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Comparison of Physical and Numerical Dam-Break Simulations

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Abstract

Laboratory data obtained from model studies of an existing dam under three different failure scenarios are presented. Moreover, the numerical failure simulations of the same dam were performed by employing two state-of-the-art numerical models, namely, SMPDBK and DAMBRK, both developed at the National Weather Service (NWS) in the United States. Comparison of the measured and computed results indicate that both numerical models predict peak flood elevation with somewhat reasonable accuracy. However, the results of the more comprehensive dambreak model, (DAMBRK) were closer to the measurements than those of the SMPDBK model, as expected. The SMPDBK model, when compared to the DAMBRK model, underestimates the peak water elevations more because of its simpler algorithm. Moreover, there exist large differences for the peak water surface occurrence times between the physical model and the numerical model predictions, especially in SMPDBK. This is attributed to the high sensivity of the numerical models to the bottom friction of the channel.

Key Words: Dambreak, Floods, Forecasting, Numerical and Physical Model.

Fiziksel ve Nümerik Baraj Yıkılma Benzeşimlerinin Kıyaslanması

Özet

Halen faaliyette olan bir barajın laboratuvar model çalışmalarında üç farklı yıkılma senaryosu kullanılarak elde edilen sonuçlar sunulmaktadır. Ayrıca, Amerika Birleşik Devletlerinin Milli Hava Servisinde (National Weather Service) geliştirilmiş olan iki nümerik model SMPDBK ve DAMBRK kullanılarak aynı baraja ait yıkılma benzeşimleri nümerik olarak gerçekleştirilmiştir. Nümerik model sonuçları ile fiziksel model sonuçlarının kıyaslanması sonucunda her iki nümerik modelin de, pik taşkın kotlarını kabul edillebilir bir mertebede tahmin ettiği görülmektedir. Fakat, daha gelişmiş bir model olan DAMBRK modelinin sonuçları, beklenildiği gibi SMPDBK modelinin sonuçlarına kıyasla, ölçümlere daha yakındır. SMPDBK modeli daha basit bir algoritması olduğu için pik taşkın kotlarını, DAMBRK modeline kıyasla, olması gerekenden daha küçük hespalamaktadır. Ayrıca fiziksel modelde ölçülen pik su kotların oluştuğu zamanlar ile her iki nümerik modelin hesapladığı, özellikle SMPDBK modelinin hesapladığı pik su kotlarının oluştuğu zamanlar arasında büyük farklar görülmektedir. Bu durum her iki modelin de mansaptaki nehir yatağındaki pürüzlülüğe karşı çok hassas olmalarına bağlanmaktadır.

Anahtar Sözcükler: Baraj yıkılması, tahmin, nümerik ve fiziksel modeller.

1. Introduction

At the beginning of 1997, the number of large dams in Turkey had reached 190, and construction is either underway or being planned for many more. Some of these dams are so located that they may constitute a major threat to people living downstream. In general, dams are safe structures. However, there is always a possibility of a dam holding up a large volume of water, to fail due to a breach. A rapidly moving floodwave caused by failure of a large dam may result in a catastrophe for people living nearby.

According to Ellingwood et al. (1993), dams of significant size fail in the U.S. at an average rate of more than one per year. Johnson and Illes (1976) have listed some past dam failures and classified them. Gray (1974) reported about 54 dams built in the U.S. before 1950, and stated that the Bureau of Reclamation should consider corrective measures to bring safety standards up to date. De Almeida and Franco (1994) stated that, "a global floodplain management will have to include emergency evacuation procedures and guidelines for the human and economic occupancy as long as an upstream dam and reservoir be a potential risk. These guidelines must be based on a special flood zone ordinance prepared according to a dam break flood anaysis".

In the early 1960's there was a growing awareness worldwide of the grave consequences of a potential dam failure. It was recognized then that such a failure would cause significant damage to property and even more importantly, loss of human lives. Thus, the major hydraulic laboratories of the world undertook physical model studies on dam break simulations, such as those performed at Waterways Experimental Station (WES) in the U.S. (1961a, 1961b), which were used later extensively by researchers to verify their numerical models, including Basco (1987, 1989), Alam and Bhuiyan (1995), and others. Costly and time consuming physical models were common at the time since both computers and computing techniques of that period were not advanced enough.

An early study of such nature conducted in Turkey was the Cubuk-1 dam failure simulations on physical model (T. Acatay, 1964), in the DSI (State Hydraulic Works of Turkey) Hydraulic laboratory, Ankara. The Cubuk-1 dam, the first concrete gravity dam of Turkey, is located on Cubuk stream 12 km north of the city of Ankara, the capital of Turkey. It was built in 1936 to supply drinking water for Ankara. At the time, the major reason for undertaking this model study was the notion that dams would be the first targets in war time and that destruction of a dam would be catasrophic for the people living downstream. With this in mind, the Cubuk-1 dam was selected for failure simulations on the physical model in order to prepare inundation maps of the residential areas downstream. It was thought that the data obtained from the physical modeling of the Cubuk-1 dam could be used for comparative analysis of the numerical models for sudden dam failures.

Consequently, the major goal of the present study (Kasap, A., 1996) was to test two widely used numerical models by using the Cubuk-1 data. The results obtained in the experiments in various failure scenarios will be compared with those obtained from the numerical model simulations in the same scenarios. The numerical models employed were the NWS Simplified Dam Break Model (SMPDBK) and the NWS Dam-Break Flood Forecasting Model (DAMBRK), both developed at the National Weather Service in the USA, by Wetmore and Fread, (1984), and by Fread, D.L., (1977, 1988), respectively. These models were selected based on the study performed by Wurbs (1986) in which it was concluded that the DAMBRK and SMPDBK were the optimal models for adoption by the Military Hydrology Program of the U.S. This conclusion was reached after an extensive comparative evaluation of some leading dam-break models representative of the current state of the art in use.

2. Physical Model Studies

2.1. Selection of the Scale of the Physical Model

The physical model was built in 14 months at the Hydraulic Laboratory of the DSI Technical Research and Quality Control Department. Topographical maps of 1/25000 scale were used without distortion. The optimum model scale was found to be 1/500 since the downstream area modeled was approximaetly 25 km in length, which meant a downstream channel 50 m in length in the model. Furthermore, preliminary considerations of surface tension effects on the measurements would not permit any smaller model scale.

2.2. Construction Style of the Physical Model

In the construction of the physical model, iron sheets of 1 mm thickness were used for the cross sections. In order to simulate the natural topography as closely as possible, a cross-section was placed at intervals of 50 cm in the model. Because 1 mm in the hydraulic model corresponded to 50 cm in the prototype, the cross sections were inserted in their proper places, with an accuracy of 1/10 mm sought. The sections between the cross secctions were filled and compacted with clayey sand, and the upper parts were covered by concrete.

