ORIGINAL ARTICLE

The interaction between tide and salinity barriers

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Abstract

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Presently, there is a number of salinity barrier utilization and this kind of structure becomes more common in estuarine areas. However, the construction of barrier at the river mouth or inside the river results in amplification of tide due to creation of standing tide at the barrier. This standing tide creates two major problems, namely, the overspill of salinewater during high water and bank erosion during low water along the tidal reach downstream of the barrier. In this study, the analytical model is developed to determine the river hydraulic behaviors which affects by tide, river flow and barrier structure of the Bang Pakong River, Thailand. The analytical model of tide and river flow of the Chao Phraya River is adopted and adjusted to determine the tide characteristics modified by river flow. Moreover, the analytical model of tide and salinity barrier would then be developed by cooperating of the analytical model of tide and river flow interaction together with tidal flow cooscillating tide theory. It is found from this study that the analytical model of the Chao Phraya River which is suitable for high freshwater discharge underestimates damping modulus and friction slope which requires adjustment for low freshwater discharge of the Bang Pakong River. The analytical model of tide and salinity barrier can be finally used to predict the water level downstream of the barrier. The model overestimated the water level fluctuation during the unsteady flow from upstream which may be because of the assumption of steady flow condition in the model development due to limited data available after the construction.

Key words : analytical model, numerical model, salinity barrier, tide reflection

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บทคัดย่อ

สุภัทท์ วงศ์วิเศษสมใจ และ ภัทรชนก ศรีวิหค
การกระทำระหว่างคลื่นน้ำขึ้นน้ำลงกับคันกั้นน้ำเค็ม
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คันกั้นน้ำเค็มมีการใช้งานในหลายรูปแบบและปัจจุบันมีการก่อสร้างหลายแห่ง แต่การก่อสร้างคันกั้นน้ำเค็มที่ปากแม่น้ำหรือในแม่น้ำทำให้คลื่นน้ำขึ้นน้ำลงสะท้อนกลับเพิ่มความสูงของระดับน้ำอีกเท่าตัวและลดระดับน้ำลงต่ำสุดไปอีกเท่าตัว เป็นผลให้น้ำเค็มล้นตลิ่ง และตลิ่งพังทลายในช่วงน้ำลง ในการศึกษานี้ได้พัฒนาแบบจำลองทางทฤษฎีเพื่อจำลองการกระทำระหว่างคลื่นน้ำขึ้นน้ำลงกับอัตราการไหลของน้ำและคันกั้นน้ำเค็มในแม่น้ำบางปะกง โดยปรับปรุงแบบจำลองทางทฤษฎีของแม่น้ำเจ้าพระยาของการกระทำระหว่างคลื่นน้ำขึ้นน้ำลงกับอัตราการไหลของน้ำ แล้วนำไปพัฒนาให้ใช้ได้กับการกระทำระหว่างคลื่นน้ำขึ้นน้ำลงกับคันกั้นน้ำเค็ม โดยใช้หลักการสะท้อนกลับของคลื่นเมื่อปะทะกับคันกั้นน้ำเค็ม จากการศึกษาพบว่าแบบจำลองทางทฤษฎีของแม่น้ำเจ้าพระยาซึ่งใช้ได้ดีมากกับแม่น้ำเจ้าพระยาซึ่งมีอัตราการไหลของน้ำจืดสูงใช้ได้ไม่ค่อยดีกับแม่น้ำบางปะกง ซึ่งมีอัตราการไหลของน้ำจืดต่ำมาก จึงต้องมีการปรับแก้ค่าสัมประสิทธิการสลายตัวและค่าความลาดเทของระดับผิวน้ำ เพื่อนำผลมาพัฒนาแบบจำลองทางทฤษฎีของการกระทำระหว่างคลื่นน้ำขึ้นน้ำลงกับคันกั้นน้ำเค็มก็ได้ผลไม่สู้ดีเพราะมีข้อมูลจำกัด

สาขาวิศวกรรมแหล่งน้ำและการจัดการ สำนักวิชาวิศวกรรมโยธา สถาบันเทคโนโลยีแห่งเอเชีย ตู้ ปณ.4 อำเภอคลองหลวง จังหวัดปทุมธานี 12120

The Bang Pakong Diversion Dam is located at Amphur Muang, Chachoengsao province which is 62 km from the Bang Pakong river mouth as shown in Figure 1. The construction of the dam was completed in November 1999 and its operation started in January 2000. There are three floodgates and two regulating gates. All the gates are supposed to be closed during the low flow period in the dry season to stop salinewater intrusion and store freshwater for water supply and irrigation purposes. Soon after the commissioning of the dam, it was found that the closure of all the gates resulted in an excessive amplification of tidal fluctuations, and caused sliding of the river banks at several locations downstream of the dam as well as overspill of salinewater and flooding problems. Consequently, The Royal Irrigation Department decided to fully open all the gates, thus deferring the use of the dam. This demonstrates that designing and evaluating the salinity barrier operation require incisive comprehension of the river hydraulic behavior under the natural condition as well as under various gate operation scenarios in order to prevent the unfavorable effects while attempting to meet the main purposes. In practice, water level downstream of the barrier should have been predicted before barrier construction and operation (JICA, 2001).

Theoretical Considerations

1. Mathematical Description of Tides with Friction

Ippen and Harleman (1966) investigated analytically the characteristics of damped tides with a linear frictional force. The treatment is restricted to the case of rectangular cross-section of width b, of local mean depth h, and of horizontal mean surface, to which temporal elevation changes $\pm\eta$ are referred. Amplitudes of the tidal waves are assumed small as compared to depth and velocities of flow as uniform over the depth.

