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〈論 文〉

댐파괴 모형의 비교평가
Comparative Evaluation of Dam-Break Models

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요 약

세개의 대표적 댐파괴 모형인 HEC－1, DAMBRK, SMPDBK를 먼저 이론적인 견지에서 분석하였고 그 다음 관측자료가 있는 Teton댐 파괴에 적용하였다.

그것으로부터 얻은 결론은 다음과 같다 : (1) 수문학적 추적방법을 사용하는 HEC－1은 본 연구에서의 거의 모든 경우에 안정한 해를 산출해 낸다 ; (2) dynamic wave 추적방법을 사용하는 DAMBRK는 세 모형중에서 가장 정확하다 ; (3) 일반화된 dynamic 추적관계를 사용하는 SMPDBK는 가장 경제적이고 쉽게 적용할 수 있다.

Abstract

Three representative dam-break models, HEC－1, DAMBRK, and SMPDBK were analyzed respectively in their theories and then applied to the failure of Teton Dam for which some observed data exist.

From the results of this study, it can be concluded that:(1) HEC－1, which uses the hydrologic routing method, produces stable solutions for almost all the cases that were tested in this study ; (2) DAMBRK, which uses the dynamic routing method, is most accurate among the three models ; (3) SMPDBK, which uses the generalized dynamic routing relationships, is most economical and easily applicable.

1. Introduction

Catastrophic flash flooding occurs when a dam is breached and the impounded water escapes through the breach into the downward valley. The potential for the flooding has recently been brought to the nation's attention, and in many countries it has been required by law to analyze the flood wave caused by the potential dam failure and to map the floodplain. In Korea, notable dam failures occurred at

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Hyogi-Ri in Namwon-Gun on July 13, 1961(DongA-Ilbo, 1961) and Mushim-Chun in Boun-Gun on July 23, 1980(HanKook-Ilbo, 1980), both of which brought large damages. Unfortunately, no technical reports exist on such floods. So, it is required to predict the potential damages caused by dam-break and mitigate them if they should occur.

There are not a few models which have been developed, among which the representative models are HEC-1, DAMBRK, and SMPDBK. These medels are evaluated comparatively in their theories and then applied to the Teton Dam failure which occurred in Idaho 1976 and has the flood data accumulated following the failure.

2. Review of Previous Studies

Previous studies on dam break include analytical, physical, numerical, and statistical models.

Ritter(1892) obtained an analytical solution for the problem of instantaneous release of water assuming that the flow is frictionless. Dressler(1952) developed a "dam-break function" using the perturbation techniques to correct the resistance effect on the whole flow region except the negative wave front. US Army Corps of Engineers(1960, 1961) used physical models to conduct an extensive investigaton of floods resulting from suddenly breached dams. Cristofano(1965) attempted to model the partial, time dependent breach formation in earthen dams. US Army Corps of Engineers(1981) developed the program HEC-1 which simulates the precipitation-runoff process and routes flood hydrograph using the hydrologic method. Fread(1984) developed the NWS DAMBRK model which assumes the rate of growth of the breach to be time-dependent with either rectangular, triangular, or trapezoidal shape and uses hydraulic methods for routing flood hydrograph. Wetmore and Fread(1983) developed the simplified model SMPDBK which assumes the breach shape rectangular and routes downstream using dimensionless peak flow routing graphs which were developed using the DAMBRK model. MacDonald and Jennifer(1984) collected data on a number of historical dam failure. Based on the result of their analysis, they developed graphical relationships to predict characteristics of erosion-type breaches and relationship to estimate peak outflow from dam failures.

Wurbs(1987) presented a comparative evaluation of the alternative state-of-the-art methods for predicting the flow characteristics of a flood wave which results from a breached dam.

In Korea, Ko(1986) applied the model HEC-1 to the potential failure of Paldang Dam. Han(1986) developed a dam-break flood forecasting model and applied it to the failures of Teton Dam and Hyogi-Ri dam.

3. Description of Dam-Break Models

3.1 Breach Simulation

A model must include some mechanism for representing the flow of water from the breached dam. However, most existing models do not have capabilities for predicting breach characteristics. Rather, the user must determine the characteristics independently of the model. Three models, HEC-1, DAMBRK, and SMPDBK can simulate the breach by overtopping. DAMBRK also has an additional option for simulating a piping failure.

