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SECTION 7

WATERSHED AND RESERVOIR MODELING

WASH LOAD COMPONENT FOR SEDIMENT YIELD MODELING

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INTRODUCTION

Abstract

A total sediment yield model was developed to distinguish between two sources of sediment yield--sheet and rill erosion and concentrated flow erosion. It uses the Universal Soil Loss Equation (USLE) (USDA 1978) and easily calculated hydrologic parameters for individual storm events. It is similar in form to other sediment yield models (Williams and Bernt 1977; Lee and Molnau 1979; Johnson et al 1985) except that it is a two-component model. They are all single storm event models that require surface runoff volume and peak rate of runoff for each sediment-producing event summed over time to determine annual yields. An error analysis was made for all four sediment yield models.

Watershed Descriptions

The model was developed from two geomorphically distinct data sets--one with predominantly silt and clay soils (Palouse-area) and the other with a greater percentage of fine sand-fraction sediments (Reynolds Creek). Slight amounts of gravel-material bed load (<5%) in both sets were ignored. Only the suspended load portion of the total sediment yield during storm events was included in these analyses. An average of an additional 13% of suspended sediment was measured at the Reynolds Creek locations between storms.

Data sets were available for four locations in the Palouse-area watersheds (Lee 1979) and for four locations in the Reynolds Creek watershed. The Palouse-area watersheds are predominantly cropland and contain mostly loess deposits which produce large amounts of silt and clay sediments. The Reynolds Creek Experimental Watershed was established in 1960 to study problems caused by floods and associated sediment yields from sagebrush rangelands in the intermountain United States (Robins et al. 1965).

Table 1 gives some general statistical information by watershed location. Only eight locations--four in each land use category--were available for the analysis, but they include a large number of sediment-producing events. The Palouse watershed sediment sources were mostly from sheet and rill erosion associated with the respective storm event. The Reynolds Creek watersheds sediment sources were mostly from the beds and banks of concentrated flow channels associated with storms.

Drainage areas and composite USLE parameters (l & s) and factors (K, L, S, C, & P) are shown in Table 2. The soil erodibility factors (K) are in original English units as per Agricultural Handbook Number (AHN) 537 (USDA 1978); the slope-length parameters (l) are in meters; the drainage areas (Da) are in hectares. The slope-steepness parameters (s), slope-length factors (L), slope-steepness factors (S), cover and management factors (C), and support practice factors (P) are non-dimensional. The values shown are described in Lee and Molnau (1979) for the Palouse-area watersheds and in Johnson et al

(1985) for the Reynolds Creek watersheds.

Table 1. Event-Time Statistics

Location ID	Location Name	Land Use	-Event Dates- first	last	No. Years	No. Events	Events per Year
1	Thompson	cropland	10/71	9/73	2	11	5.5
2	Pitzen	cropland	10/38	9/41	3	58	19.3
3	Naylor	cropland	10/38	9/41	3	52	17.3
4	Missouri	cropland	10/34	9/40	6	217	36.2
P	Total number of events Palouse-area watersheds					338	24.1
5	Mountain	rangeland	10/68	9/83	15	419	27.9
6	Macks Cr.	rangeland	10/67	9/32	16	171	10.7
7	Tollgate	rangeland	10/66	9/83	17	368	21.6
8	Outlet	rangeland	10/66	9/83	17	279	16.4
R	Total number of events Reynolds Creek watersheds					1237	19.0
C	Total number of events combined watersheds					1575	19.9

The C factors for the Palouse-area watersheds were varied seasonally (Lee 1979). Only time-averaged values are shown in Table 2. All the USLE factors were constant over time for the Reynolds Creek watersheds during the analyses. It was found that a single-component model developed from one data set did not perform well with the other. The two-component model performed better than each single-component model for every data set and subset.

Table 2. Time-Averaged USLE Parameters and Factors

Location	Da ha	K *	*l* m	*s* m/m	C -	P -	KLSCP -	KLS -
Thompson	3.32	0.330	83.8	0.120	0.286	0.9	0.20178	0.784
Pitzen	59.4	0.320	112.1	0.117	0.471	1.0	0.40692	0.864
Naylor	71.8	0.320	109.7	0.104	0.342	0.6	0.16166	0.788
Missouri	7123.	0.320	190.8	0.108	0.414	0.8	0.35321	1.066
Mountain	40.5	0.205	72.	0.150	0.006	1.0	0.00367	0.612
Macks Cr.	3175.	0.197	117.	0.220	0.010	1.0	0.01086	1.086
Tollgate	5444.	0.197	128.	0.220	0.008	1.0	0.01130	1.027
Outlet	23,372.	0.205	116.	0.200	0.011	1.0	0.01130	1.027

The product of the slope-length and slope-steepness factors for the Palouse-area watershed locations (cropland) was computed according to equation 1.

$$LS = (SQRT(s/22.13)) * ((11.16*\sin(\arctan(l)))^{0.7}) \qquad \text{Eq. 1}$$

where

l = slope-length parameter (meters);
s = slope-steepness parameter (m/m); and
LS = product of slope-length and slope-steepness factors (-).

The LS products for the Reynolds Creek watershed locations (rangeland) were computed according to equation 2 (McCool 1982). It differs from equation 1 with respect to the 0.7 exponent for the slope-steepness factor component.

$$LS = (SQRT(s/22.13)) * (11.16*\sin(\arctan(l))) \qquad \text{Eq. 2}$$

Where the parameters and variables are as previously defined.

REGRESSION ANALYSIS

Initially, the data was analyzed using equation 3.

-sheet and rill erosion component- + ---concentrated flow component----

$$S_y = \{a * (Q^b) * (q_p{}^c) * (D_a{}^d) * KLSCP\} + \{e * (Q^f) * (q_p{}^g) * (D_a{}^h) * C_f KLS\} \qquad \text{Eq. 3}$$

Where: S_y = sediment yield (Mg/ha);
Q = surface runoff volume (mm);
q_p = peak rate of surface runoff (mm/hr);
D_a = drainage area (ha);
K,L,S,C,P are USLE factors as per AHN 537;
C_f is a concentrated flow factor that was assumed equal to 1;
a to d are regression coefficients for the sheet and rill component; and
e to h are regression coefficients for the concentrated flow component.

The results showed that d was very nearly zero, and therefore, the drainage area term could be eliminated from the sheet and rill erosion component; i.e., the sheet and rill component is truly a unit-area function. This implies that wash load is a mostly a function of sheet and rill erosion.

Also, h was very nearly equal to f+g-1 which indicated that the concentrated flow component is a function of total flow. Therefore, the drainage area term in the concentrated flow component was set always to cause the concentrated flow component to be total flow; i.e., h=f+g-1. This implies that the concentrated flow component is not a function of upland sheet and rill erosion, but more likely a function of bed-material load. It is known that Reynolds Creek carried an average an additional of 13% of suspended sediment between storms--obviously bed-material load.

The data was tested for nonlinearity and significance of the individual independent variables. Equation 4 was the general form used.

$$\begin{aligned} S_y = \{a * [Q^{(b+d*\ln(Q*q_p))}] * [qp^{(c+d*\ln(Q*q_p))}] * KLSCP\} + \\ \{e * [Q^{(f+h*\ln(Q*q_p))}] * [qp^{(g+h*\ln(Q*q_p))}] * C_f KLS * \\ [D_a{}^{(f+g-1+2*h*\ln(Q*q_p))}]\} \qquad \text{Eq. 4} \end{aligned}$$

Where the parameters and variables are as previously defined.

The analysis indicated that the d, f, and h coefficients were nearly zero and their associated terms had negligible effect on the results; i.e., the nonlinearity terms in both components--$d*\ln(Q*q_p)$ and $h*\ln(Q*q_p)$--and the runoff volume term of the concentrated flow component--Q^f--was not significant.

The final form selected for further analysis was equation 5.

$$\underbrace{S_y = \{a * [Q^b] * [q_p^c] * KLSCP\}}_{\text{-sheet and rill component-}} + \underbrace{\{d * [q_p^e] * [D_a^{(e-1)}] * C_f KLS\}}_{\text{---concentrated flow component-}} \qquad \text{Eq. 5}$$

Where: a to c are regression coefficients for the sheet and rill component; d & e are regression coefficients for the concentrated flow component; all other parameters and variables are as previously defined.

