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STATE OF ILLINOIS
HENRY HORNER, Governor

MODEL STUDY OF SPILLWAY CHARACTERISTICS

1935-37

A joint publication by the University of Illinois Engineering Experiment Station and the Illinois State Water Survey.

ISSUED BY

DEPARTMENT OF REGISTRATION AND EDUCATION

STATE WATER SURVEY DIVISION

Urbana, Illinois

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WEST FRANKFORT, ILLINOIS

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BY J. J. DOLAND, T. E. LARSON, and C. O. REINHARDT.

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URBANA, ILLINOIS

RESERVOIR WEST FRANKFORT, ILLINOIS

Note ridge in front of spillway.

MODEL STUDY
OF SPILLWAY CHARACTERISTICS

WEST FRANKFORT, ILLINOIS

BY

J. J. DOLAND, T. E. LARSON, AND C. O. REINHARDT.*

INTRODUCTION

One of the functions of the State Water Survey is to collect hydrological data for localities where impounding reservoirs have been built for storage of water. To derive the maximum value from these studies it is necessary to obtain complete data for each locality.

One of the problems involved in the study of surface water resources in Illinois is the calibration of reservoir spillways. The recent interest in this country in the use of models for this purpose (1) suggested a series of such studies of typical reservoirs in the State.

Arrangements were made for cooperation between the Civil Engineering Department, the Theoretical and Applied Mechanics Department, and the State Water Survey in financing a project involving model studies of reservoir spillways in the State of Illinois. The tests were made in the laboratory of the T. & A. M. Department by members of the Water Survey staff, under the direction of Professor W. J. Putnam and Professor J. J. Doland.

The authors wish to express their appreciation of the assistance given by Professor W. J. Putnam in making arrangements for the use of the hydraulic laboratory facilities and in the installation of the model and appurtenances used in this study.

AIMS AND PURPOSES

Each of the three cooperating agencies had a well defined interest in the West Frankfort Reservoir Spillway Model. By using the head-discharge curves obtained from the model studies, the State Water Survey expected to be able to make use of the hydrological data that is being collected on the Reservoir. The available data on the West Frankfort Reservoir consists of rainfall, temperature, pumpage, and continuous stage records. Since the spill from a reservoir is an important

* J. J. Doland, Professor of Civil Engineering; Dr. T. E. Larson, Chemist, Illinois State Water Survey: and C. O. Reinhardt, Assistant Engineer ot the Illinois State Water Survey.

factor, if reliable estimates of run-off, and evaporation, are to be made, the lack of any gaging station below the dam renders the data unusuable for quantitative calculations until these values are known. Within limits imposed by the variable roughness of the approach channel, head-discharge curves have been plotted which should give values sufficiently accurate to warrant their use in completing the hydrological studies. The danger of extrapolating quantitative results of model studies to the performance of the prototype is recognized. However, at the present time the model studies afford the only quantitative data available, and will be used with a full appreciation of the limitations and probable inaccuracy.

The Theoretical and Applied Mechanics and Civil Engineering Departments were interested in the educational value of such a study. This model is the first experimental work in applying the principles of hydraulic similitude to an actual hydraulic structure to be carried on at the University of Illinois. Considerable interest has been shown by the students and faculty, and the participation in the project by the two departments of the engineering school has been justified.

PROBLEM

The municipal reservoir spillways in Illinois of primary interest to the State Water Survey are those of West Frankfort, Bloomington, Lake Bracken at Galesburg, Centralia, Carbondale, and Staunton. Since the hydrological data for West Frankfort were more complete, and the performance of the spillway and resulting erosion offered such an important and interesting problem, this structure was chosen first for tests.

EQUIPMENT

The initial equipment was designed by M. A. Churchill and T. E. Larson in November, 1935. The equipment consisted of an 8'x18' welded steel tank with a position for various models of spillways, discharge weir, baffles, and tailwater regulator. The model of the spillway was built to a 1:20 undistorted scale. The topography of the actual spillway approach was paved to scale with cement in the model box, by T. E. Larson and C. O. Reinhardt. A rock fill of medium gravel was placed in the box below the model for observing erosive effects.

Water was supplied through an 8-inch pipe with a 2-inch by-pass pipe for final adjustment of flow. Orifice plates used to measure the flow were calibrated by weighing the discharge. A calibrated 90° V-notch weir was later used for convenience and to obtain data for lower discharges. An A. B. McIntire gage reading to 0.001 of a foot was mounted on a movable T frame in order that elevations could be taken at any point in the tank or on the model.

