วารสารทางวิชาการ วสท. ปี 2532 เล่ม 1 หน้า 63–75
Technical J. of EIT 1989, 1, 63 – 75

อิทธิพลของน้ำทะเลหนุนและน้ำหลาก ต่อระดับน้ำในแม่น้ำเจ้าพระยา

Effect of Tide and River Flow on Flood Level of the Chao Phraya River

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บทคัดย่อ

การกระทำระหว่างน้ำขึ้นน้ำลงกับน้ำหลากในแม่น้ำนั้นเป็นเรื่องที่ซับซ้อน จึงยังไม่ใคร่มีผู้สนใจศึกษา ส่วนใหญ่มักจะเป็นการศึกษาลักษณะการเคลื่อนตัวของน้ำขึ้นน้ำลงในแม่น้ำและอ่าวซึ่งไม่มีน้ำหลาก น้ำหลากมีอิทธิพลในการยกระดับน้ำปานกลางให้สูงขึ้น และลดระดับความสูงของน้ำขึ้นน้ำลงที่ปากแม่น้ำ เป็นผลให้ยากแก่การระบายน้ำจากตัวเมืองลงสู่แม่น้ำในช่วงน้ำสูง การวิจัยนี้มีจุดมุ่งหมายที่จะศึกษาการกระทำระหว่างน้ำขึ้นน้ำลงกับน้ำหลาก ด้วยการวิเคราะห์ข้อมูลระดับน้ำที่บันทึกได้ที่สถานีวัดน้ำห้าแห่งในแม่น้ำเจ้าพระยาตอนล่าง การไหลของน้ำในแม่น้ำช่วงนี้มีการเปลี่ยนแปลงตลอดเวลาอันเป็นผลจากการขึ้นลงของน้ำ ดังนั้นจึงต้องใช้วิธีคำนวณโดยแบบจำลองคณิตศาสตร์ น้ำขึ้นน้ำลงในแม่น้ำนั้นมีหลายคาบและเมื่อกระทำต่อน้ำหลากจึงเกิดเป็นปรากฏการณ์ที่ซับซ้อน ดังนั้นจึงต้องใช้วิธีแยกศึกษาการกระทำของน้ำขึ้นน้ำลงเป็นคาบ ๆ เพื่อกำหนดลักษณะอิทธิพลของน้ำหลากในการสลายน้ำขึ้นน้ำลง การหน่วงลดความเร็วของน้ำขึ้นน้ำลง และการยกระดับน้ำปานกลาง ค่าที่กำหนดได้นี้สอดคล้องกับค่าที่ได้จากแบบจำลองทางกายภาพ

Summary

Studies on interaction of tide and river flow are scarce while studies on tidal characteristics in canals and gulfs are common. The river flow raises the mean water level but damps the tide from the river mouth which makes drainage of rain water from the city to the river more difficult and causes flooding at high water. This study is to find the interaction of tide and river flow from an analysis of 5 tidal records along the Chao Phraya River. The time-dependent flow in the lower reach of the river is computed from a numerical model. The complicated interaction of the river flow with various constituents of tide is simplified to its interaction with individual constituents obtained from a harmonic analysis. The dependences on river flow of damping modulus, reduction of celerity and raising mean water level are then determined from the current results as well as from another set of results from a tidal flume.

Introduction

A tide is a long wave having various constituents (with periods of about 1/2 day, 1 day, 1/2 month, 1 month, 1/2 year, 1 year, etc.) which interact simultaneously with river flow (with a cycle of about 1 year). As a result of this interaction, water levels along the river are varied as functions of distance for each given tide and river flow. The river flow damps the tide from the river mouth and raises the mean water level. Studies on interaction of tide and river flow are scarce. Ippen and Harleman (1966) reported results of tests in a tidal flume at the Waterways Experiment Station, Vicksburg, U.S.A. on the interaction of tide with river flow. Recently, Godin (1985) analyzed the tidal records of three rivers in Canada to determine the influences of fresh water discharge on the time of travel and on the range of the tides. Low correlation coefficients were obtained because the resultant tides, instead of individual constituents were used.

Bangkok, located on a low lying alluvial deposit, is flooded every year by the tide and river flow of the Chao Phraya River. Due to land subsidence caused by the pumping of groundwater, the flooding is more severe and more frequent than ever. In the past decade, severe floods have occurred in 1975, 1978, 1980 and 1983. In each of these floods, the overall damage to both the public and private sectors was more than 1,000 million Baht. In particular, the flood damage to Bangkok and the five adjoining provinces in 1983 estimated by the National Statistic Office was about 6,600 million Baht. In addition to the high tide and large river discharge, the 1983 flood was caused by heavy rainfall too. As a result of the 1983 flood, the Thai Government spent about 1,000 million Baht in 1984 and about 500 million Baht in 1985 for temporary protective measures, including construction of dikes, pumping stations, etc. Several studies have been conducted to provide a permanent solution to the flooding of Bangkok, i.e. Preliminary Study on Flood Protection Drainage Project in Eastern Suburban Bangkok, JICA (1984), Flood Routing and Control Alternative of the Chao Phraya River for Bangkok, AIT (1985), Bangkok Flood Control and Drainage Project – City Core, NEDECO (1986). Bangkok Flood Protection – Chao Phraya 2, THAI AUSTRAIN CONSORTIUM-AIT (1986) and Master Plan for Flood Protection and Drainage of Thonburi and Samut Prakan

West, NEDECO (1987). These studies have recommended practical solutions to the flooding problem but do not show clearly nor comprehensively the exact magnitudes of flooding due to the tide and river flow.

