - z')^{5/3} \quad (11)$$

Therefore, the left overbank conveyance (K_L') is determined as follows:

$$K_L' = (1.486/0.12) \times 1{,}200 \times (y - 12)^{5/3} \quad (12)$$

For y = 15 and 18 ft, K_L' = 92,730 and 294,400 ft^3/sec, respectively. The right overbank conveyance (K_R) is determined as follows:

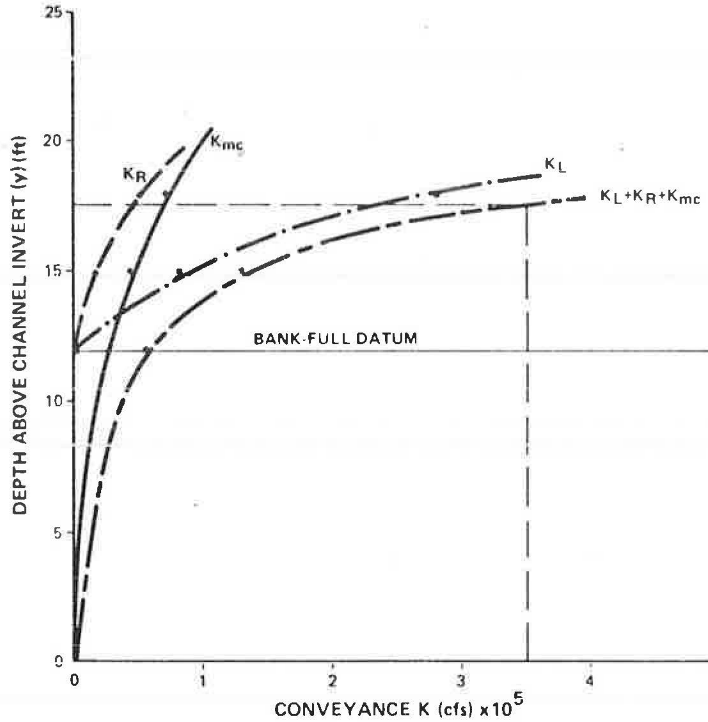
$$K_R = (1.486/0.09) \times 200\,(y - 12)^{5/3} \quad (13)$$

For y = 15 and 18 ft, K_R = 20,607 and 65,461 ft^3/sec, respectively.

Plots of K_{mc}, K_L', K_R, and total K as a function of depth y are shown in Figure 4, where

$$K = K_{mc} + K_L' + K_R \quad (14)$$

Figure 4. Conveyance versus depth for cross section.



y_n is determined for the depth that satisfies Equation 8; i.e., $K = 12{,}000/0.0014^{1/2} = 3.1\text{x}10^5$ ft³/sec. From Figure 4, the depth at which the value of K computed above is satisfied is 17 ft. Therefore, y_n = 17 ft.

Step 2

Determine the weighted Froude number from the conveyance and area distribution derived in step 1 and Equations 2 and 3. From Figure 4, the conveyance distribution is as follows: $K_L' = 2.0 \times 10^5$ ft³/sec, $K_R = 3.5 \times 10^4$ ft³/sec, and $K_{mc} = 7.0 \times 10^4$ ft³/sec.

From Equation 3,

$$\alpha = \Big([(2.0 \times 10^5)^3/(1{,}200 \times 5)^2] + [(3.5 \times 10^4)^3/(200 \times 6)^2] + \{(7 \times 10^4)^3/[60 \times 6 + 1/2\,(60 \times 12)]^2\}\Big) / \{(3.1 \times 10^5)^3 \div [5(1{,}200 + 200 + 60) + 1/2(60 \times 12)]^2\} = 2.07 \quad (15)$$

From Equation 2,

$$F_r = \Big((2.07 \times 12{,}000^2 \times 1{,}460)/\{32.2(1{,}460 \times 5) + [1/2(60 \times 12)]\}^3 \Big)^{1/2}$$

$$F_r \simeq 0.17 \quad (16)$$

Step 3

Determine $\Delta K_0/K_0$.

From Equation 9,

$$\Delta K_0 = (1.486/0.12) \times 960 \times 5^{5/3} = 1.74 \times 10^5 \text{ ft}^3/\text{sec} \quad (17)$$

From step 2,

$$K_0 = 3.05 \times 10^5 - 7 \times 10^4 \text{ ft}^3/\text{sec} = 2.35 \times 10^5 \text{ ft}^3/\text{sec} \quad (18)$$

$$\Delta K_0/K_0 = 1.74/2.35 = 0.74 \quad (19)$$

Step 4

Determine $\Delta H/H'$. Figure 5 [taken from the user's manual by Traille and others (1, p. 37)] provides an estimate of the surcharge for values of F_r, K_{mc}/K, and $\Delta K_0/K_0$ determined in steps 1-3. From step 1, $K = 3.1 \times 10^5$ ft³/sec and from step 2, $K_{mc} = 7.0 \times 10^4$ ft³/sec.