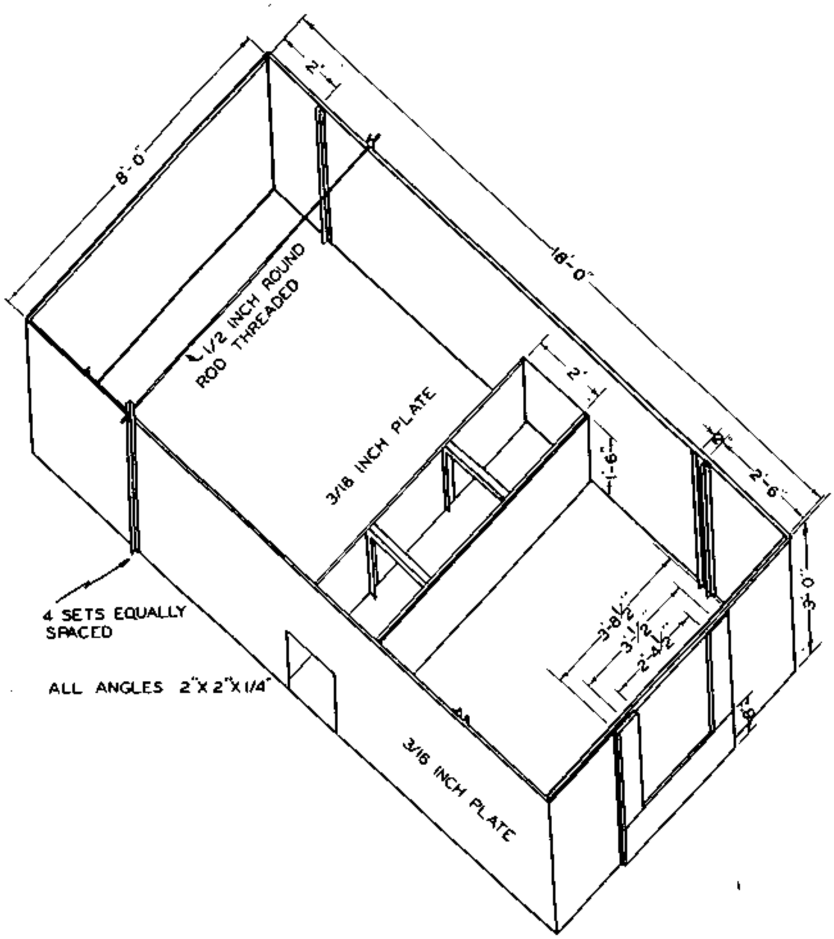


FIG. 1. WELDED STEEL TANK

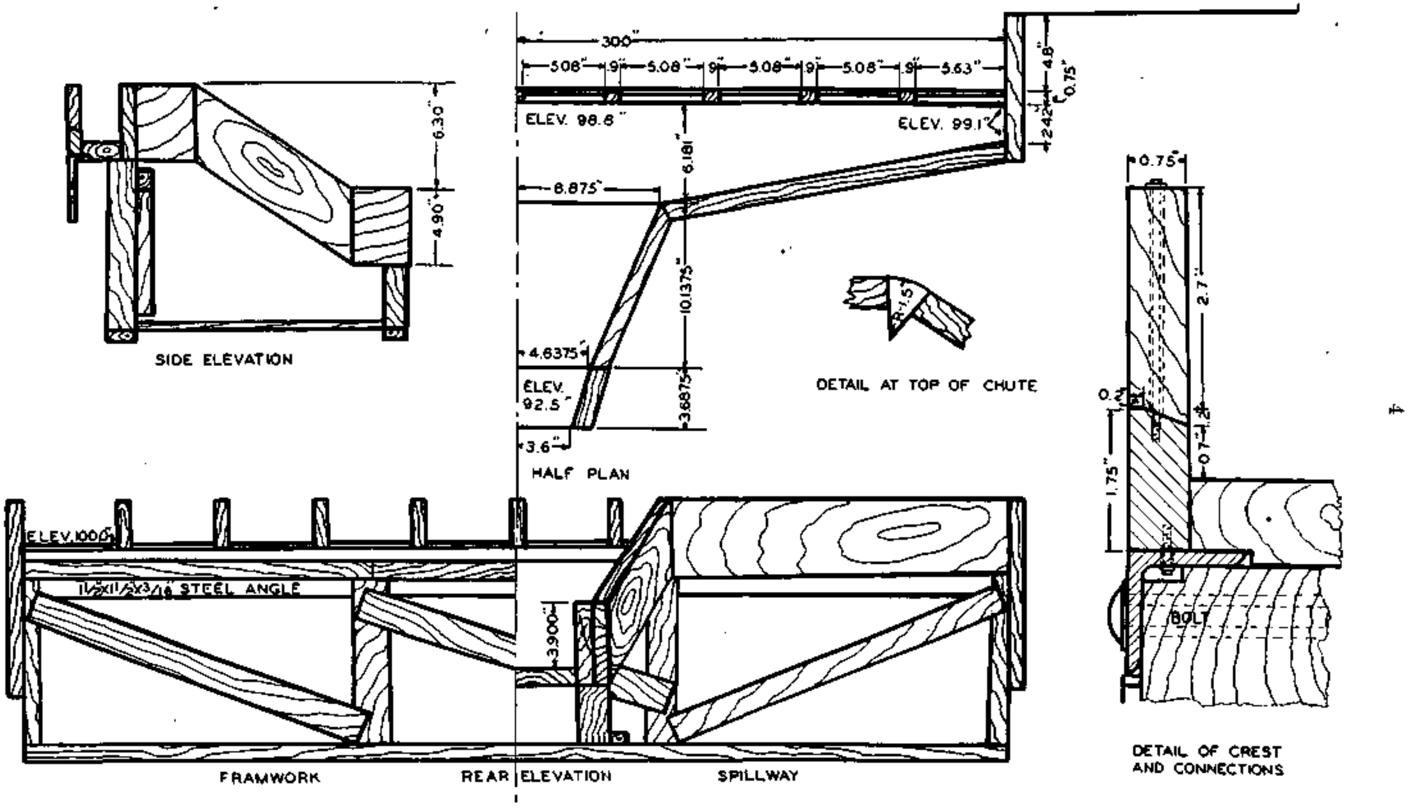
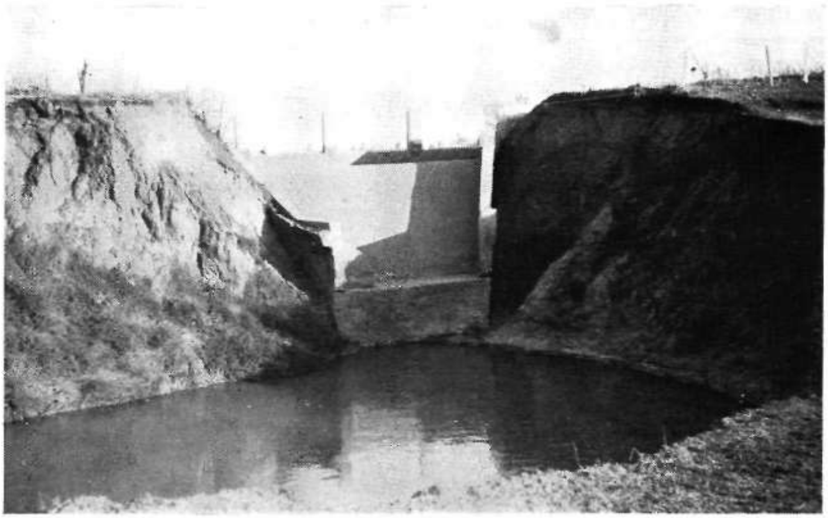
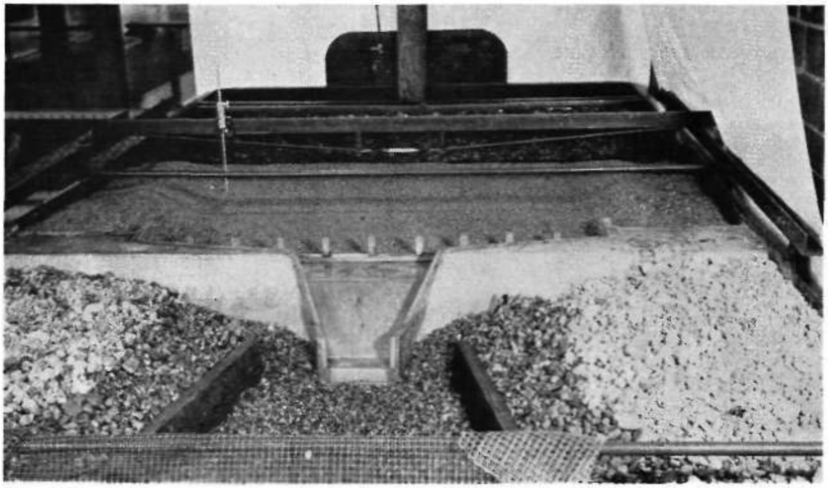