2.3. Test Performed on the Physical Model

For the physical model tests, it was assumed that the dam failed instantaneously since the motivation for the physical model study was to see how the flood wave moved downstream after the hypothetical destruction of the Cubuk-1 dam in a potential war situation. The instantaneous failure of the dam was simulated by suddently lifting a gate which functioned like the dam body.

The tests were performed at three reservoir water elevations (RWSE): 1) 906.25 m, full capacity 2) 900.00 m, 2/3 capacity; and finally, 3) 895.00 m, 1/2 capacity.

2.4. Description and Results of the Tests

An electronic recording instrument was installed at each cross section location downstream of te dam (i.e., a total of 7) to measure the water surface elevations during the transient event. The first two columns of Table 1 show the locations and thalweg elevations of those downstream stations. Note that the first station is the dam location. The downstream channel bed was dry prior to the tests. As soon as the metal gate simulating the dam body was lifted, the recording instruments started recording the water surface elevations on continuously fed graph paper. Consequently, the time history of the water surface elevation at each cross section was obtained from the instant at which the dam failure commenced until a sufficient time period elapsed in which the flood wave peak occurred and attenuated at each of the seven stations. The collected data for the time variations of the water surface elevations were scaled up to the prototype and are shown in Table 1 for all of the initial reservoir water surface elevations tested.

The values obtained from all of the tests are plotted in Figures 1 through 3, which show the time history of the water surface elevations at stations 2 through 7 for Test 1, Test 2, and Test 3, respectively.

Table 1 presents the peak values of the measured water depths and water surface elevations, as well as their occurrence times at all stations for Tests 1, 2, and 3, respectively. Figures 4 and 5 were obtained using the data from Table 1. Fig. 4 shows the peak water surface elevations at all the measurement station, and Fig. 5 presents the occurrence times of those peak water surface elevations.

It is clear in Figs. 1 through 3 that the curves corresponding to each measurement station run parallel to the time axis, as expected, until the flood wave arrives at a given station, at which point it suddenly jumps to a maximum value in a short time interval, and after passing the peak value (i.e. the crest) it starts decreasing. Each curve can be divided into three regions:

Region 1) The horizontal part of the curve which is parallel to the time axis. This part shows the time elapsed for the flood wave to arrive at a measurement station.

Region 2) The rising limb part of the curve which depicts the increase of the magnitude of the flood wave in height.

Region 3) The falling limb of the curve showing the attenuation of the flood wave.

Table 1. Peak Values Measured in the Experiments on the Physical Model

			TEST 1 (RWSE=906.25 m)			TEST 2 (RWSE=900.00 m)			TEST 3 (RWSE=895.00 m)		
X-S No.	Distance D/S (km)	Thalweg Elev. (m)	Peak Depth (m)	Peak Elev.(m)	Time to Peak (hr)	Peak Depth (m)	Peak Elev. (m)	Time to Peak (hr)	Peak Depth (m)	Peak Elev. (m)	Time to Peak (hr)
1	0.000	884.20	22.05	906.25	0.00	15.80	900.00	0.00	10.80	895.00	0.00
2	1.625	880.50	15.00	895.50	0.10	10.50	891.00	0.06	5.35	885.85	0.21
3	3.400	872.40	5.65	878.05	0.32	4.60	877.00	0.29	4.25	876.65	0.48
4	5.575	863.40	8.35	871.75	0.38	5.00	868.40	0.59	4.05	867.45	0.56
5	7.625	855.10	11.65	866.75	0.69	5.25	860.35	0.83	5.00	860.10	0.80
6	9.850	848.15	6.63	854.78	0.72	5.00	853.15	0.83	4.10	852.25	0.89
7	13.150	837.75	3.37	841.12	0.94	4.60	842.35	1.25	3.00	840.75	1.53
8	14.750	834.70	3.30	838.00	1.21	3.60	838.30	1.38	3.00	837.70	2.83

Table 2. Comparison of Physical Model and SMPDBK Model Results Using Various Manning n Values for Dambreak Simulation at RWSE=906.25 m.

			Physical Model			SMPDBK Model n=0.045			SMPDBK Model n=0.050			SMPDBK Model n=0.060		
X-S No.	Distance D/S (km)	Thalweg Elev. (m)	Peak Depth (m)	Peak Elev.(m)	Time to Peak (hr)	Peak Depth (m)	Peak Elev. (m)	Time to Peak (hr)	Peak Depth (m)	Peak Elev. (m)	Time to Peak (hr)	Peak Depth (m)	Peak Elev. (m)	Time to Peak (hr)
1	0.000	884.20	22.05	906.25	0.00	13.87	898.07	0.03	14.09	898.29	0.03	14.47	898.67	0.03
2	1.625	880.50	15.00	895.50	0.10	9.40	889.90	0.07	9.47	889.97	0.07	9.75	890.25	0.07
3	3.400	872.40	5.65	878.05	0.32	7.20	879.60	0.18	7.35	879.75	0.19	7.56	879.96	0.21
4	5.575	863.40	8.35	871.75	0.38	10.34	873.74	0.31	10.51	873.91	0.33	10.93	874.33	0.37
5	7.625	855.10	11.65	866.75	0.69	8.40	863.50	0.42	8.62	863.72	0.45	9.02	864.12	0.50
6	9.850	848.15	6.63	854.78	0.72	13.78	861.93	0.52	14.21	862.36	0.56	14.97	863.12	0.63
7	13.150	837.75	3.37	841.12	0.94	8.36	846.11	0.89	8.45	846.20	0.97	8.58	846.33	1.11
8	14.750	834.70	3.30	838.00	1.21	3.64	838.34	1.14	3.67	838.37	1.24	3.66	838.36	1.47

Table 3. Comparison of Physical Model and SMPDBK Model Results Using Various Manning n Values for Dambreak Simulation at RWSE=900.00 m.