For tide entering a uniform channel of infinite length with the channel extending from the ocean entrance at $x = 0$ to infinity, the initial and maximum amplitude is a_0, which according to the known solution is exponentially damped with progress of the tide into the channel by damping

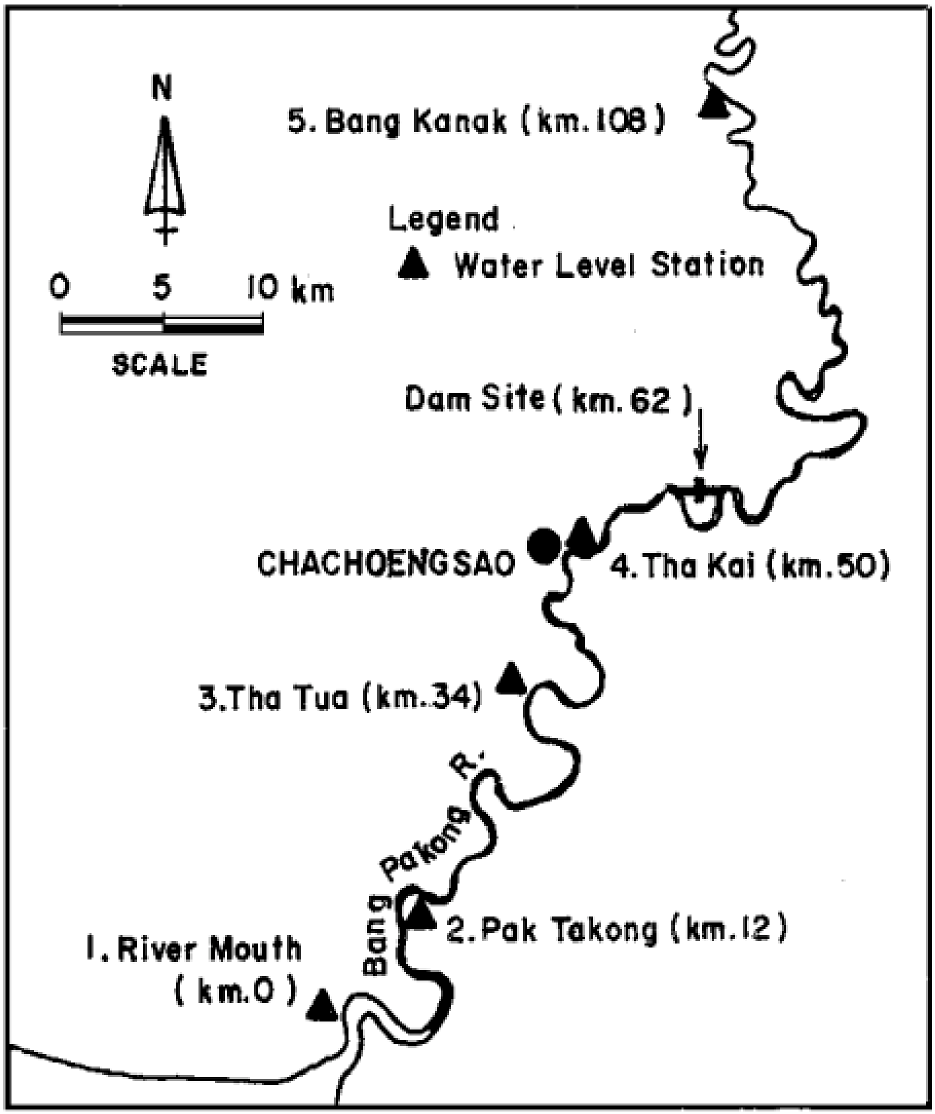


Figure 1. The Bang Pakong River.

modulus μ. The solution is

$$\eta(x,t) = a_o e^{-\mu x} \cos(\sigma t - kx) \quad (1)$$

where $\sigma = 2\pi/T$ is tidal angular frequency, T = period of tide, $k = 2\pi/L$ is tide number, L = length of tide. For frictionless fluid, $\mu = 0$, Eq(1) is simplified as $\eta = a_0 \cos(\sigma t - k_0 x)$ where $k_0 = 2\pi/L_0$ is tide number of frictionless fluid and L_0 = length of tide of frictionless fluid; the celerity of tide $C_0 = L_0/T$. Friction imposes an additional phase shift, since $k>k_0$ and hence $L<L_0$ for $C<C_0$. It is noted that the ratio μ/k must be constant throughout the channel. The velocity, u, can be expressed as:

$$u(x,t) = \frac{a_0}{h} C_0 e^{-\mu x} \frac{k_0}{\sqrt{\mu^2 + k^2}} \cos(\sigma t - kx + \alpha) \quad (2)$$

where α is phase angle of tidal velocity. For cooscillating tides, the incident tide η_i and reflected tide η_r are functions of x and t and are at all times equal at the reflecting station at $x = 0$. The time t is measured from the occurrence of high water at $x = 0$, at which time $\eta_i = \eta_r = a_0$ and at $x = -L/4$ (a quarter of tidal length) $\eta_i = \eta_r = 0$ (See Figure 2). The tidal elevation at any time t and at any station x in the channel is therefore

$$\eta = \eta_i + \eta_r = a_0[e^{-\mu x}\cos(\sigma t - kx) + e^{\mu x}\cos(\sigma t + kx)] \tag{3}$$

where the phase angle of tidal velocity can be expressed as:

$$\alpha = \tan^{-1}\left(\frac{\mu}{k}\right) \tag{4}$$

and tidal celerity of frictionless fluid is

$$C_0 = \sqrt{gh} = \frac{L_0}{T} = \frac{\sigma}{k_0} \tag{5}$$

2. Analytical Model of Tide and River Flow Interaction

Vongvisessomjai and Rojanakamthorn (2000) developed an analytical model of interaction of tide and river flow. The unsteady one-dimensional equation of continuity and the equation of motion are used to describe the interaction of tide and river flow taking into account the convective inertia force and bottom frictional force. The perturbation method is used to linearize this nonlinear problem. The actual water surface and flow velocity are the resultant of the zeroth order solution representing the steady flow solution from the freshwater discharge and the first order solution representing the resultant of all tide constituents. The general hydrodynamic characteristics can be subsequently developed to the following forms.