FIG. 2. DETAILS OF WEST FRANKFORT SPILLWAY MODEL





WEST FRANKFORT, ILLINOIS

Top: Spillway structure, note eccentricity of scour hole and undermining of the spillway.

Bottom: Model of spillway approach channel as in Test No. 7, note movable gage.

Figure 1 shows details of the welded steel tank used in the studies and Figure 2 shows details of the West Frankfort spillway model. The model spillway was constructed of white pine wood covered with zinc and welded water tight to the steel tank. The crest was machined from brass and the posts for flash boards are of wood, bolted to the crest.

TESTS

An inspection of the West Frankfort Reservoir spillway and approach channel raises a number of questions regarding the efficiency of the structure, and the seasonal effect of vegetation growing in the approach channel. In the following tests the effect of the various items has been shown graphically by means of head-discharge curves, all plotted on Figure 3. To facilitate a visual picture of conditions these curves have been plotted and comparison made using prototype dimensions. The frictional resistance in the wide shallow approach channel is the predominant force acting upon the water, and therefore a friction formula (Manning's) was used to compute discharge scale relationships between model and prototype. If it is assumed that the ratio of the frictional factor, n, of the model to the prototype equals one, then $Q_m \propto l^{8/3}$ (2), where 1 is the ratio of the linear dimensions. This relationship was used to compute the prototype discharge from the model discharge.

Since this is a model of an existing structure, the attempts to improve the efficiency of the structure were limited to those which could be practically and economically installed in nature. Therefore, nothing was tried that would have necessitated rebuilding the spillway, or excessive construction costs.

A ridge, the crest of which is approximately at spillway elevation, keeps the approach channel dry for several hundred feet in front of the spillway except when the latter is discharging. Consequently, the channel is heavily overgrown with reeds and grass. The seasonal variation in the physical characteristics of growth causes such a wide variation of the probable frictional resistance that it is impossible to set up a condition in the model which will exactly represent prototype conditions. Therefore, an extremely rough bed simulating prototype topography, and a relatively smooth bed in a level approach channel were tested as limiting conditions. (Tests Nos. 2 and 7). If the model is to be used to determine quantitatively the discharge from the reservoir with the topography of the prototype channel as it is at the present time, it is recommended that the discharge be interpolated between the curves obtained in Tests Nos. 1 and 2, Fig. 4. These tests may therefore be considered to cover a range of conditions between a smooth