This study is to find the interaction of tide and river flow of the Chao Phraya River from the analysis of 5 tidal records along the river in the years 1980 and 1983. The time-dependent flow in the lower reach of the river will be computed from a numerical model. The complicated interaction of the river flow with the various constituents of tide will be simplified to its interaction with individual constituent through a harmonic analysis; damping of tidal amplitude and reduction of celerity for each constituent of tide are thus determined. The effects of river flow on the damping modulus, celerity of the tide and the raising of the mean water level are then presented.

Description of The Chao Phraya River

The Chao Phraya River is the most important river in Thailand. Its catchment area is 162,000 km^2 which occupies most of the northern region and the central part of Thailand, and its length is about 1,000 km. It drains into the Gulf of Thailand where a strong mixed tide prevails at the river mouth.

The geometry in the lower reach of this river is rather uniform having an averaged width of about 400 m, an averaged depth of 9.2 m and a rather flat slope of about 2×10^{-5}. Tides recorded at 5 stations in this lower reach in 1980 and 1983 are used in this study. Observed tides at these 5 stations from April 5-18, 1983 (Fig. 1a) reveal a small damping in the dry season, while those from October 5-18, 1983 (Fig. 1b) show a large damping in the rainy season. A clear damping of the tide from August to October 1983 is shown in Fig. 1c which is a plot of the maximum and the minimum instantaneous water levels or the upper and lower envelopes of the water levels. It can be seen from these figures that at the river mouth (Station 1), where the river drains into the Gulf of Thailand, the tide is not damped out by the river flow even in the rainy season as compared with that in the dry season. Due to the damping, the range of tide at the upstream station is smaller than that of the downstream station at a particular time when the river flow is the same. When the river flow increases from the dry season to the rainy season, the damping increases while the mean water level raises up. As a result, the high water level is higher than the river banks and it causes flooding.

Methodology

The methodology to obtain the magnitude of river flow which is the most important parameter of this study will be first presented. The principle of harmonic analysis to obtain amplitudes and phases of tides as well as their mean water levels (affected by the river flow) from recorded water levels is then described. Analytical description of tide with linear frictional force is finally summarized to serve as useful guides for subsequent analyses.

Computation of River Flow

Figure 1 shows clearly that the damping of the tide is strongly influenced by the river flow. Due to the tidal influence, the flow in the lower reach of the river, which is the superposition of river flow and tidal flow, varies with time and location, therefore, is not presently available. However, there is a rating curve at Bangsai for the high fresh water flow when the tidal influence is small. The flow in the river is computed by a finite difference model using water levels recorded at the river mouth (Station 1) and at Bangsai (Station 5) as boundary conditions. The rating curve at the upstream boundary and water levels at other 3 stations are used to calibrate the model, the Single Reach Model of the Asian Institute of Technology (1985). Due to gradual changes of width and depth of the river in the lower reach, an incremental distance of 3 km is used and the constant Manning n = 0.028 is obtained from the calibration of the model.

Harmonic Analysis of Tide

Since the tide is composed of various constituents which interact simultaneously with the river flow, the resulting records of water levels in the river show rather complicated patterns. Individual interaction of each constituent of the tide in each month is then obtained by a harmonic analysis of the hourly water levels of the 5 stations. The characteristics of the individual constituents of the tide can then be correlated with the river flow. Four predominant constituents of the tide are analyzed for their characteristics; they are:

Principal lunar M_2 with a period = 12.4206 hours;
Principal solar S_2 with a period = 12.0000 hours;
Luni-solar declinational K_1 with a period = 23.9346 hours; and
Large lunar declinational O_1 with a period = 25.8194 hours.

The harmonic analysis of tides is developed from the following two premises:

1. The resultant tide at any location is composed of a finite number of constituents, each with its own periodicity, phase angle and amplitude; and
2. The constituents are each simple-harmonics in time and are mutually independent.

$$n_r(t) = \triangle h + \sum_{i=1}^{N} a_i \sin\left[\frac{2\pi t}{T_i} + \delta_i\right] \qquad (1)$$

in which $n_r(t)$ is the resultant tide recorded as a function of time t at a particular location and it is composed of N constituents. The periods T_i of the i-constituents are known from knowledge of astronomical computations. The mean water level $\triangle h$, the amplitudes a_i and phase δ_i are evaluated from a discrete hourly $n_r(j)$ for j = 1 to M as follows:

$$\triangle h = \frac{1}{M}\sum_{j=1}^{M} n_r(j) \qquad (2)$$

$$a_i = 2\left[\left\{\frac{1}{M}\sum_{j=1}^{M} n_r(j)\sin\left[\frac{2\pi j}{T_i}\right]\right\}^2 + \left\{\frac{1}{M}\sum_{j=1}^{M} n_r(j)\cos\left[\frac{2\pi j}{T_i}\right]\right\}^2\right]^{1/2} \qquad (3)$$

and

$$\delta_i = \tan^{-1}\left[\frac{\sum_{j=1}^{M} n_r(j)\cos\left[\frac{2\pi j}{T_i}\right]}{\sum_{j=1}^{M} n_r(j)\sin\left[\frac{2\pi j}{T_i}\right]}\right] \qquad (4)$$

Analytical Description of Tide with Linear Frictional Force

Ippen and Harleman (1966) investigated analytically the characteristics of damped tides with a linear frictional force. Their useful expressions are summarized as follows:

The water surface fluctuation

$$n(x,t) = a(x)\cos(\sigma t - kx) = a_0 \exp(-\mu x)\cos(\sigma t - kx) \qquad (5)$$

in which a(x) = amplitude of tide at a distance x [km] from the river mouth; a_0 = amplitude of tide at the river mouth; μ = damping modulus [km^{-1}], $\sigma = 2\pi/T$ = angular velocity of tide; T = period of tide; $k = 2\pi/L$ = tide number; and L = length of tide.