The water surface fluctuation

$$\eta(x,t) = \eta_0 + \eta_1 \tag{6}$$

where

$$\eta_0 = S_{fo}x + \Delta h_0$$

$$\eta_1 = \sum_{i=1}^{N} a_{oi} e^{(-\mu_i x)} \cos\left(\frac{2\pi t}{T_i} - k_i x + \delta_{oi} - \frac{\pi}{2}\right)$$

$$S_{fo} = \frac{\eta_0(x)}{x} = \frac{U_f^2}{C_z^2(h + \eta_0(x))} \tag{7}$$

The total discharge is the sum of freshwater discharge Q_f and tidal discharge Q_T

$$Q(x,t) = Q_f + Q_T \tag{8}$$

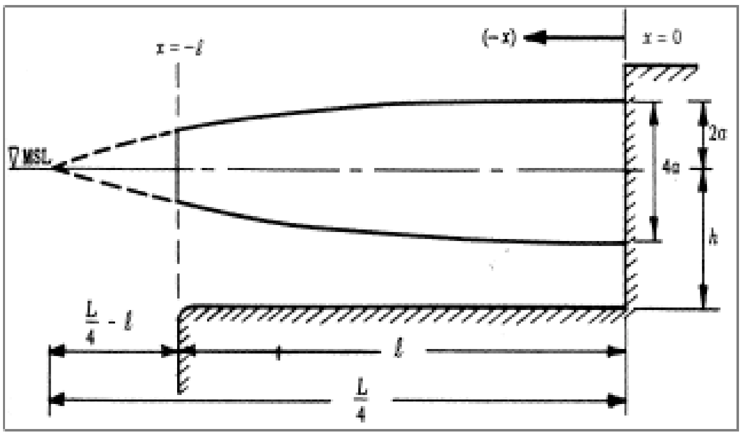


Figure 2. Tide entering channel with reflecting end.

where

$$Q_T = A_e \sum_{i=1}^{N} \frac{a_0}{h} C_0 e^{(-\mu_i x)} \frac{k_0}{\sqrt{\mu_i^2 + k_i^2}} cos(\frac{2\pi t}{T_i} - k_i x + \alpha_{oi} - \frac{\pi}{2})$$

$$\alpha_{oi} = \delta_{oi} + \xi \qquad (9)$$

in which S_{fo} is friction slope, Δh_0 is mean water level at the river mouth station, η_0(x) is mean water level at distance x compared to mean water level at the river mouth, Q_f is freshwater discharge, U_f is freshwater velocity, A_e is effective cross sectional area which computed from average discharge and average flow velocity, δ_{oi} is phase of water surface fluctuation of tide for constituent i at the river mouth, ξ is phase difference between water surface fluctuation and tidal flow at the river mouth.

Normally, phase of water surface fluctuation of tide for constituent i at the river mouth, δ_{oi} and phase of tidal flow for constituent i at the river mouth, α_{oi} can be obtained from harmonic analysis of time series of water level and time series of discharge at the river mouth, respectively. However the results from harmonic analysis of water level, δ_{oi} and discharge at the river mouth station, α_{oi}, are generally computed based on sine function whereas the analytical solution Eq.(6) and Eq.(8) are derived based on cosine function. Therefore, phase parameter from harmonic analysis requires adding the term $-\pi/2$ before applying to the analytical solution. The magnitude of μ_i and k_i in Eq.(6) and Eq.(8) can be determined from the analytical model of tide and river flow interaction with the known values of Froude Number, U_f/C_0 and constant value, $g/(C_z^2 k_0 h)$. The values of μ and k can be obtained by trial and error using the following two equations:

$$\frac{\mu}{k_0} = \frac{g}{C_z^2 k_0 h}\left[\frac{3}{2}\frac{k}{k_0}\left(\frac{U_f}{C_0}\right)^2 - \frac{U_f}{C_0}\right] \Bigg/ \left[\frac{k}{k_0} - \frac{k}{k_0}\left(\frac{U_f}{C_0}\right)^2 + \frac{U_f}{C_0}\right] \qquad (10)$$

$$\frac{k}{k_0} = \frac{C_0}{C} = \frac{\sqrt{1+\left(\frac{\mu}{k_0}\right)^2 - 2\left(\frac{\mu}{k_0}\frac{U_f}{C_0}\right)^2 + \left(\frac{\mu}{k_0}\right)^2\left(\frac{U_f}{C_0}\right)^4 - \frac{3g}{C_z^2 k_0 h}\frac{\mu}{k_0}\left(\frac{U_f}{C_0}\right)^2\left[1-\left(\frac{U_f}{C_0}\right)^2\right]} - \frac{U_f}{C_0}}{1-\left(\frac{U_f}{C_0}\right)^2} \qquad (11)$$

These two equations show that the Froude number which is the ratio of freshwater flow velocity and tidal celerity affects the amplitude damping of tide and tide number or celerity of tide propagating into the river.

Analysis and Results

1. Analytical Model of Tide and River Flow in the Bang Pakong River

The mathematical solutions are applied to the tidal reach of the Bang Pakong River from Bang Kanak to river mouth approximately 108 km apart. The time interval of one month is taken for this analysis in order to minimize the error due to the harmonic analysis and for convenience. The mean monthly discharge and river flow velocity in the tidal reach of the river is assumed to be constant for whole month. Four predominant constituents of the tide in the Bang Pakong River are used in the analysis and their characteristics are shown in Table 1.

According to the analytical model of tide and river flow interaction, (Vongvisessomjai and Rojanakamthorn, 2000), harmonic analysis is firstly applied to the existing water level and

Table 1. Tidal constituent at the Bang Pakong River Mouth.