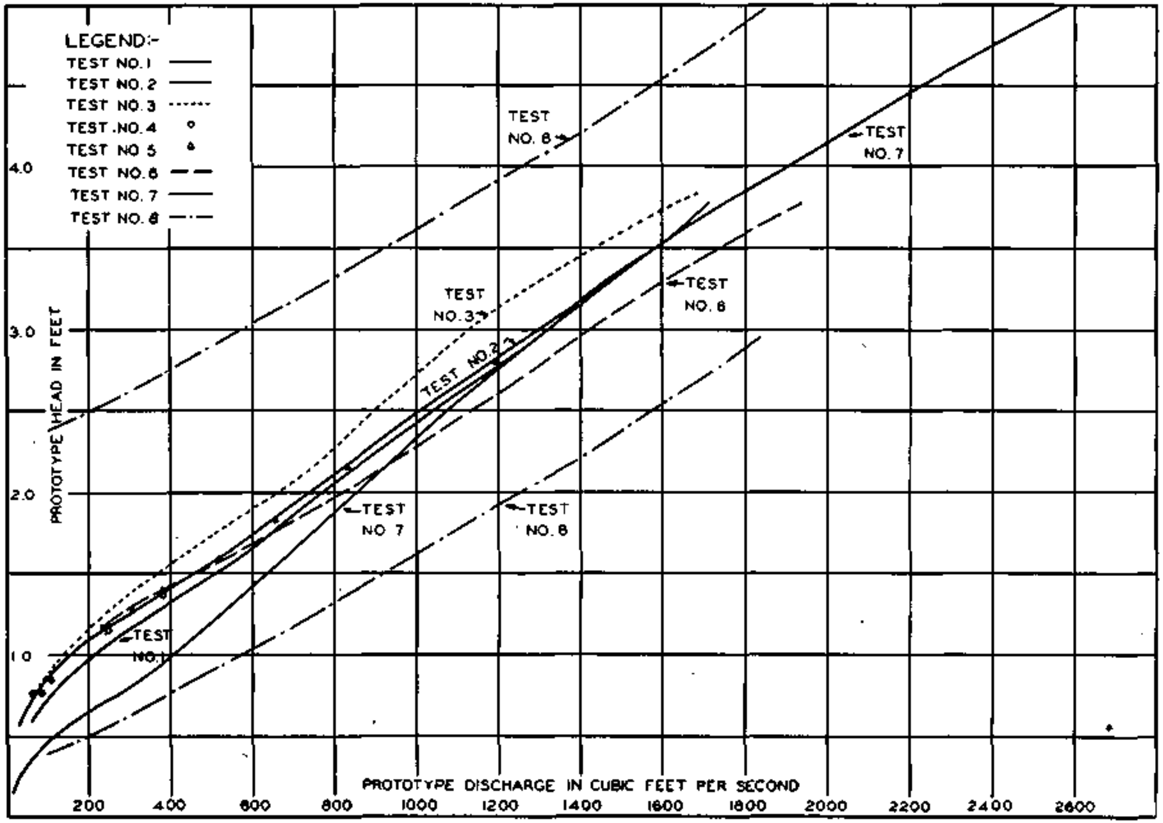


FIG. 3. HEAD—DISCHARGE CURVES WEST FRANKFORT MODEL STUDY

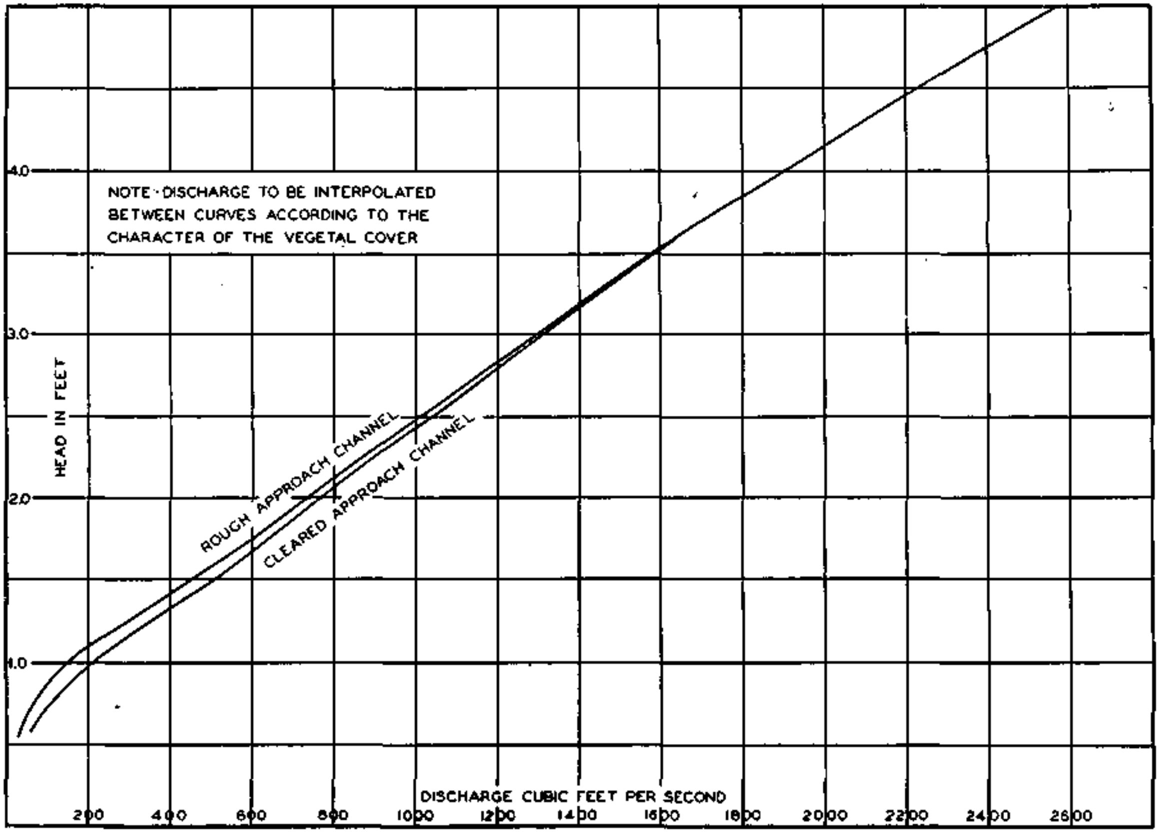


FIG. 4. RECOMMENDED HEAD-DISCHARGE CURVES FOR WEST FRANKFORT SPILLWAY

approach channel devoid of vegetation and one thoroughly covered with grass, reeds, and brush.

TEST No. 1

In this test the approach channel in front of the spillway was paved with smooth concrete mortar. Profile templates plotted from soundings taken at ten-foot intervals along lines perpendicular to the spillway on the center line of each post were used to mould the concrete to the configuration of the prototype. In the prototype the posts on the spillway crest are slotted to receive flash boards. However, it was thought that this would have relatively little effect upon the discharge; so the posts used in this test had smooth sides. The above assumtion was justified by a later test, Test No. 5.

Test No. 1 was run primarily to observe the action of the spillway under, what might be called, standard conditions to provide a basis for comparison of all subsequent tests. This should not be taken to mean that this test truly represents the prototype performance at any particular season.

Observations during this test indicated the probability of a shifting control. At low stages, up to about 0.8 foot head, the high ground in front of the spillway causes noticeable ripples and concentrates the flow in the central and southern section of the spillway. For heads ranging between 0.8 to 1.5 feet the distribution of flow is more uniform and the crest of the spillway is probably the control. Above a 1.5 foot head, decided eddies occur in the collection channel opposite the panels at either end. The effectiveness of these panels decreases with the head and for high heads the surface currents are flowing from the collection channel into the lake. However, evidence that these four panels do not have a zero efficiency is submitted in Test No. 3. For these high heads the apparent control section is the throat of the collection channel. The limits of head in the above discussion of the various control sections should not be taken as an exact limit, because of the fact that the transition from one to the other is not a sudden but a gradual change and no definite point of complete change could be noted in the tests.

TEST NO. 2

Test No. 2 was run to determine quantitatively the effect of roughness in the approach channel. The approach channel was roughened by applying a thin coat of neat cement stucco, to the existing concrete. All other conditions remained the same as for Test No. 1.

