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A translation wave model: Güneycedere case study

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Abstract

This study proposed a flood routing model which was derived from Saint-Venant (SV) equations. It can be called translation wave model (TWM). In this model, bed slope term and friction slope term were ignored in the momentum equation of SV equations. This means, the difference between bed slope and friction slope are relatively small compared to other terms in the SV equations. This approach is similiar to the one in kinematic wave model (KWM), but in KWM inertia and pressure terms are neglected. In this study, governing equations for the proposed model were derived and solved numerically by using an explicit scheme. Then, validation of the proposed model was obtained through real flood data that belong to an actual creek reach in Isparta Province, Turkiye. The creek reach was between two stream gauging stations and the inflow and outflow hydrographs of a real flood event were available. Also, KWM was implemented for this creek reach using this real flood event. Thus two simulated outflow hydrographs; one that belongs to KWM and another that belongs to TWM were created. Then the two simulated outflow hydrographs were compared by differences in peak discharge, time to peak flow and hydrograph volume. Since KWM fails to predict attenuation and dispersion in outflow hydrographs, relative error of peak flow in KWM is calculated bigger than in TWM (2,19%>-0,27%). Relative error of time to peak flow in TWM is calculated as 0,00% while it is calculated -2,50% in KWM and the two models failed to provide volume conservation. Also, TWM and KWM were evaluated by the statistical parameters; Root Mean Square Error (RMSE), Mean Absolute Error (MAE) and Nash-Sutcliffe Efficiency (NSE). The results were in acceptable range but KWM gave better results since the creek reach had a steeper slope than average ($S_0 \geq 0.005$). Finally, for comparison, an inflow hydrograph from literature was routed with KWM and TWM in a rectangular channel.

1 Introduction

Flood or flow routing is a method to predict time and magnitude of flood/flow in a river or a channel from available upstream inflow data. Flood routing is classified into two types; hydrologic routing and hydraulic routing. In hydrologic routing, flow is only time dependent while in hydraulic routing flow is space and time dependent (Chow et al., 1988). In hydrologic routing; contiunity equation and a relation between inflow, outflow and storage is used to solve the routing problem. Solution process is relatively simple and results are satisfactory in general (Shaw, 2005). In hydraulic routing topographical data is needed to solve complex equations while in hydrologic routing there is less need of topographical data (Zhang et al., 2016).

https://doi.org/10.5194/hess-2022-192
Preprint. Discussion started: 24 May 2022

Hydrology and Earth System Sciences
Discussions

In 1848 Barré de Saint-Venant first put forward a solution to hydraulic flood routing problem. However, there is evidence in the literature that it was derived first by Lagrange as early as 1781 (Stoker, 1948). In the mentioned solution, "contiunity equation" and a statement of Newton' s Second Law "momentum equation" are solved for a differential volume of one-dimensional flow. In order to obtain an analytical solution, various approximations to the SV equations have been proposed, because of difficulties in analytical solution of the complete model, called a dynamic wave model. On the other hand, for many problems, a full solution of the SV equations is unnecessary and a variety of simplified methods exist (Heatherman, 2008). KWM is the simplest form of SV equations and it fails to predict attenuation and dispersion in outflow hydrographs. This model is used when downstream backwater effect is insignificant (Lighthill and Witham 1955a, 1955b). Another form of SV equations is diffusion wave model (DWM). This model is not suitable for reaches that have dramatically varying cross sectional areas and for very small slopes (Heatherman, 2008). In this study another simplified form of SV equations is proposed. It can be called TWM. This model can be considered as a nonkinematic model because of its neglected terms in SV equations. Since KWM is suitable for steep slopes (Henderson, 1966), TWM can be suitable for mild slopes.

In this study, firstly basic equations of TWM were derived. Secondly, numerical solution of the model was described. Then applicability of the model was studied by routing an observed inflow hydrograph for a creek reach length of 1764 m with a trapezoidal shape in general. The creek reach was between two gauging stations and was in Isparta province of Turkiye. Also equations of KWM were given in this study and the observed inflow hydrograph was routed by KWM in HEC-HMS. Results of the two model were compared with the observed outflow hydrograph.

2 Study site and data

2.1 Study site

Güneycedere Creek is in Isparta Province of Turkiye. Catchment area of Güneycedere Creek basin covers 102 km^2. Study area is located in lower reach of this creek. The creek flows northwestly toward Lake Eğirdir which is also known as the "Seven Colored Lake" in Isparta.