Constituent	Period (hr)	k_0 (m^{-1})
Principal lunar M_2	12.4206	0.0000142
Principal solar S_2	12.0000	0.0000147
Luni-Solar declinational K_1	23.9346	0.0000074
Large lunar declinational O_1	25.8194	0.0000068

discharge data for decomposing each tidal constituent. The interaction simultaneously of each tidal constituent with the river flow can then be determined. The geometry of the Bang Pakong River are shown in Figure 3 and hydraulic parameters of the Bang Pakong River can be defined and summarized as follow

1) Average area of the tidal reach, A=3,400 m^2 (at 0 m MSL)

2) Average water depth, h=10 m (at 0 m MSL)

3) Chezy's C_z = 44.5 $m^{1/2}/s$ or Manning n = 0.033 $s/m^{1/3}$

4) Range of Froude Number $|U_f / C_0|$ < 0.015

5) $g/(C_z^2 k_0 h)$ = 35, 34, 67 and 73 for the constituents M_2, S_2, K_1 and O_1 respectively

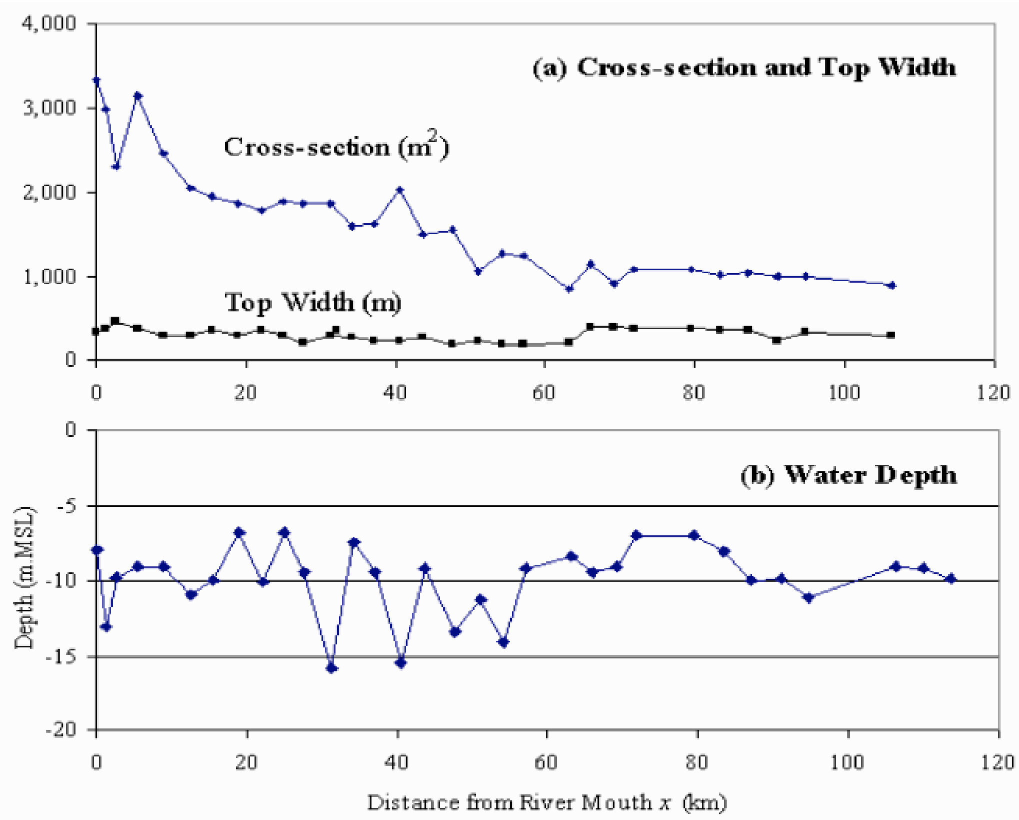
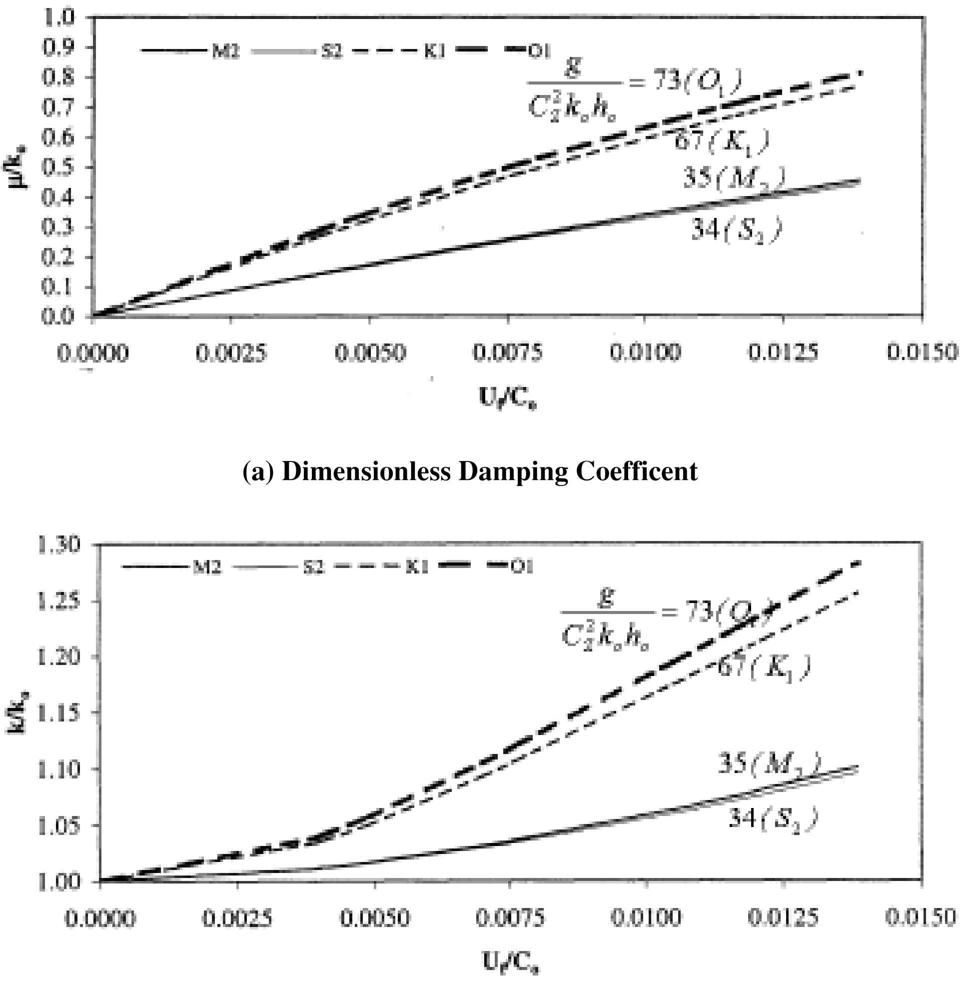


Figure 3. Geometry of the Bang Pakong River.