The tidal velocity

$$u(x,t) = \frac{a_0}{h} c_0 \exp(-\mu x) \frac{k_0}{\sqrt{\mu^2 + k^2}} \cos(\sigma t - kx + \alpha) \quad ..(6)$$

in which h = water depth; $k_o = 2\pi/L_o$ = tide number at the rive mouth; L_o = length of tide at the river mouth

$$c_0 = \sqrt{gh} = \frac{L_0}{T} = \frac{\sigma}{k_0} \quad ....(7)$$

The relationship of celerity of tide

$$\frac{c_0}{c} = \frac{k}{k_0} = \sqrt{1 + \left[\frac{\mu}{k_0}\right]^2} \quad ....(8)$$

The phase angle of tidal velocity

$$\alpha = \tan^{-1}\left(\frac{\mu}{k}\right) \quad ....(9)$$

The maximum tidal velocity of Eq. 6 when $\cos(\sigma t - kx + \alpha) = 1$ at the river mouth ($x = 0$) is

$$u_0 = \frac{a_0}{h} c_0 \quad ....(10)$$

Note that Eqs. 5 and 6 are solutions of linear governing equations, therefore, the superposition of the various constituents of tide is possible.

Analysis and Results

River Flow

Using the observed hourly water levels at Stations 1 and 5 shown in Fig. 1 as the downstream and upstream boundary conditions respectively, the hourly discharges at the downstream and upstream boundaries are computed. The hourly discharges at the downstream boundary show larger fluctuations than those at the upstream boundary which are consistent with their respective hourly water level fluctuations (Fig. 1). When the hourly discharges in a day are averaged to get the mean daily discharges, the semi-diurnal and diurnal fluctuations are eliminated; the daily discharges at Stations 1 and 5 are almost the same, therefore, the mean monthly discharges for all the stations along the lower reach of the river would have smaller differences or they could be assumed to be the same.

Harmonic Analysis

The observed hourly water levels of Stations 1 – 5 in April, August, September and October 1980 and 1983 are used to compute $\triangle h$, a_i and δ_i by Eqs. 2 – 4 respectively. The monthly data are selected for this analysis in order to minimize the errors of $\triangle h$, a_i and δ_i and the mean monthly discharge or the river flow velocity U_F in the whole lower reach of the river could be assumed to be constant.

These results will be used to determine: i) the damping or the attenuation of tidal amplitude; ii) the reduction of the tidal celerity; and iii) the variation of the mean monthly water levels along the river.

The attenuation of the four predominant constituents M_2, S_2, K_1 and O_1 of the tide along the river in April, August, September and October 1980 is shown in Fig. 2a, while that in 1983 is shown in Fig. 2b. The attenuation can be expressed in exponential form in the same manner of Eq. 5 as,

$$a(x) = a_0 \exp(-\mu x) \quad ....(11)$$

It can be seen from these two figures that the slope of the plot or the damping modulus of the semi-diurnal tide is about twice that of the diurnal tide for the same river flow. Since the wavelength of the semi-diurnal tide at the river mouth is about half that of the diurnal tide, the product of the damping modulus and the wavelength of the tide at the river mouth L_0 for a given river flow velocity U_F is about the same for the semi-diurnal tide and the diurnal tide. The results of the analysis are summarized in Table 1. This dimensionless damping modulus is found to correlate well with a Froude number $U_F/\sqrt{gh}$ shown in Fig. 3a as

$$\mu/k_0 = 30(U_F/\sqrt{gh}) \quad \text{for } U_F/\sqrt{gh} > 0.022 \qquad .....(12)$$

$$= 0.66 \quad \text{for } U_F/\sqrt{gh} \leq 0.022$$

in which $k_0 = 2\pi/L_0$ = tide number at the river mouth; and h = mean water depth = 9.2 m. The above relationship indicates that the damping modulus increases when

Table 1 Present Results

Tide (1)	Period Month/Yr (2)	u_0(Eq.10) (m/s) (3)	U_F (m/s) (4)	$U_F/\sqrt{gh}$ (5)	μ (km^{-1}) (6)	μ/k_0 (7)	c/c_0 (8)
M_2	4/1980	0.674	0.175	0.0184	0.0084	0.570	0.645
M_2	8/1980	0.656	0.369	0.0388	0.0124	0.840	0.608
M_2	9/1980	0.688	0.528	0.0556	0.0166	1.120	0.531
M_2	10/1980	0.659	0.760	0.0800	0.0327	2.210	0.408
M_2	4/1983	0.654	0.165	0.0174	0.0110	0.740	0.645
M_2	8/1983	0.614	0.299	0.0315	0.0115	0.780	0.576
M_2	9/1983	0.648	0.363	0.0382	0.0142	0.960	0.566
M_2	10/1983	0.628	0.650	0.0684	0.0258	1.740	0.575
S_2	4/1980	0.365	0.175	0.0184	0.0087	0.570	0.684
S_2	8/1980	0.366	0.369	0.0388	0.0107	0.700	0.670
S_2	9/1980	0.391	0.528	0.0556	0.0210	1.370	0.636
S_2	10/1980	0.297	0.760	0.0800	0.0315	2.060	1.764
S_2	4/1983	0.381	0.165	0.0174	0.0110	0.720	0.686
S_2	8/1983	0.334	0.299	0.0315	0.0132	0.860	0.705
S_2	9/1983	0.389	0.363	0.0382	0.0138	0.900	0.649
S_2	10/1983	0.347	0.650	0.0684	0.0262	1.710	0.685
K_1	4/1980	0.516	0.175	0.0184	0.0068	0.880	0.561
K_1	8/1980	0.636	0.369	0.0388	0.0081	1.050	0.572
K_1	9/1980	0.487	0.528	0.0556	0.0118	1.530	0.525
K_1	10/1980	0.532	0.760	0.0800	0.0277	3.600	0.430
K_1	4/1983	0.574	0.165	0.0174	0.0072	0.940	0.557
K_1	8/1983	0.644	0.299	0.0315	0.0085	1.100	0.604
K_1	9/1983	0.604	0.363	0.0382	0.0087	1.130	0.510
K_1	10/1983	0.634	0.650	0.0684	0.0212	2.750	0.531
O_1	4/1980	0.319	0.175	0.0184	0.0056	0.790	0.706
O_1	8/1980	0.317	0.369	0.0388	0.0092	1.300	—
O_1	9/1980	0.375	0.528	0.0556	0.0122	1.720	0.766
O_1	10/1980	0.455	0.760	0.0800	0.0249	3.510	1.604
O_1	4/1983	0.391	0.165	0.0174	0.0067	0.940	0.579
O_1	8/1983	0.416	0.299	0.0315	0.0087	1.230	0.682
O_1	9/1983	0.530	0.363	0.0382	0.0095	1.340	0.594
O_1	10/1983	0.531	0.650	0.0684	0.0213	3.000	4.099