			Physical Model			SMPDBK Model n=0.045			SMPDBK Model n=0.050			SMPDBK Model n=0.060		
X-S No.	Distance D/S (km)	Thalweg Elev. (m)	Peak Depth (m)	Peak Elev.(m)	Time to Peak (hr)	Peak Depth (m)	Peak Elev. (m)	Time to Peak (hr)	Peak Depth (m)	Peak Elev. (m)	Time to Peak (hr)	Peak Depth (m)	Peak Elev. (m)	Time to Peak (hr)
1	0.000	884.20	15.80	900.00	0.00	10.51	894.71	0.02	10.67	894.87	0.02	10.97	895.17	0.02
2	1.625	880.50	10.00	891.00	0.06	6.47	886.97	0.04	6.71	887.21	0.06	6.73	887.23	0.07
3	3.400	872.40	4.60	877.00	0.29	4.89	877.29	0.22	5.07	877.47	0.25	5.18	877.58	0.27
4	5.575	863.40	5.00	868.40	0.59	6.94	870.34	0.41	6.56	869.96	0.45	6.80	870.20	0.52
5	7.625	855.10	5.25	860.35	0.83	5.56	860.66	0.61	5.68	860.78	0.66	5.90	861.00	0.75
6	9.850	848.15	5.00	853.15	0.83	7.64	855.79	0.73	7.90	856.05	0.79	8.35	856.50	0.90
7	13.150	837.75	4.60	842.35	1.25	5.61	843.36	1.29	5.68	843.43	1.40	5.71	843.46	1.66
8	14.750	834.70	3.60	838.30	1.38	2.24	836.94	1.34	2.26	836.96	1.47	2.22	836.92	1.77