Using the above geometry and hydraulic parameters, Eqs.(10) and (11) are used to compute μ/k_0 and k/k_0 as plotted in Figure 4. The observed water levels variation with time at various station in the Bang Pakong River are shown in Figure 5. From harmonic analysis of water level and discharge at the river mouth, it can be determined that the value of phase difference between water level and discharge parameter (ξ) is constant for semi-diurnal tide component M_2, S_2 and diurnal tide component K_1, O_1 for every season.

For the Bang Pakong River, this constant value is approximately -2.4 radian for semi-diurnal tide components, M_2 and S_2, and approximately -2.1 radian for diurnal tide components, K_1 and O_1.

The correlation of Froude number and dimensionless damping modulus coefficient together with the correlation of Froude number and dimensionless tide number computed by the analytical model of tide and river flow interaction in the Bang Pakong River were made by Srivihok



(a) Dimensionless Damping Coefficent

(b) Dimensionless Tide Number

Figure 4. Analytical solution of effect of Froude number to: (a) Dimensionless damping coefficient and (b) Dimensionless tide number.

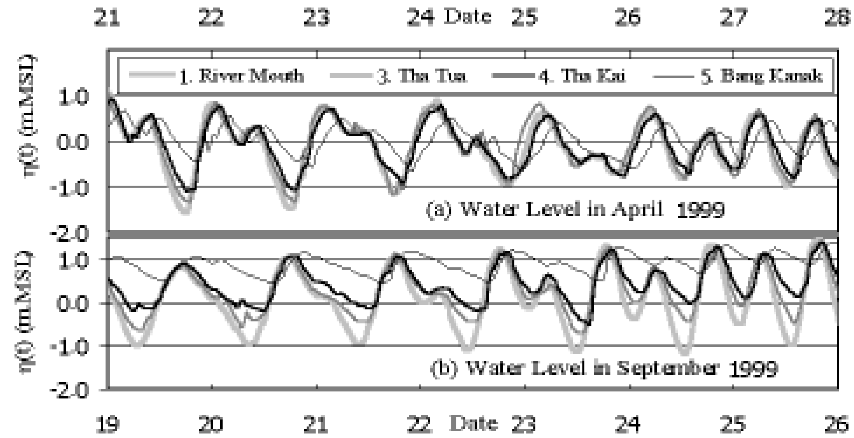


Figure 5. Observed tide variation with time in the Bang Pakong River.

(2002).

The observed hourly water levels of five stations which are River Mouth , Pak Takong, Tha Tua, Tha Kai Bang Kanak station along the Bang Pakong River in January1999 to January 2000 are used to compute the mean water level Δh, the amplitude a_i and phase δ_i. The amplitude damping modulus coefficient of each constituent in each month can be calculated from slope of the semi-log graph between distance from river mouth and tidal amplitude at that distance. The attenuation can be expressed in exponential form which is

$$a(x) = a_0 \exp(-\mu x) \qquad (12)$$

The attenuation of tide in the Bang Pakong River are shown in Figure 6 and can be observed that the amplitude damping modulus coefficient (μ) of component K_1 generally shows the largest value while that of component M_2 is the second. It is

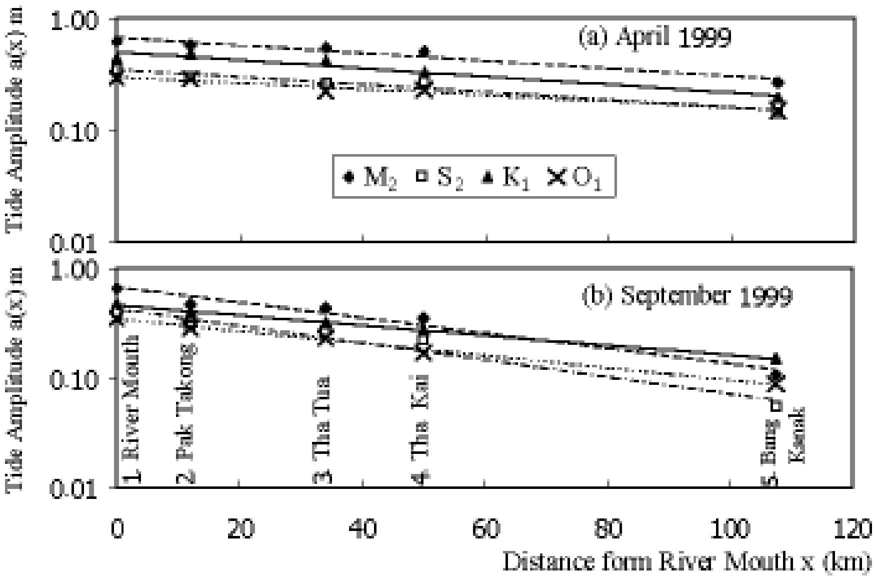


Figure 6. Attenuation of tide in the Bang Pakong River.

also found that the S_2 component gives the least amplitude damping modulus coefficient.

Normally, both phase and the amplitude of the tide are modified by the river flow. This can be seen from the harmonic analysis results, which the phases δ_i of the tides at each recording station are different. The celerity of each constituent of the tide in each month is then determined:

$$C = \frac{2\pi\Delta x}{\left[(\delta_i)_0 - \delta_i\right]T_i} \tag{13}$$

then

$$\frac{C}{C_0} = \frac{k_0}{k} = \frac{2\pi\Delta x}{\left[(\delta_i)_0 - \delta_i\right]T_i\sqrt{gh}} \tag{14}$$

where $(\delta_i)_0$ is the phase of the tide for constituent i at the river mouth with the period T_i, δ_i is the phase of the tide at the inner station with the distance Δx from the river mouth and h is the mean water depth of the river. Analysis results can be seen in Srivihok (2002).