There are several reasons to choose this creek reach for this study:

https://doi.org/10.5194/hess-2022-192
Preprint. Discussion started: 24 May 2022

. In the study area, there are two stream gauging stations. Thus, observed inflow hydrograph and observed outflow hydrograph are available. Distance between upper gauging station (D09A601) and lower gauging station (D09A602) is 1764 metres in length (Fig. 1, Fig. , 2. and Fig. 3).

. There is no lateral inflow or outflow between the stations along the creek reach.

. There is no abrupt changes in the cross sectional areas along creek the reach.

. Slope of the creek reach is relatively a mild slope when compared to other creeks that have gauging stations.

Figure 1 Study area © Google Earth 2018

Figure 2 Stream gauging station D09A601

Figure 3 Stream gauging station D09A602

2.2 Data

The inflow hydrograph and the outflow hydrograph chosen for this study belong to flood event occured in 2018 spring. The two hydrographs have good hydrograph shapes. Base flow of the creek reach is assumed to be the minimum flow in the inflow hydrograph and is 7.19 m^3/s. Elapsed time of the two hydrographs is 9 hours. They have single peak flows. Time to peak flow in the inflow hydrograph is 3 hours and lag of time to peak in the outflow hydrograph is 0.33 hours.

In this study, a 1:1000 scale digital topographical map of the study area was used. Based on the topographical map, bed slope between upper and lower gauging stations was calculated as 0.006236. The value of Manning roughness coefficient (n) was derived from the well-known Manning equation. “n” roughness coefficient was the average value for the whole reach and it was calculated as 0.037. Since the creek bed is lined with gravels and stones, the calibrated value of “n” is compatible with “Manning roughness coefficients for various open channel surfaces” (Chow et al., 1988).

https://doi.org/10.5194/hess-2022-192
Preprint. Discussion started: 24 May 2022

3 Methodology

3.1 Governing equations

SV equations solve both the continuity and momentum equations for a differential volume of one-dimensional flow, where the forces on the control volume are limited to the effect of gravity, pressure variation, and friction or roughness of the channel walls. Mass is conserved in the solution and the effect of acceleration within the control volume and momentum flux across the upstream and downstream faces are considered (Heatherman, 2008).

Neglecting wind shear and eddy losses, the continuity equation is stated as:

$$\frac{\partial A}{\partial t}+\frac{\partial Q}{\partial x}=0 \tag{1}$$

where A is cross-sectional area of flow, t is time, Q is discharge and x is longitudinal distance of the control volume. The continuity equation is same for all forms of SV equations.

The momentum equation can be stated as:

$$a\left[\frac{\partial Q}{\partial t}+\frac{\partial\left(\frac{Q^2}{A}\right)}{\partial x}\right]+b\left[gA\frac{\partial y}{\partial x}\right]+c\left[gA\left(S_0-S_f\right)\right]=0 \tag{2}$$

term1 term2 term3 term4 term5

where g is gravitational acceleration, y is depth of flow, S_0 is bed slope and S_f is frictional slope.

In Equation 2, term 1 arises from temporal acceleration, term 2 from convective acceleration, term 3 from net pressure forces on the control volume, term 4 from gravitation force and term 5 from friction force.

https://doi.org/10.5194/hess-2022-192
Preprint. Discussion started: 24 May 2022

In the momentum equation (Eq. 2), if a=1, b=1, and c=1, then momentum equation of the dynamic wave model is obtained. If a=0, b=1, and c=1, then momentum equation of the diffusion wave model is obtained.

3.1.1 KWM

In the momentum equation (Eq. 2) if a=0, b=0, and c=1, then momentum equation of the kinematic wave model is obtained in Eq. (3).

$$S_o = S_f \tag{3}$$

which means that the flow is uniform and a function of depth or channel' s average cross-sectional area.

The momentum equation of KWM can be written as:

$$Q=\alpha A^{\beta} \tag{4}$$

where α and β are the kinematic wave model parameters. Substituting Eq. (4), in Eq. (1) yields an expression for solving for Q as the only dependent variable (Chow et al., 1988):

$$\frac{\partial Q}{\partial x}+\alpha\beta Q^{\beta-1}\left(\frac{\partial Q}{\partial t}\right)=0 \tag{5}$$

3.1.2 TWM

In Equation 2, if a=1, b=1, and c=0, then momentum equation of TWM is obtained:

$$\frac{\partial Q}{\partial t}+\frac{\partial\left(\frac{Q^2}{A}\right)}{\partial x}+gA\frac{\partial y}{\partial x}=0 \tag{6}$$

https://doi.org/10.5194/hess-2022-192
Preprint. Discussion started: 24 May 2022

In the momentum equation of TWM, inertia and pressure terms are included while gravitation and friction terms are neglected. This model can be considered as a nonkinematic model. Since kinematic wave model is suitable for steep slopes, TWM can be suitable for mild slopes.