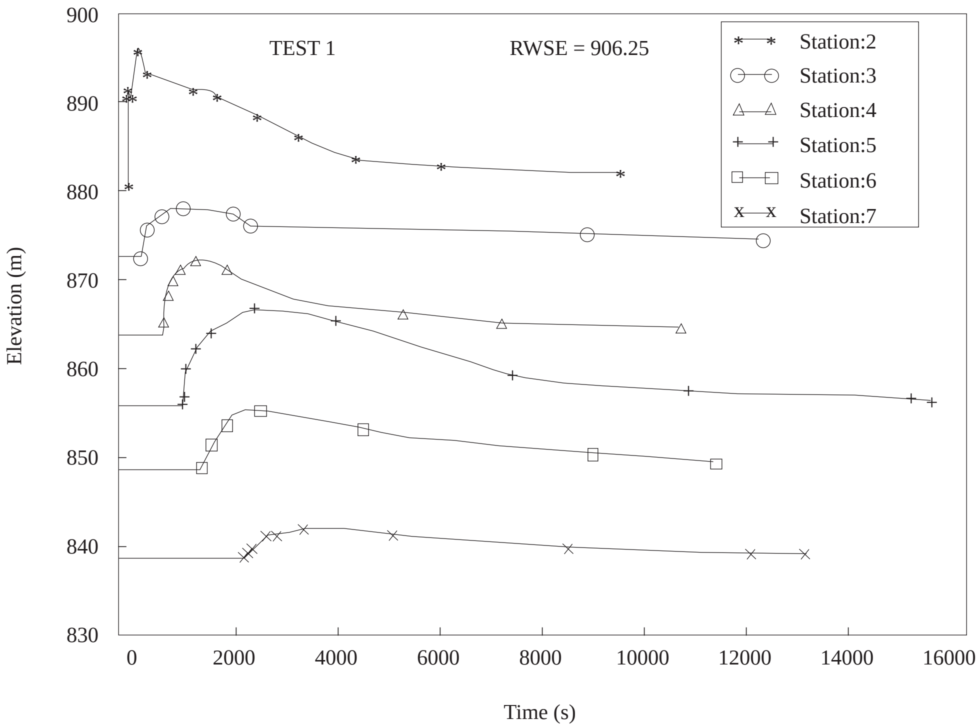

Figure 1. Time variation of water elevations

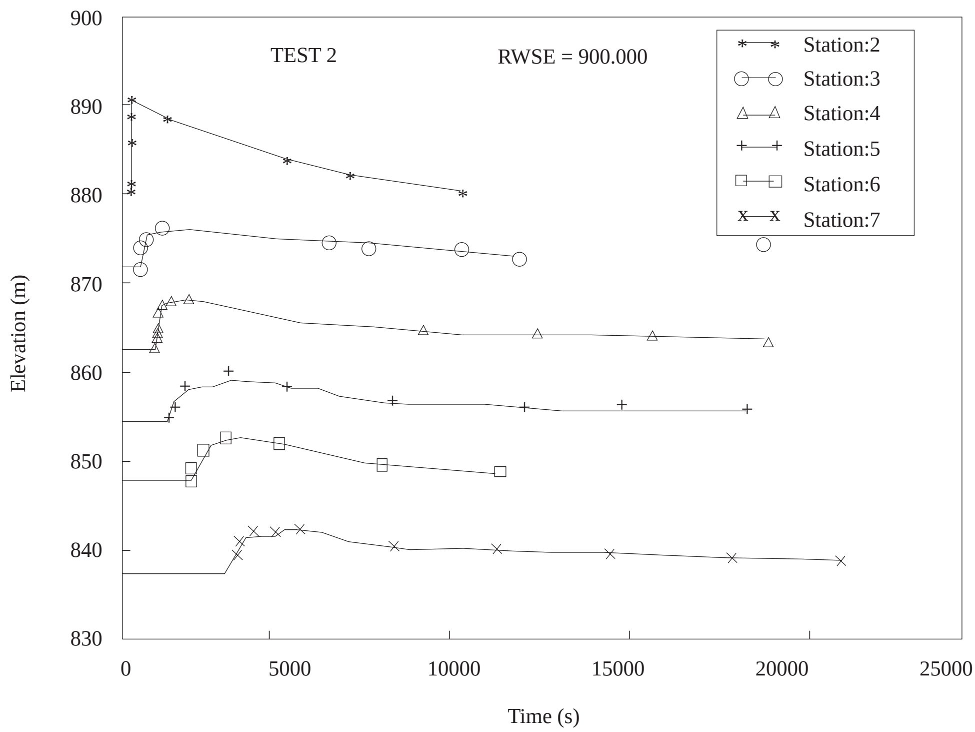

Figure 2. Time variation of water elevations

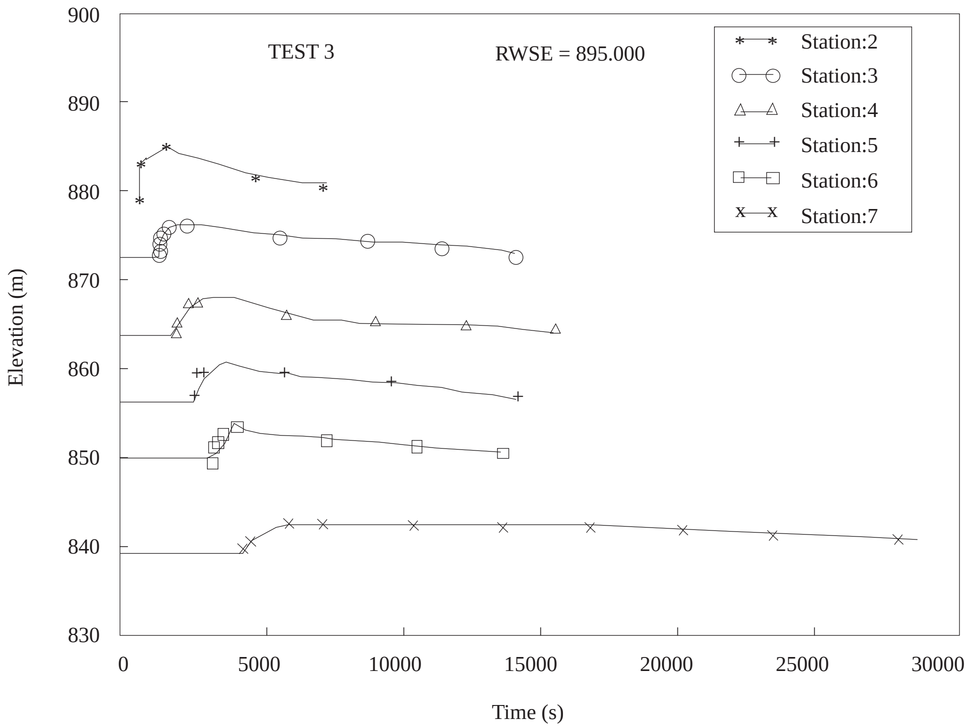

Figure 3. Time variation of water elevations

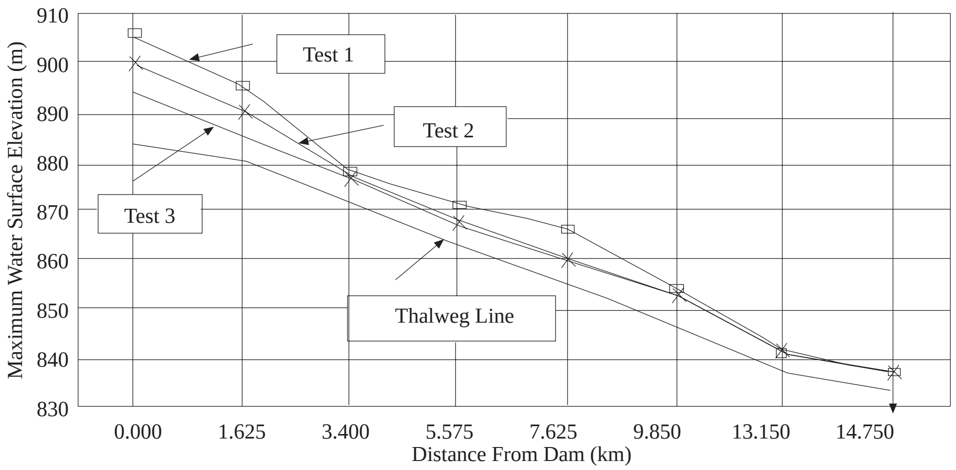

Figure 4. Peak water surface elevations measured at downstream stations

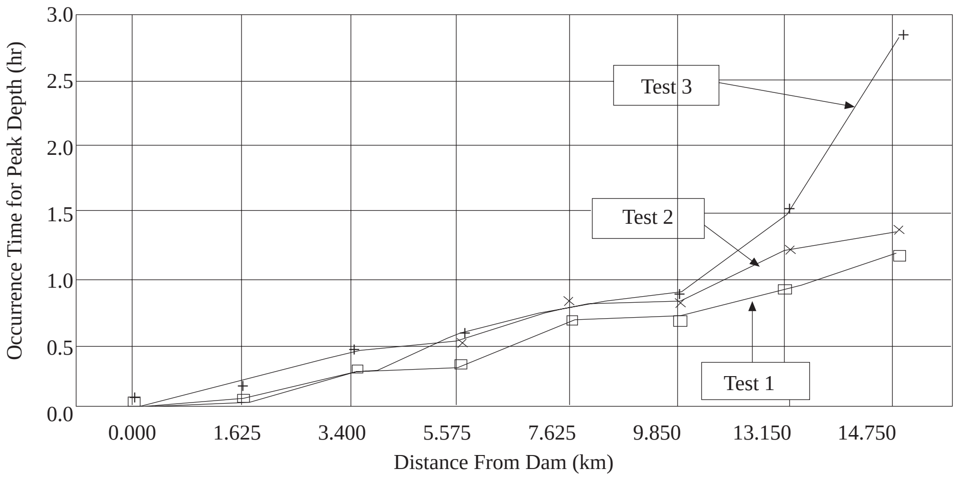

Figure 5. Occurence times for peak depths at downstream stations

The total flood duration time is equal to the time required for the floood wave to rise and subsequently to fall. If the curves are analyzed carefully, it can be seen that the flood wave reaches its crest in a much shorter time interval at the stations near the dam and also attenuates more quickly to its minimum value at the same stations. Naturally, the farther the location from the dam, the longer the flood wave arrival time.

In stations located farther from the dam, the flood wave takes longer both to reach its maximum value and to drop to its minimum value. Therefore, flooding lasts longer at these stations. In other words, the region in which the farther stations are located becomes the accumulation zone of the flood. Comparison of Figures 1 through 3 reveals that the flood arrival time is progressively longer at each successive station, as was expected for the lower reservoir water surface elevations of Tests 2 and 3.

3. Results and Discussion

Both the DAMBRK model and SMPDBK model were used in the numerical failure simulation analyses of Cubuk-1 dam on Ankara Cubuk stream. The same failure scenarios that were used previously in the physical model simulations were employed in the numerical simulations as well, as indicated below:

Case 1: The dam fails instantaneously when the RWSE is 906.25 m.

Case 2: The dam fails instantaneously when the RWSE is 900.00 m.

Case 3: The dam fails instantaneously when the RWSE is 895.00 m.

For each case, the maximum water surface elevations, peak occurrence times and peak discharges were calculated by the models. Since the physical model simulation results are available for the above scenarios, they will be compared with the numerical simulation results obtained from the SMPDBK and DAMBRK models in the following sections.

4. SMPDBK Model Analysis

As stated previously, the SMPDBK model was developed by Wetmore and Fread (1984) at the National Weather Service (NWS) of the USA. This model produces information needed for determining the areas threatened by dam-break flood waters while substantially reducing the amount of time, data, computer facilities, and technical expertise required in employing more sophisticated unsteady flow routing models such as the DAMBRK model. The SMPDBK model can easily be processed on a PC with a minimal amount of data. The user may within minutes predict the dam-break floodwave peak flows, peak flood elevations, and peak travel times at selected downstream points. This capacity for providing results quickly and efficiently makes the SMPDBK a useful forecasting tool in a dam failure emergency when warning response time is short, data are sparse, or mainframe computer facilities are inaccessible. The SMPDBK model is also useful for pre-event dam failure analysis by emergency management personnel engaged in preparing disaster contingency plans when the use of other flood routing models is precluded by limited resources.

The SMPDBK model calculates the maximum outflow at the dam first, then evaluates how this flow will be reduced as it moves from the dam location to the downstream locations specified by the user. The flood wave routing through the downstream channel, which is assumed to be prismatic, is accomplished using dimensionless curves obtained previously from the NWS DAMBRK model runs (Fread, D.L., 1977). All of these calculations utilize input data supplied by the user. While the model supplies default values for many of the input variables, the most accurate results are produced when the most realistic data are entered.