The "best fit" as determined by a combination of least squares and calibration for percent average annual values is shown as equation 6.

$$\underbrace{S_y = \{0.22 * [\, Q^{0.68}] * [q_p^{0.95}] * KLSCP\}}_{\text{------ sheet \& rill component ------}} + \underbrace{\{0.0021 * [q_p^{1.61}] * [D_a^{0.61}] * C_f KLS\}}_{\text{------ concentrated flow component ---}} \qquad \text{Eq. 6}$$

Where the parameters and variables are as previously defined.

Equations 3 through 6 are all power-curve math equations. Therefore, log transformations produce standard linear regression equations, including equation 4. A standard least squares solution was made for the respective coefficients for equations 3 through 5. The regression coefficients for the sheet and rill erosion component were determined from the Palouse-area watersheds data set adjusted for concentrated flow. The regression coefficients for the concentrated flow component were determined from the Reynolds Creek watersheds adjusted for sheet and rill erosion. The data was weighted by the reciprocal of the respective location's number of years of record. This simply weights each location's data according to its average annual sediment yield. After determining the least squares solution, the leading coefficients of each component--a & e for equations 3 & 4 and a & d for equation 5--were calibrated so that the average of the mean percent error of the average annual sediment yield for each watershed grouping was zero.

A nonlinear regression analysis was also made, but the results were biased too heavily towards the larger events which lead to large percent errors.

The estimated wash load in Table 3 is the ratio of the estimated sheet and rill sediment yield to the estimated total sediment yield in equation 6.

Table 3. Sediment Yield Statistics for Equation 6

		----- Sediment Yield (Mg/ha)----				Std. Err	Estimated
		--Observed---		--Estimated--		of Est.	Wash Load
	Location	Mean	Std. Dev.	Mean	Std. Dev.	(Mg/ha)	(%)
	Thompson	6.927	2.473	5.189	3.654	2.132	98
Palouse	Pitzen	5.369	2.888	4.603	2.420	0.933	95
	Naylor	1.618	0.730	2.371	1.457	0.823	96
	Missouri	1.781	1.075	1.635	0.857	0.372	65
	Mountain	0.173	0.046	0.256	0.057	0.045	31
Reynolds Cr.	Macks Cr.	0.530	0.560	0.222	0.162	0.436	4
	Tollgate	0.617	0.381	0.745	0.173	0.281	3
	Outlet	0.489	0.421	0.426	0.309	0.389	2

ERROR ANALYSIS

Equation 7 is the Lee and Molnau equation that was developed from the Palouse-area data for cropland only.

$$S_y = 0.218 * [(Q*q_p)^{0.80}] * KLSCP \qquad \text{Eq. 7}$$

Where the parameters and variables are as previously defined.

Equation 8 is the Johnson et al equation that was developed from the Reynolds Creek data for rangeland only.

$$S_y = 0.1030 * [(Q*q_p)^{0.75}] * [D_a^{0.50}] * KLSCP \qquad \text{Eq. 8}$$

Where the parameters and variables are as previously defined.

Equation 9 is the MUSLE equation that was developed by Williams and Bernt (1977) for the Southwest.

$$S_y = 1.5862 * [(Q*q_p)^{0.56}] [D_a^{0.12}] * KLSCP \qquad \text{Eq. 9}$$

Where the parameters and variables are as previously defined.

An error analysis was made for equations 6 through 9. Table 4 shows the observed average annual (mean) sediment yield and standard deviation about the mean for each location. Both the average annual mean error and standard error of estimate about the mean are shown for each model. The errors are presented as a percentage of the average annual sediment yield; i.e., the sediment yields (both the observed and the predicted) for each storm event were summed at each location and divided by its respective number of years of record. Then the percent error was determined by dividing the difference between the predicted and observed by the observed. The watershed averages are the column totals divided by the number of locations.

Table 4. Average Annual Error Statistics

Location	Obs. Sed. Yd.		Error (%)							
	Mean	Std. Dev.	Mn	SE	Mn	SE	Mn	SE	Mn	SE
	(Mg/ha)		Eq. 6		Eq. 7		Eq. 8		Eq. 9	
Thompson	6.927	2.473	-25	86	-9	109	-41	59	115	227
Pitzen	5.369	2.888	-14	32	10	43	238	256	576	423
Naylor	1.618	0.730	47	113	102	224	590	881	1249	1117
Missouri	1.781	1.075	-8	35	-9	35	3304	2638	1521	818
Palouse-area w/s averages			0	66	24	103	1023	958	865	661
Mountain	0.173	0.046	48	96	-35	79	68	99	308	171
Macks Cr	0.530	0.560	-58	78	-98	98	-38	64	-55	84
Tollgate	0.617	0.381	21	74	-94	98	91	65	12	83
Outlet	0.489	0.421	-13	92	-98	99	41	68	-41	85
Reynolds Cr w/s averages			0	85	-81	94	40	74	56	106
combined averages:			0	76	-28	98	532	516	460	384

Figures 1 through 4 are plots of the average annual and average storm event predicted versus observed sediment yield for each of the four models (equations 6 through 9). Average annual is the accumulated total sediment yield divided by the respective number of years. Average storm event is the accumulated total sediment yield divided by the respective number of storms. The average storm event is always less than the average annual value. Each plotting point corresponds to the location ID in Table 1.

