EXPERIMENTS ON FLOOD-WAVE PROPAGATION IN COMPOUND CHANNEL

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ABSTRACT: Data related to floodplain inundation by fast-rising floods are limited and needed. Eighteen data sets are provided from experiments conducted in a symmetrical compound channel that has a trapezoidal main channel. Three flood types were investigated: long, moderate, and short duration floods (LDF, MDF, and SDF). The data include water depths, velocities, and bed shear stresses measured in straight and meandering reaches. Data show that a flat portion occurred in the MDF and LDF hydrographs when the floodplain was being inundated. Flood peaks stretch exponentially along the channel and are more pronounced in the meandering reach. An SDF has the fast decay rate, followed by MDF and LDF. Overbank front speeds of the SDF and MDF can be predicted by the moving speed of a surge, with the surge height being the difference of the bankfull depth from that of the base flow. The tail of a synthetic hydrograph should be modified according to the channel storage so that it can be used as an inflow boundary condition.

INTRODUCTION

The cross section of a natural river is usually composed of a deep channel and adjacent shallow floodplains. The floodplain may be a wetland and is frequently a wild life habitat. A two-stage channel or compound channel has been proposed as a design cross section to retain large parts of the natural environment unchanged (Ackers 1993). Flow in a compound channel is very complicated. Townsend (1967), Knight (1983, 1987), and Wormleaton and Merrett (1990) pointed out that at a low floodplain depth the large velocity difference between the main channel and the floodplain induced a strong shear layer and a lateral momentum transfer (LMT) across the interface between the main channel and the floodplain. The floodplain might act as a storage or active region when a flood passes through it. The peak of the flood wave attenuates along the channel and a hysteresis occurs in the stage-discharge curve, Ivanova (NERC 1975; Tingsanchali 1976, 1988; Cunge et al. 1980; Yen 1987). Flows in a compound channel are usually turbulent but can be locally in laminar flow condition, if the depth and the local velocity are both small. The high flow velocity in the main channel can also induce a recirculating flow in the floodplain (Plate 1).

The complexities in compound channel flow are not handled easily in experimental and computational works. Rice (1974) conducted unsteady flow experiments and noted that the test flume (44 ft) was too short to experience the attenuation of the model flood that they provided. Stephenson and Kolovopoulos (1990) and Abida and Townsend (1992) both noticed the effect of the lateral momentum transfer to the unsteady flow prediction. They included an empirical apparent shear stress expression in their calculation and demonstrated that the inclusion of the momentum exchange improved the accuracy of their calculations. Lai and Yen (1993a) used a 2D turbulent flow model to simulate a synthetic flood wave propagating through the SERC FCF and the Ashida (1990) meandering channel for flood hydrographs having shorter duration in rising limb and larger unsteady parameters. The model floods were categorized into short, moderate, and long-lasting-limb (SDF, MDF, and LDF hereafter) floods. Three key features were introduced to characterize them. They are discussed in this paper.

DEFINITION AND DESCRIPTIONS OF FLOOD WAVE

Fig. 1(a) depicts a typical cross section of a compound channel reach and the frequently used notations. The depth hydrographs at two sections and the possible water level at one of the sections are shown in Figs. 1(b and c). At time ➁ the water depth is the base flow $h_0$. The water level rises at time ➂ as the flood enters the reach. At time ➃ the bank is full; water starts inundating the floodplain until time ➄, and the hydrograph at this period shows a milder rising slope. If the left and right floodplain have the same bed elevations, the inundation starts evenly at both sides in a straight reach. However, in a meandering reach the flood will overbank first at the concave side because of the water surface superelevation at that side. As the flood overbanks, the water level rises continually and reaches $h_e$ at $t_e$ or time ➅, then it falls down from time ➆ and ➇. The slope of the hydrograph at time ➆–➇ is generally steeper than that at time ➅–➆. A freefall may occur at the edge of the main channel when the water volume stored in the floodplain flowing into the main channel and the water level of the later is lower than the bankfull stage. Some of the floodplain properties, such as its width and vegetation pattern, will affect the feature of the hydrograph. In some cases, if the flood...
strength and the storage in the main channel are limited, floodplain inundation is achieved very quickly and the time interval $\bar{t} = \bar{t} - \bar{t}$ is short. From this observation, it seems convenient to categorize flood patterns into three types and to characterize them individually. They are

1. SDF: time intervals $\bar{t} - \bar{t}$ are all very short.
2. MDF: time interval $\bar{t} - \bar{t}$ is short.
3. LDF: time intervals $\bar{t} - \bar{t}$ are long and can be distinguished in a hydrograph.