The following input variables, which are in English units, were used in the present study when employing the SMPDBK model:

* type of dam: concrete gravity dam.

* dam crest elevation or reservoir elevation when breaching begins: 2973.26 ft (906.25 m); 2952.76 ft (900.00 m); 2936.35 ft (895.00 m), in Cases 1 through 3 respectively.

* final bottom elevation: 2900.92 ft (884.20 m).

* reservoir storage volume: 3717.91 acre-ft (4,586,000 m^3); 702.88 acre-ft (867,000 m^3); 15.40 acre-ft (19,000 m^3), in Cases 1 through 3, respectively.

* reservoir storage area: 249.60 acres (1,010,000 m^2); 75.37 acres (305,000 m^2); 8.40 acres (34,000 m^2), in Cases 1 through 3, respectively.

* final breach width: 360.89 ft (110 m).

* time required for breach formation: 0.0

Note that a value of zero was entered in the input data of SMPDBK in the simulations for the breach formation time. This causes the program to compute the breach formation time by using the following dimensionally non-homogeneous empirical equation:

$$t_f = \frac{H}{40} \tag{1}$$

where t_f (breach formation time) is in minutes and H (the height of water depth in the reservoir) in feet. The breach formation times for Cases 1 through 3 are computed to be t_f=1.81, t_f=1.30, and t_f=0.89 minutes, respectively.

channel topwidth vs. elevation data or two or more downstream river cross sections are required. In our cases, eight cross sections were utilized. They were obtained from maps with a scale of 1/25000 supplied by the DSI. Five pairs of the channel topwidth vs. elevation data were used for each cross section in the input data when running SMPDBK.

the reach length between each cross section and the dam (Table 1).

The SMPDBK model was employed for three different uniform Manning roughness values (i.e. n=0.045, n=0.050, and n=0.060) in order to determine the sensitivity of the results to the roughness coefficient n.

Tables 2, 3, and 4 present both the measured physical model data and the predicted results of the SMPDBK model simulations in a tabulated form for Cases 1 through 3, respectively. As seen in the tables, increasing the Manning n does not change the computed peak water surface elevations significantly.

On the other hand, Manning n has a significant effect on the peak occurrence times, as shown in these tables, corresponding to Case 1, Case 2, and Case 3, respectively. In general, Manning n=0.050 yields the best agreement between the computed and measured values at most of the cross sections for Case 1 and Case 2. In Case 3, n=0.045 yields the closest peak time values to those of physical model.

5. DAMBRK Model Analysis

DAMBRK is a much more sophisticated numerical model than SMPDBK in that it employs a more elaborate numerical scheme to simulate a flood wave moving downstream in a valley. The governing equations of the model are the complete one-dimensional Sain-Venant equations of unsteady flow which are coupled with internal boundary equations representing the rapidly varied (broad-crested weir) flow through structures. In addition, suitable external boundary equations at the upstream and downstream ends of the routing reach are utilized. The system of equations is solved by a non-linear weighted four-point implicit finite-difference method. The model of two main, namely: (1) a description of the dam failure mode, i.e., the temporal and geometrical description of the breach; and (2) a hydraulic computational algorithm for determining the time history of the outflow through the breach as affected by the breach description, reservoir inflow, reservoir storage characteristics, spillway outflows, and downstream tailwater elevations; and for routing of the outflow hydrograph through the downstream valley in order to account for changes in the hydrograph due to valley storage, frictional resistance, downstream bridges or dams. The model also determines the resulting water surface elevations (stages) and flood-wave travel times.

In addition to the data used in employing the SMPDBK model, more detailed information (such as a table of surface area vs. corresponding water surface elevation in the dam reservoir) were used. The same cases were analyzed as in the SMPDBK with the same Manning roughness values, n=0.045, 0.050 and 0.060.

Similarly, Tables 5, 6, and 7 present both the measured physical model data and the predicted results of the DAMBRK model simulations in tabulated form for Cases 1 through 3, respectively. As seen in the tables, as the value of n is increased, computed peak depths increase at the cross sections to some degree. This is due to the fact that a higher n value means more roughness, and hence more resistance to the flow. This, in turn, means less discharge passing a particular cross section. Consequently, this increases the water surface elevations at the stations and causes the peak depths to occur for longer times. It is clear that the Mannig roughness has a more significant effect on the predicted travel time of the peak flood wave. However, the increase in the peak depths with the increasing n value is not so substantial. In other words, peak occurrence time magnitudes are significantly more sensitive to the roughness parameter than are the peak water surface elevations. It appears that the Manning n=0.050 is the most reasonable choice in the numerical analysis for the peak occurrence times in connection with the physical model measurements.

6. Physical Model versus Numerical Models

In order to show the differences more clearly between the measured values of the physical model and the computed values of both of the numerical models, SMPDBK and DAMBRK together, Figures 6 through 11 were presented. In these comparisons, a Manning roughness coefficient of n=0.050 was selected in the numerical simulations. Figures 6, 7, and 8 show the peak water surface elevations at the downstream stations for the three cases of the initial reservoir water surface elevations, namely, 906.25, 900.00, and 895.00, respectively, while Figures 9, 10, and 11 show the occurrence times of the peak water surface elevations at the same stations for the respective initial reservoir water surface elevations. As seen clearly in Figs 6 through 8, the DAMBRK model predicts the peak water elevations better than the SMPDBK model. As to the time of the peak water elevations shown in Figs 9 through 11, it can be said that although SMPDBK predicts somewhat more accurately at some stations, overall DAMBRK predictions approach the physical model results more closely. However, there are some large differences between the physical model and the numerical model results at some stations regarding the time to peak figures (Figs 9 through 11). In particular, at the stations located 7.625, 13.150, and 14.750 km from the dam, deviatons were wider from the physical model. It is thought that these differences were due to the fact that the arrival times of the peak water surfaces (ie, the flood peaks) were very sensitive to the Manning n value selected. In addition, there is an uncertainty about the selected n value as to how well it really describes the friction on the physical model. It is recommended that the selection of the n value be supported by statistical analysis in future studies.

7. Summary and Conclusions

Data obtained from the laboratory model studies of an existing dam under three different failure scenarios were presented. Moreover, the numerical failure simulations of the same dam were performed by employing two state-of-the-art numerical models, namely, NWS SMPDBK and DAMBRK. Comparison of the measured and computed results indicate that both numerical models predict the peak flood elevations (or peak water dephts) with reasonable accuracy. However, the results of the more comprehensive model DAMBRK, were more accurate than those of SMPDBK, as expected. SMPDBK underestimated the peak water elevations because of its simpler algorithm.

However, there exist large differences for the peak water surface occurrence times between the physical model and the numerical model predictions, especially of SMPDBK. This is attributed to the high sensitivity of the numerical models to the bottom friction of the channel.