When a dam failure emergency occurs and the warning response time is short, little data are available and large computer facilities are inaccessible. In this case, using the internally set default values, SMPDBK is capable of producing approximate flood forecasts after reading only the dam height, reservoir storage volume, and depth vs. width data.

3.2 Reservoir Routing

Reservoir routing is accomplished by either the hydrologic storage or dynamic routing methods. HEC-1 uses the hydrologic routing method and DAMBRK uses the hydrologic and/or dynamic routing methods. In this study, DAMBRK uses the hydrologic routing method. SMPDBK has the other routines which calculate the maximum breach outflow and the depth using broad-crested weir flow equation.

3.3 Downstream Routing

HEC-1 has five methods for downstream routing. They are Muskingum, modified Puls, working R & D, average-lag, and kinematic wave methods. Among them, modified Puls method is used in this study(Lee, 1988). In the method, a storage indication is computed by

$$SI_i = C_o\left(\frac{S_i}{\Delta t}\right) + \frac{O_i}{2} \tag{1}$$

where SI_i is the storage indication in cfs, S_i is the storage in the routing reach for a given outflow in acre-ft, O_i is the outflow from routing reach in cfs, C_o is the conversion factor from acre-ft / hr to cfs, Δt is the time interval in hours, and i is a subscript indicating corresponding values of storage and outflow. Storage indication at the end of each time interval is given by

$$SI_2 = SI_1 + \bar{I} - O_1 \tag{2}$$

where $\bar{I}$ is average inflow in cfs, and subscripts 1 and 2 indicate beginning and end of the current time interval. The outflow at the end of the time interval is interpolated from a table of storage indication vs. outflow. Storage is then computed by

$$S = \left(SI - \frac{O}{2}\right)\frac{\Delta t}{C_o} \tag{3}$$

When storage data are given, stages are interpolated from the computed storages.

DAMBRK uses the St. Venant equations which consist of the conservation of mass equation

$$\frac{\partial Q}{\partial x} + \frac{\partial (A + A_o)}{\partial t} - q = 0 \tag{4}$$

and the conservation of momentum equation

$$\frac{\partial Q}{\partial t} + \frac{\partial (Q^2 / A)}{\partial x} + gA\left(\frac{\partial h}{\partial x} + S_f + S_e\right) - qL = 0 \tag{5}$$

where Q is discharge, A and A_o is the active and inactive flow areas, respectively, q is the lateral flow per linear distance along the channel, h is the water surface elevation, S_f is the friction slope, S_e is the expansion-contraction slope, L is the momentum effect of lateral flow, g is the acceleration of gravity, and x is the longitudinal distance along the channel. The friction slope is evaluated from Manning's equation.

$$S_f = \frac{n^2 Q \mid Q \mid}{2.21 A^2 R^{4/3}} \tag{6}$$

where n is Manning's n and R is hydraulic radius. The expansion-contraction slope is defined as

$$S_e = \frac{k_e \Delta (Q / A)^2}{2g \Delta x} \tag{7}$$

where k_e(Morris and Wiggert, 1972) is the expansion-contraction coefficient varying from 0 to ±1. The momentum effect of lateral flow has the following form : (1) lateral inflow, $L = 0$: (2) seepage lateral outflow, $L = -0.5qQ / A$; and (3) bulk lateral outflow, $L = -qQ / A$. Eqs.(4) and (5) constitute a system of partial differential equations of the hyperbolic type. Of the various numerical schemes, Preissmann scheme(4-point implicit) with time weighting factor $\theta = 0.6$ is used.

SMPDBK uses dimensionless curves which were developed using the DAMBRK model. These curves are grouped into several families. The X-coordinate of the curves is the ratio of the downstream distance (from the dam to a selected cross-section) to a distance computed using the equations given by Wetmore and Fread(1983). The Y-coordinate of the curves is the ratio of the peak flow at the selected cross-section to the computed peak flow at the dam.

4. Model Applications to Teton Dam Failure

Teton Dam failed on June 5, 1976, when the reservoir was at El.5301.7 ft, 3.3 ft below the spillway sill. The Report to US Department of the Interior and State of Idaho(1976) concluded that the failure resulted from piping. Of the three models, only DAMBRK has the capability to treat piping failure, so it is assumed in this study that the failure resulted from overtopping.