Three key features can be used to describe a flood wave. They are the shapes of a stage hydrograph, the maximum flood celerity, and the stage-discharge ($h - q$) relationship at one section. The peak discharge and the shape of a stage hydrograph are related, and their transformations or peak attenuation along the reach are considered as one feature. The first key feature, the shape of a hydrograph, is closely related to the testing or calibrating of a numerical model. Cosine, log-Pearson III, or the two parameter gamma functions have been used as synthetic hydrographs for the upstream boundary condition of river flow computational models. For example, the cosine time function used by Mozayeny (1969) and Tingsanchali (1988) was

\[
h(x_o, t) = h_b + \frac{(h_h - h_b)}{2} \left[1 - \cos \left(\frac{2\pi t}{T}\right)\right] 0 \leq t \leq T \quad (1a)
\]

\[
h(x_o, t) = h_b \quad t < 0 \text{ or } t > T \quad (1b)
\]
where $T$ = period of the cosine function. The two-parameter-gamma-function synthetic hydrograph used by Lai and Yen (1993a) has the following form:

$$h(t, t) = h_0 + (h_p - h_0)\sin\left(\frac{\pi}{T}t\right); \quad t = \tau_p$$  \hspace{1cm} (2a,b)

where $\tau_p$ = time instant of the peak flow; and $\alpha$ = parameter related to channel property. The hydrograph changes further downstream and the peak flow attenuates. Lighthill and Whitham (1955) noted that the decay rate of a flood peak along a prismatic channel having a constant boundary roughness was

$$\frac{h_p}{h_0} = \exp\left[\frac{1}{2} \frac{gS_p}{F_o \sqrt{gh_b}} (2 - F_o)(\tau_p - \tau_0)\right]$$  \hspace{1cm} (3)

Eqs. (1)–(3) provide guides to examine hydrographs in compound channels.

The second key feature, the maximum flood celerity, was shown by Cunge et al. to be the celerity of the overbank flow front. The dynamic wave speed $V_{\text{dynam}}$ for a base flow may be described as

$$V_{\text{dynam}} = v_1 \sqrt{gh_b}$$  \hspace{1cm} (4)

If the overbank flow front is assumed to be nearly vertical and moves as a positive surge (Fig. 2), the celerity of the front may be derived as

$$V_{\text{surge}} = v_1 \sqrt{\frac{A_{\text{obk}}}{2A_{\text{obk}}}}$$  \hspace{1cm} (5)

Intuitively, (5) would provide a key value for checking the overbank front speed of an MDF or an SDF, because both floods have nearly vertical fronts. We will examine these front's moving speeds based on the present experimental data.

**EXPERIMENTAL SETUP**

The experimental system was designed using two criteria so that the key features described above could be identified and measured. The criteria were as follows: (1) Flood duration should be appropriate so that the front of the flood can be observed in the test section, and (2) the inundating flow and the transverse velocity near dike should be weak so that a plunging flow on the dikes does not occur. The following experimental setup was used.

**Test Flume and Experimental Section**

The test flume was located in the River Engineering Laboratory of Cheng-Kung University (NCKUF). It was 35 m in length and had a cement finished measuring section of 18 m. This measuring section was composed of a 5.1 m straight reach and a 11.2 m sinusoidal reach, as shown in Fig. 3(a). Langbein and Leopold’s (1966) meander channel equation:

$$2p_s = u \sin (\theta_{\text{max}})$$  \hspace{1cm} (6)

was used to generate the bends of the main channel, in which the wave length of the bend, $L$, and the maximum deflecting angle, $\theta_{\text{max}}$, were 200 cm and 30°, respectively. The cross section of the main channel was a trapezoid with a side slope $S_z$ of 1:1. Half of the bottom width $b_{mt}$ and the bankfull depth of the main channel $h_{mc}$ were both $5 \pm 0.3$ cm. The straight reach was symmetric, and its relative width $B/b_{mt}$ was 8.6. The channel bed was cement finished and the average longitudinal slope of the floodplain was 0.001. The roughness coefficients Manning’s $n$ were estimated from numerical calculations to be 0.012 and 0.013 for the main channel and the floodplain, respectively. In all the experiments the coordinate origin was set at the left corner of the entrance marked by “o” in Fig. 3(a).