From the analytical model comparing with observed data analysis, it can be investigated that the analytical model gives the underestimated of dimensionless damping modulus coefficient and dimensionless tide number when compared with the value form observed data analysis. By the nature, river discharge, bottom friction and changing of the river geometry are such the complex and implicit factors to damp tide amplitude and to decrease of tide celerity. However the analytical model of tide and river flow interaction (Vongvisessomjai and Rojanakamthorn, 2000) which emphasized the river flow effect, can apply well for the Chao Phraya River which has high freshwater discharge but shows the limitation in case of no freshwater discharge ($U_f = 0$). When freshwater discharge is zero, the result from analytical solution gives zero of damping coefficient and unchanged of tide celerity.

For the rather low magnitude of freshwater discharge from upstream flowing to tidal reach of the Bang Pakong River, the bottom friction is the dominant factor for tide amplitude damping and reducing of tide celerity in the Bang Pakong River. Therefore, the adjustment of the analytical model is necessary to improve the analytical model performance for applying in the Bang Pakong River.

The correlation of the observed dimensionless damping modulus coefficient, μ/k_0 and those from analytical solutions for the same magnitude of freshwater flow is determined for adjusting the analytical result. The correlation of dimensionless damping modulus coefficient can be expressed as:

$$\left(\frac{\mu}{k_0}\right)_{observed} = a_1\left(\frac{\mu}{k_0}\right)_{analytical} + b_1 \tag{15}$$

and the correlation of the observed dimensionless tide number, k/k_0 with those from analytical solutions are also shown as follows:

$$\left(\frac{k}{k_0}\right)_{observed} = a_2\left(\frac{k}{k_0}\right)_{analytical} + b_2 \tag{16}$$

The obtained parameters are listed in table 2

Table 2. Parameter for adjusting analytical solution.

Parameter	For M_2 and S_2	For K_1 and O_1
a_1	1.53	2.08
b_1	0.41	0.35
a_2	2.76	0.89
b_2	-1.66	0.35

Moreover, the zero order solution of the analytical model which represents to the mean water level have to adjust. Normally, the zeroth order solution can be calculated by Chezy's equation as shown in Eq.(7) with the assumption of steady flow of freshwater discharge and constant freshwater flow velocity along the tidal reach of the river but in the reality, the freshwater flow velocity and the water depth are not consistent along the river. The freshwater velocity, U_f substituted in the Chezy's equation is calculated from mean monthly discharge and the area of the river mouth at mean sea level.

Generally, the surface slope approachs mean sea level at the river mouth for any magnitude of freshwater discharge, while at the upstream, when cross sections area smaller, water level and flow velocity considerably depend on freshwater discharge. Therefore the zeroth order solution from Chezy's equation can be adjusted by comparing it with the observed water surface slope along the considered reach from river mouth to the Bang Kanak Station.

The correlation between theoretical and observed friction slope, S_f is used for adjusting the zeroth order solution. It can be fitted by linear regression in Figure 7 and the equation is:

$$(S_f)_{observed} = 10.6\,(S_f)_{Chezy} \tag{17}$$

2. Analytical Model of Tide and Salinity Barrier Interaction

Water surface elevation is composed of incident tide and reflected tide including effect of friction. The tidal elevation can be computed superposition of incident tide η_i and reflected tide η_r by:

$$\eta(x,t) = \eta_i + \eta_r \tag{18}$$

where

$$\eta_i = a_0 e^{(-\mu x)} \cos\left(\frac{2\pi t}{T} - kx + \delta_0 - \frac{\pi}{2}\right)$$

$$\eta_r = (a_0 e^{-\mu x_b}) e^{-\mu_r (x_b - x)} \cos\left[\frac{2\pi t}{T} - k_r(2x_b - x) + \delta_0 - \frac{\pi}{2}\right]$$

in which a_0 is the tide amplitude at the river mouth, μ is the tide amplitude damping coefficient in case of damping by bottom friction and river flow and μ_r is the tide amplitude damping coefficient in case of damping by the bottom friction only, x_b is the distance from river mouth to the salinity barrier, x is the distance from river mouth to the analysis station, δ_0 is the phase δ_i at the estuary from harmonic analysis of water level at the river mouth. Consequently, Eq.(18) can properly describe the specific single tide constituent excluded the effect from freshwater flow.

In the real situation, freshwater flow is taken into account as well as effects of N tide constituents, thereby the general expression of the actual water level fluctuation effected by salinity

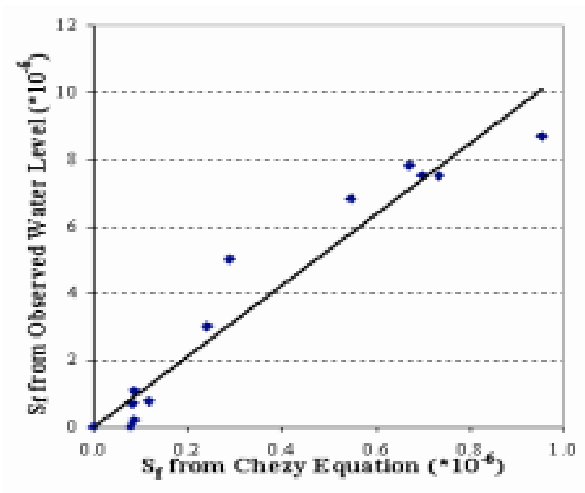


Figure 7. Adjustment of the zeroth order solution.