Table 2 Results from WES Tidal Flume

(h = 15.24 cm and T = 600 s)

μ/k_0 (1)	$U_F/\sqrt{gh}$ (2)	a_0/h (3)	c/c_0 (4)	Symbol (5)
0.54	0	0.10	0.78	0
0.54	0.005	0.10	0.82	0
0.58	0.005	0.10	0.75	0
0.65	0.005	0.15	0.75	Φ
0.73	0	0.20	0.71	⊖
0.73	0.005	0.20	0.73	⊖

the fresh water flow velocity increases. The dimensionless damping modulus $\mu/k_o = 0.66$ when the Froude number $U_F/\sqrt{gh} \leq 0.022$ for a small river flow velocity U_F; this is the effect of frictional force. Note that the data of μ/k_0 obtained by Ippen and Harleman (1966) from tests in a tidal flume at the Waterways Experiment Station at Vicksburg for very low Froude numbers as listed in Table 2 are also plotted in Fig. 3a. These flume data have the same magnitude of μ/k_o as the present results.

It can be seen from Eq. 1 that the phase of the tide δ_i controls the times of occurrence of high and low waters, and it is, in general, independent of the amplitude of the tide a_i for the weak effects of the river flow. However, both the phase and the amplitude of the tide are modified by the river flow.

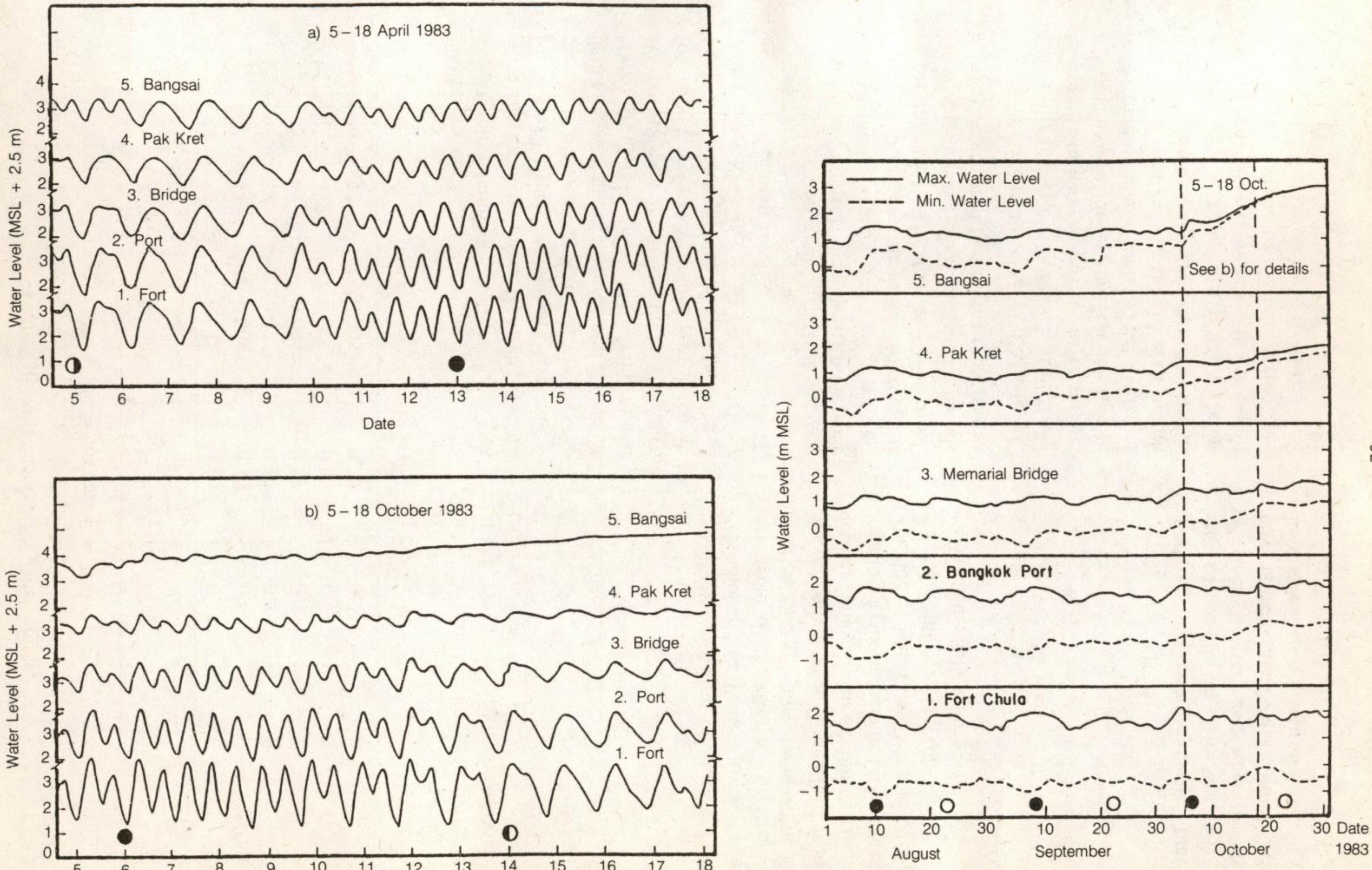
It is evident, by equating Eq. 1 to Eq. 5, that the phases of the tides δ_i at different recording stations would be different, i.e. it would take a longer time for the tide to travel to the inner station. The celerity of each constituent of the tide in each month is then determined, $c = 2\pi(\triangle x)/\{[(\delta_i)_o - \delta_i]T_i\}$, in which $(\delta_i)_0$ = phase of the tide for constituent i at the river mouth with the period T_i; and δ_i = phase of the tide at the inner station with the distance $\triangle x$ from the river mouth. When the obtained celerity is normalized by $c_0 = \sqrt{gh}$, the ratio of the celerities are