**Instrumentation**

The model test conditions for generating SDF, MDF, and LDF were set based on several preliminary tests. The test con-
ditions were controlled by the water storage R1 and R2 and the gates W1 and W2 shown in Fig. 3. Water was supplied by the centrifugal pump P1 with a discharge capacity of 0.1 m³/s. Water level, velocities, and bed shear stress were measured at the locations shown in Fig. 3(b). The accuracy of the level gauge is within 2 mm, and that of the velocimeter and the minute shear meter were within 0.5% of the range measured. The shear meter had an acting area of 5 × 5 mm². They were all calibrated before each test run. Signal outputs of these instruments were connected to an AD-DA controller board inside an IBM PC, and commercial software (Labtech Notebook 7.3, Laboratory Technologies Corporation) was used to control the entire data acquisition procedure. The sampling rate for the measurements of velocity, bed shear stresses, and water level was 5 Hz. A moving averaged procedure was used to smooth the data for presentation.

In addition to the local velocity measured by the velocimeter, the spatial distribution of velocity field during a flood event was obtained by flow visualization (FV) technique. Two systems were used. Main components of the system for some earlier experimental runs were a video camera, a high density VHS tape recorder, and an image grabber (VHSG). Several plastic balls with a diameter of 5 mm were painted in different colors and were released as the flow tracers during an experiment. Time interval of each recorded image frame was 1/30 s and the paths of the flow were traced through consecutive images. This earlier system gave only the paths of flow but was unable to obtain accurate local flow velocities because the time interval between two images was not stable. The second FV system, HDVC, which had higher image-scanning rates (200 images/s) and much stabler image intervals, was thus used for some later experimental runs. A small number of polystyrene bits were used as the tracer. The recorded digital images were analyzed by a multiframe image-processing technique similar to the method suggested by Adrian (1991).

RESULTS AND ANALYSES

There were 18 experiments conducted, and the inlet hydrograph characteristics for these experiments are summarized in Table 1. In the table, the subscripts p, b, and r or f represent the peak, the base, and the rising or falling stages, the Froude number, was computed based on the flow velocity and depth, and V_{dynam} and V_{surge} were calculated by (4) and (5), respectively. The results are analyzed as follows.

First Feature: Hydrograph Shape and Peak Attenuation

Hydrograph Shape

The shapes of depth hydrographs and their transformation along the test channel were drawn in Figs. 4−6 for LDF (test j25d), MDF (j25b), and SDF (j25c), respectively. Figs. 4(a and b) show the velocity, depth hydrographs, and the discharge through the upstream weir (W1) of the LDF test run j25d. Locations of the measurement are indicated in the legends. The horizontal line in Fig. 4(b), showing a depth of 5 cm, is the bankfull stage. The graphs indicate that the inlet depth hydrographs follow the generating schemes well. An LDF has a gently rising limb from depth h_{p} to h_{mc}, while that of an MDF has a steep limb from h_{p} to h_{mc} and a moderate one from h_{mc} to h_{b}. However, an SDF rises sharply from h_{p} to h_{b}. Values of h_{p} decrease along the channel, and the recession limbs of all the hydrographs are rather flat from h_{mc} to h_{b}. The slope of the recession limbs increase as the flow depth falls below h_{mc}. In some rising limbs of the hydrographs there is a flat region at a level just over the bankfull stage. This flat region is more pronounced at the downstream sections. The velocity plots demonstrate that the velocity in the channel increases as the flood front reaches a section. Its rising rate decreases a little at the depth just over h_{mc}. This implies that the velocity in the channel increases as the flood front reaches a section. Its rising rate decreases a little at the depth just over h_{mc}. This implies that the rising limb decreases.