The following breach parameters were used to reconstruct the downstream flooding due to the failure of Teton Dam(Fread, 1984) : t_f(failure time) = 1.25hrs, B_w(breach width) = 150 ft, z(side slope of breach) = 0, H_b(final breach elevation) = 0 ft, H_f(water surface elevation at the dam site when breach begins) = 261.5 ft, and Q_o(initial discharge) = 13,000 cfs. Cross-sectional properties at 12 locations (11 reaches) shown in Fig. 1 along the 59.5 mile reach of the Teton-Snake River Valley were considered.

There are three points, 8.5 miles (section No. 3), 43.0 miles (section No. 10), and 59.5 miles (section No. 12) downstream of the dam which have estimated peak discharge and observed time to peak. The results of model applications are shown in Figs. 2, 3, 4, 5, and 6.

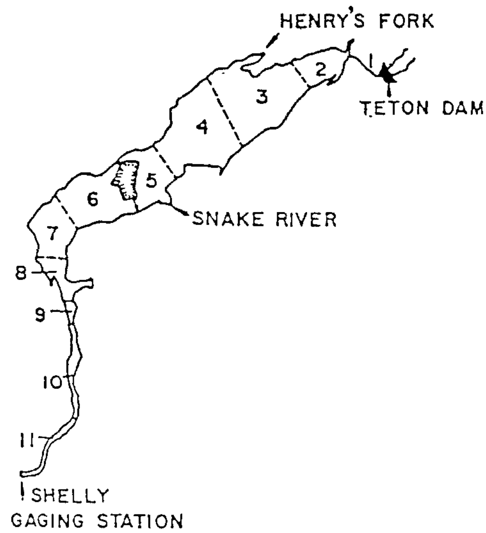

Fig. 1 Teton-Snake River Basin

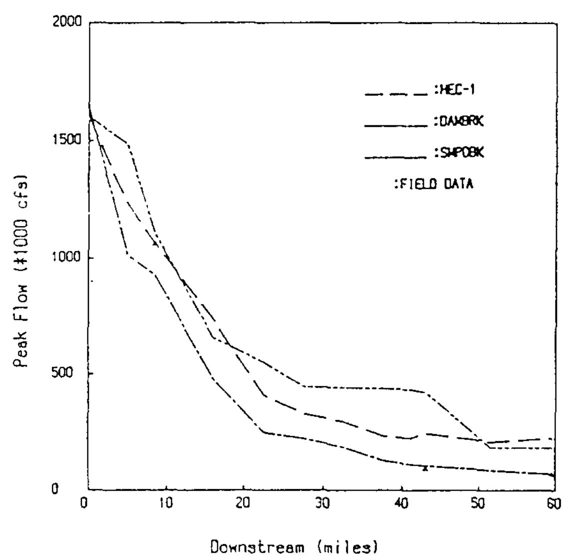

Fig. 2 Peak Flow for Teton Dam Failure.

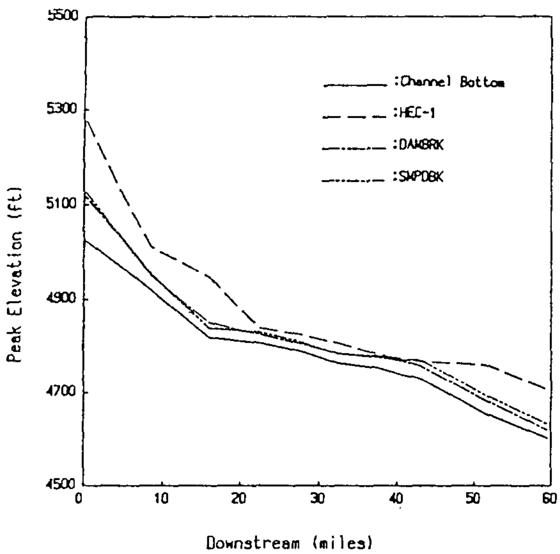

Fig. 3 Peak Elevation for Teton Dam Failure.