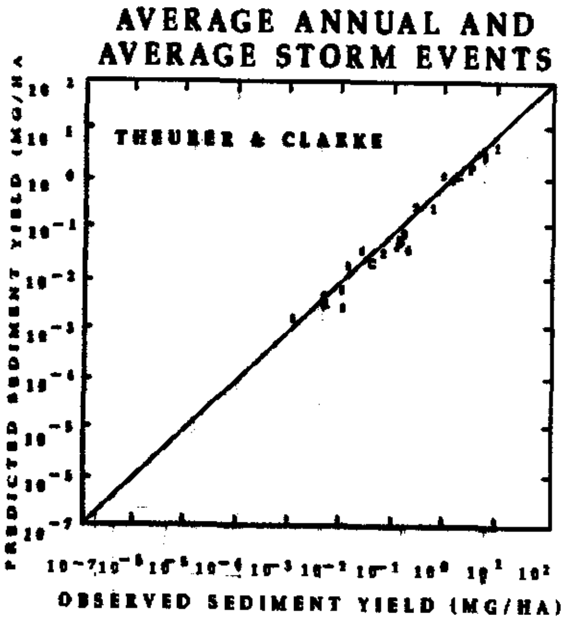

Figure 1

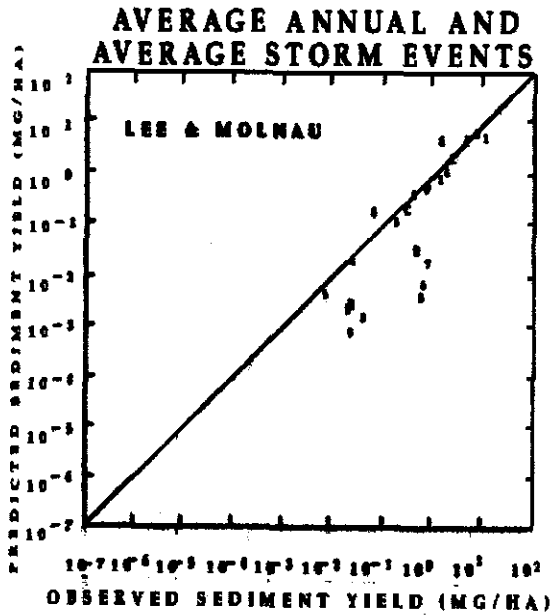

Figure 2

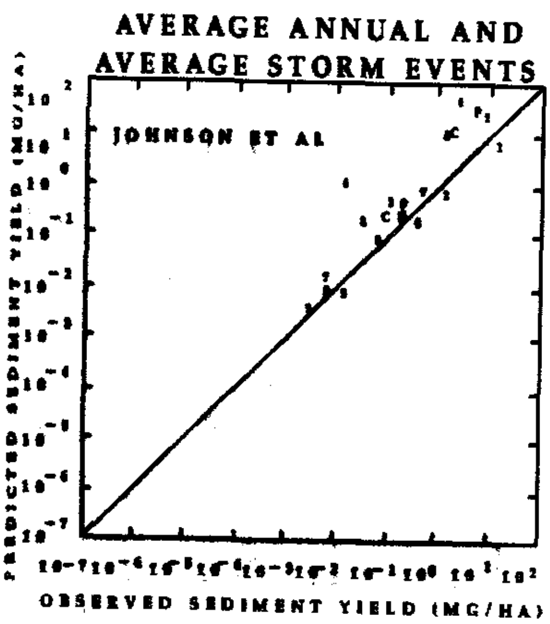

Figure 3

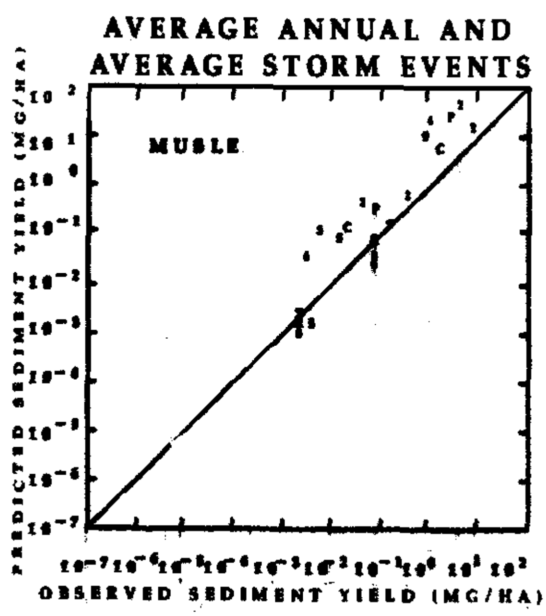

Figure 4

It can be seen easily from Table 4 and Figures 1 through 4 that the two-component model (equation 6) is better than each of the other models at each location, the two watershed groupings, and for the combined totals.

Table 5 shows the observed individual storm event mean and standard deviation about the mean for each location. The mean error and standard error of estimate about the mean are also shown for all four models. The error statistics are presented in units of sediment yield per unit of drainage area. Table 5 shows that the two-component model also is better than the other models on an individual storm basis at each location.

Table 5. Storm Event Averages Error Statistics

Location	Obs. Sed. Yld.		Error (Mg/ha)							
	Mn	SD	Mn	SE	Mn	SE	Mn	SE	Mn	SE
	(Mg/ha)		Eq. 6		Eq. 7		Eq. 8		Eq. 9	
Thompson	1.259	1.054	-0.316	0.909	-0.423	1.145	-0.522	0.624	0.444	2.390
Pitzen	0.278	0.657	-0.040	0.212	0.027	0.284	0.661	1.681	1.600	2.778
Naylor	0.093	0.175	0.045	0.198	0.095	0.393	0.550	1.544	1.166	2.064
Missouri	0.049	0.179	-0.004	0.062	-0.004	0.062	1.627	4.716	0.749	1.462
Mountain	0.006	0.009	0.003	0.008	-0.002	0.007	0.004	0.009	0.019	0.015
Macks Cr	0.050	0.171	-0.029	0.133	-0.048	0.168	-0.019	0.109	-0.027	0.144
Tollgate	0.028	0.082	0.006	0.060	-0.027	0.080	0.026	0.053	0.004	0.068
Outlet	0.030	0.140	-0.004	0.096	-0.029	0.103	0.012	0.071	-0.012	0.089

Figures 5 & 6 are plots of the individual storm event predicted versus observed sediment yield for equations 6 & 9 respectively.

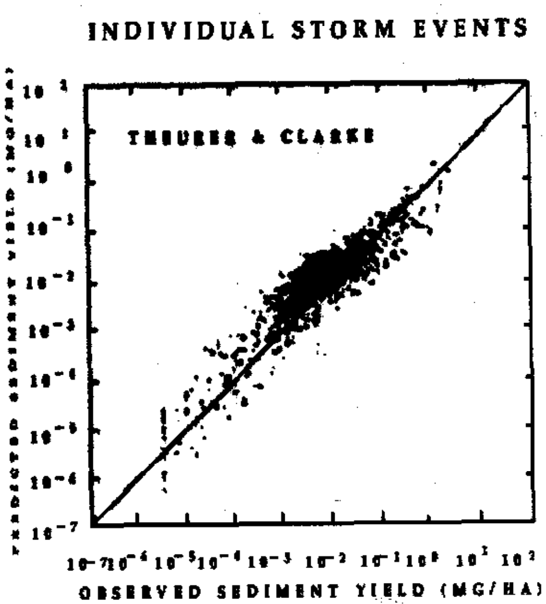

Figure 5

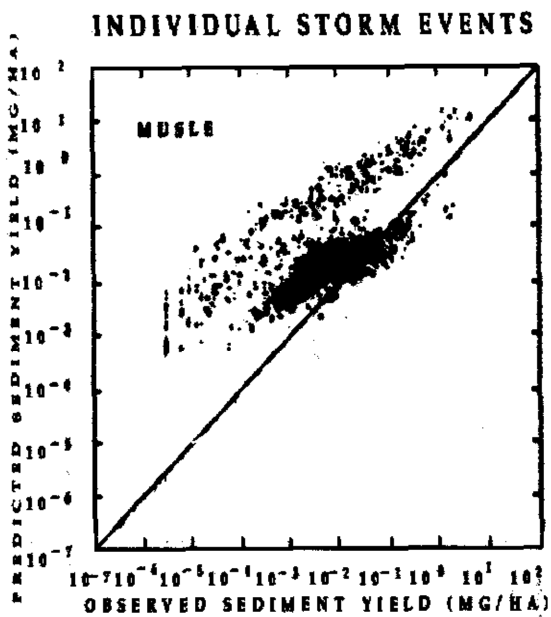

Figure 6

INTERPRETATIONS AND CONCLUSIONS

Sheet and rill erosion produce mostly fine sediment--the finer fraction of which is wash load. Therefore, wash load generally originates from sheet and rill erosion and is always associated with the respective storm event.

The concentrated flow component of sediment yield is mostly bed-material load. Bed material does not contain a significant amount of wash load. Furthermore, while the ultimate source of the bed-material load is upland erosion, it tends to move in waves downstream--not entirely by a single storm.

The sheet and rill erosion component is closely associated with USLE while the fundamental physical processes underlying the concentrated flow component is not. There is more scatter in the two-component model error statistics for the Reynolds Creek than for the Palouse-area locations (see Table 4).

The two-component model is a good predictor of total sediment yield (and also better than the other three) where wash load is the dominant source of sediment yield because of its strong sheet and rill erosion prediction capabilities. Equation 6 can be used in the Palouse-area geomorphic region as a good model for total sediment yield estimations.

Better bed-material transport models exist than the concentrated flow component of the recommended model (Eq. 6) or any of the other three regression-type models (equations 7 through 9). Applicable bed-material models could be substituted for the concentrated flow component and used with the sheet and rill component to predict total sediment yield.

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Hydraulic Model Studies of Tien-lun Reservoir Desiltation

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Abstract

Experience on hydraulic flushing of sediment deposits through low-level sluices to clear reservoir sedimentation in Tien-lun reservoir is described for studies conducted both in models and in prototype. Problems related to the geometry design of the desilting sluices,uncertainty of desilting capacity and abrasion/ erosion of concrete structure are discussed .

Introduction

In central Taiwan there are many rivers characterized by rather extreme composition of coarse sediments ranging from gravel to cobble. Tachia River is one of the typical examples , where a series of dams , from upstream to downstream,Te-chi,Chin-shan,Kukuan and Tien-lun were built. In this series of dams and reservoirs , hydraulic flushing through low-level sluices was applied at Kukuan and Tien-lun Dam. The problems related to their operation are the geometry of sluices,uncertainty of desilting capacity, blockage of gates by sediments and abrasion/erosion of concrete structures. In order to solve these problems several undistorted hydraulic models,all with 1:40 scale were built at different stages for Tien-lun Dam. Froude law was adopted to determine the model scales. In modelling the sediment behavior, for coarse sediments, they were scaled down in linear scale. For fine sediments, they were scaled down by using artificially light materials of equivalent fall velocity. Experiments were conducted for sluices with different geometries and dimensions to determine their desilting capacity and optimal shapes . Deficiencies in design and operation are discussed and remedial measures which were undertaken to repair damage and overcome the deficiencies are described.