For a trapezoidal channel, the cross-sectional area of the channel is given by:

$$A= (B+ zy)\, y \tag{7}$$

where B is bottom width and z is the inverse of the side slope of the channel. In Equation (7) z=0 for rectangular channels, and B= 0 for triangular channels.

Using Eq. (7), if B is constant the partial derivative can be written as:

$$\frac{\partial A}{\partial x} = T\frac{\partial y}{\partial x} \tag{8}$$

where T= B+ 2zy as width of water surface.

Substituting Eq. (8) into Eq. (6), Eq. (9) is achieved:

$$\frac{\partial Q}{\partial t} + \frac{\partial\left(\frac{Q^2}{A}\right)}{\partial x} + gA\frac{1}{T}\frac{\partial A}{\partial x} = 0 \tag{9}$$

Equation (9) can be rearranged as:

$$\frac{\partial Q}{\partial t} + \frac{1}{A}2Q\frac{\partial Q}{\partial x} - \frac{Q^2}{A^2}\frac{\partial A}{\partial x} + \frac{gA}{T}\frac{\partial A}{\partial x} = 0 \tag{10}$$

Multiplying both sides with A^2, Eq. (10) becomes:

$$A^2\frac{\partial Q}{\partial t} + 2QA\frac{\partial Q}{\partial x} - Q^2\frac{\partial A}{\partial x} + \frac{gA^3}{T}\frac{\partial A}{\partial x} = 0 \tag{11}$$

https://doi.org/10.5194/hess-2022-192
Preprint. Discussion started: 24 May 2022

Equation (11) can be rearranged as:

$$A^2\frac{\partial Q}{\partial t}+2QA\frac{\partial Q}{\partial x}+\left[\frac{gA^3}{T}-Q^2\right]\frac{\partial A}{\partial x}=0 \qquad (12)$$

Equation (12) can be rewritten as:

$$\alpha\frac{\partial Q}{\partial t}+\beta\frac{\partial Q}{\partial x}+\gamma\frac{\partial A}{\partial x}=0 \qquad (13)$$

where:

$$\alpha= A^2 \qquad (14)$$

$$\beta= 2QA \qquad (15)$$

$$\gamma=\left(gA^3/T\right)-Q^2 \qquad (16)$$

In definition, T= B+ 2zy for trapezoidal channels, T= B (z= 0) for rectangular channels, and T= 2zy for triangular channels.

Equation (1) and Eq. (13) can be solved bu using initial and boundary conditions. For a triangular inflow hydrograph, the inital conditions can be written as:

$$Q(x,0)= Q_0 \qquad (17)$$

$$A(x,0)= A_0 \qquad (18)$$

Where Q_0 is the base constant flow, and A_0 is cross-sectional area corresponding to the base flow.

The upstream boundary condition can be written as for $0< t < t_p$:

https://doi.org/10.5194/hess-2022-192
Preprint. Discussion started: 24 May 2022

$$Q(0,t) = Q_0 + \left(\frac{Q_p - Q_0}{t_p}\right)t \tag{19}$$

where Q_p is peak flow and t_p is time to peak.

For t_p 0< t < t_b the upstream boundary condition is defined as:

$$Q(0,t) = Q_p - \left(\frac{Q_p - Q_0}{t_b - t_p}\right)t \tag{20}$$

Where t_b is duration of the inflow hydrograph.

Finally at the end of the inflow hydrograph the boundary condition becomes for t> t_b:

$$Q(0,t)= Q_0 \tag{21}$$

3.2 Numerical solution

In order to solve the governing equations of the model, an explicit difference method is used (Abbot and Basco, 1989). Applying this finite difference method, as backwards in space and forward in time, Equation (13) and Eq. (1) can be written respectively as follows:

$$Q_i^{j+1} = Q_i^j - \Delta t\left(\beta_i^j\left(Q_i^j - Q_{i-1}^j\right) + \gamma_i^j\left(A_i^j - A_{i-1}^j\right)\right) / \left((\Delta x)\alpha_i^j\right) \tag{22}$$

$$A_i^{j+1} = A_i^j - \Delta t\left(Q_i^{j+1} - Q_{i-1}^{j+1}\right) / \Delta x \tag{23}$$

where Δx ve Δt are space and time intervals, respectively.