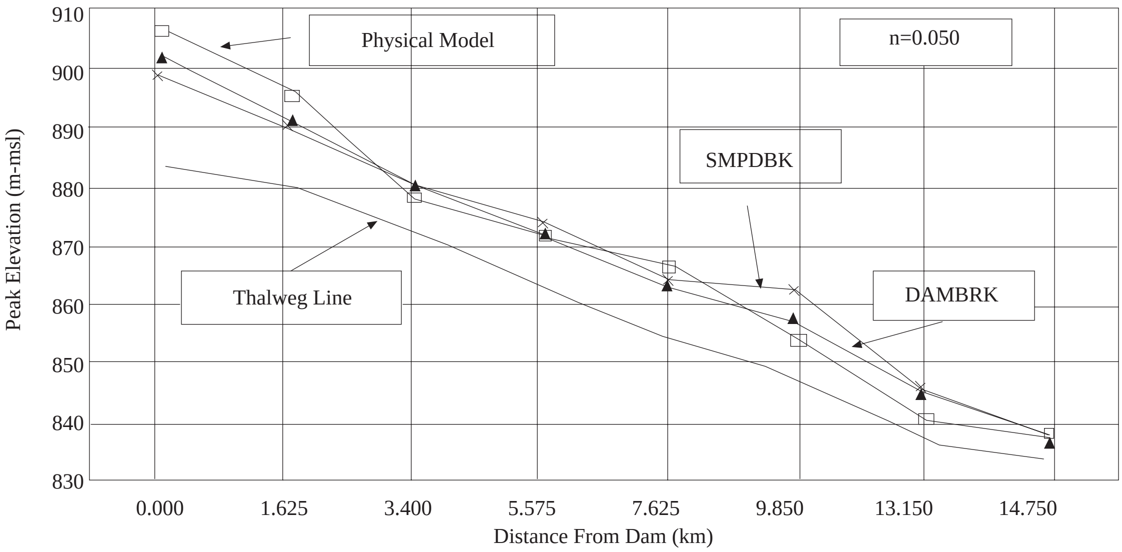

Figure 6. Peak water surface elevation vs. Distance downstream for RWSE=906.25 m.

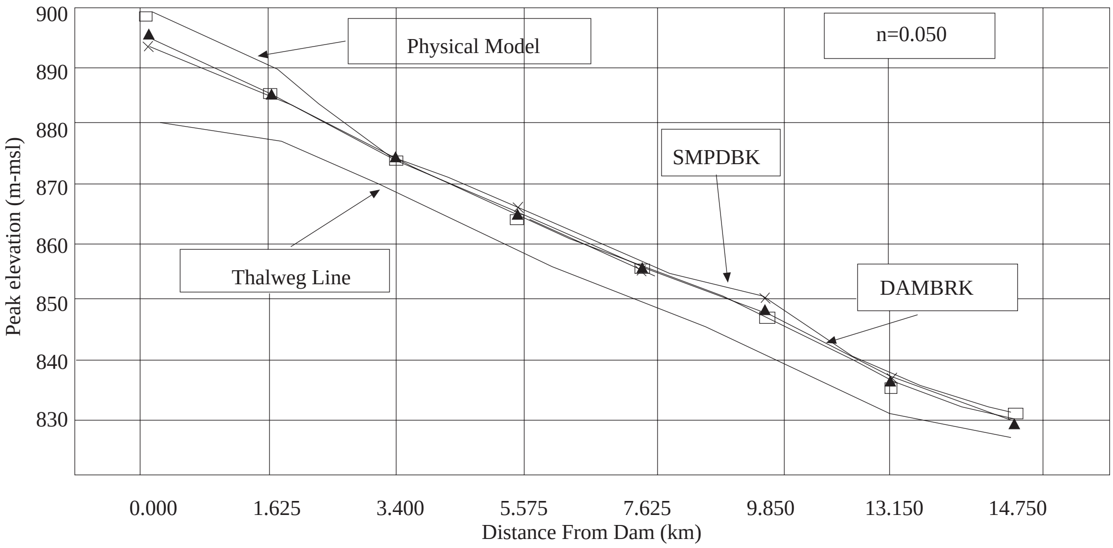

Figure 7. Peak water surface elevation vs. Distance downstream for RWSE=900.00 m.

Assuming that the physical model was built in such a way that it simulated the prototype conditions quite well, then it may be argued that, for the present study, the DAMBRK model can be employed for failure simulations with the acceptable results for engineering purposes.

In general, the DAMBRK model is best suited for pre-emergency dam-break analyses since it requires detailed input data for an accurate analysis, as well as technical expertise to use it. It can be used reliably to prepare inundation maps of the regions which may be subject to potential dam-break floods.

On the other hand, the SMPDBK model is better suited for real-time predictions of the behavior of dam-break flood waves in the cases where a dam-failure is imminent and there is not sufficient time for a comprehensive dam-failure anaysis. Thus, the SMPDBK model is not recommended for comprehensive dam-break failure analyses because of its much simpler algorithm.

Table 4. Comparison of Physical Model and SMPDBK Model Results Using Various Manning n Values for Dambreak Simulation at RWSE=895.00 m.

			Physical Model			SMPDBK Model n=0.045			SMPDBK Model n=0.050			SMPDBK Model n=0.060		
X-S No.	Distance D/S (km)	Thalweg Elev. (m)	Peak Depth (m)	Peak Elev.(m)	Time to Peak (hr)	Peak Depth (m)	Peak Elev. (m)	Time to Peak (hr)	Peak Depth (m)	Peak Elev. (m)	Time to Peak (hr)	Peak Depth (m)	Peak Elev. (m)	Time to Peak (hr)
1	0.000	884.20	10.80	895.00	0.00	6.50	890.70	0.01	6.62	890.82	0.01	6.85	891.05	0.01
2	1.625	880.50	5.35	885.85	0.21	2.63	883.13	0.08	2.59	883.09	0.08	2.47	882.97	0.11
3	3.400	872.40	4.25	876.65	0.48	1.62	874.02	0.37	1.61	874.01	0.41	1.58	873.98	0.49
4	5.575	863.40	4.05	867.45	0.56	1.59	864.99	0.75	1.59	864.99	0.83	1.58	864.98	0.99
5	7.625	855.10	5.00	860.10	0.80	1.75	856.85	1.03	1.75	856.85	1.13	1.70	856.80	1.37
6	9.850	848.15	4.10	852.25	0.89	1.58	849.73	1.44	1.57	849.72	1.59	1.50	849.65	1.95
7	13.150	837.75	3.00	840.75	1.53	1.28	839.03	2.34	1.26	839.01	2.62	1.17	838.92	3.25
8	14.750	834.70	3.00	837.70	2.83	0.72	835.42	2.90	0.70	835.40	3.29	0.61	835.31	4.07

Table 5. Comparison of Physical Model and DAMBRK Model Results Using Various Manning n Values for Dambreak Simulation at RWSE=906.25 m.