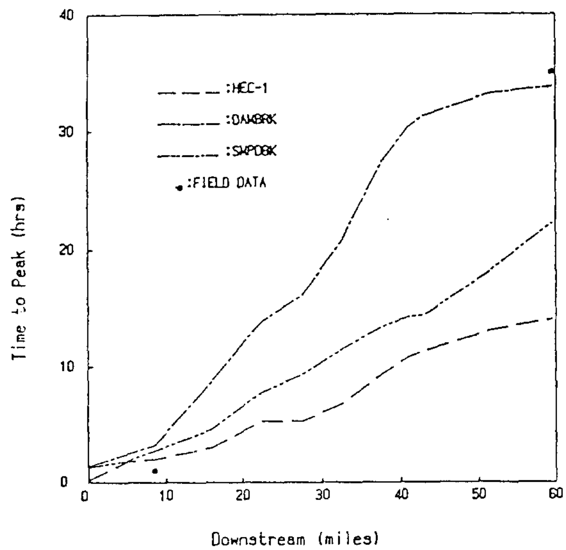

Fig. 4 Time to Peak for Teton Dam Failure.

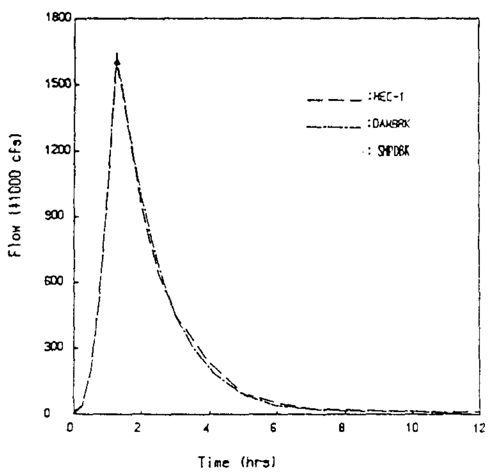

Fig. 5 Reservoir Outflow for Teton Dam Failure.

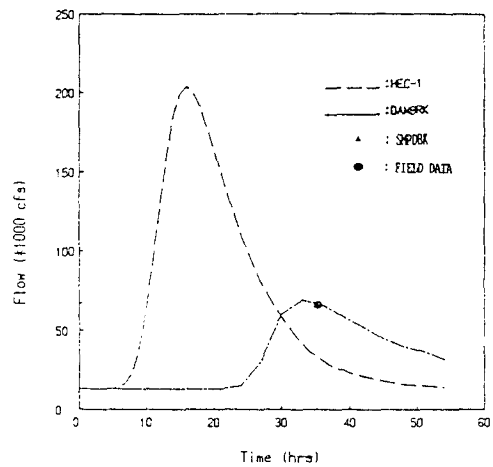

Fig. 6 Hydrograph at Shelly Gaging Station for Teton Dam Failure.

At the immediate downstream of Teton Dam (Fig. 5), three models yielded similar values of peak flow and time to peak. But, as the flood wave moved downstream, increasing differences among the three models were found. At Shelly Gaging Staion (Fig. 6), compared with the observed data, DAMBRK yielded +3% error of peak flow and -3% error of time to peak, while SMPDBK yielded +170% error of peak flow and -37% error of time to peak and HEC-1 yielded +230% error of peak flow and -60% error of time to peak. These results show that DAMBRK is most accurate, while SMPDBK is the second and HEC-1 is the last.

CPU time was checked as an indicator of the computer running time. When executed with MV 8000, SMPDBK, HEC-1, and DAMBRK required 1.9 seconds, 23.4 seconds, and 191.9

seconds, respectively. SMPDBK is capable of producing approximate flood forecasts with the least amount of data. In these respects, SMPDBK is most economical and easily applicable.

5. Sensitivity Analysis and Discussions

From the data of Teton Dam and Teton-Snake River Basin, the three models were compared using the sensitivity analysis of the parameters. The following parameters were considered : (1) failure time (t_f) ; (2) breach width(B_w) ; (3) final breach elevation (H_b) ; (4) elevation of water at the dam site when breach begins(H_f) ; (5) initial discharge(Q_o) ; and (6) Manning's n. The results are shown in Tables 1, 2, 3, 4, 5, and 6.