In the calculation procedure, first each pair of α_i^j, β_i^j and γ_i^j values can be readily calculated from Eq. (14), Eq. (15) and Eq. (16) using the known initial and boundary data at starting point of (i, j), then one can obtain Q_i^{j+1} from Eq. (22). Finally using Q_i^{j+1}, A_i^{j+1} can be calculated from Eq. (23). This technique will be repeated for successive values of (i, j). In the procedure,

https://doi.org/10.5194/hess-2022-192
Preprint. Discussion started: 24 May 2022

Hydrology and Earth System Sciences
Open Access
Discussions

discharge that is leaving the downstream boundary of a channel segment, enters to the upstream boundary of the next segment and establishes the upstream boundary condition for flow on this next segment.

3.3 Application

In this study, applicability of the TWM is investigated in a gauged creek reach. In addition, KWM is applied to this reach and the results of the two model are compared with the observed data.

Also for comparison, some data were selected from literature (Akan and Yen, 1981). In order to use the TWM, a rectangular channel with a width of 5 m is used. The initial values of the A, corresponding to inflow hydrograph values were calculated by using Manning equation. For given values of of Q, n, S_0 and B (5 m); water depth and consequently A can be found by trial and error method.

3.3.1 Güneycedere Creek application

All of the required physical components to perform KWM in HEC-HMS are calculated based on the available flow and topographical data.

In Figure 4, length is the distance between upper and lower gauging stations and it is 1764 meters. Slope is the average bed slope between the two stations and it was calculated as 0.006236. “n” roughness coefficient is the average value for the whole reach and it was calculated as 0.037. Bottom width is the average width of the bottom through the reach. In 2000, a bank protection project was constructed within the lower basin of Güneycedere Creek including the study area. The project consists of grading the Güneycedere Creek bank to a 2V:5H slope along 2500 meters of the eroded bank, and it is made of riprap (Fig. 2. and Fig. 3.). However, according to the cross section views created by Netcad computer program in the creek reach, side slope was taken as 1V:3H in average.

https://doi.org/10.5194/hess-2022-192
Preprint. Discussion started: 24 May 2022

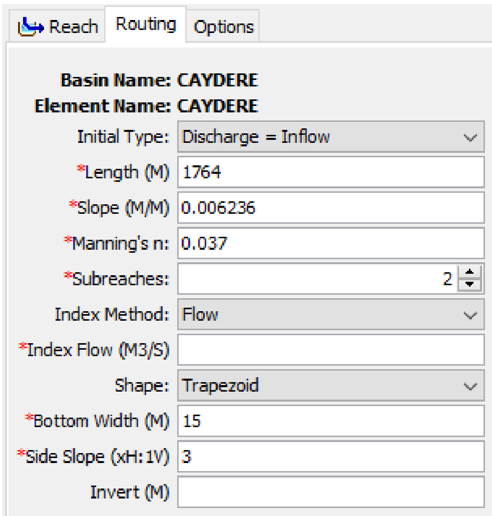

Figure 4 © HEC-HMS component editor for KWM

The peak flow in the inflow hydrograph of upstream gauging station D09A601 is 13,10 m^3/s. After routing the inflow hydrograph with KWM in HEC-HMS, the outflow hydrograph is computed. This simulated outflow hydrograph belongs to the downstream gauging station D09A602 and the peak flow in the outflow hydrograph is computed as 13,08 m^3/s. Observed peak outflow in the downstream gauging station D09A602 is 12,80 m^3/s. Time to peak flow in the simulated outflow hydrograph is 15 minutes, while time to peak flow in the observed outflow hydrograph is 20 minutes. Observed total outflow volume is 319.509 m^3 while computed total outflow volume with KWM is 322.845 m^3. In Figure 5, observed outflow hydrograph and KWM' s simulated hydrograph are given.