$$\frac{c}{c_0} = \frac{k_0}{k} = \frac{2\pi(\triangle x)}{\sqrt{gh}\ [(\delta_i)_0 - \delta_i]\ T_i} \qquad \text{.........................(13)}$$

The celerity ratios thus obtained for Station 5 are tabulated in Table 1 using the mean water depth of the river h = 9.2 m. The theoretical celerity ratios computed by Eq. 8, using the values of μ/k_0 obtained from the preceding analysis, have almost the same magnitudes as those computed from the phases of the tides. When they are correlated with the Froude number, $U_F/\sqrt{gh}$, as shown in Fig. 3b, their relationship is

$$\begin{aligned} \frac{c}{c_0} = \frac{k_0}{k} &= 1 - 8.2(U_F/\sqrt{gh}) \quad \text{for } U_F/\sqrt{gh} > 0.024 \\ &= 0.80 \quad \text{for } U_F/\sqrt{gh} \leq 0.024 \end{aligned} \qquad \text{.....................(14)}$$

The above relationship indicates that the celerity of the tide decreases when the river flow velocity U_F increases when the Froude number is greater than 0.024. But the celerity ratio approaches to a constant of 0.80 when



c) Envelopes of Maximum and Minimum Water Levels

***Fig. 1** Observed Tides in a) Dry Season b) Rainy Season and c) Envelopes of Maximum and Minimum Water Levels*

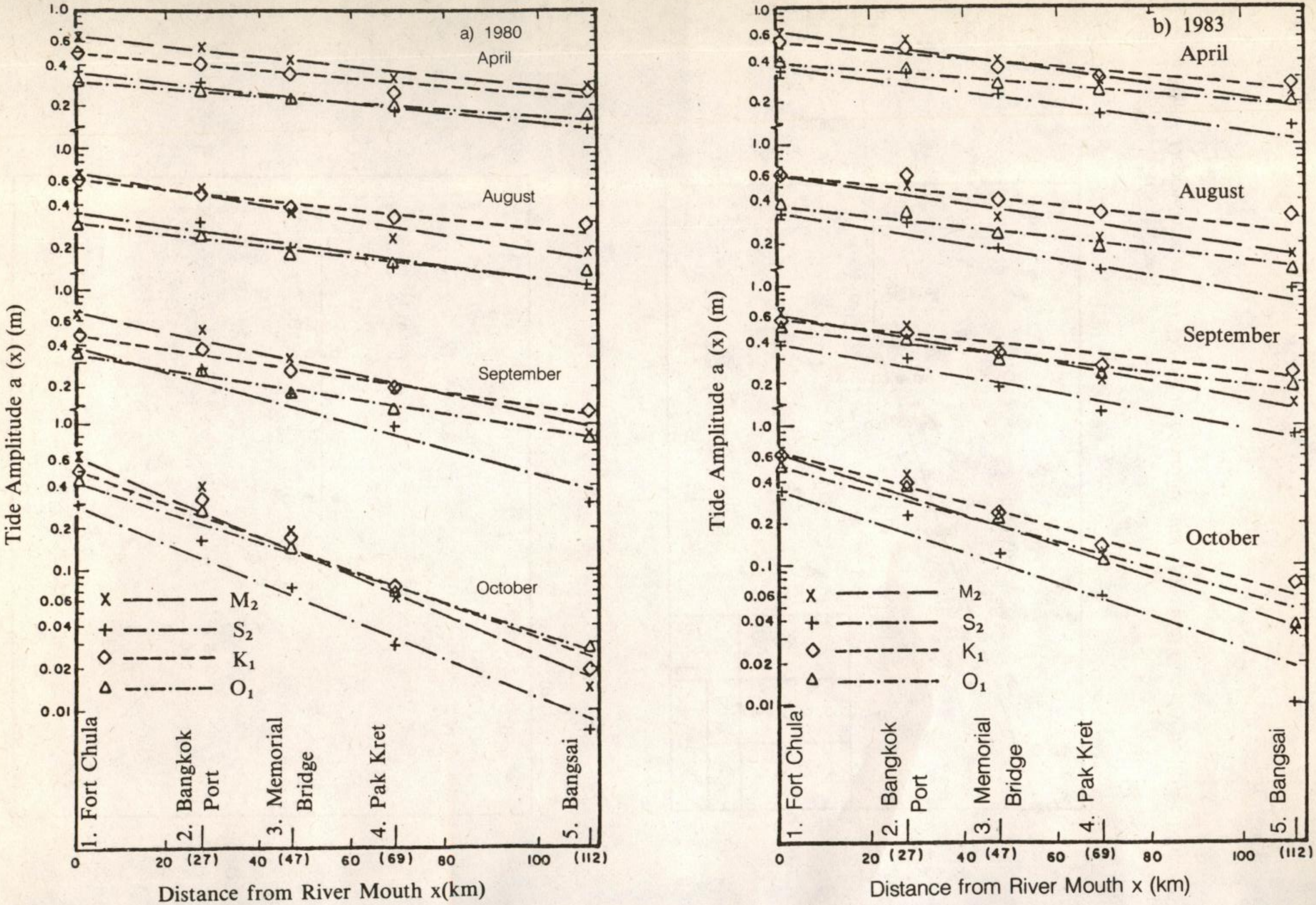


Fig. 2 Attenuation of Tides in a) 1980 and b) 1983

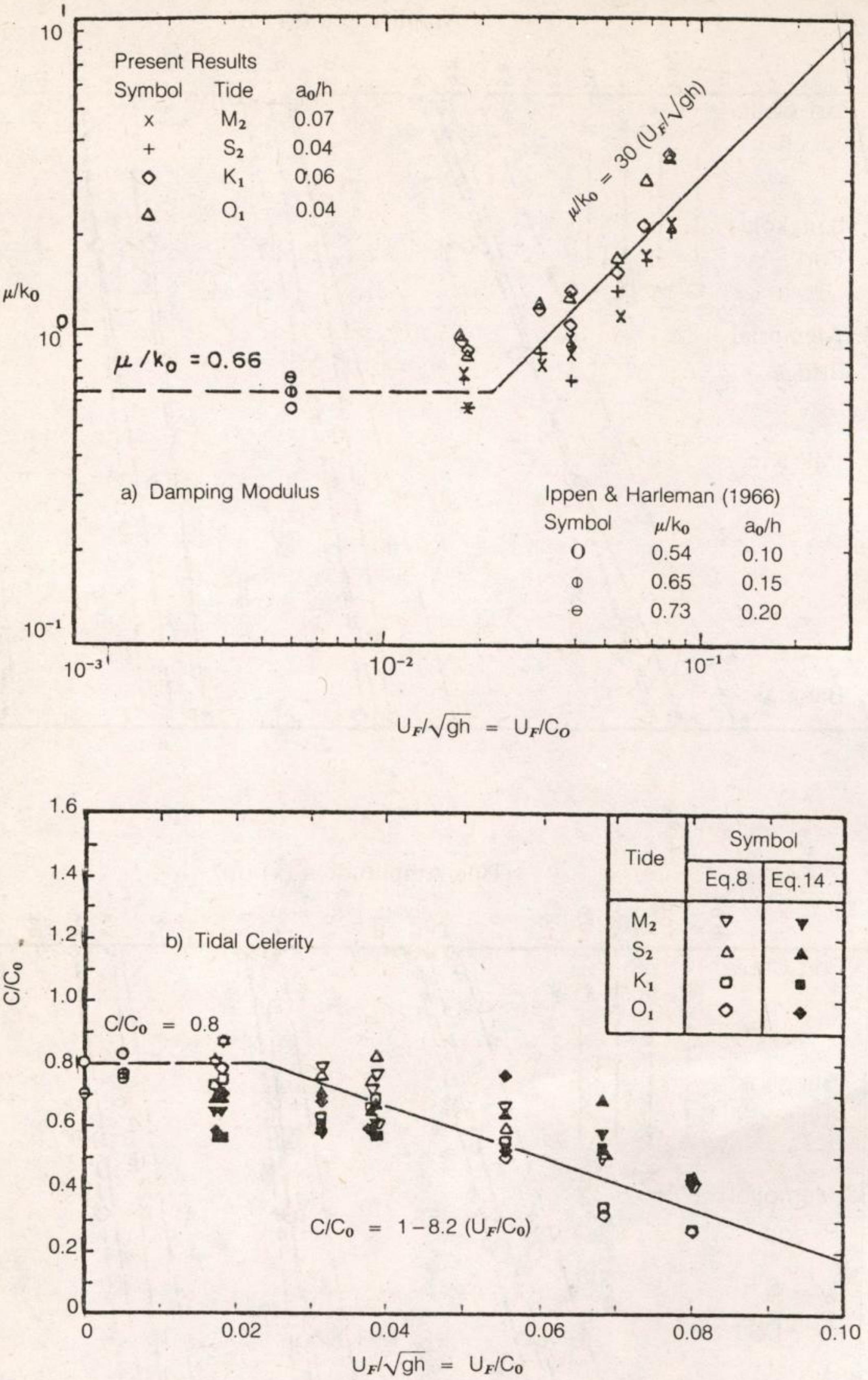
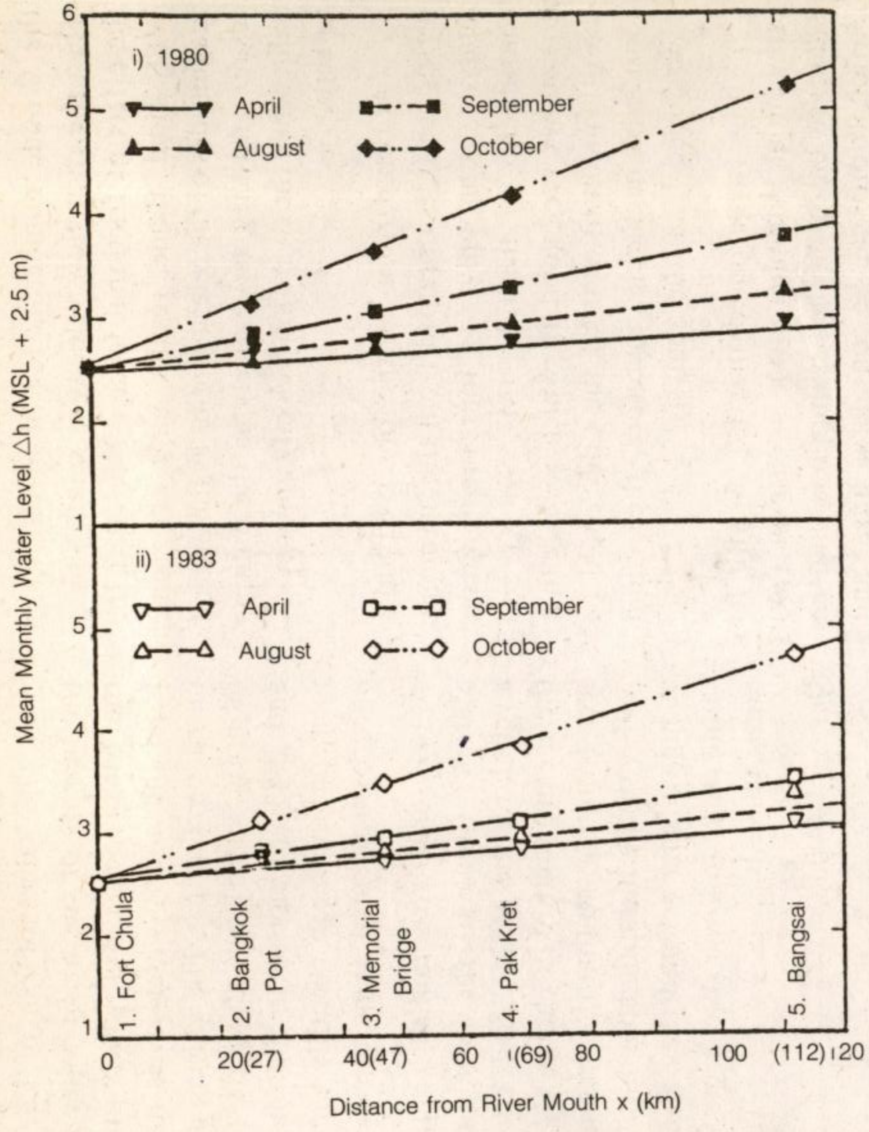
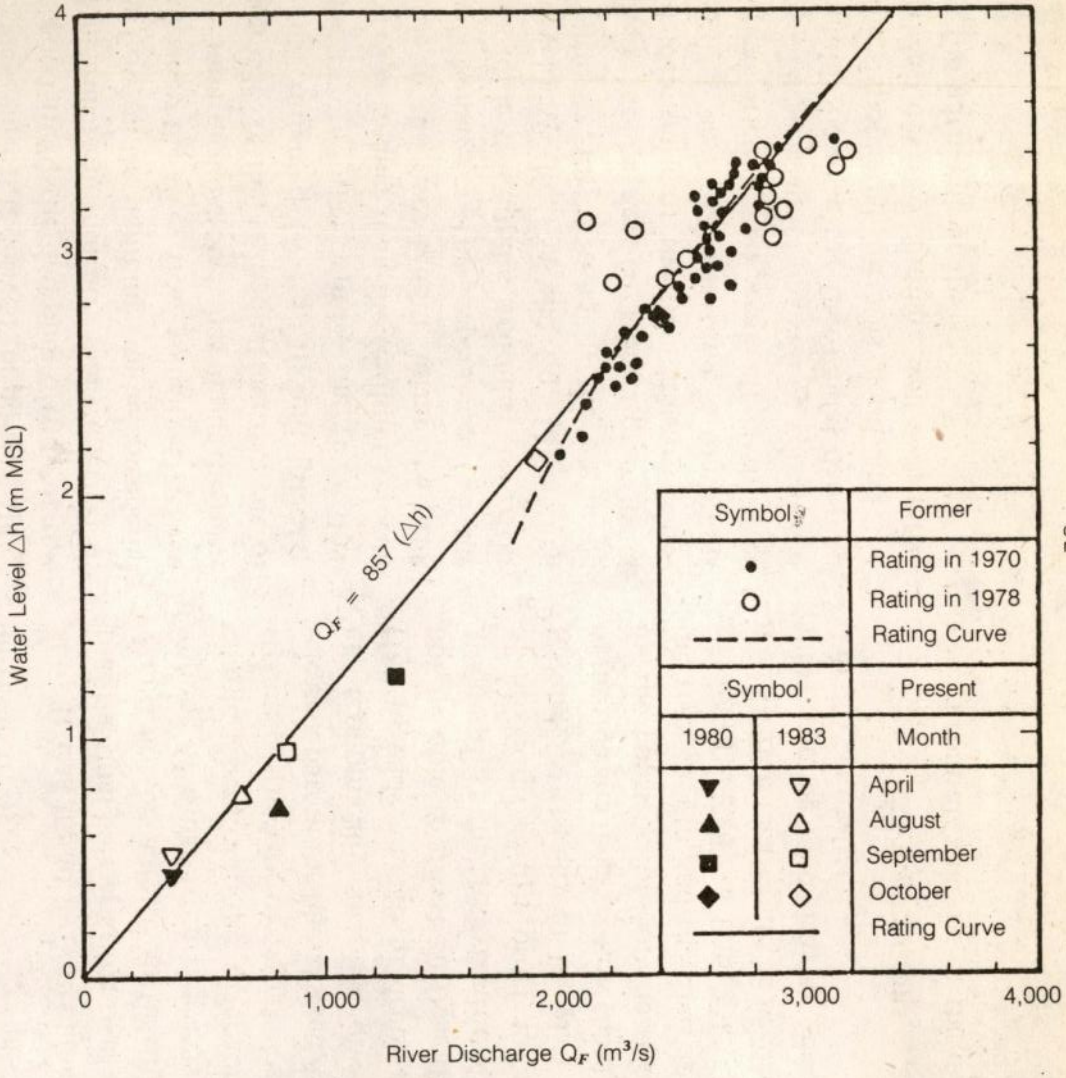