			Physical Model			DAMBRK Model n=0.0045			DAMBRK Model n=0.050			DAMBRK Model n=0.060		
X-S No.	Distance D/S (km)	Thalweg Elev. (m)	Peak Depth (m)	Peak Elev.(m)	Time to Peak (hr)	Peak Depth (m)	Peak Elev. (m)	Time to Peak (hr)	Peak Depth (m)	Peak Elev. (m)	Time to Peak (hr)	Peak Depth (m)	Peak Elev. (m)	Time to Peak (hr)
1	0.000	884.20	22.05	906.25	0.00	17.39	901.59	0.03	17.25	901.45	0.03	17.10	901.30	0.03
2	1.625	880.50	15.00	895.50	0.10	10.06	890.56	0.09	10.20	890.70	0.09	10.40	890.90	0.11
3	3.400	872.40	5.65	878.05	0.32	7.39	879.79	0.18	7.41	879.81	0.19	7.46	879.86	0.22
4	5.575	863.40	8.35	871.75	0.38	8.97	872.37	0.27	9.02	872.42	0.30	9.15	872.55	0.35
5	7.625	855.10	11.65	866.75	0.69	7.70	862.80	0.41	7.72	862.82	0.46	7.79	862.89	0.54
6	9.850	848.15	6.63	854.78	0.72	9.81	857.96	0.57	9.94	858.09	0.62	10.13	858.28	0.73
7	13.150	837.75	3.37	841.12	0.94	7.17	844.92	0.72	7.20	844.95	0.79	7.27	845.02	0.94
8	14.750	834.70	3.30	838.00	1.21	2.52	837.22	0.91	2.53	837.23	1.01	2.56	837.26	1.19

Table 6. Comparison of Physical Model and DAMBRK Model Results Using Various Manning n Values for Dambreak Simulation at RWSE=900.00 m.

			Physical Model			DAMBRK Model n=0.045			DAMBRK Model n=0.050			DAMBRK Model n=0.060		
X-S No.	Distance D/S (km)	Thalweg Elev. (m)	Peak Depth (m)	Peak Elev.(m)	Time to Peak (hr)	Peak Depth (m)	Peak Elev. (m)	Time to Peak (hr)	Peak Depth (m)	Peak Elev. (m)	Time to Peak (hr)	Peak Depth (m)	Peak Elev. (m)	Time to Peak (hr)
1	0.000	884.20	15.80	900.00	0.00	12.89	897.09	0.02	12.70	896.90	0.02	12.59	896.79	0.02
2	1.625	880.50	10.50	891.00	0.06	7.14	887.64	0.06	7.05	887.55	0.07	7.05	887.55	0.08
3	3.400	872.40	4.60	877.00	0.29	5.55	877.95	0.39	4.56	876.96	0.17	4.57	876.97	0.23
4	5.575	863.40	5.00	868.40	0.59	6.85	870.25	0.44	5.55	868.95	0.44	5.63	869.03	0.54
5	7.625	855.10	5.25	860.35	0.83	6.28	861.38	0.60	5.00	860.10	0.66	5.06	860.16	0.80
6	9.850	848.15	5.00	853.15	0.83	7.95	856.10	0.78	5.87	854.02	0.88	6.03	854.18	1.05
7	13.150	837.75	4.60	842.35	1.25	6.37	844.12	0.94	5.37	843.12	1.11	5.45	843.20	1.33
8	14.750	834.70	3.60	838.30	1.38	2.18	836.88	1.16	1.80	836.50	1.37	1.83	836.53	1.63

Table 7. Comparison of Physical Model and DAMBRK Model Results Using Various Manning n Values for Dambreak Simulation at RWSE=895.00 m.

			Physical Model			DAMBRK Model n=0.045			DAMBRK Model n=0.050			DAMBRK Model n=0.060		
X-S No.	Distance D/S (km)	Thalweg Elev. (m)	Peak Depth (m)	Peak Elev.(m)	Time to Peak (hr)	Peak Depth (m)	Peak Elev. (m)	Time to Peak (hr)	Peak Depth (m)	Peak Elev. (m)	Time to Peak (hr)	Peak Depth (m)	Peak Elev. (m)	Time to Peak (hr)
1	0.000	884.20	10.80	895.00	0.00	7.96	892.16	0.01	7.93	892.13	0.01	7.89	892.09	0.01
2	1.625	880.50	5.35	885.85	0.21	5.51	886.01	0.12	5.63	886.13	0.12	5.35	885.85	0.15
3	3.400	872.40	4.25	876.65	0.48	3.52	875.92	0.18	3.54	875.94	0.20	3.50	875.90	0.24
4	5.575	863.40	4.05	867.45	0.56	4.17	867.57	0.39	4.27	867.67	0.44	4.16	867.56	0.51
5	7.625	855.10	5.00	860.10	0.80	3.79	858.89	0.59	3.86	858.96	0.65	3.75	858.85	0.77
6	9.850	848.15	4.10	852.25	0.89	4.13	852.28	0.82	4.24	852.39	0.89	4.09	852.24	1.07
7	13.150	837.75	3.00	840.75	1.53	4.26	842.01	1.05	4.34	842.09	1.16	4.23	841.98	1.38
8	14.750	834.70	3.00	837.70	2.83	1.42	836.12	1.34	1.45	836.15	1.47	1.41	836.11	1.77

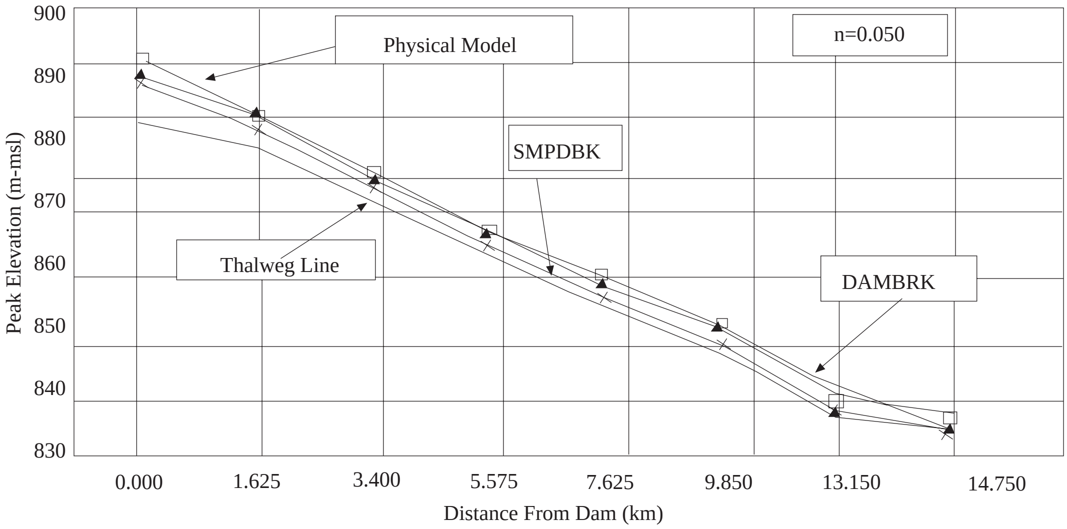

Figure 8. Peak water surface elevation vs. Distance downstream for RWSE=895.00 m.