Flood Peak Attenuation

Flood peak depth h_{p} at different sections for all the experiments are normalized with h_{p,0} and plotted with a normalized distance X in Fig. 7. X is defined as X = x/(h_{p,0}/S_{0}), where x is the distance of a section from the inlet. The dashed lines are the exponential fitted lines through the data, and have the following form:

\[ h_{p}/h_{p,0} = Ae^{-bX} \]  

(7)

Values of the coefficients A are 1.03, 1.04, and 0.97, and B are 0.42, 0.58, and 0.97 for the LDF, MDF, and SDF, respectively. Although the data are rather scattered, the fit lines show that the flood peak attenuates exponentially. Eq. (3) is plotted as the solid line in Fig. 7 and it shows that this equation gives a faster decay rate than that of the experiments. This implies that if the inflow depth hydrograph is the same, the depth of flood peak in a single channel has a faster decay rate than that in a compound channel.

Comparison of Measured and Synthetic Inflow Hydrographs

Using twice (tp − t₀) as the period T in (1), the inflow hydrographs for LDF and MDF are generated and plotted as the curve “2” in Figs. 8(a and b). The measured hydrographs “1” of a seventh-order data-fitted polynomial (2) is a triangular curve (based on t₀, t_{p}, t_{r}, and t_{br}), “5,” and the synthetic inflow, (2), used by Lai and Yen (1993a) are also plotted as the line indicated by “3.” It is clear from the comparison that the synthetic inflow hydrographs “2,” “3,” and “5” do not give a good representation of the measured data. On the falling limb, the difference between the synthetic hydrograph and the data is
FIG. 4. Depth and Velocity Hydrographs of Long Duration Flood (LDF)—Case j25d

FIG. 5. Depth and Velocity Hydrographs of Moderate Duration Flood (MDF)—Case j25b
large. This is because the water volume stored in the wide floodplain needs time to flow further downstream, which the synthetic curves cannot take into account. On the other hand, one might extend the duration $T$ of (1) to a longer time, or use a different sinusoidal curve for the falling limb (as that used by Tingsanchali), in order to reduce the error on the recession limb. The former would cause larger differences on the rising limb and the latter results in a discontinuous point at $t_p$. In a comparison test run, $m_{22e}$, the control gate $W_2$ at the downstream end was adjusted so that the recession took a shorter time. The comparison is plotted in Fig. 8(c) and shows that the synthetic curve gives a better representation than the previous two cases, in Figs. 8(a and b). This implies that the falling limb of the inflow hydrograph is affected by the downstream control and the flow is not parabolic. The fitted seventh-order polynomial curve seems to give better representation; however, it is less useful since the inflow condition is already known.

The previous comparison shows that none of the synthetic hydrographs are suitable for use as an inflow condition for compound channel unsteady flow computation. Further study of this problem therefore is needed. The previous discussion and the analytical analyses of Yen (1987) and Tingsanchali (1988) suggest that the variables: $t_r$, $t_{br}$, $t_{dur}$, $h_b$, $h_p$, $F_0$ given in the first row of Table 1, and the floodplain properties, such as the width $B$ and the roughness distribution, are possible parameters to characterize the inflow hydrograph.

**Second Feature: Overbank Front Speed**

**Overbank Flow Visualization at Channel Bend**

The visualized paths of wave front at the channel bend $V$, using the VHSG and HDVC image systems, are shown in Figs. 9(a and c). The symbol “⊗” with marked numbers in Fig. 9(a) are the measuring locations of the water level, velocity, and bed shear stress. The symbol “x” in Fig. 9(a) are the traced points of the surface floaters. The interval between the two marked points is 1/30 s. The photograph in Fig. 9(b), indicating by $t = 0.0$ s, is the HDVC image that was taken when the flood just started to inundate over the floodplain. The wavy free surface at both sides of the floodplain can be observed. Fig. 9(c) shows the processed result of four consecutive HDVC images that spans from 0 to 0.144 s. The surface
velocity, therefore, can be estimated from the coordinates of the tracers. As one example, the speed of the tracer marked with the number “8” is estimated to be 0.6 m/s. The simulated overbank flow pattern at bends IV–VI calculated by a 2D model (see Lai and Liu 1998), is shown in Fig. 9(d) to assist in interpreting the flow image. From the tracer paths in Figs. 9(a and c) and the calculated result in Fig. 9(d), one observes how the flood flow inundates across the floodplain and interacts with the flow in the main channel. The flow starts inundating the floodplain at the concave side of the bend. It drifts downstream, returns, and merges with the main channel flow at the transition between the sequential bends.