Fig. 3 Dependence of a) Damping Modulus and b) Tidal Celerity on River Flow



a) Variation of Mean Monthly Water Levels along the River



b) Rating Curve at Bangsai

Fig. 4 a) Variation of Mean Monthly Water Levels along the River and b) Rating Curve at Bangsai

the Froude number is less than 0.024, or when the river flow velocity is small; this is the effect of frictional force. Note that the WES tidal flume results have the same magnitudes of the celerity ratios as the present results and they serve nicely as the limiting value when the river flow velocity approaches to zero.

A comparison of the observed tides versus the tides obtained from the harmonic analysis using only the four predominant constituents of the tides (M_2, S_2, K_1 and O_1) shows very good fit which reveals that the other constituents of the tides could be considered negligible, therefore, they could be excluded from the analysis.

It is to be noted that Eq. 5 with its exponential damping $a(x) = a_0 \exp(-\mu x)$ is derived based on the assumptions that the frictional force can be considered linear and the damping is not explicitly dependent on river flow since it is derived for no river flow. The good fit of the damping of tide of this exponential form indicates that the effect of river flow on the damping can be considered weak since the river flow velocity U_F, even in the month of October, has about the same magnitude as the individual component of tide u_0 computed by Eq. 10 and listed in Table 1. In other words, the river flow velocity is smaller than the resultant velocity of tide at the river mouth during the peak river discharge. The dependence of the dimensionless damping modulus on river flow velocity (Fig. 3a and Eq. 12) is then established. Eq. 1 used in the harmonic analysis can be derived from Eq. 5 for a fixed location x of the tidal station, therefore, it is based on the same assumptions of linear friction and weak effect of river flow. These assumptions can be considered acceptable because the fit of tidal damping to the exponential form is good and the observed tides also compare well with those from the harmonic analysis.

The values of $\triangle h$ of Eq. 1 represent the heights of the mean water levels above the Mean Sea Level which are computed from the hourly water levels in any month based on Eq. 2. The raising of the mean monthly water levels along the river in 1980 and 1983 are plotted in Fig. 4a. It can be seen that the mean monthly water levels are linearly distributed along the river.

In order to establish a rating curve at Station 5 (Bangsai), the mean monthly water level above the MSL ($\triangle h$) is correlated with the mean monthly discharge $\bar{Q}_F$. It can be seen from Fig. 4b that the present data agree well with the instantaneous measured values at high discharges and they also extend the rating curve to lower discharges nicely. The rating curve is

$$\bar{Q}_F = 857 \ (\triangle h) \qquad (15)$$

in which units of Q_F and $\triangle h$ are m^3/s and m respectively.

It was found by Vatcharasinthu (1977) that the tidal barrier at the river mouth alone would not be able to sufficiently drain the volume of water from the severe flood discharge, therefore, the diversion canal upstream of the city of Bangkok is needed to release a part of the high discharge. Similarly, an economically feasible diversion canal (180 m wide and 10 m deep) alone would not be able to divert sufficiently (less than 2,000 m^3/s) the 100-year design discharge of 3,600 m^3/s. In order to alleviate the flooding of the city of Bangkok, the effects of the tide and the river flow should be reduced by the constructions of the tidal barrier at the river mouth and the diversion canal upstream of the city of Bangkok. In addition, a navigation lock would be needed for cargo-ships coming and leaving Bangkok Port (km 27).

Conclusions

From an analysis of the interaction of

tide and river flow, the following conclusions can be drawn:

1. The complicated interaction of river flow with various constituents of tide could be simplified to its interaction with individual constituents obtained from a harmonic analysis; four predominant constituents were found sufficient to represent the resultant tide.
2. The river flow did have effects on raising the mean water levels along the river; the river flow together with the bottom friction damped the amplitude and reduced the celerity of tide in the river. However, the interaction of river flow and tide was found to be weak, therefore, the analytical expressions of damped tide by the linear frictional force without river flow, Eqs. 5 and 8 derived by Ippen and Harleman (1966), were found to suitably fit the field data. The obtained parameters describing the damping of tidal amplitude, the reducing of tidal celerity and the raising of mean water level were later correlated with the river flow velocity normalized by the celerity of tide without friction.
3. The dimensionless damping modulus of each constituent of tide was found to follow the same empirical expression, Eq. 12. The damping modulus depended on the Froude number, with the fresh water velocity and the mean water depth along the river as the characteristic velocity and length respectively, i.e. the higher fresh water velocity resulted in the higher damping of the tide; the dimensionless damping modulus of the WES tidal flume tested by Ippen and Harleman (1966) agreed with the current results.
4. The reduced celerity of each constituent of tide was found to follow the same empirical expression, Eq. 14. The ratio of the celerities depended also on the Froude number, i.e. the higher river flow velocity resulted in the higher reduction of the celerity.
5. The raising of the mean water level at the upstream boundary was found linearly dependent on the mean monthly fresh water discharge, Eq. 15, and the raising of the mean water levels from the river mouth to the upstream boundary was also found linearly distributed, Fig. 4a.
6. The raising of the mean water level due to the fresh water flow was found to be the major cause of the flooding along the river especially when it was superimposed by high tide from the river mouth. The raising of the mean water level and the reduction of the tidal range caused difficulty in draining the rain water in the city to the river.

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