The effect of the increased roughness is clearly shown by the spread of the two head discharge curves. For a 0.70 foot head the discharge was only 60.0% of the discharge for the same head with a smooth approach channel, the discharge for a head of 1.25 feet was 84.0% of

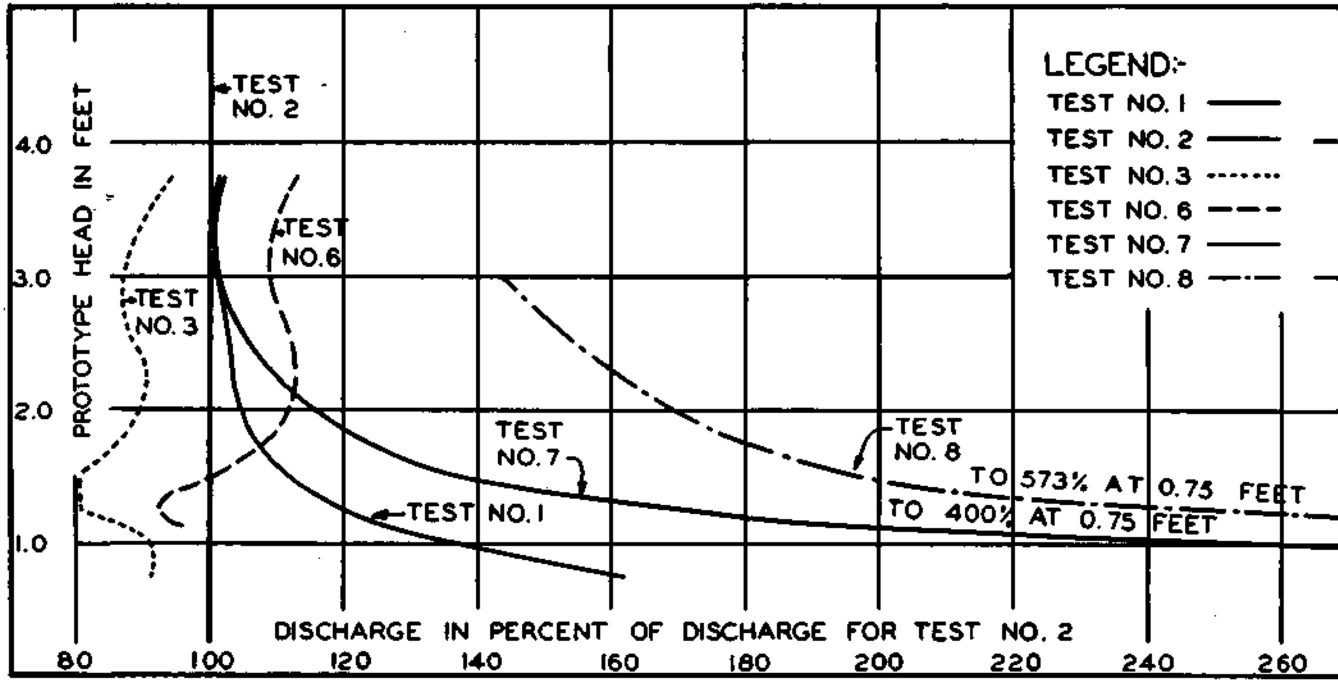


FIG. 5. RELATIVE HEAD-DISCHARGE CURVES COMPARED WITH TEST NO. 2

the discharge for a smooth bed (Test No. 1). The convergence of the two curves continued until for a 3.00 foot head the discharge is 99.0% of the former discharge.

The convergence of these curves would seem to indicate that, for heads above 3.00 feet, contraction of the collection channel has reduced the efficiency of the spillway, to such an extent that the frictional resistance of the approach channel is of no importance. This phenomena is again emphasized in Test No. 7.

The distribution of flow and eddies noted in Test No. 1 were repeated in this test. However, the high ground in front of the spillway was a more effective control. Since the approach channel remained unchanged for Tests Nos. 2-6, all subsequent comparisons are made with Test No. 2, and are plotted in Fig. 5.

TEST NO. 3

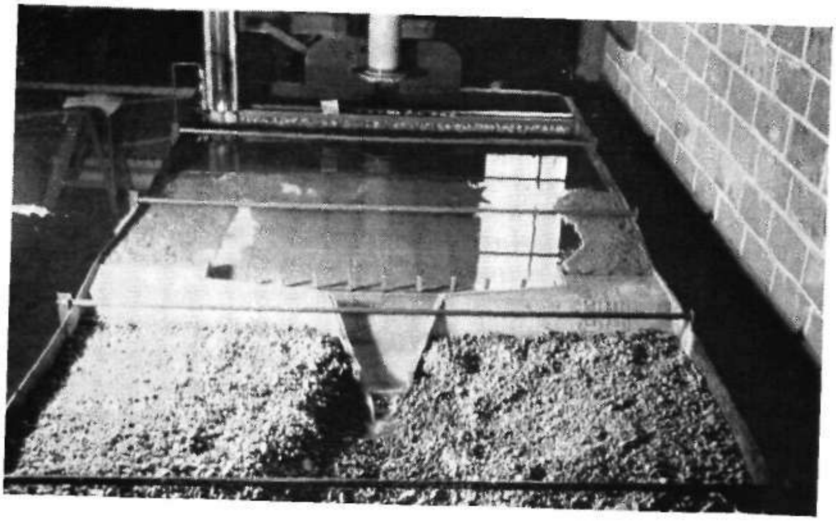
For high heads the eddy currents at the ends of the spillway were so marked that the discharge was apparently carried by the center six panels. To determine the effectiveness of the two end panels upon the total discharge these four panels were blocked. With this exception the conditions were the same as for Test No. 2 and all comparisons are made with that test.

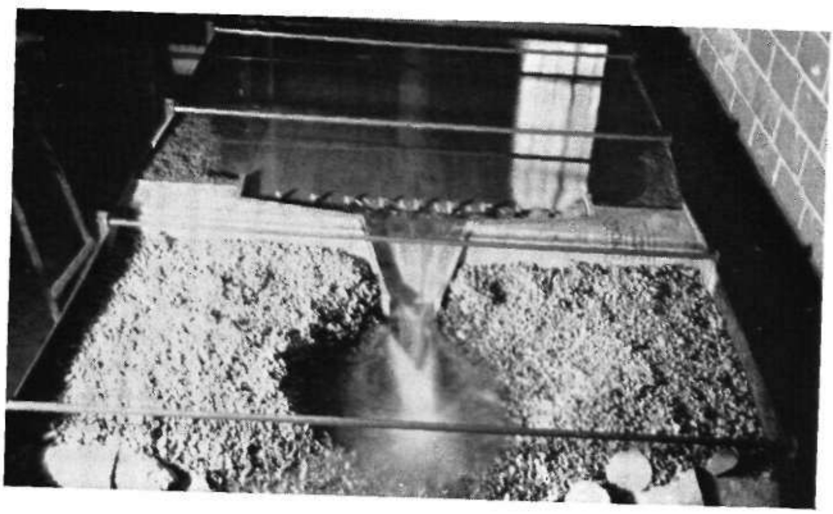
The discharge for a head of 0.75 feet is 91.2% of the discharge for the same head for the conditions of the Test No. 2. Below this head the curves converge as the high ground in front of the spillway controls the discharge. Above the head of 1.00 feet the curves rapidly diverge until for a head of 1.25 feet the discharge is 80.4% of the discharge in the preceding test. This divergence is almost constant between 1.25 feet and 1.50 feet. The curves then converge until the discharge for a head of 2.25 feet is 90.8% of the discharge for a similar head in Test No. 2. The curves spread again and the discharge for a head of 2.90 feet is 87.0% of the discharge in Test No. 2. Above a head of 2.90 feet the curves converge rapidly until the discharge is 94.8% of the former discharge for a head of 3.75 feet.