barrier can be express as:

$$\eta(x,t) = (S_{fo}x + \Delta h_0) + \Sigma\eta_{incident} + \Sigma\eta_{reflected} \tag{19}$$

where

$$S_{fo} = \frac{U_f^2}{C_z^2(h+\eta_0(x))}$$

$$\sum \eta_{incident} = \sum_{i=1}^{N} a_{oi} e^{-\mu_i x} \cos\left[\frac{2\pi t}{T_i} - k_i x + \delta_{oi} - \frac{\pi}{2}\right]$$

$$\sum \eta_{reflected}$$

$$= \sum_{i=1}^{N} (a_{oi} e^{-\mu_i x_b}) e^{-\mu_{ri}(x_b - x)} \cos\left[\frac{2\pi t}{T_i} - k_{ri}(2x_b - x) + \delta_{oi} - \frac{\pi}{2}\right]$$

From Eq.(19), the first term is the mean water level performing the steady flow of discharge released from upstream and the last two terms are the summation of the resultant of incident and reflected tide constituents. This expression can be used to describe the water level fluctuation in the downstream of the salinity barrier for fully close operation and partially close operation with the constant released discharge from the salinity barrier.

In case of partially close operation, it is assumed that the incident tide amplitude and celerity are damped by the discharge released from the salinity barrier including the bottom friction, while the reflected tide is damped only by the bottom friction during traveling back to the river mouth. μ and k are the tide amplitude damping modulus and tide number computed from the freshwater discharge released from upstream together with bed resistance, while μ_r and k_r are the reflected tide amplitude damping modulus and tide number when only effect of bed resistance is considered. In case of fully closed gate operation, $\mu = \mu_r$ and $k = k_r$ can be substituted in Eq.(19).

3. Application of Analytical Model of Tide and Salinity Barrier in the Bang Pakong River

One of the objectives of this study is to apply and examine the applicable of the analytical of tide and salinity barrier. Therefore, the analytical model of tide and salinity barrier interaction has been applied to compute the water level in the tidal reach downstream of the Bang Pakong Diversion Dam which was proposed for preventing the salinewater intrusion in the Bang Pakong River.

The Bang Pakong Diversion Dam was started the operation since 6^{th} January 2000 and the available records and information during January 2000 were selected to apply to the analytical model of tide and salinity barrier interaction.

The required data for the analytical model are:

1) Water level at the river mouth with considering four main tide constituents, M_2, S_2, K_1 and O_1 for the Bang Pakong River in computing a_{oi} and δ_{oi}.

2) Discharge released from the diversion dam in computing U_f and U_f/C_o

3) Adjustment of dimensionless amplitude damping modulus coefficient (Eq.15)

4) Adjustment of dimensionless tide number (Eq.16)

5) Observed water level along the Bang Pakong River

4. MIKE-11 HD Model Utilization

The test operation of the salinity barrier in August 1999 revealed the problem of riverbank collapse and overspill of salinewater at several locations along the Bang Pakong River. Therefore, when the Royal Irrigation Department decided to permanently keep some of the regulating gates open and permit the tides to propagate upstream for reducing the strong adverse effects downstream of the dam. Due to this, the discharge at the dam could not be observed. Normally the discharge from the dam could be measured by the discharge gauge and then automatically recorded. However, the discharge gauge could not be operated to measure discharge because of permanent opening of some gate during January 2000.The recorded discharge at the dam site is shown in Figure 8(a). For this reason, MIKE 11 model was adopted for estimating the discharge at released from the dam site instead of the use of measured discharge data

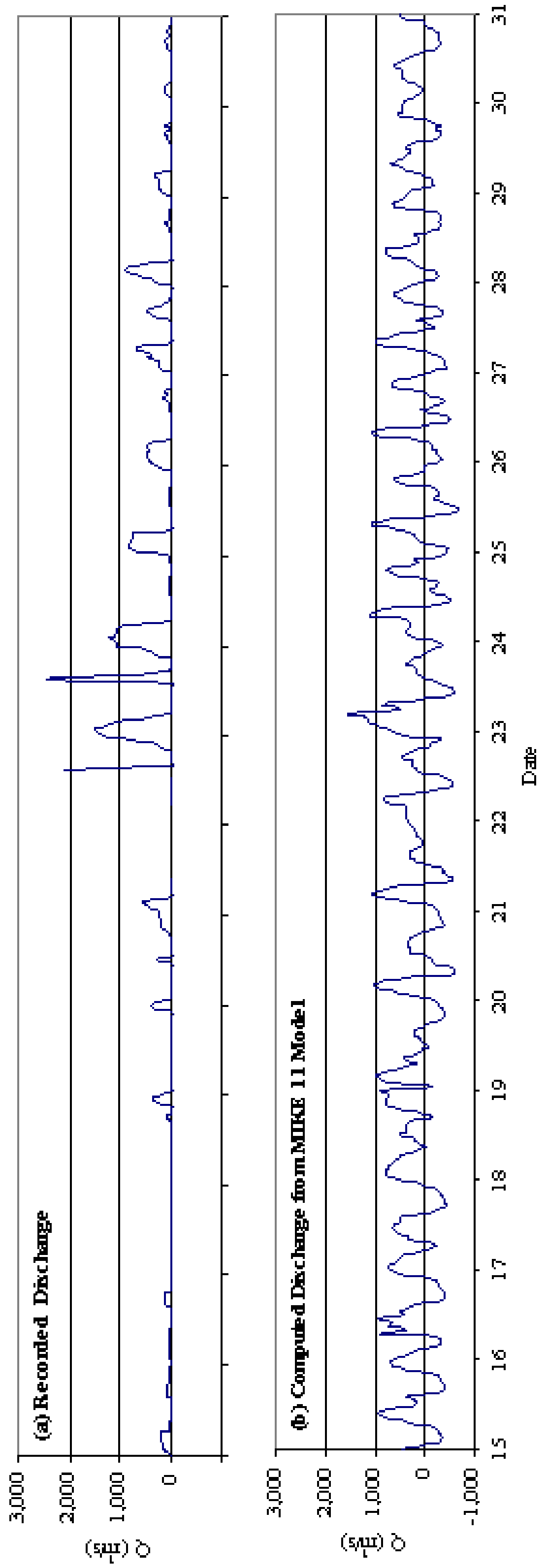


Figure 8. Recorded and computed discharges at dam site in January 2000.