**Wave Front Speeds and Travel Distance**

Accurate prediction at the instant of bankfull flow at a channel section is needed for an effective flood warning system. Eqs. (4) and (5) are suggested for this prediction. From experimental data, we obtained the instants of the overbank flow $L_{ov}$ and the distance between sections $S_{ov}$. By knowing $V_{ov}$ and $V_{ov}$ from (4) and (5), the travel distances $S_{ov}$ are computed and plotted against $S_{ov}$ and shown in Fig. 10 as the symbols “+” and “O”. The broken lines marked with “d” and “s” are their linear fitted lines and the solid lines with the symbol “p” are the perfect predictions, respectively. This com-
Third Feature: Stage-Velocity Relation

The variation of bed shear stress, $\tau_{mc}$, $\tau_{fp}$, with main channel velocity $U$, and the relation of $U$ with the main channel water depth $h$ during the rising and falling stage of a flood are examined using the LDF data of the test runs n21a and m25a. Figs. 11(a–d) show the velocity and water depth hydrographs, the correlation of $h-U$ and $\tau-U$ of the test runs, respectively. In the figures, $U_p$, $h_p$, and the notations ‘$\bar{X}$, $\bar{X}'’$ are marked to locate the instants of the peak and the overbank flows. The $h-U$ curves for case n21a and m25a in Fig. 11(c) show that they have loop patterns. At a specified depth the velocity has a higher value at the rising stage than at the falling stage. There is a spike in the rising curve at the range of the relative depth, $h_r = h_p/H$, between 0.1 and 0.2. The peak of the spike is the highest velocity $U_p$, and the relative depth corresponding to it is 0.14. Above this relative depth the velocity increases with the depth until it reaches $h_r$. The relations of the bed shear stress $\tau_{mc}$, $\tau_{fp}$ to the main channel velocity, $U$, in Fig. 11(d) show that they form two narrow loops, and there are two steps observed on the $\tau_{mc}-U$ curve. The first step occurs at just before the overbank stage and the second one occurs at the stage having a relative depth of around 0.1. The value of the $\tau_{mc}$ at these two steps almost does not vary with the velocity. The second step also forms a small triangular loop that corresponds to the spike in Fig. 11(c). In general it is noticed that at a specified velocity value, $\tau_{mc}$ has a higher value during the rising stage than the falling stage. A reverse trend for the $\tau_{fp}$ curve is observed.

Limitation of Data Sets

There are already a few published unsteady flow data that can be used to validate numerical models. This paper adds 18 more laboratory results to the existing data bank. The present data sets are categorized into three patterns based on the duration of the rising stage. The wide floodplain with a relative width of 8.6 was chosen primarily to prevent the flow from interfering with the impact return flow from the dike, and the overbank process can be observed in the floodplain. If detailed boundary shear stress data and hydrographs at both of the main channel and floodplain along the channel are needed, such as those required by a refined numerical model, the present data sets may be useful.

It should be noted that in a natural environment the channel roughness varies place to place. The data sets given in Table 1 are the results for the smooth compound channel experiments. More studies that are concerned with unsteady flow in compound channel with roughened floodplain thus are needed. Furthermore, if (5) is used to predict the occurrence of overbank flow for a real river, one should understand that the varying boundary roughness of the river would retard the moving
speed of a wave front at the rougher floodplain. This causes a larger portion of the flow to go to the smoother reach and may result in a faster-moving front at the smoother reach or main channel. For design purposes, a safety factor should be considered in responding to this circumstance.