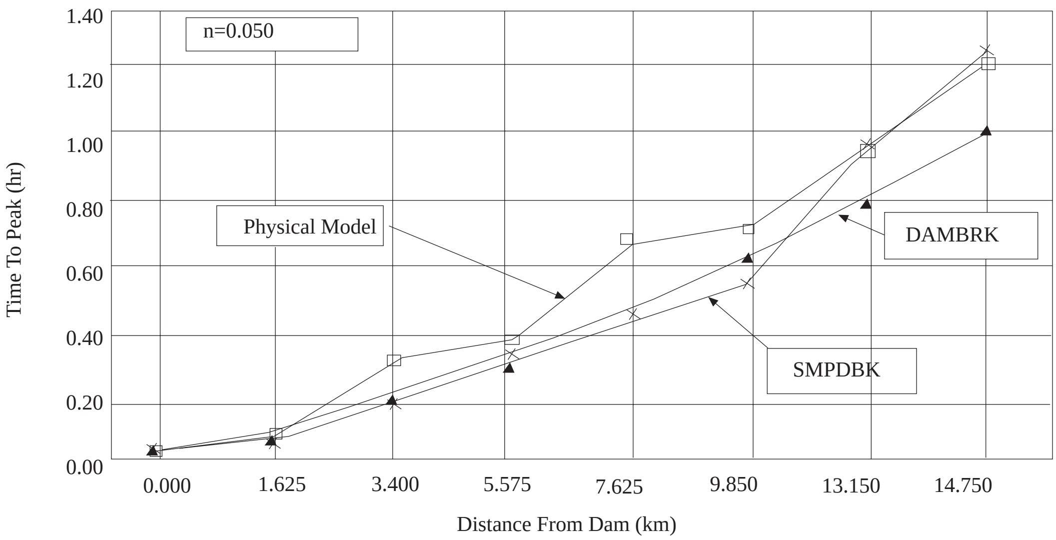

Figure 9. Peak occurence time vs. Distance downstream for RWSE=906.25 m.

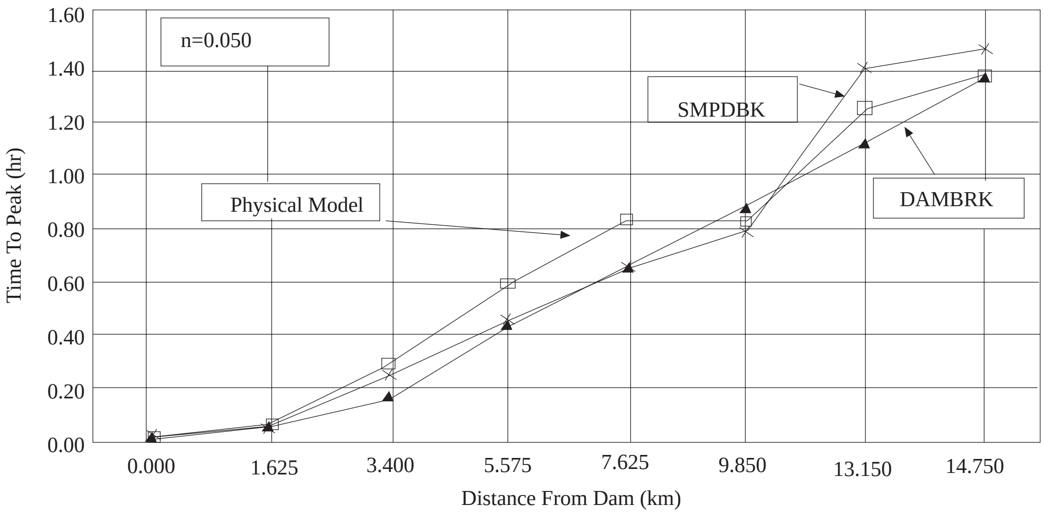

Figure 10. Peak occurence time vs. Distance downstream for RWSE=900.00 m.

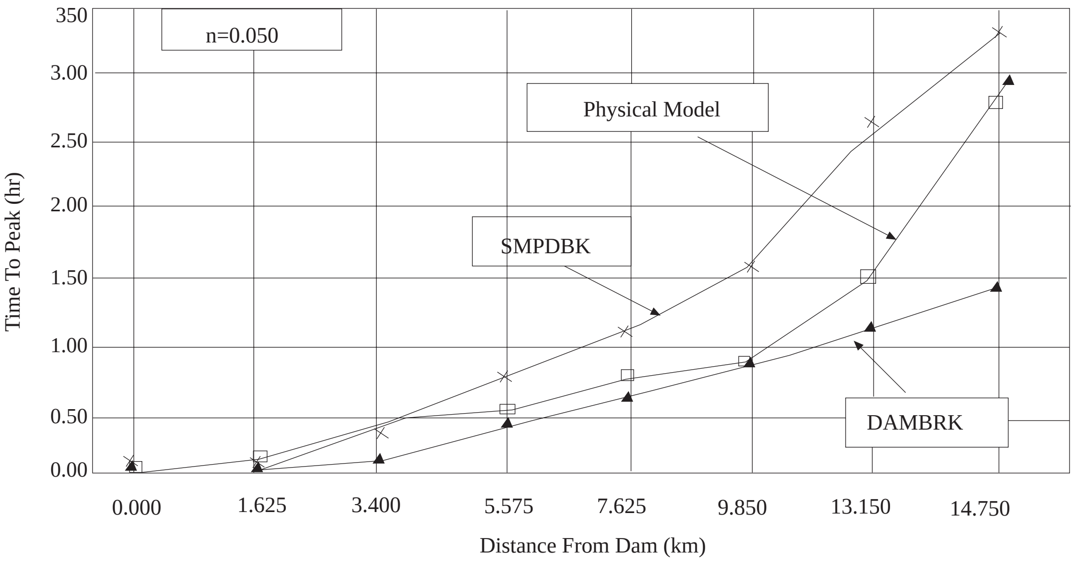

Figure 11. Peak occurence time vs. Distance downstream for RWSE=895.00 m.

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9. Appendix II. Notation

The following symbols are used in this paper.

H	=	the height of water depth in the reservoir,
n	=	Manning roughness coefficient,
t_f	=	breach formation time.

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