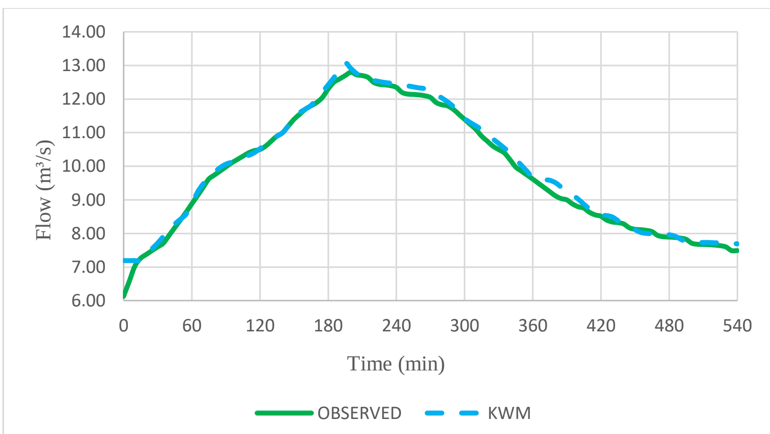

Figure 5 Observed outflow hydrograph and outflow hydrograph with KWM

https://doi.org/10.5194/hess-2022-192
Preprint. Discussion started: 24 May 2022

The TWM calculations are performed on a grid placed over x-t plane. The x-t grid is a network of points defined by taking distance increments of length Δx and time increments of duration Δt (Chow et al.,1988). On our x-t grid, upstream boundary condition on time line is composed of the inflow hydrograph. This inflow hydrograph belongs to the upstream gauging station D09A601. Initial boundary condition on the distance line is composed of base flow. As mentioned before, base flow of the creek reach is assumed to be the minimum flow in the inflow hydrograph and is 7.19 m^3/s.

In order to satisfy the courant condition as $c\Delta t<\Delta x$, in which c is the wave celerity, Δx and Δt were set equal to various values and then for various values, TWM was applied to the creek reach and it was seen that when Δx =450 m and Δt=120 s, computational stability was maintained and courant condition was satisfied. TWM x-t grid is given in Fig. 6.

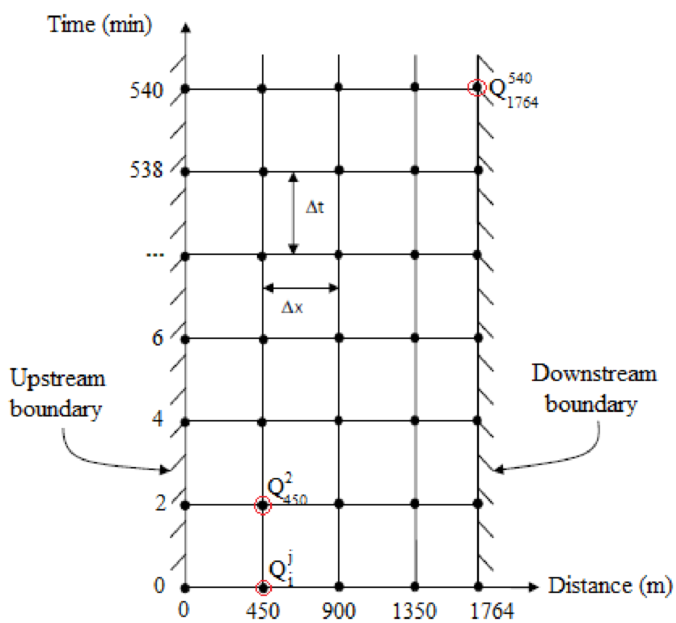

Figure 6 Güneycedere TWM grid on an x-t plane

Starting from the upper gauging station, distance of the station' s location was assumed to be KM:0+000 and was assumed to be the first point, labeled as number 1. Moving downstream

https://doi.org/10.5194/hess-2022-192
Preprint. Discussion started: 24 May 2022

and considering "Δx =450 m", other locations are specified along the 1764 metres long creek reach respectively; 0+450 km (2), 0+900 km (3), 1+350 km (4) and 1+764 km (5). At these points of the reach, cross section views were created in Netcad computer program. On the TWM grid, distance points were denoted by index i and time points were denoted by index j.

Applying the finite difference method, as backwards in space and forward in time and using Eq. (22), Q_i^{j+1} (Q_{450}^{2}) was calculated. Then Q_i^{j+2} (Q_{450}^{4}), Q_i^{j+3} (Q_{450}^{6}), Q_i^{j+4} (Q_{450}^{8}), …, and Q_i^{j+n} (Q_{450}^{540}) were calculated in the same way. While calculated flows formed the discharges that were leaving the first segment of the creek reach they also formed the upstream boundary condition for the next segment. In this way, the flows that belonged to the last point on the x-t grid were calculated. This point represents the location of the downstream gauging station in the reach and calculated flows of this point form the outflow hydrograph.