Tien-lun Dam Project

Tien-lun Dam, a concrete gravity dam of 33.8-m high, controls a drainage area of 797 km^2 with spillway capacity of 4,500 cms. As the sediment concentration is high in the sluice, hydraulic structures are usually subject to the abrasion/erosion of the sediment-laden flow . Hence, the shape of sluices, especially the sluiceway conveyance structure and energy dissipator is critical in the hydraulic design of the structure. Tien-lun dam, put into operation in 1952 as a firm power plant , was initially constructed as an un-controlled, free overflow structure. However,in 1956,it was changed to a peak power plant and the dam crest was lowered 1.2 m and the spillway crest was changed into a

Plate 1 Operation of Tien-lun Sluice under 400 cms Flow

Plate 2 Damages of Tien-lun Sluiceway after 400 cms Flow

gated structure. The modified structure consists of a 5-bay spillway designing for 3,700 cms , and 2-bay sluiceway with design discharge of 800 cms to prevent sediments from entering the intakes to the powerhouse tunnel.

Sediment Flushing Technique

Towards the end of the water supply season or before the beginning of the high flow season, a reservoir normally retains some water; this water can be used to desilt sediment deposited from the previous years. When the desilting sluices are opened the water level in reservoir begins to fall, and flow towards the sulices is generated , The reservoir can be flushed by either pressurized condition or free flow condition. Both prototype and model operations showed that flushing under free flow is capable of transporting a much greater sediment load than when flushing under pressurized conditions . The applicability of this kind of flushing can be further extended if there is a series of dams in a river system. In addition to the natural flow addition flow can be made available from the upstream reservoir system. Dams in the Tachia River,and particularly the Tien-lun and the Kukuan Dam, are typical examples in which such a desilting practice is performed . However , sluicing of the sediment also induces many difficult problems to be solved hydraulically.

Plate 1 shows the operation of Tien-lun sluiceway under 400 cms flow.

Sluiceway Geometry Problems

Operation of the original sluiceway induced tremendous difficulty in maintaining the dam apron and sluiceway invert in shape. The outlet of the original sluiceway is joined with hard rock canyon. The side wall of the sluiceway curves along the edge of the canyon wall, wide and clear at the crest and tapering toward the toe of the apron. As originally designed, the sluices were effective in moving material from the intake area. However, as a result, the concave side of the sluiceway received direct impact of both sediment and water . The flow was reflected back to the apron of the right spillway bays. The combined effect of water and sediment eroded the underlying materials . Erosion cavitation along the sluiceway chute was severe, and damage of the right bay apron was critical to the stability of the dam structure. As result of a survey from 1957 to 1961, a hole eroded up to about 15 m in depth and 5,000 m^3 in volume was noted . The maximum sluiceway discharge in that period was 400 cms. Plate 2 shows the damage of tien-lun sluiceway after 400 cms flow.

In order to improve this condition , a 1:40 model of the sluiceway was constructed in 1959 ,and it was found in addition to the reasons aforesaid, the main reason for such damage was due to the fact that the design head of the sluiceway has been raised following the modification of the dam structure. As a conquence,the nappe of the flow did not coincide with the sluiceway surface and induced negative pressure along the chute.

In the first effort of modifying the design, the Ogee shape was changed into a superelevated chute type structure with mild slope. The flow direction was straightened and a flip was added to spread the flow as far away as possible. However , model studies showed that, with the comparatively low head available, this design was such that cross waves downstream from the bays reduced velocities and consequently discharge capacity. As a result of further studies,the shape of the pier nose between the sluiceway bays was streamlined and a rounded side wall which gradually decreased the height from upstream to downstream was constructed to carry some of the water off the side of the sluiceway chute. Discharge in the sluiceway acted as a spatially, gradually varied flow decreasing its momentum from upstream to downstream, thus, reducing the impact force at the downstream river bed where the spillway apron joined together and tremendous erosion had been recorded in the original layout of the dam structure,Plate 3. The slope of the rounded side wall was adjusted so that enough momentum could be obtain to carry out the sediment flushing out through the two sluiceway bays and enable more bed material to be transported from in front of the intakes. Reconstruction of the prototype , with these features, was completed in Jan. 1961. Survey of 1962 to 1966 indicated considerable improvement in the downstream apron, however, minor abrasion of concrete surface was still noted. In these period, a maximum flood of 943 cms which is larger than design capacity , 800 cms, was released through the sluiceways.

In 1973 the upstream Te-chi dam , an 180-m high arch dam was completed. With more water becomes available, the frequency of sluiceway operation increased and appreciable damage has been observed. In the latter rehabilitation work the sluiceway chute walined with steel plate embedded with wasted railway I-beam. The steel plate resisted the erosion and no major damage has been observed.

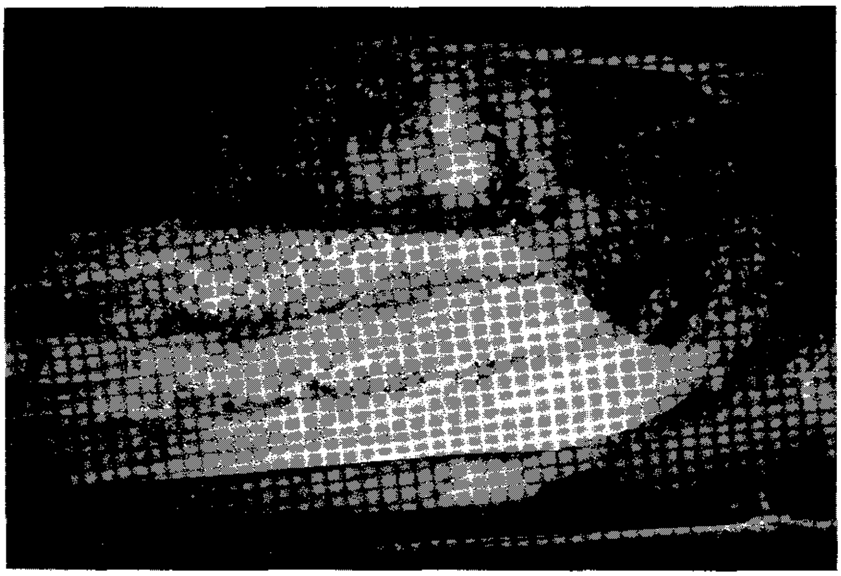

Plate 3 Spatially varied flow chute has been proposed to cope with cavitaion/erosion by using 1:40 scaled model

Discharge Calibration Curves

Operation of the Tien-lun reservoir has been in considerable difficulty because of the effect of silting on the discharge rating of the spillway and sluiceway . The hydraulic model was further used to calibrate the rating curves for different stages of sedimentation. Model observation showed that slight decrease in discharge coefficient has resulted for a medium water head. This is due to the fact that after truncation of the dam crest, an apron type of approach has been left in the upstream face of the dam. This induces eddy currents and disturbs the approach flow, consequently, reduces the spillway flow capacity. Test results also showed that after sedimentation of the reservoir, the spill capacities for higher stage increase rapidly and hence,was considered to be on the safe side. Since then new calibration curves have been used in the reservoir regulation and operation .

Desilting Capacity of Sluices

With the optimal shape and layout implemented in the model, the model was further operated to determine the optimal operating mode and the desilting capacity of the sluiceway. Data were collected to determine sediment movement and redistribution of substantial sediment deposits within the reservoir area. Results indicated that under storage condition , there existed has no flushing effect. In addition, the relative efficiency of sediment withdrawl by reservoir drawdown varied considerably with fluctuation of reservoir storage and thus affect the distribution of sediment within the reservoir. Sediment desilting is greatest at empty reservoir storage and will be limited if a minimum storage is to be maintained. Numerous sediment transport equations have been proposed for the prediction of sediment load or concentration in rivers. Using the model desilting data,it was found that the parameters Vs/w and V^3/gdw were significant, and the desilting capacity of the sluice may be expressed in the form of

$$Cw = 0.024\ (V^3/gdw)^{2.66} \qquad (1)$$

or

$$Cw = 181.4\ (Vs/w)^{2.54} \qquad (2)$$

where Cw = the sediment concentration,in kg/m^3
V = the flow velocity in the tunnel,in m/sec
s = the energy gradient of flow in the tunnel
d = the depth of flow in the tunnel,in m
w = the falling velocity of the particles, in m/sec

In addition to being a function of the sluice geometry, the desilting capacity of a sluice also depends upon the flushing discharge and the scour length of the sediment deposits. The following empirical formula is developed to include these parameters.

$$C_w = 1170 \frac{B^{2/5} L^{2/5}}{Q^{2/5}} (0.56 s_b^{3/2} - 0.0003) \qquad (3)$$

where C_w = the desilting sediment concentration, in kg/m3
B = width of desilting flow, in m
L = length of desilting reach, in m
$s_b = \Delta H/L$ = slope between the location where desilting begins and the sill of the bottom outlet, see Fig.1

The gorge under studied are relatively narrow and flow is relatively two-dimensional, hence,the sediments are deposited in a concentrated from and consequently can be flushed effectively.