If the discharge through each panel was proportional to the length of the panel, these four' end panels should carry 40% of the total discharge of the total spillway. However, it was found that they are most effective for heads from 1.25 to 1.50 feet, and even then they carry only 19.6% of the total discharge for that head and thus have a maximum efficiency of only 49%.

TEST NO. 4

In the prototype the collection channel in front of the spillway has neither sufficient slope nor cross-sectional area to carry the flow if the discharge from each panel were proportional to its length. With this





WEST FRANKFORT SPILLWAY MODEL

Approach channel molded to prototype conditions.

Top: Low head discharge

Bottom: High head discharge

in mind the collection channel was contracted from the third point to the end. The width was made to vary from zero at the end to the width of the prototype channel at the third point. The depth was also varied from the elevation of the spillway crest at the end to prototype depth at the third point. The first few points for heads between 0.77 and 1.36 feet fell so close to the curve obtained in Test No. 2 that no further observations were made. These points are plotted in Figure 3, but to avoid confusion the curve was not drawn in.

TEST No. 5

As was noted in the discussion of Test No. 1 the posts on the spillway crest are grooved to receive flash boards. In all previous tests this detail had been neglected, and the effect on the efficiency of the structure had been questioned. With the exception of grooving the posts all conditions were identical with those of Test No. 2. Except for very low flows the velocity of the water was sufficient to jump the relatively narrow grooves. Observations for heads between 1.15 and 2.80 feet plotted so close to the former curve that no further observations were made. These are plotted but no curve was drawn through them.

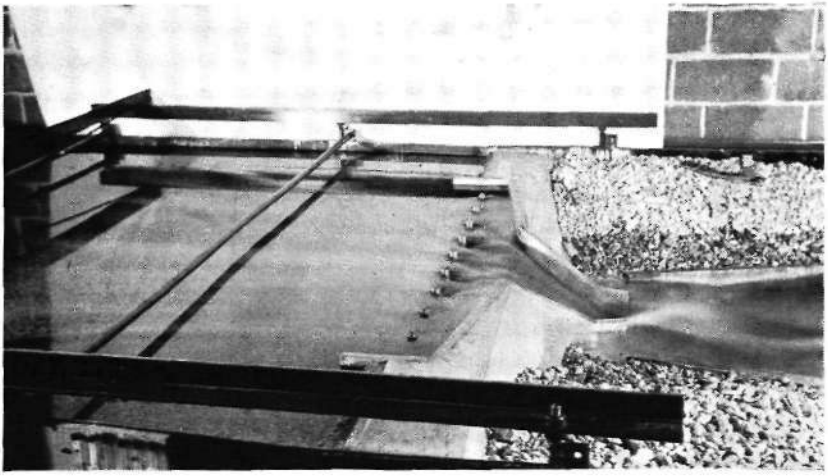
TEST No. 6

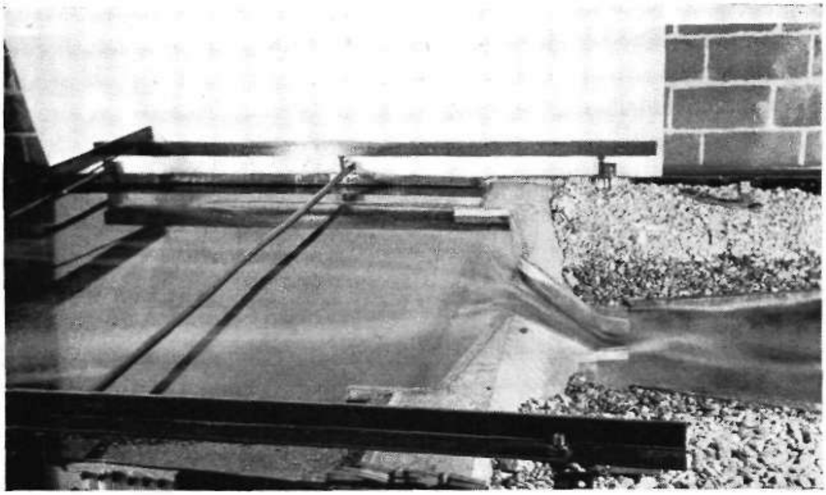
In this test all posts were removed from the crest of the spillway. This increase in the effective width of the spillway should have increased the discharge for any given head. However, for heads below 1.48 feet the discharge was decreased. For example, for a head of 1.35 feet the discharge was 92.4% of the discharge in Test No. 2. This is possibly due to the increased friction over the weir and the definite control established by the unevenness of the approach channel. For heads above 1.48 feet the curve diverged rapidly until the discharge for a head of 2.30 feet was 112.8% of the discharge for the same head in Test No. 2. The percentage increase remained practically constant for all heads above 1.80 feet.

TEST NO. 7

For this test the approach channel was cleared and made horizontal with an elevation 0.7 feet lower than the spillway crest. This condition eliminates the upstream control and thus increases the effective width of the spillway and its efficiency for low flows. Before this test was started a 90° triangular weir notch was installed to measure the discharge. With this arrangement a wider range of discharges could be measured than with the orifices used in the preceding tests.

For heads under 0.5 feet the observed points scattered. The scatter was probably caused by the fact that for the low flows equilibrium was attained only after a long interval of time. However, an apparent





WEST FRANKFORT SPILLWAY MODEL
Approach channel level

Top: Spillway as in prototype
Head: 3.98 Discharge: 1890 c. f. s.

Bottom: Posts removed from spillway crest
Head: 4.08 Discharge: 1890 c. f. s.
Note inefficiency of side channels.

equilibrium was attained in from 10 to 15 minutes, and a number of observations were made after this interval had elapsed.