In rectangular, trapezodial and triangular channels, Eq. (23) enables to do calculations without the need of creating cross section views on each points on the x-t grid. Since this study was on a natural creek and since the 1:1000 scale digital topographical map of the study area was available, cross section views were created at each point by Netcad computer program easily. For various water depths corresponding to various discharges in the cross sections; α_i^j, β_i^j and γ_i^j were calculated using macros created in excel specific for this study.

The peak flow in the inflow hydrograph of upstream gauging station D09A601 is 13,10 m^3/s. After routing the inflow hydrograph with TWM, the outflow hydrograph is computed. This simulated outflow hydrograph belongs to the downstream gauging station D09A602 and the peak flow in the outflow hydrograph is computed as 12,77 m^3/s. Observed peak outflow in the downstream gauging station D09A602 is 12,80 m^3/s. Time to peak flow in the simulated outflow hydrograph is 200 minutes. This value is equal to the one in the observed outflow hydrograph. Observed total outflow volume is 319.509 m^3 while computed total outflow volume with TWM is 312.990 m^3. In Figure 7, observed outflow hydrograph and TWM' s simulated hydrograph are given.

https://doi.org/10.5194/hess-2022-192
Preprint. Discussion started: 24 May 2022

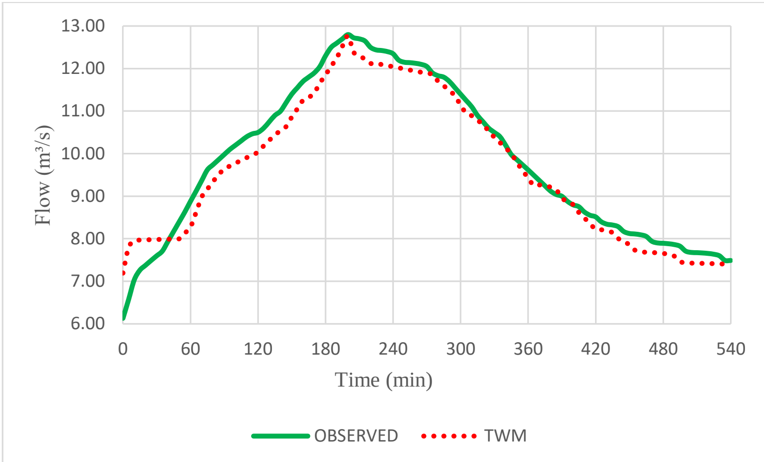

Figure 7 Observed outflow hydrograph and outflow hydrograph with TWM

Observed outflow hydrograph and outflow hydrographs with TWM and KWM are given in Fig. 8.

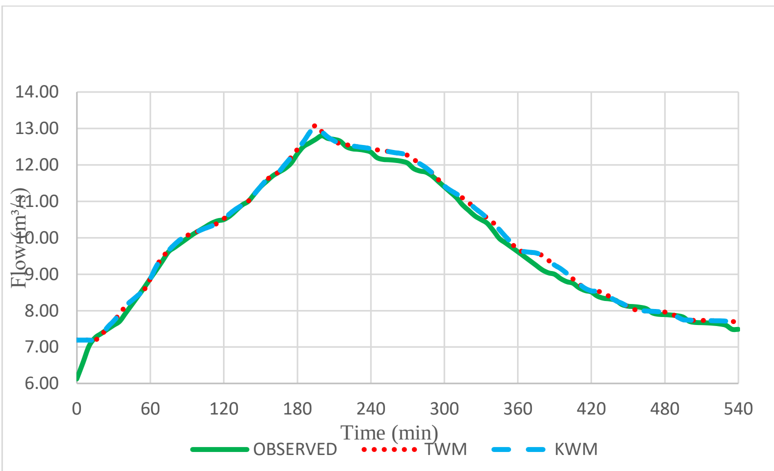

Figure 8 Observed outflow hydrograph and outflow hydrographs with TWM and KWM

3.3.2 Application of KWM and TWM with literature data

As an example of the application of the KWM in HEC-HMS, it was applied to a rectangular channel with a width of 5 m and a bottom slope of 0.005, and the Manning roughness coefficient of 0.012. Reach length (L) of the channel was assumed to be 1000 m. In the inflow hydrograph

https://doi.org/10.5194/hess-2022-192
Preprint. Discussion started: 24 May 2022

Hydrology and Earth System Sciences
Discussions

peak flow is 12.00 m^3/s, baseflow is 3.00 m^3/s. Elapsed time of the inflow hydrograph is 2400 s (40 min) and time to peak flow is 600 s (10 min).