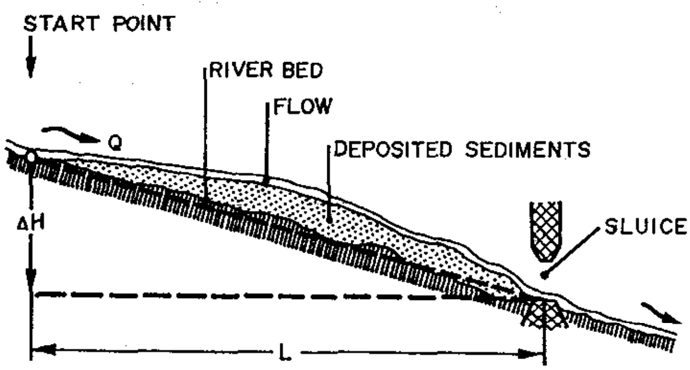

Fig. 1. Definition Sketch of Desilting

Conclusions

The Tien-lun Dam Project, the most significant desilting achievement of coarse sediments in Taiwan, illustrates the benefits that are derived from modern day technology of hydraulic model study. Hydraulic model was effectively used to overcome unfavour conditions induced by sediment-laden flow, and subsequent prototype operation has verified the expected results. The above desilting experience in prototype and model reservoirs has indicated that hydraulic flushing is an efficient technique for the removal of sediment deposits . Finally a set of desilting capacity equations is suggested for preliminary estimates of desilting efficiency.

THE WISCONSIN NONPOINT MODEL (WINHUSLE)

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ABSTRACT

The Wisconsin Nonpoint sediment yield model (WINHUSLE) estimates instream sediment yields throughout rural watersheds and sediment delivery from individual fields to designated downstream points. WINHUSLE runs on either a single event or average annual basis, under either existing or alternative land management practices.

From the user's perspective, the model has several notable attributes. Four of the more significant are: 1) it uses input data based on fields and hydrologic areas rather than grid cells, 2) it uses geomorphic relationships to derive default hydrogeomorphic parameter values, 3) it uses the Soil Conservation Service's (SCS) Computer Assisted Management Planning System (CAMPS), and 4) it uses a nationally available weather simulator.

Sediment yield calculations within the model are based on runoff volume and peak discharge estimates (based on SCS Technical Release 55 [TR-55]), and the use of a unit-area, Universal Soil Loss Equation (USLE) related, runoff and peak discharge regression equation. The model is applicable to any agricultural area, but must be calibrated to any hydrogeomorphic region where it is used.

INTRODUCTION

The Wisconsin Nonpoint model (WINHUSLE) estimates sediment yield from fields and hydrologic areas to designated downstream receiving waters. The original model, known as WIN, was developed by the Wisconsin Department of Natural Resources (WDNR) in 1986 and 1987 under a grant from the Environmental Protection Agency (Baun and Snowden, 1988).

Between 1987 and 1990, WIN was applied to over 100,000 fields on over 2,000 square miles of land in WDNR's Nonpoint Source Program. In this program, the model has been used to estimate watershed sediment loads, to help set sediment reduction goals and management strategies, to identify critical sediment source areas (and hence land owner eligibility for state cost share assistance), and to evaluate the effectiveness of field specific management practices.

In the fall of 1989, a team of state and national SCS staff evaluated the WIN model. Since then, WDNR and SCS staff have revised the model and made significant changes in the way that data is collected and sediment yield is calculated. In the original WIN model, runoff was routed from field to field to stream, and sediment delivery was a function of USLE, volume of runoff, and runoff travel time. In the revised WINHUSLE model, fields are associated with hydrologic areas, runoff is routed through hydrologic areas, and sediment delivery is a function of USLE, volume of runoff and peak flow rate. Conceptually WIN and WINHUSLE are quite similar. In implementation, they're quite different.

This paper is about the revised WINHUSLE model. It includes a description of the model from the user's perspective, and a description of the analytical components of the model.

USER'S PERSPECTIVE

WINHUSLE is a distributed parameter model. Distributed parameter models are typically very data intensive and therefore quite time consuming, and WINHUSLE is no exception. However, several aspects of the model combine to make it relatively manageable. The most significant features include polygonal input data, the use of geomorphic relationships to develop hydrogeomorphic parameters, a tie to SCS's CAMPS database, and the use of a nationally avaiable weather simulator.

Polygonal Input Data

Input data for WINHUSLE consists of data collected for and based on irregularly shaped fields and hydrologic areas. There are several advantages to polygonal based input data compared to grid cell input data. First of all, it allows for more homogenous data collection, with uniform land cover, land management, land ownership, and flow routing. Secondly, fields are a common unit of data collection, so much of the required data may have already been collected by someone else, or could be used by someone else. Third, the size and shape of fields corresponds to the diversity of the landscape. Where there's a lot of diversity (and considerable detail is desired for accurate modeling) they tend to be smaller, and vice-versa. Additionally, fields are the unit of farm management decisions, so the output of the model can readily be used in developing farm management plans. Lastly, polygonal input data is the basis of GIS and automated data entry.

WINHUSLE input data consists primarily of three components: 1) a sediment detachment (erosion) related field inventory, 2) a sediment delivery related hydrologic drainage inventory, and 3) a means to overlay the first two. WINHUSLE requires 100 percent inventories of both the fields and hydrologic areas within the watershed being modeled.

The field inventory consists of delineating field boundaries and collecting the usual USLE parameters (K, L, S, C and P [collectively KLSCP]) for each field (Wischmeier and Smith, 1978). Also the crop rotation of cropped fields must be specified. The user defines the acceptable crop codes, and sets the Manning's N and SCS runoff curve numbers for each crop.

The hydrologic drainage inventory consists of delineating hydrologic areas and collecting data for each area. Input parameters include drainage area, channel characteristics (length, slope, Manning's N and flow depth), and ID of the hydrologic area that it drains to. In lieu of inventoried channel characteristics, the model will default to hydrogeomorphically derived values (see next subsection).

The division of a watershed into hydrologic areas is a subjective process. On one extreme, a watershed could be considered a single hydrologic area, or on the other extreme, a watershed could be divided into myriads of absurdly small hydrologic areas. The smaller the hydrologic areas, the more accurate the field specific estimates of sediment delivery, but the more cumbersome the data collection. A balanced delineation of hydrologic areas is desired that will yield reasonably accurate sediment yield estimates, without requiring an unreasonable amount of data collection.

Each field (or subfield) is associated with the hydrologic area in which they are contained. The model then uses the USLE data for each field in the hydrologic area to compute composite USLE and runoff values, and to calculate sediment delivery.

Hydrogeomorphic Parameters

Several parameters are required for computation of peak discharges within hydrologic areas. They include channel length, slope, hydraulic radius and Manning's N. The user can enter inventoried values directly into the database, or they can rely upon hydrogeomorphic relationships to obtain default values for these parameters.