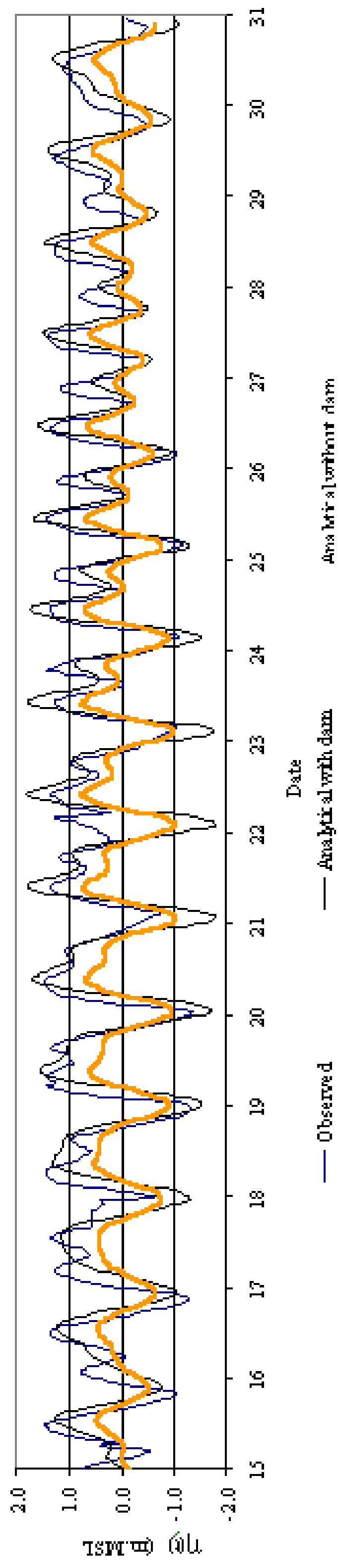


Figure 9. Observed and analytically computed water levels at dam site in January 2000.

(DHI Water and Environment, 2000).

Boundary conditions of MIKE-11 HD module were the upstream hourly observed water level at the dam site (km 62) and the downstream hourly observed water level at the Bang Pakong River Mouth (km 0). The computation was carried out for the whole mouth, January 2000. To evaluate the model performance, the computed water level along the Bang Pakong River from MIKE 11 were compared with the observed hourly water level at Pak Takong, Tha Tua and Tha Kai Stations.

The computed discharge from MIKE 11 was shown in Figure 8(b). It can be seen that, in January 2000, the dam operation could be classified as partially open operation because some discharge was allowed to flow through the regulating gate. The mean monthly discharge at the dam site was the representative of the steady flow component in the analytical model. The mean monthly discharge, 130 m^3/s and Eq.(19) were used for determine the water level downstream of the dam during dam operation. The variables of the Eq. (19) were shown in Table 3.

The hourly water level at the dam site in January 2000 for without dam and for real operation are in Figure 9.

Conclusion

This study attempts were made to analyze the river hydraulic behaviors affected by the tide and river flow and salinity barrier operation in the Bang Pakong River by applying with adjustment the analytical model of tide and river flow interaction derived by Vongvisessomjai and Rojanakamthorn (2000) as well as the developed analytical model of tide and salinity barrier. The conclusions of this study are as follows:

1. Analytical model of tide and river flow interaction of Vongvisessomjai and Rojanakamthorn (2000) provided the underestimated results of dimensionless damping modulus coefficient (μ/k_0) and dimensionless tide number (k/k_0). Because of the low range of river flow of the Bang Pakong River, the frictional effect was more dominated than the effect from river flow. Adjustment of the above two dimensionless parameters were made for meaningful application.

2. The analytical model of tide and salinity barrier interaction was developed in this study by the superposition of three components, the steady flow component from upstream flow, incident tide and reflected tide with the assumption of amplitude and celerity of incident tide being damped by the lumped effects of bottom friction and upstream flow, while the amplitude and celerity of reflected tide were damped by only the bottom friction. The damping modulus coefficient and tide number changed by only bottom friction could be calculated by the adjusted analytical model in case of no freshwater flow.

3. The analytical model of tide and salinity barrier was applied in the real situation of the Bang Pakong River. It was found that the unsteady flow condition of the upstream flow introduced some inaccuracies in of the analytical model. The analytical model gave over-estimated results during high upstream flow. The high upstream flow con-

Table 3. Variable of the Analytical Model of Tide and Salinity Barrier.

Tide	a_{oi} (m)	δ_{oi} (rad)	U_f (m/s)	$\frac{U_f}{C_0}$	$\frac{\mu}{k_0}$ incident	$\frac{k}{k_0}$ incident	$\frac{\mu}{k_0}$ reflected	$\frac{k}{k_0}$ reflected
M_2	0.514	0.868	0.039	0.004	0.606	1.121	0.414	1.095
S_2	0.246	3.170	0.039	0.004	0.599	1.119	0.414	1.095
K_1	0.719	4.512	0.039	0.004	0.848	1.261	0.353	1.235
O_1	0.342	1.385	0.039	0.004	0.885	1.265	0.353	1.235

Note that a_{oi} and δ_{oi} are obtained from harmonic analysis of water level at the river mouth

tributes to of the higher tidal amplitude damping and raising of the mean water level in that particular high discharge times. Phase difference of analytical model and observed data during high upstream flow was noticed. Because the higher flow causes retarding celerity of tide, therefore the observed tide would move more slowly than the computed tide from the analytical model based on mean monthly water level.

4. The error from analytical model was higher in the station near the barrier, which highly affected by changing of upstream flow, and, lower at the stations near the river mouth, where the impact of reflected tide was low.

5. For the use of mean monthly discharge from the dam during January 2000, though the mean discharge had almost the same magnitude as the natural flow in this month, a higher fluctuation of water level downstream of the dam still occurred which may be due to the additional effect from the reflection of tide at the close gate of the dam.

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