From the analysis, it can be concluded that : As t_f becomes longer, peak flow becomes smaller and time to peak is lagged by nearly the same change of t_f ; Larger B_w, lower H_b, higher

Table 1. Sensitivity to Failure Time(t_f) for Teton Dam Failure.

t_f(hrs)		Section NO.1		Section NO.3		Section NO.10		Section NO.12	
		Flow (*10³ cfs)	Time (hrs)	Flow (*10³ cfs)	Time (hrs)	Flow (*10³ cfs)	Time (hrs)	Flow (*10³ cfs)	Time (hrs)
0	HEC-1	1692	0.25	987	1.25	197	13.00	184	15.50
	DAMBRK	error	error	error	error	error	error	error	error
	SMPDBK	error	error	error	error	error	error	error	error
0.01	HEC-1	1694	0.25	990	1.25	203	13.00	190	15.75
	DAMBRK	error	error	error	error	error	error	error	error
	SMPDBK	1976	0.01	1325	1.42	505	12.49	177	20.87
0.1	HEC-1	1760	0.25	1044	1.25	209	12.75	194	15.50
	DAMBRK	1935	0.10	965	2.22	114	29.93	75	32.55
	SMPDBK	1945	0.10	1307	1.52	497	12.62	175	20.96
1	HEC-1	1664	1.00	1080	2.00	220	13.25	204	15.75
	DAMBRK	1710	1.00	937	2.95	107	30.74	70	33.38
	SMPDBK	1670	1.00	1146	2.46	434	13.99	180	21.91
2	HEC-1	1435	2.00	1028	2.75	218	14.00	202	16.50
	DAMBRK	1458	2.00	889	3.80	102	31.71	67	34.53
	SMPDBK	1422	2.00	984	3.50	375	15.45	182	23.20

Table 2. Sensitivity to Breach Width(B_w) for Teton Dam Failure.

B_w(ft)		Section NO.1		Section NO.3		Section NO.10		Section NO.12	
		Flow (*10³ cfs)	Time (hrs)	Flow (*10³ cfs)	Time (hrs)	Flow (*10³ cfs)	Time (hrs)	Flow (*10³ cfs)	Time (hrs)
10	HEC-1	129	1.25	116	4.75	83	24.75	82	27.75
	DAMBRK	129	1.25	108	4.87	36	46.43	27	49.83
	SMPDBK	142	1.25	96	3.83	39	23.74	39	30.78
100	HEC-1	1140	1.25	817	2.25	200	14.25	196	34.67
	DAMBRK	1168	1.25	715	3.54	97	32.04	64	23.29
	SMPDBK	1148	1.25	794	2.82	308	15.60	175	27.75
500	HEC-1	3066	1.25	1883	1.75	232	12.25	214	14.75
	DAMBRK	3074	1.13	1403	2.44	116	29.63	77	32.14
	SMPDBK	2244	1.25	1479	2.63	538	13.39	188	21.85
1000	HEC-1	3348	1.00	2174	1.50	232	12.00	214	14.50
	DAMBRK	3487	0.94	1460	2.25	118	29.38	78	31.90
	SMPDBK	1266	1.25	876	2.78	337	15.14	182	22.88

Table 3. Sensitivity to Final Breach Elevation (H_b) for Teton Dam Failure.

H_b(ft)		Section NO.1		Section NO.3		Section NO.10		Section NO.12	
		Flow (*10³ cfs)	Time (hrs)	Flow (*10³ cfs)	Time (hrs)	Flow (*10³ cfs)	Time (hrs)	Flow (*10³ cfs)	Time (hrs)
5088	HEC-1	1103	1.25	758	2.25	178	14.50	167	17.25
	DAMBRK	1136	1.25	657	3.48	86	33.14	57	35.91
	SMPDBK	1155	1.25	939	2.79	327	15.44	132	24.27
5138	HEC-1	735	1.25	506	2.50	136	15.50	128	18.25
	DAMBRK	756	1.25	478	3.55	66	35.57	45	38.75
	SMPDBK	793	1.25	602	2.92	224	16.49	106	26.04
5188	HEC-1	414	1.25	283	2.75	95	17.25	90	20.25
	DAMBRK	424	1.25	288	3.65	46	38.19	33	41.03
	SMPDBK	457	1.25	358	3.11	143	17.65	81	28.29
5238	HEC-1	155	1.25	107	3.50	46	21.25	46	23.50
	DAMBRK	156	1.25	112	3.73	26	38.93	21	42.76
	SMPDBK	176	1.25	115	2.75	73	21.56	54	33.61

Table 4. Sensitivity to Water Surface Elevation (H_f) at the Dam Site when Breach Begins for Teton Dam Failure.