CONCLUSIONS

Three key features, i.e., the hydrograph shape, the overbank wave front, and the $h-U$ relations, are used in drawing the following conclusions from the present study of flood flows in concrete compound channels:

1. The rising limb of a hydrograph at a section is controlled by the flow conditions upstream. The falling limb is affected by the channel properties downstream of the section. There may be a flat region in the rising limb that starts at the overbank stage and lasts until the velocity in the main channel reaches a peak value, or the time when the inundating wave approaches the channel levee. In the present smooth channel study, both the velocity and the bed shear stress in the main channel reach their highest value at the stage that has a relative depth of around 0.1.

2. In a straight reach the overbank flow occurs evenly on both sides of the floodplains. In a meandering reach the overbank flow starts sequentially at the concave side of bends. The overbank flow drifts downstream, returns, and merges with the main channel flow at the transition between the sequential bends.

3. The peak of a flood attenuates exponentially along the channel. If the inflow hydrograph is the same, the depth of the flood peak, $h_p$, of a single channel has a faster decay rate than those of a compound channel. The decay rate of $h_p$ of SDF is the fastest among the three flood types, followed by MDF and LDF.

4. The inflow boundary condition should be chosen cautiously for an unsteady flow model of a compound channel, especially when it has a wide floodplain. A control or the amount water stored in the floodplain downstream of the inflow section can affect the falling limb of the inflow hydrograph.
5. The occurrence of overbank flow of MDF in the smooth compound channel can be predicted by taking the wave front as a dynamic wave or a surge wave. The overbank instant for SDF can be estimated from the moving speed of a surge front, with the surge height being the difference of the stages at the overbank and the base flows.

6. At a specified flow depth the velocity has a higher value on the rising limb than it has on the falling limb. At a velocity value the bed shear stress in the main channel also has a higher value at the rising stage.

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APPENDIX I. REFERENCES


APPENDIX II. NOTATION

The following symbols are used in this paper:

\( A_{obf} \) = flow area of base flow;

\( A_{mak} \) = flow area of main channel at overbank stage;

\( b_{w} \) = top half width of main channel;

\( D_{w} \) = bottom half width of main channel;

\( F_{n} \) = Froude number of base flow;

\( g \) = gravitational acceleration;

\( H \) = overall depth of channel;

\( h_{p} \) = water depth at peak of a flood;

\( h_{w} \) = water depth of main channel;

\( h_{s} \) = water depth of steady uniform base flow;

\( n \) = Manning’s roughness coefficient;

\( q_{0} \) = base flow discharge;

\( q_{o} \) = peak discharge of flood at entrance;

\( S_{mc} \) = distance along main channel between two sections;

\( S_{obf} \) = calculated moving distance of surge or dynamic wave;

\( S_{opk} \) = average slope of channel;

\( T \) = duration or period of synthetic flood hydrograph;

\( t_{dur} \) = duration of overbank flow = \( t_{obf} - t_{obr} \);

\( t_{obf} \) = time of overbank flow at rising flood (see Fig. 1);

\( t_{r} \) = rising time of hydrograph;

\( t_{p} \) = peak time of model flood;

\( U \) = measured main channel velocity;

\( U_{p} \) = peak velocity in main channel during flood;

\( u, v \) = velocities in \( x, y \) directions;

\( v_{o} \) = mean velocity of base flow;

\( V_{dyn} \) = moving speed of dynamic wave front [Eq. (4)];

\( V_{surr} \) = moving speed of surge front [Eq. (5)];

\( w^{*} \) = boundary shear velocity;

\( X \) = \( x(h_{p}/S_{obf}) \); dimensionless distance for wave front;

\( x, y \) = coordinates of channel (origin is at lower left corner of \( C_{2} \) in Fig. 2);

\( Z_{obk} \) = depth of centroid of main channel at overbank stage, measured downward from free surface; and

\( \tau \) = bed shear stress.

Subscripts

\( b \) = base flow;

\( fp \) = values in flood-plain;

\( mc \) = values in main channel;

\( obk \) = at overbank flow stage;

\( obf \) = overbank stage on the falling limb;

\( obr \) = overbank stage on the rising limb;

\( p \) = hydrograph peak; and

\( r \) = rising stage.