The first hydrogeomorphic process relates channel length (in feet) to drainage area. The equation relating channel length to drainage area (Leopold et. al., 1964) is:

$$\text{Length} = 153.127 * \text{Acres}^{0.6} \qquad (1)$$

Other hydrogeomorphic processes relate the flow depth, Manning's N and channel slope to channel length, given a channel depth function and the Manning's channel N and elevation values at the upper and lower watershed extremes (Theurer, 1990). The hydrogeomorphic equation for flow depth assumes that the channel hydraulic radius is equal to the bank full flow depth. The estimate of flow depth ($D_{(L)}$) at any channel length (L) is given by:

$$D_{(L)} = \text{Delta} * (\text{L}/\text{Alpha})^{(\text{Epsilon}/0.6)} \qquad (2)$$

$$\text{where Epsilon} = (\ln(D_1)-\ln(D_{10})) \,/\, (\ln(1)-\ln(10)) \qquad (3)$$

where D_1 and D_{10} are the average channel flow depths (feet) of one and ten square mile watersheds

$$\text{and where Delta} = 2/(640^{\text{Epsilon}}) \qquad (4)$$

Epsilon must be calibrated for each new hydrogeomorphic area.

The hydrogeomorphic equation for Manning's N is an exponential curve fit equation to collocate extreme watershed channel roughness end points. The model assumes that the open channel flow Manning's N value at the upper extreme of the watershed is 0.05, and at the lower extreme is 0.035. The equation is dependent upon the hydraulically most distant channel length (L_s) for the entire area or watershed. The equation for the channel Manning's N ($N_{(L)}$) value at any channel length (L) is given by:

$$Tau = (\ln(0.05)-\ln(0.035))/L_s, \text{ and} \quad (5)$$

$$N_{(L)} = 0.05 * \exp(-Tau*L) \quad (6)$$

The hydrogeomorphic equation for channel slope is an exponential curve fit equation to collocate extreme watershed elevation end points. The user must enter the uppermost and lowermost elevations of the watershed ((Z_u) and (Z_l) respectively). The equation is dependent upon the hydraulically most distant length (L_s) for the entire area or watershed. The equation for channel slope ($S_{(L)}$) at any channel length (L) is given by:

$$Mu = (\ln(Z_u)-\ln(Z_l))/L_s, \quad (7)$$

where Z_u is the uppermost watershed elevation and Z_l is the lowermost watershed elevation, and

$$S_{(L)} = Z_u * Mu * \exp(-Mu*L) \quad (8)$$

SCS's CAMPS System

Another advantage of the WINHUSLE model is that it is tied directly to SCS's CAMPS system. CAMPS is database system related primarily to farm inventories and farm plans. Currently, there are two versions of CAMPS nationwide: a DOS version and a UNIX version. DOS CAMPS outnumbers UNIX CAMPS about 50 to 1 in Wisconsin, so WINHUSLE currently runs on DOS CAMPS. The model uses an interface program called R-Bridge to access the DOS CAMPS R-Base files.

WINHUSLE is tied to CAMPS through CAMPS' "state and local options". In the process, several tables were expanded or added to the CAMPS database. The USLE table was expanded to include geographic identifiers and a second set of variables for "before and after" soil loss and sediment delivery analysis. By using the same table, the data that's collected for USLE soil loss analysis for farm plans is also used, without any conversion, for sediment delivery analysis, and vice versa.

In addition to the USLE table, another table was added for the hydrologic area inventory, and a third table was added to relate fields to hydrologic areas. Several other tables were added to faciltate or validate data entry.

By having the model tied directly to a database, the user can harness the power and functions of the database system to collect, edit, analyze, summarize and report the data.

In another year or two, the SCS will develop a new UNIX version of CAMPS and phase out both current DOS and UNIX versions. It is anticipated that both the "state and local options" package, and the WINHUSLE model itself, will need to be rewritten for the new system.

Weather Simulator

The model can be run in either a single design event or average annual mode. In either case, it uses rainfall files that are generated by the Water Erosion Prediction Project's (WEPP) Climate Generator (Nicks and Lane, 1989). This weather simulator will produce statistically derived rainfall files for any area of the United States.

ANALYTICAL COMPONENTS

Model Configuration

When the model is run and the data read in, the "record" of each field and hydrologic area contains all of the inventory data, plus several other variables calculated by the model, plus "pointers" to records of other upslope and downslope hydrologic areas. By knowing which hydrologic area flows to which, and which fields are in which hydrologic area, the model can recreate the geomorphology and hydrology of the watershed.

The use of pointers allows the watershed to be analyzed in either a top-down or bottom-up sequence or traversal. In a top-down traversal, all of the hydrologic areas above any other area are analyzed before the given area is analyzed. In a bottom-up traversal, all of the hydrologic areas below any other area are analyzed before the given area is analyzed. Depending on the task at hand, the model uses both top-down and bottom-up traversals.

In the following discussion of sediment yield, reference is made to both local and composite values for several variables. Local values refer to the calculations of the variable for a single hydrologic area. Composite values refer to the calculations of the variable inclusive of all other hydrologic areas above a point (top-down) or below a point (bottom-up). Composite area, time of concentration, runoff volume, runoff weighted KLSCP, peak discharge rate and sediment yield are calculated inclusive of upslope areas from the top-down. On the other hand, the composite sediment delivery from an area to a downstream point is calculated inclusive of downslope areas from the bottom up.

Sediment Yield Predictions

Sediment yield in the WINHUSLE model uses a hydrogeomorphic process, based on a unit-area, USLE-related, runoff and peak discharge calibrated power curve (Theurer and Clarke, 1991). WINHUSLE uses the procedures described in TR-55 (USDA, 1986) for estimating the runoff volume and peak discharge rate for each event. Because of the widespread familiarity and availability of these procedures, the specific equations governing these procedures are not repeated here.

In estimating sediment yield, the model first determines the local and composite times of concentration for each hydrologic area. The time of concentration is the estimate of the runoff travel time from the hydraulically most distant point in the individual (local) and collective (composite) hydrologic areas. By knowing which areas flow to which, the model can estimate the greatest hydraulic distance to any point.

Calculations of time of concentration along the hydraulically most distant path is divided into three segments. For the first 300 feet of travel, runoff flows as sheet flow. Sheet flow velocity is computed as a function of the 2-year, 24-hour rainfall, the slope of the hydrologic area, and Manning's N of the land cover. The Manning's N value of each hydrologic area is derived by taking the area weighted Manning's N values of the land cover or crop rotations of the fields within the hydrologic area.

From 300 to 1500 feet of travel, sheet flow aggregates into and travels as shallow concentrated flow, and velocity is computed as a function of the slope of the hydrologic area. Beyond 1500 feet of travel runoff aggregates into and travels as open channel flow, and velocity is calculated as a function of the slope, hydraulic radius and Manning's N of the channel.

For all three segments, the flow distance divided by the flow velocity yields the travel time through the segment. Time of concentration is obtained by summing the travel time estimates for the three segments.

After calculating time of concentration, the model calculates the runoff volume and KLSCP value for each hydrologic area. The SCS runoff curve number of each field is obtained based on the land cover and hydrologic soil group. The runoff curve number, combined with the rainfall volume, yields runoff volume from each field. The runoff volume from each hydrologic area is derived by summing the runoff volumes of the fields or subfields within it.

The KLSCP value of each hydrologic area is based on the KLSCP values of the fields within it. A runoff weighted KLSCP value is obtained by multiplying the KLSCP of each field by the volume of runoff from the field, and then dividing the total of these calculations by the total runoff from the area.

Once the runoff volumes and KLSCP are established for each hydrologic area, they are summed and passed on down to the area below it (top-down) to derive a composite volume and KLSCP for each area.

Peak discharge at any point is a function of the volume of runoff (inches) and the time of concentration (Tc). During event calculations, the unit peak discharge rate is calculated from the time of concentration, the initial abstraction of the rainfall, and the SCS regional rainfall distribution pattern (I, IA, II or III). Finally, the peak discharge rate at is obtained by multiplying the unit peak discharge rate by the drainage area.

The event specific, unit-area sediment yield (Sy, in tons/acre) at any point is given by:

$$Sy = a * Q^b * qp^c * KLSCP \qquad (9)$$

where

a, b and c are calibration parameters,

Q is the inches of runoff,

qp is the peak discharge rate, and

KLSCP is the product of K, LS, C and P.

For any hydrologic area, the local sediment yield is calculated using equation (9) and local values of Q, qp and KLSCP. Likewise, the composite sediment yield (from the combined area above and including each hydrologic area) is calculated using equation (9) and the composite values of Q, qp and KLSCP.

The sediment yield predicted by the model employs an accounting system (based on conservation of mass) for hydrologic areas in parallel and series. The sediment yield at the bottom of each of two areas in parallel (i.e., they both flow into the same downslope area) is additive at their confluence. The total sediment out of the two areas into the downslope area is simply the sum of the sediment from the two areas.