H_f(ft)		Section NO.1		Section NO.3		Section NO.10		Section NO.12	
		Flow (*10³ cfs)	Time (hrs)	Flow (*10³ cfs)	Time (hrs)	Flow (*10³ cfs)	Time (hrs)	Flow (*10³ cfs)	Time (hrs)
5288.5	HEC-1	1601	1.25	1062	2.00	243	11.25	222	14.00
	DAMBRK	1646	1.25	927	3.14	105	30.96	69	33.82
	SMPDBK	1602	1.25	1106	2.72	418	14.35	181	22.21
5338	HEC-1	1997	1.25	1417	2.00	305	12.75	280	15.25
	DAMBRK	2177	1.25	1379	3.02	160	27.60	108	30.20
	SMPDBK	1953	1.25	1154	2.69	476	13.78	227	21.23
5388	HEC-1	2409	1.25	1753	2.00	306	12.25	362	14.75
	DAMBRK	2750	1.25	1881	2.96	250	24.84	164	27.22
	SMPDBK	2287	1.25	1194	2.69	525	13.45	252	20.71

Table 5. Sensitivity to Initial Discharge (Q_o) for Teton Dam Failure.

Q_o (*10^3cfs)		Section NO.1		Section NO.3		Section NO.10		Section NO.12	
		Flow (*10^3cfs)	Time (hrs)	Flow (*10^3cfs)	Time (hrs)	Flow (*10^3cfs)	Time (hrs)	Flow (*10^3cfs)	Time (hrs)
0.1	HEC-1	1594	1.25	1044	2.00	199	13.75	184	16.50
	DAMBRK	error	error	error	error	error	error	error	error
	SMPDBK	1589	1.25	1098	2.72	415	14.37	181	22.24
1	HEC-1	1595	1.25	1046	2.00	201	13.75	186	16.50
	DAMBRK	error	error	error	error	error	error	error	error
	SMPDBK	1590	1.25	1098	2.72	415	14.36	181	22.23
10	HEC-1	1600	1.25	1051	2.25	204	13.75	188	16.50
	DAMBRK	1645	1.25	925	3.13	104	31.29	69	33.90
	SMPDBK	1599	1.25	1104	2.72	417	14.34	181	22.22
100	HEC-1	1651	1.25	1139	2.00	331	12.25	312	14.75
	DAMBRK	1679	1.25	989	3.13	317	28.23	57	31.36
	SMPDBK	1689	1.25	1157	2.70	438	14.19	179	22.17

Table 6. Sensitivity to Manning's n for Teton Dam Failure.

n		Section NO.1		Section NO.3		Section NO.10		Section NO.12	
		Flow (*10^3cfs)	Time (hrs)	Flow (*10^3cfs)	Time (hrs)	Flow (*10^3cfs)	Time (hrs)	Flow (*10^3cfs)	Time (hrs)
0.02	HEC-1	1601	1.25	1381	1.75	346	8.75	324	10.00
	DAMBRK	error	error	error	error	error	error	error	error
	SMPDBK	1602	1.25	1586	1.86	545	8.17	534	9.21
0.06	HEC-1	1601	1.25	1071	2.00	140	20.50	130	24.75
	DAMBRK	1646	1.25	934	3.14	68	49.24	46	53.80
	SMPDBK	1602	1.25	1084	2.75	303	19.01	62	38.70
0.10	HEC-1	1601	1.25	886	2.50	93	34.50	86	38.75
	DAMBRK	error	error	error	error	error	error	error	error
	SMPDBK	1602	1.25	800	3.57	204	30.21	43	39.35
0.14	HEC-1	1601	1.25	761	2.75	72	42.00	67	52.00
	DAMBRK	error	error	error	error	error	error	error	error
	SMPDBK	1602	1.25	791	4.34	134	40.31	40	75.34

H_f, or larger Q_o produces larger peak flow and shorter time to peak ; As the flood wave moves downstream, the above effects on peak flow decrease but the effects on time to peak do not decrease ; Manning's n has little effect on peak flow, but it has a large effect on time to peak.

HEC-1 produces stable solutions for almost all the cases considered in this study, but DAMBRK and SMPDBK do not always produce stable solutions. This results from the fact that HEC-1 uses hydrologic routing methods.

DAMBRK could not produce the flood waves by an instantaneous dam failure (Table 1), which can be inferred as follows : DAMBRK uses "through computation" methods which are based on the principle that the numerical method should take care of the shock without any special programming for it. But the methods cannot give a reasonable solution for large shock. So in case of large shock, shock-fitting method or pseudo-viscosity method should be used(Cunge, 1975 : Cunge et al., 1980). DAMBRK could not produce the flood wave for small initial discharge(Table 5) or extreme Manning coefficients(Table 6), which proves that the model is sensitive to both initial and boundary data in spite of the implicitness of the numerical scheme.