After routing the inflow hydrograph with KWM in HEC-HMS, the outflow hydrograph is computed. The peak flow in the simulated outflow hydrograph is computed as 11.55 m^3/s. Time to peak flow in the simulated outflow hydrograph is 14 min.

Using the same data (inflow hydrograph, n, S_0, L and B), TWM was applied to the rectangular channel. In order to satisfy the courant condition, Δx =200 m and Δt=5 s were used in the computations. After routing the inflow hydrograph with TWM, the outflow hydrograph is computed. The peak flow in the simulated outflow hydrograph is computed as 10.9 m^3/s. Time to peak flow in the simulated outflow hydrograph is 12.08 min.

Inflow hydrograph and simulated outflow hydrographs are given in Fig. 9.

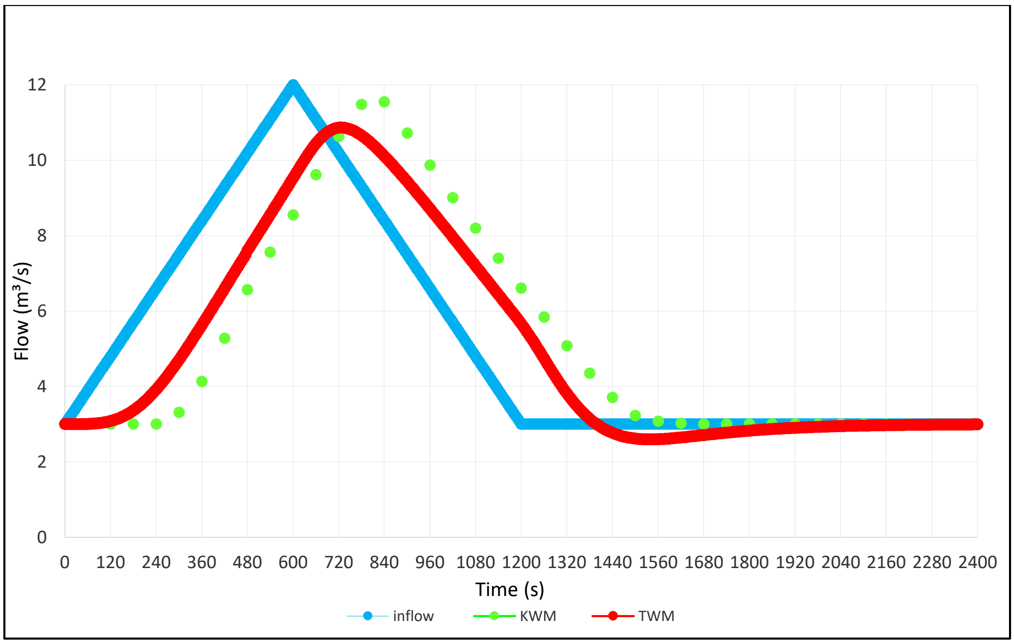

Figure 9 Inflow hydrograph (Akan and Yen, 1981) and simulated outflow hydrographs

3.4 Statistical analyses

Since observed inflow and observed outflow hydrographs in Güneycedere Creek reach were available in this study, we could be able to compare the simulated KWM and TWM hydrographs with the observed outflow hydrograph. Above, in Fig. 8, it was clear that the simulated KWM and TWM hydrographs had good agreement with the observed outflow hydrograph. Although visual comparison of the observed and simulated outflow hydrographs gave a positive opinion about the accuracy of the KWM and TWM in the study area, statistical analyses were needed to support this opinion. The root-mean-square error (RMSE) in Eq. (24), the mean absolute error (MAE) in Eq. (25) and the Nash–Sutcliffe model efficiency coefficient (E) in Eq. (26) were calculated to determine the difference between the observed and simulated hydrographs.

$$\mathrm{RMSE}=\sqrt{\frac{\sum_{i=n}^{n}(Q_c-Q_o)^2}{n}}\quad i=1, 2, 3,\ldots, n \tag{24}$$

$$\mathrm{MAE}=\frac{\sum_{i=1}^{n}|Q_c-Q_o|}{n}\quad i=1, 2, 3,\ldots, n \tag{25}$$

$$E=\frac{\sum_{i=1}^{n}(Q_o-Q_{mo})^2-\sum_{i=1}^{n}(Q_o-Q_c)^2}{\sum_{i=1}^{n}(Q_o-Q_{mo})^2}\quad i=1, 2, 3,\ldots, n \tag{26}$$

RMSE, MAE and E values were given in Table 1.