Hydrologic areas in series however pose a different situation. The sediment yield at the lower end of two areas in series (say where area "A" flows to area "B") is not additive because of the deposition losses of sediment from the upper area as it flows across the lower area. In this case, the sediment yield from the combined areas AB will <u>not</u> equal the sediment yield from A plus the sediment yield from B.

The model however can calculate the local sediment yield out of areas A and B independently, as well as the composite sediment yield out of the combined areas AB. By the conservation of mass then, the sediment yield from A delivered through B equals the composite sediment yield from AB minus the local sediment yield from B. By knowing the sediment yield into a downslope area, and the local and composite sediment yield out of the area, the sediment deposition of the upslope area(s) in the downslope area can be determined.

After the sediment yield is determined for each hydrologic area, the sediment yield is proportioned back to the fields within each area according to the volume of runoff and KLSCP value of each field.

The model outputs the event specific local and composite sediment yield (tons) for each hydrologic area, and the estimated sediment yield (t/ac) from each field delivered to the outlet of the watershed. If the model is being run on a single design event basis, it outputs the results of that one event. If the model is being run on an average annual basis, the model tallies the sediment yield for each area and field for each event for every year of rainfall. When all of the events are done, it divides them by the total number of years of rainfall, and outputs the average annual data.

CONCLUSIONS

WIN has proven to be an effective tool for Wisconsin's Nonpoint Source Program and WINHUSLE even more promise. Hopefully, chemical yield, flow and delivery through impoundments, and a tie to a GIS, can be added to the model in the future.

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MATHEMATICAL MODEL OF SEDIMENT TRANSPORT

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ABSTRACT

Based on the principle of nonequilibrium transport of nonuniform sediment a mathematical model of fluvial process and reservoir sedimentation has been developed. The transport equation system for each size group, which is related with the size grade of carrying capacity and of bed material, is much more universal with only very few parameters to be calibrated. Besides, some important aspects, such as the variation of roughness and the variation of 2-D geometry of cross section are taken into consideration. The model has been checked by more than ten sets of field data.

INTRODUCTION

This is a one-dimensional, steady, movable boundary, open channel flow and sediment transport model designed to simulate the large scale reservoir sedimentation and fluvial process over long section period. The main feature of the model is that the transport equation is based on the principle of nonequilibrium transportation of nonuniform sedimentation. (Han 1980 a,b;He & Han 1986). The model has been verified by more than ten sets of field data. Each of these sets was collected along a rather long river reach over a long period of time (Han 1980 a,b; Han & He 1987).

BASIC PRINCIPLE of NONEQUILIBRIUM TRANSPORTATION

Comparing with the other material diffused or transported in the flow the sediment is much more nonuniform. The settling velocities could be different as large as several ten thousand times. Since the finer particles are easy to scour up and the coarser are easy to settle down, the exchange of moving states between finer and coarser particles always exists even in the steady flow and thus the process appears as a nonequilibrium one. In this process the size grade of concentration or transport rate and bed material has strong influence on total amount of concentration and transport rate and thus has to be considered theoretically. Based on the concept of exchange intensity the system of transport equation of suspended load for each size group is derived in the following form,

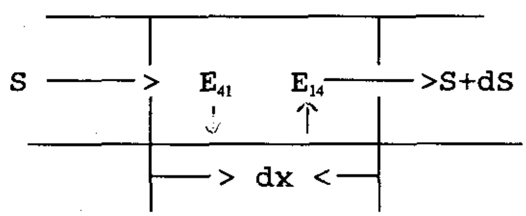

Fig.1 Sediment Exchange

$$\frac{dS_L}{dx} = \frac{\alpha w_L}{q} (S_L - S_L^*), \quad (L = 1,2,\ldots,m_L) \tag{1}$$

where m_L is the number of the size group, w_L the settling velocity of Lth size group, S_L and S_L^* the concentration and carrying capacity of each size group, q the flow discharge through unit width and α an empirical constant. The terms at the right side $E_{41L}=(\alpha w_L/q)S_L$ and $E_{14L}=(\alpha w_L/q)S_L^*$ are the exchange intensities from suspended to rest and from rest to suspended respectively. The physical meaning of eq.(1) is clearly illustrated in Fig.1. The difference between sediment discharge through outlet and inlet section is just equal to the amount of sediment scoured from bed material, which is expressed as the difference of exchange intensities from and to bed material. Summarizing eq.(1) turns to the equation of total concentration,

$$\frac{dS}{dx} = \frac{\alpha w}{q} (S - S^*) \tag{2}$$

where S^* is the total amount of carrying capacity and the empirical formula of total carrying capacity

$$S^* = K \left(\frac{V^3}{ghw}\right)^m \tag{3}$$

is available in practice, K is a constant, h the water depth, g gravity acceleration and w the average settling velocity. The equations of size grade of concentration is presumed for different situation. For intensive deposition they are calculated by the formulae

$$P_{4L} = P_{4L0} (1 - \Gamma)^{(wL/w0-1)} \tag{4}$$

where P_{4L} and P_{4L0} are the size grade at outlet and inlet section, Γ the percentage of deposition $\Gamma = (S - S_0) / S_0$, S and S_0 the sediment concentration at outlet and inlet section respectively and w_0 the solution of the following equation

$$P_{4L0}(1 - \Gamma)^{(wL/w0-1)} = 1 \tag{5}$$

According to eqs.(4) and (5) it is obviously that suspended load is getting finer in the process of deposition.

In the case of intensive erosion the size grade of concentration turns to

$$P_{4L} = \frac{1}{1-\Gamma} \left(P_{4L0} - \frac{\Gamma}{\Gamma^*} R_{1L0} \left(1-(1-\Gamma^*)^{(wL/w'0)}\right)\right) \tag{6}$$

where R_{1L0} is the size grade of bed material within layer h_0, Γ^* the percentage of scouring,

$$\Gamma^* = (h - h_0)/ h_0$$

w_0' the solution of the equation

$$\frac{1}{1-\Gamma} (1 - \frac{\Gamma}{\Gamma^*} \Sigma R_{1L0} (1-(1-\Gamma^*)^{(wL/w'0)})) = 1 \qquad (7)$$

and h and h_0 the scouring and efficient depth of bed material respectively. Eqs.(6) and (7) are also the expressions of armoring process.

The third case is for weak erosion and deposition. The detail of the theory of nonequilibrium transportation has been described in refs. of the authors (Han 1980 a,b; Han & He 1987;He & Han 1986).

BASIC ASSUMPTION and FUNCTION of MODEL

Reservoir sedimentation or fluvial process is a long term process. The numerical solution can be solved at successive discrete time period, in which the flow is assumed as steady and the sediment concentration is not so high to influence the structure of the flow. Therefore two step procedure is available. The first is a backwater step to obtain water surface, elevation, flow depth and velocity with the known condition of flow discharge and of the water elevation at outlet section and the fixed bed surface. And second is a sweep downstream. Solving the system of transport equations yields the concentration of suspended load and transport rate of bed load and their size grade. Then sediment continuous equation is used to obtain the total amount and volume of deposition or scouring for each reach and to adjust the cross section geometry. The cycle is repeated with the updated geometry. The flow chart of the model is shown in Fig.2. The following is a brief description of the function of the model.