For a head of 0.6 feet the discharge was 453% of the discharge for the same head in Test No. 2. The two curves converge rather rapidly until for a head of 3.0 feet the discharge is 101.5% of the former discharge. This is a further confirmation of the existence of a control formed by the collection channel.

TEST No. 8

In all previous tests, the spillway weir was submerged, and the discharge decreased for a wide range of heads. The inadequate size of the side channels prevented the water from flowing away from the crest and thus lessened the effective head and the efficiency of the structure.

In this test flash boards were added to raise the elevation of the spillway crest two feet. All other conditions were identical with Test No. 7. The head-discharge curve for this test has been plotted as observed and also as the observed discharge plotted against the observed head minus two feet. All comparisons are made with the latter curve to show the effect of providing an adequate collection channel.

For a head of 0.6 feet the discharge is 738% of the discharge for Test No. 2.

The percentage discharge decreases somewhat and at 1.0 feet is 307% of the discharge in Test No. 2. The curve continues to approach the curve for Test No. 2 until for a head of 3.0 feet the discharge is 143.8% of the former discharge.

This again indicates definitely that the collection channel is not of sufficient cross-sectional area to take care of the discharge.

During this test the water was at least 2.7 feet deep in all parts of the approach channel. The increase in minimum depth of the flowing water made it seem possible that the predominate force acting on the water might be a gravitational force. If the predominate force acting on the water is gravitational then $Q_m \propto l^{5/2}$, where 1 is the ratio of the linear dimensions. To determine whether this condition existed, the data from Tests Nos. 7 and 8 were plotted in model dimensions. When plotted, the results indicated that friction rather than gravity was the predominate force acting.

CONCLUSION

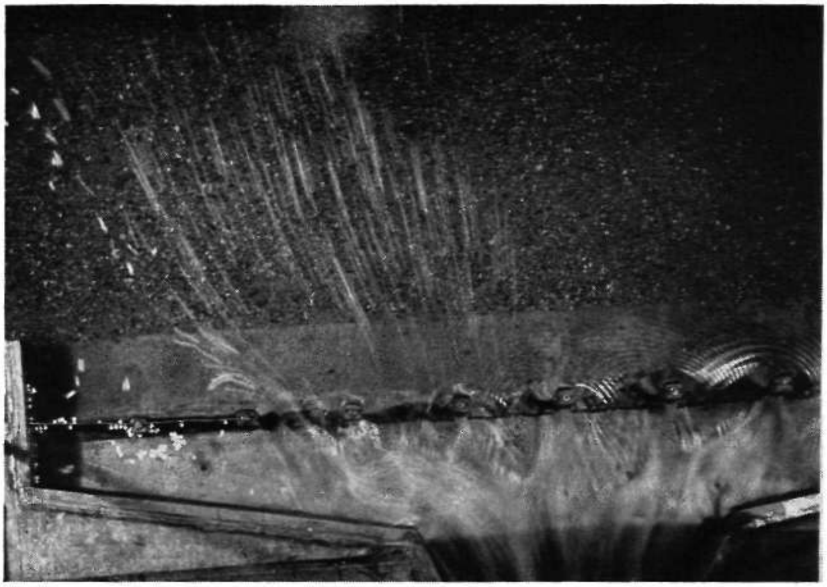
These tests have in general shown the inefficiency of a spillway structure of this type. The existence of three distinct control sections in the prototype, precludes any possibility of computing an empirical coefficient of discharge. Since roughness of the prototype approach channel varies with the vegetal cover, and thus with the season of the year, it is impossible to obtain a head-discharge curve which can be taken as absolute. For this reason it is recommended that the curves obtained in Tests Nos. 1 and 2 (Fig. 4) be considered as limiting cases for existing prototype conditions. By combining these curves with reasonable judgment as to the condition of the vegetal cover, a fairly accurate determination, of the spill from the reservoir could be obtained. However, if prototype conditions could be altered so that they were similar to the conditions in the model at the time of Tests Nos. 7 or 8, the respective curves could be used to obtain a reasonably good estimate of the discharge. The curves plotted are probably inaccurate for low heads, because of the effect of surface tension and viscosity on the thin flowing sheet of water, and the increased probability of inaccuracies in measuring very small discharges with a triangular weir notch. For high and medium heads the curves should provide a relatively close estimate when applied with good judgment.

Studies made on this model clearly point out the waste of material, and the possible danger to the structure and property below the dam if improperly designed side channel spillways are used. Test No. 3 shows the maximum efficiency of the four end panels to be only 49%. The maximum efficiency of the two end panels is no doubt much less than this figure. The consequences of this reduced efficiency are obvious if the run-off should at any time be equal to the maximum run-off used in the design calculations. For example, for a head of 3.75 feet the four end panels carry only 5% of the total discharge. This reduces the capacity of the spillway to about 63% of the probable design capacity for that head. The above calculations assume that the side channel is of adequate capacity to carry the discharge of the center six panels.

If topographical conditions demand the installation of a side channel spillway the slope, depth and cross-sectional area of the collection channel should be carefully calculated. The neglect of careful calculation may result in a considerable portion of the length of the crest functioning at reduced efficiency.

The water enters the channel with considerable velocity at right angles to the axis of the spillway channel. This transverse velocity has no component in the final direction of flow down the channel. The force necessary to change the direction of the water is supplied by the fall between the point of inlet and the outfall of the channel. Assuming a uniform distribution of velocity across the channel, the total applied energy is equal to the total discharge multiplied by the average drop of all the particles of water. Only part of this energy is available for producing velocity in the direction of flow. The remaining energy is dissipated as friction.

WEST FRANKFORT SPILLWAY MODEL
Approach channel level
Note surface current directions, distribution of flow, and relative velocities
Top: Test No. 7 Head: 0.71 feet Discharge: 240 c. t. s.
Bottom: Test No. 7 Head: 4.46 feet Discharge: 2200 c. t. s.



WEST FRANKFORT SPILLWAY MODEL
Approach channel level
Note surface current direction, and distribution of flow
Top: Two end panels blocked as in test No. 3.
Head: 4.52 feet Discharge: 2200 c. f. s.
Bottom: Test No. 8 Head: 4.32 feet Discharge: 1470 c. f. s.