$$K_{mc}/K = (7.0 \times 10^4)/(3.1 \times 10^5) = 0.22 \quad (20)$$

From Figure 5 at $K_{mc}/K = 0.22$, $\Delta K_0/K_0 = 0.70$, and for $F_r = 0.1$, $\Delta H/H' = 0.7$, where H' is the depth of flow above bank-full stage or 17 minus 12.

$$\Delta H = 0.7 \times 5.0 = 3.5 \text{ ft} \quad (21)$$

Example 2: Maximum Encroachment for a Given Permissible Increase in Flow Level

Determining the maximum encroachment for a specified rise in water-surface elevation (for example, 1.0 ft) requires first determining the following ratio:

$$\Delta H/H' = 1/5 = 0.20 \quad (22)$$

From Figure 5 for $\Delta H/H' = 0.20$, and K_{mc}/K as determined in Example 1, step 4, $\Delta K_0/K_0 = 0.29$. For the rectangular approximation to the overbank, the conveyance can be assumed to be linearly distributed in each overbank. Therefore, the allowable constriction width T_Z for $\Delta K_0/K_0 = 0.29$ is computed as follows:

$$T_Z/W_L = (\Delta K_0/K_0)(K_0/K_L') \quad (23)$$

or

$$T_z = 0.29 \times [(2.4 \times 10^5)/(2.0 \times 10^5)] \times 1{,}200 = 417 \text{ ft} \quad (24)$$

Figure 5. Changes in water-surface elevation.

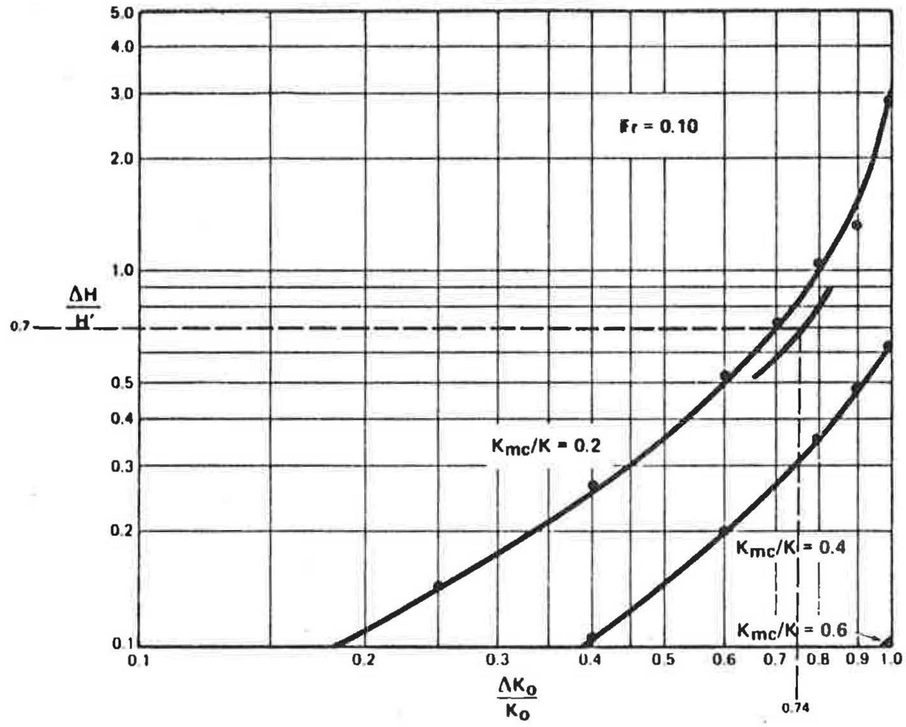


Figure 6. Dimensionless plot of peak-flow changes due to lateral encroachment: case 1.C.1.