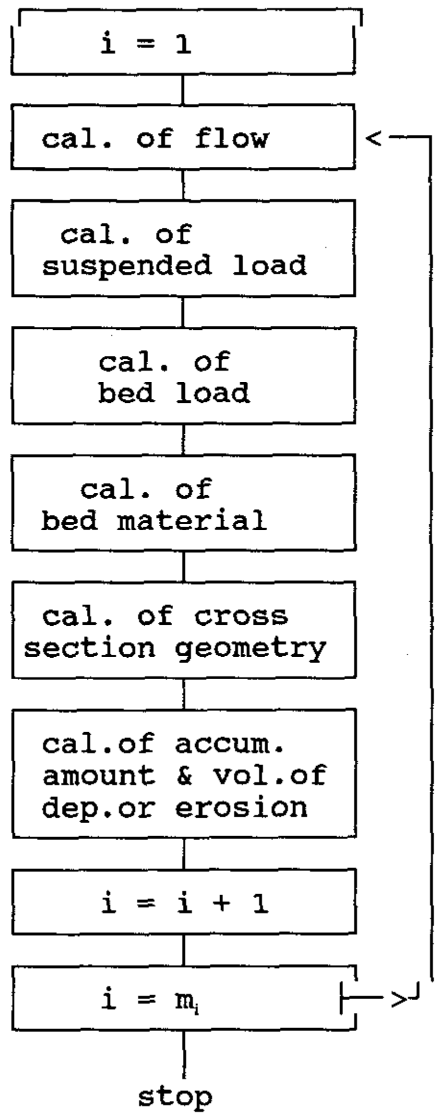

Fig.2 Flow Chart of Model

(1) Determination of roughness and computation of flow. The 1-D momentum equation of the flow

$$\frac{dH}{dx} + j_x + \frac{1}{2g}\frac{dV^2}{dx} = 0 \qquad (8)$$

is used to calculate the water elevation, where H is the water elevation, V=Q/A average velocity, Q discharge, A area of cross section and $j_x = n^2Q^2B^{4/3}/A^{10/3}$ the energy slope and B the width of the water surface of the cross section.

Determination of roughness coefficient is a main problem in computing river flow. For the alluvial river usually the hydrological data of water level and discharge are used to calibrate the Manning coefficients. In a reservoir in the process of sedimentation the roughness turns to be smaller since the bed material is getting finer and river bank getting more smooth. The following formula of interpolation is adopted to estimate the variation of roughness in the sedimentation process of reservoir

$$n^{3/2} = n_k^{3/2} + (n_0^{3/2} - n_k^{3/2}) \left(\frac{a_k - a}{a_k} \right) \qquad (9)$$

where a_k is the deposit area of cross section when reservoir sedimentation is in equilibrium, a is the real deposit area, n_0 the roughness before reservoir in operation and n_k the roughness of reservoir in equilibrium. Some principles are developed to present a more reasonable value of n_0 and n_k (Han & He 1987,1988).

(2) Calculation of concentration and size grade of suspended load

In practical computation the given distance between two cross sections Dx is not small enough so that the carrying capacity has to be assumed as a linear variable along the river reach and then the solution of the difference equation comes to be

$$S = S^*_0 + (S_0\Sigma P_{4L0}\mu_L - S^*_0\Sigma P_{4L0}\mu_L)$$

$$+ (S^*_0\Sigma P_{*4L0}\mu_L - S^*\Sigma P^*_{4L}\mu_L) \qquad (10)$$

where

$$\mu_L = \exp \left(\frac{\alpha_L w_L (B_0 + B) Dx}{Q_0 + Q} \right) \qquad (11)$$

$$\beta_L = \frac{Q_0 + Q}{\alpha_L w_L (B_0 + B) Dx} (1 - \mu_L) \qquad (12)$$

the parameters with and without subscript 0 express that of outlet and inlet section respectively. The size grade of suspended load are computed by eqs.(4),(5),(6) and (7) respectively.

(3) Calculation of transport rate and size grade of bed load

For bed load the step length is much smaller than that of suspended load, so the transport of bed load turns to equilibrium in a short distance. As a result, the transport rate is assumed to equal to the carrying capacity, but is still computed separately for different size group due to the significant difference among them. The equations of uniform bed load derived by the authors are used in the model (Han & He 1988). An efficient size grade P_{1L} is defined as

$$P_{1L} = \frac{P_{bL0}Q_{b0}Dt + P_{1L1}r_s'h_0DxB}{Q_{b0}Dt + r_s'h_0DxB} \tag{13}$$

to calculate the transport rate of each size group, and the total group has a transport rate as,

$$q_b = \Sigma\ q_{bL} = \Sigma\ q_b(L)/P_{1L} \tag{14}$$

where $q_{bL} = q_b(L)/P_{1L}$ is the transport rate of Lth size group, Q_{b0} and P_{bL0} the transport rate and size grade of income bed load, h_0 the disturbed depth of bed material, P_{1L1} the size grade of bed material, B the section width and r_s' the porosity specific gravity of deposit.

The velocity of pebble ranges only from several hundreds to several thousands meters per year and thus should be used to estimate the transport distance of pebble bed. A simplified formula adopted in the model is

$$V_{sL} = \frac{q_b(L)}{m_0r_sD_L} \tag{15}$$

where $m_0 = 0.4$ is the density index of rest particle at bed surface, r_s the specific density of particle, and D_L the particle size.

(4) Calculation of depth and size grade of bed material

In fluvial process usually only deposition occurs at flood plain, whereas in main channel aggradation and degradation appear alternatively. The size grade within the stable width of cross section, which expresses the average width of main channel along a certain river reach, will influence the sediment scouring in main channel and therefore should be estimated. The total depth is divided into several layers. Each of them has a fixed depth Dh, with the exception that the layer on bed surface sometimes less than Dh and the deepest layer might be equal to several times of Dh if the depth is too large for the given stored cells of bed material layer in computer. During a certain time period only an efficient depth will influence sediment scouring and Dh usually is assumed to equal to it.

The detail formulae of the variation of bed material is referenced in authors' paper (Han & He 1987 a,b).

(5) Calculation of cross section geometry

The cross section geometry is expressed by the function of section area $F_A(t,x,z)$ and section width $F_B(t, x, z)$, where t is time, x distance of section from original point, z water level, F_A the section area under z and F_B the width at the water surface. The transverse distribution of sedimentation over main channel and flood plain is quite different and effects the water stage and sedimentation significantly, so even in 1-D model it should be taken into account. According to the reservoir morphology three cases are considered separately,

i) The deposited and scoured sediment are distributed in equal depth along the wetted perimeter under the water stage during deposition or within stable width when scouring;

ii) The sedimentation level varies horizontally;

iii) The cross section is enlarged widely and deeply simultaneously.

The first case corresponds to wide section; the second to narrow section; and the third is available when the section is gradually scoured to a stead one. The detail is in reference (Han & He,1987 b).

INITIAL and BOUNDARY CONDITIONS and OUTPUT RESULTS

The initial and boundary conditions required are

(1) time intervals Dt_i, ($i = 1,2,...,m_i$), where m_i is the total number of intervals;

(2) length intervals Dx_j, ($j = 1,2,...,m_j$), the distance between neighboring sections, where m_j is the total number of cross sections;

(3) Initial cross section geometry f_{A0jk}, f_{B0jk} and f_{Z0jk}, where j is the index of cross section and $k = 1,2,...,m_k(j)$ the index of elevation;

(4) inflow discharge of main channel Q_i, where i is the index of time interval;

(5) water stage at outlet section H_{i0} ;

(6) inflow concentration of suspended load S_i and transport rate of bed load q_i of main channel;

(7) size grade of inflow concentration of suspended load P_{4L0} and of transport rate of bed load P_{bL0} of main channel;

(8) original thickness and size grade of bed material h_{0j} and $\bar{R}_{0Lj}$, which are especially necessary for alluvial river and river course downstream a dam site;

(9) the inflow (outflow) discharge, incoming (outgoing) concentration of suspended load and transport rate of tributary (diversion flow), if there is any;

(10) original roughness coefficient and the formulae of roughness variation in the sedimentation process;

(11) specific field data of water stage-flow discharge relationship for roughness acquisition and information on flow discharge-sediment relationship for determining formulae of carrying capacity.

Fig.3 Variation of Accumulative Depoosition in Sanmenxia Reservoir from 1967 to 1968

The main output results are

(1) water elevation H_{ij};

(2) concentration and size grade of suspended load S_{ij} and P_{4Lij};

(3) transport rate and size grade of bed load q_{ij} and P_{bij};

(4) thickness and size grade of bed material h_{ijk} and R_{Lijk};

(5) accumulative amount and volume of sedimentation Q_{sij} and V_{sij};

(6) geometry parameters of cross section F_{Aijk}, F_{Bijk} and F_{Zijk}.

VERIFICATION of MODEL

More than ten sets of field data have been used to verify the model. The items for comparing include water level, concentration and size grade of suspended load, armoring process of bed surface and accumulative amount of sedimentation and most are in good agreement (Han 1980 a,b; Han & He 1987). The verification for variation of accumulative deposition in Sanmenxia Reservoir is shown in Fig.3.

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