The equation expressing the hydraulic relations for side channel flow are derived by making the momentum after impact equal to the momentum before impact plus the accelerations due to external forces. The discharge is considered to be uniform across the entire length of the crest. It is therefore obvious that the usual analysis of side channel spillways can not be applied to the West Frankfort structure.

A detailed discussion of the hydraulics of side channel spillways, and the design calculations is contained in a paper by Julian Hinds (3). The theory proposed by Mr. Hinds has been verified by measurements on the existing structures and also by model studies (4).

If contraction of the collection channel is necessary because of physical limitations, the reduction in area should be compensated for by an increased slope. The greater velocities crested by the steeper slope must be then dissipated.

Erosion could be lessened or prevented by extending the paved discharge channel down stream with facilities for inducing a hydraulic jump on the apron.

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DATA

WEST FRANKFORT MODEL

$$Q_p \propto l^{8/3} Q_m, \; l = 20$$

TEST NO.	ORIFICE		MODEL		PROTOTYPE	
	Size inches	Head feet	Qm c. f. s.	Head feet	QP c. f. s.	Head feet
1	3.000	6.52	0.620	0.195	1830	3.90
		4.72	0.524	0.172	1543	3.44
		2.78	0.411	0.140	1211	2.80
	2.000	7.31	0.280	0.106	825	2.12
		5.80	0.249	0.097	734	1.94
		4.11	0.214	0.086	630	1.72
		2.81	0.178	0.075	524	1.50
	1.335	7.31	0.129	0.064	380	1.28
		5.23	0.109	0.059	321	1.18
		3.86	0.0932	0.055	274	1.10
		2.88	0.0815	0.052	240	1.04
	1.004	6.93	0.0758	0.050	224	1.00
		5.17	0.0652	0.047	192	0.94
		. 4.40	0.0600	0.045	177	0.90
		3.76	0.0558	0.044	164	0.88
		2.77	0.0478	0.041	141	0.82
	0.667	6.76	0.0346	0.035	102	0.71
		4.68	0.0283	0.033	83	0.66
		2.59	0.0209	0.029	62	0.58
2	3.000	5.79	0.580	0.189	1710	3.78
		4.53	0.520	0.170	1534	3.40
		2.82	0.410	0.142	1209	2.84
	2.000	7.33	0.280	0.108	826	2.16
		6.64	0.266	0.105	784	2.10
		5.74	0.248	0.100	732	2.00
		4.88	0.229	0.094	675	1.88
		4.06	0.210	0.089	619	1.78
		2.84	0.177	0.081	522	1.62
	1.335	6.84	0.127	0.068	374	1.36
		4.76	0.105	0.063	310	1.26
		2.68	0.078	0.057	230	1. 14
	0.667	7.39	0.036	0.042	106	0.84
		4.71	0.029	0.039	85	0.78
		2.87	0.022	0.036	65	0.72

TEST NO.	ORIFICE		MODEL		PROTOTYPE	
	Size inches	Head feet	Qm c. f. s.	Head feet	QP c. f. s.	Head feet
3	3.000	5.69	0.573	0.191	1690	3.84
		2.72	0.402	0.156	1185	3.12
	2.000	7.03	0.276	0.116	814	2.32
		6.03	0.255	0.109	753	2.18
		3.86	0.206	0.094	608	1.88
		2.64	0.172	0.085	507	1.70
	1.335	7.43	0.130	0.077	384	1.54
		4.94	0.106	0.070	313	1.40
		2.72	0.079	0.062	233	1.24
	0.667	6.58	0.034	0.043	100	0.86
		2.72	0.0215	0.037	63.4	0.74
4	1.335	7.21	0.1285	0.068	379	1.36
		2.89	0.083	0.057	244	1.14
	0.667	7.14	0.0357	0.042	105	0.84
		4.60	0.0280	0.039	83	0.78
		2.64	0.0211	0.0385	62	0.77
5	3.000	2.80	0.405	0.140	1190	2.80
	2.000	7.39	0.281	0.108	829	2.16
		4.51	0.222	0.0915	655	1.82
		2.75	0.175	0.080	516	1.60
	1.335	7.41	0.129	0.070	380	1.40
		4.66	0.104	0.064	307	1.28
		2.66	0.078	0.0575	230	1.15
6	3.000	7.51	0.660	0.195	1940	3.90
		4.87	0.530	0.162	1560	3.24
		2.81	0.410	0.131	1210	2.62
	2.000	7.21	0.279	0.101	823	2.02
		4.73	0.227	.0.089	670	1.78
		2.75	0.175	0.0785	516	1.57
	1.335	7.06	0.128	0.070	378	1.40
		4.61	0.103	0.064	304	1.28
		2.67	0.078	0.058	230	1.16

TEST NO.	WEIR		MODEL		PROTOTYPE	
	Size	Head feet	Qm c. f. s.	Head feet	QP c. f. s.	Head feet
7.	90° V-notch weir	0.652	0.900	0.255	2650	5.10
		0.610	0.760	0.226	2240	4.52
		0.571	0.641	0.199	1890	3.98
		0.567	0.625	0.194	1840	3.88
		0.549	0.565	0.183	1665	3.66
		0.516	0.480	0.161	1415	3.22
		0.474	0.400	0.136	1180	2.72
		0.329	0.158	0.057	466	1.14.
		0.274	0.101	0.039	298	0.78
		0.243	0.074	0.033	218	0.66
		0.203	0.047	0.027	160	0.54
		0.179	0.035	0.022	103	0.44
		0.174	0.032	0.022	94	0.44
		0.167	0.029	0.022	84	0.44
		0.156	0.024	0.019	69	0.38
		0.149	0.022	0.017	63	0.34
		0.130	0.015	0.015	44	0.30
		0.124	0.014	0.015	40	0.30
		0.091	0.0106	0.012	31	0.24
		0.075	0.0104	0.0095	31	0.19
		0.067	0.0103	0.0075	30	0.15
8	90° V-notch weir	0.568	0.625	0.248	1840	4.96
		0.525	0.525	0.223	1550	4.46
		0.486	0.440	0.203	1300	4.06
		0.431	0.320	0.177	944	3.54
		0.393	0.255	0.162	751	3.24
		0.322	0.158	0.143	466	2.86
		0.251	0.081	0.127	239	2.54
		0.179	0.034	0.119	100	2.38