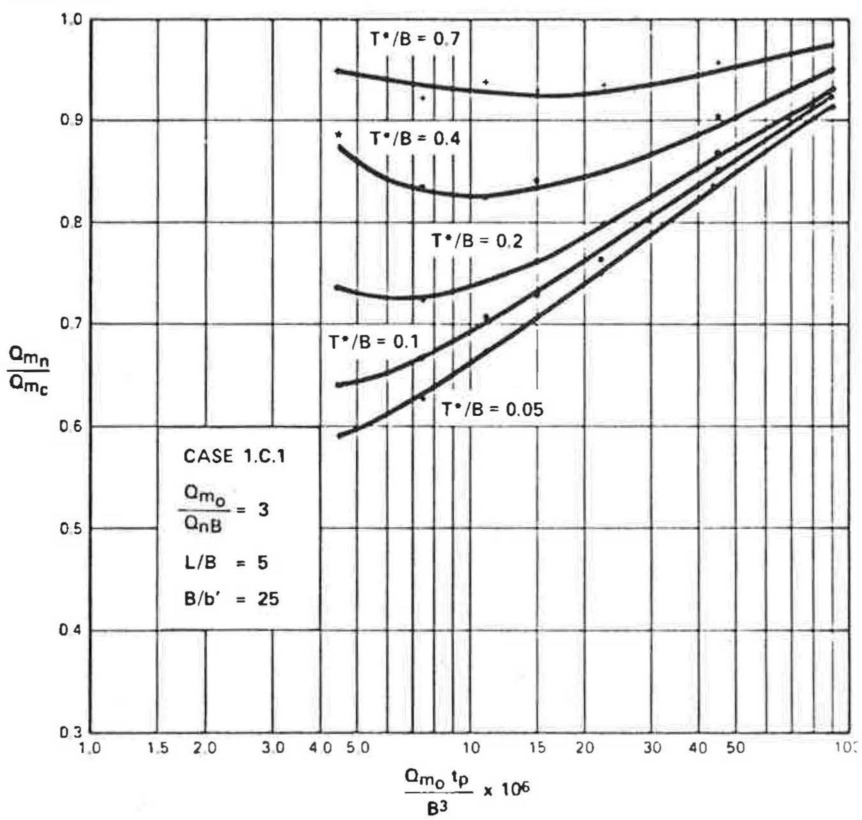


Figure 7. Dimensionless plot of peak-flow changes due to lateral encroachment: case 1.D.1.

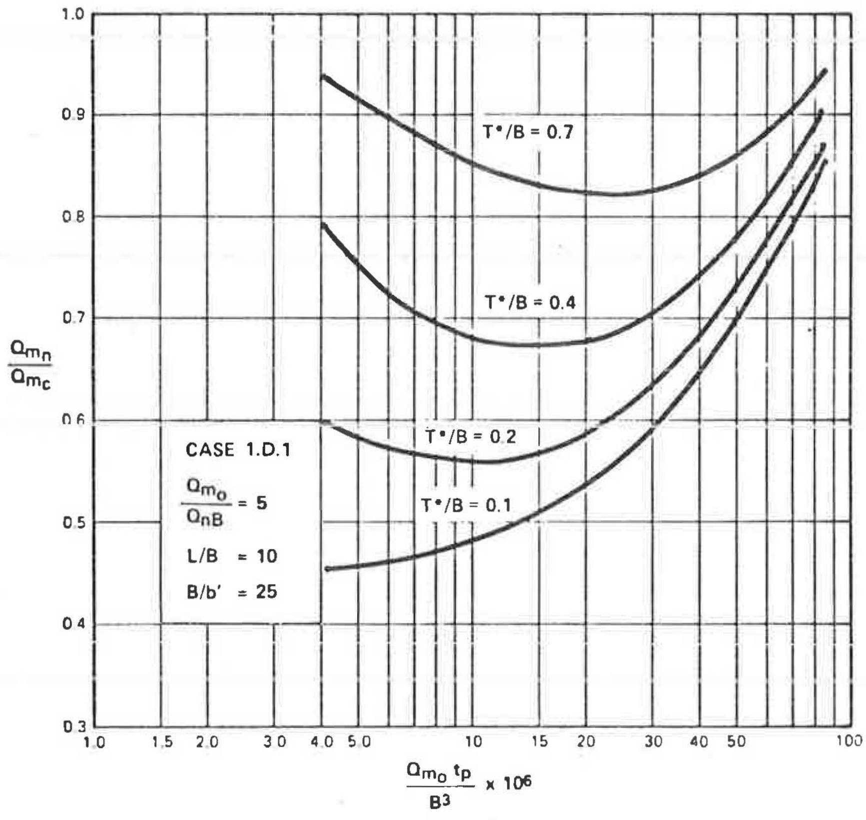
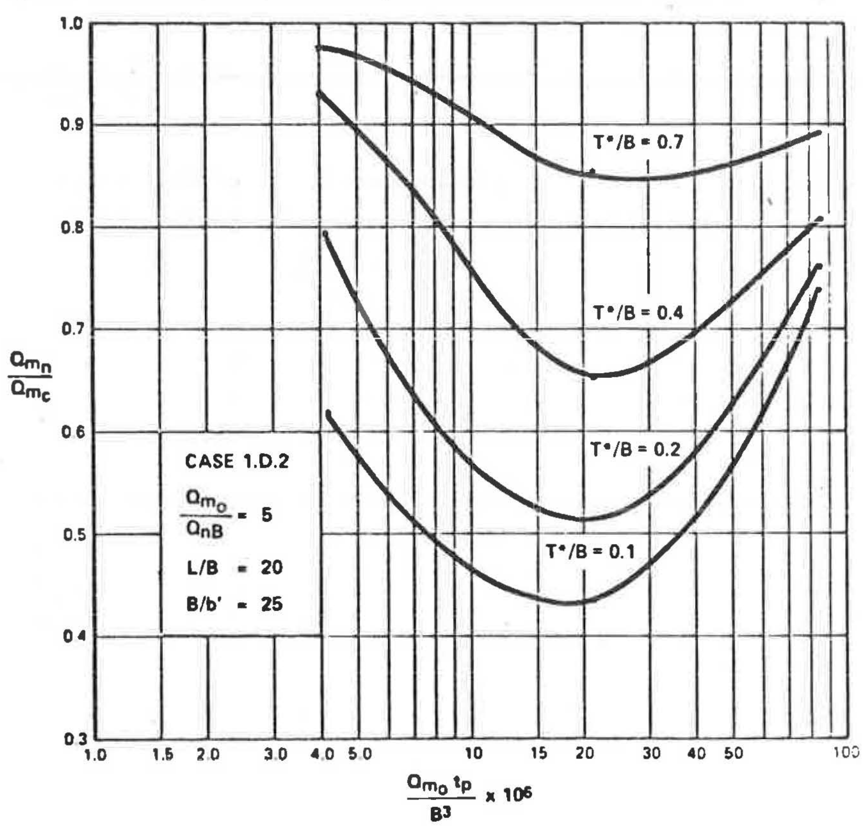