In SMPDBK the given value of B_w(= 1,000 ft) was changed to final value of B_w(= 708 ft). It was due to the model's scheme : If a given B_w is greater than 1.2 * [B(2, 1) + B_o(2, 1)] where B(2, 1) and B_o(2, 1) are the second lowest topwidths of active and inactive cross-sections immediately downstream of the dam, respectively, the final B_w is changed to 1.2 * [B(2, 1) + B_o(2, 1)] to restrict the final breach width. This seems reasonable. For concrete dams, however, total failure is possible. So, the limitation of all the cases is not convincing.

6. Conclusions and Future Studies.

The three representative dam-break models, HEC-1, DAMBRK, and SMPDBK were analyzed respectively in their theories and then applied to the failure of Teton Dam.

All the models do not have capabilities for predicting breach characteristics but can simulate the breach by overtopping. DAMBRK also has an additional option for simulating a piping failure. DAMBRK is most accurate among the three models, while HEC-1 produces stable solutions for almost all the cases considered in this study and SMPDBK is most economical and easily applicable.

The followings are suggested in order to improve the capabilities of the three models : It is necessary to better understand breach mechanism from the structural and geotechnical engineering perspective ; Erosion and sediment transport simulation should be added to dam-breach flood forecasting models ; DAMBRK could simulate an instantaneous failure if shock-fitting method or pseudo-viscosity method were included in the model ; Two-dimensional unsteady flow modeling is needed.

감사의 말

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References

Cristofano, E. A.(1965), Method of Computing Rate for Failure of Earth Fill Dam, USBR, Denver, Colorado.

Cunge, J. A.(1975), Rapidly Varying Flow in Power and Pumping Canals, Ch. 14 of *Unsteady Flow in Open Channels* edited by D. Mahmood and V. Yevjevich, WRP, pp. 539–586.

Cunge, J. A., F. M. Holly, and A. Verwey(1980), *Practical Aspects of Computational River Hydraulics*, Pitman.

DongA–Ilbo(July 14, 1961), Seoul (in Korean).

Dressler, R. F.(1952), Hydraulic Resistance Effects upon the Dam–Break Functions, *J. of Research*, National Bureau of Standards, Vol. 49, No. HY3, Washington D.C..

Fread, D. L.(1984), *DAMBRK : The NWS Dam–Break Flood Forecasting Model*, NWS, Office of Hydrology, Maryland.

HanKook–Ilbo(July 24, 1980), Seoul (in Korean).

Han, Kun Yeun(1986), *A forecasting Model for the Floodwave Propagation Resulting from Dam-Break*, Ph. D Thesis, Yunsei University (in Koeran).

Ko, Young Chan(1986), *Prediction of Peak Discharge at the Lower Han River Resulting from the Failure of Paldang Dam*, M. S. Thesis, Seoul National University (in Korean).

Lee, Changhoon(1988), *Comparative Evaluation of Dam–Break Models*, M. S. Thesis, Seoul National University.

MacDonald, T. C. and L. M. Jennifer(1984), Breaching Characteristics of Dam Failures, *J. of Hydraulic Engineering*, ASCE, Vol. 110, No. 5.

Morris, H. M. and J. M. Wiggert(1972), *Applied Hydraulics in Engineering*, Wiley, pp. 184–188.

Report to US Dept. of the Interior and State of Idaho on Failure of Teton Dam by Independent Panel to Review Cause of Teton Dam Failure(Dec. 1976), Idaho.

Ritter, A.(1892), The Propagation of Water Waves, *V. D. I. Zeitschr*, Vol. 36, No. 33.

US Army Corps of Engineers(1960, 1961), Floods Resulting from Suddenly Breached Dam, No. 2–274, US WES, Vicksburg, MISS.

US Army Corps of Engineers(Sept. 1981), *HEC–1 Flood Hydrograph Package Users and Programmers Manual*, HEC, Calif.

Wetmore, J. N. and D. L. Fread(1983), *The NWS Simplified Dam–Break Model Executive Brief*, NWS, Office of Hydrology, Maryland.

Wurbs, R. A.(Jan. 1987), Dam–Breach Flood Wave Models, *J. of Hydraulic Engineering*, ASCE, Vol. 113, No. 1.