Table 1 RMSE, MAE and NSE values

Model	**RMSE (m^3/s)**	**MAE (m^3/s)**	**NSE**
TWM	0,36	0,30	0,96
KWM	0,18	0,12	0,99

Relative errors of 1) peak flow, 2) time to peak and 3) volume are computed as shown in Eq. (27), Eq. (28) and Eq. (29).

https://doi.org/10.5194/hess-2022-192
Preprint. Discussion started: 24 May 2022

$$\sigma_{peak} = (\frac{Q_{pc}}{Q_{po}} - 1)100 \quad (27)$$

$$\sigma_{time} = (\frac{t_{pc}}{t_{po}} - 1)100 \quad (28)$$

$$\sigma_{volume} = (\frac{V_c}{V_o} - 1)100 \quad (29)$$

where:

Q_c= Computed flows in the outflow hydrograph (m^3/s)

Q_o= Observed flows in the outflow hydrograph (m^3/s)

σ_{peak}= Relative error of peak flow (%)

Q_{pc}= Computed peak outflow (m^3/s)

Q_{po}= Observed peak outflow (m^3/s)

σ_{time}= Relative error of time to peak flow (%)

t_{pc}= time to peak flow in the computed outflow hydrograph (h)

t_{po}= time to peak flow in the observed outflow hydrograph (h)

σ_{volume}= Relative error of total volume (%)

V_c=Total volume of the the computed hydrograph (m^3)

V_o=Total volume of the the observed hydrograph (m^3)

Q_{mo}= Mean of the observed outflows (m^3/s)

σ_{peak} , σ_{time} and σ_{volume} values were given in Table 2.

Table 2 Relative errors of peak flow, time to peak and volume

Model	Q_{gp} (m^3/s)	Q_{hp} (m^3/s)	σ_p (%)	t_{gp} (saat)	t_{hp} (saat)	σ_t (%)	V_g (m^3)	V_h (m^3)	σ_v (%)
TWM	12,80	12,77	-0,27	3,33	3,33	0,00	319509	312990	-2,04
KWM	12,80	13,08	2,19	3,33	3,25	-2,50	319509	322845	1,04

4. Discussion and conclusions

At the end of this study, the following concluding points can be made:

1. In this study, a translation wave model (TWM) was developed for a trapezoidal channel. This model was adapted to simulate the outflow hydrograph in a gauged creek reach and also to

https://doi.org/10.5194/hess-2022-192
Preprint. Discussion started: 24 May 2022

Hydrology and Earth System Sciences Discussions

predict the outflow hydrograph in a rectangular channel that was a special case of a trapezoidal channel.

2. The results of the TWM were compared with those of the KWM. TWM was successfully applied in Güneycedere Creek in terms of peak flow (PF) and time to peak flow (TPF). In other words, KWM gave a value that was bigger than the observed value in terms of PF, but, TWM gave a very close value to observed value. TWM gave the exact value to the observed value in terms TPF, but, KWM gave a close value to observed one. KWM results supports the fact that there is no attenuation and dispersion in outflow hydrographs when they are simulated by KWM.

2. Relative error of volume (REV) in TWM was in the form of decreasing of volume while REV in KWM was in the form of increasing of volume.

3. According to the statistical analyses that were performed in this study, KWM gave better results than TWM. Bed slope of the creek reach that was subject to this study was approximately 0,006 (≥0,005) and it was a steep slope (Soentoro, 1991). KWM' s momentum equation was composed of bed slope and friction slope terms and KWM was suitable for steep slopes. In the momentum equation of TWM bed slope term and friction slope term were neglected. Thus TWM was more accurate to use in smooth sloped river reaches. Statistical analyses' results supported this conclusion.

4. In addition, for comparison, an inflow hydrograph from literature was routed with KWM and TWM in a rectangular channel. As expected, in the KWM' s simulated hydrograph the peak flow was calculated as a very close value to the peak flow in the inflow hydrograph. In the TWM' s simulated hydrograph, the peak flow was calculated in an acceptable value ranges and the simulated hydrograph had a good shape.

As a result, TWM' s momentum equation does not include the friction and bed slope. This implies that the numerical solution of this model can be more stable and takes less time than that of any other model that includes these terms. TWM needs only one boundary condition. So this model can be solved for supercritical flows.

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Author contributions

HÇ and MEK developed the model, HÇ collected hydrological and survey data, performed the analysis and wrote the manuscript. MEK supervised and improved the study.

Competing interests

The authors declare that they have no conflicts of interest.

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