Figure 8. Dimensionless plot of peak-flow changes due to lateral encroachment: case 1.D.2.



Example 3: Change in Hydrograph Peak Flow Rate (No Overbank Conveyance)

Values for the input variables of Equation 6 are required. Determine a representative trapezoidal approximation of the channel cross-section geometry for the selected subreach. The next step establishes whether the flow condition is based on the obstruction in the floodplains. The parameters in Equation 7 are established next. These parameters can be adjusted to account for the effects of meanders and the variable overbank roughness coefficient (1).

The following data are given for a heavily vegetated overbank channel (i.e., no overbank conveyance):

Parameter	Amount
Width of cross section (B)	200 ft
Width of main channel (b')	10 ft
Depth of main channel (Z')	10 ft
Main channel roughness (n_{mc})	0.025
Main channel slope (S_0)	0.0015
Encroached top width (T*)	40 ft
Time to hydrograph peak (t_p)	0.30 hr
Inflow hydrograph peak (Q_{m_0})	2,000 ft³/sec
Reach lengths of channel and overbanks ($L_C = L_R = L_L$)	1,200 ft

Follow the next two steps to estimate the percentage of increase in the flow rate.

Step 1

Calculate values for the parameters in Equation 7, excluding the roughness ratio. Normal bank-full flow (Q_{nB}) is computed by using Equation 8:

$$\begin{aligned}
Q_{nB} &= (1.486/0.025)\{(10 \times 10)^5/[10 + 2(10)^2]\}^{1/3}(0.0015)^{1/2} \\
&= 514\ \text{ft}^3/\text{sec} \\
Q_{m_0}/Q_{nB} &= 2{,}000/514 = 4 \\
B/b' &= 200/10 = 20 \\
Q_{m_0}t_p/B^3 &= (2{,}000 \times 0.30)/200^3 = 75 \times 10^{-6} \\
L/B &= 1{,}200/200 = 6 \\
T^*/B &= 40/200 = 0.2
\end{aligned} \tag{25}$$

Step 2

Figure 6 approximately satisfies all the variable groupings except Q_{m_0}/Q_{nB}. Extrapolation of Figures 7 and 8 for L/B = 6 gives Q_{m_n}/Q_{m_c} = 0.88. From Figure 6 read Q_{m_n}/Q_{m_c} as 0.92. Averaging with 0.88 to determine the value at Q_{m_0}/Q_{nB} = 4 gives 0.90. The amount of amplification to the outflow hydrograph is therefore Q_{m_c}/Q_{m_n} = 1/0.90 = 1.11. Thus, the outflow peak of the encroached reach Q_{m_c} is magnified 11 percent.

CONCLUSIONS

The simulation results indicate that the change in water-surface profile can be expressed as a function of the valley section's conveyance distribution in the main channel and floodplains, the average flow depth in the overbanks (expressed as a depth above bank-full stage), the Froude number of the flow in the unencroached condition, and the degree of constriction (conveyance reduction) in the overbanks due to the encroachment.

Constricting the natural channel valley by encroaching laterally was found to decrease the rate of hydrograph attenuation and thus to increase the natural peak-outflow hydrograph. The characteristic shape of the inflow hydrograph, as defined by the peak-discharge rate and the time of rise of the inflow hydrograph, was found to exert considerable influence on the attenuation of a flood.

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