University of Minnesota
St. Anthony Falls Hydraulic Laboratory

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HYDRAULIC MODEL STUDY OF THE BUSSE WOODS RESERVOIR DROP STRUCTURE

by

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Conducted for

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ACKNOWLEDGEMENT

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I. INTRODUCTION

The Busse Woods Drop Structure and recreation area is located in Cook County, Illinois in Suburban Chicago. Operated as a typical Soil Conservation Service drop structure since 1976, it is now contemplated that the structure be modified to reduce flood flows downstream of the structure for floods greater than the two–year event. In order to maintain its present low flow capabilities and at the same time decrease the return frequency for high discharges associated with low frequency return interval events, it was proposed by the Illinois Department of Transportation (IDOT) that a 2.3 ft high steel box beam be installed 1.2 feet above the crest. Placement of such a beam would modify the existing weir flow into three flow regimes: 1) weir flow at low flows, 2) orifice flow at moderate flows, and 3) a combination of orifice and weir flow when the beam is overtopped at higher flows. This report contains the results of rating curve and sediment erosion tests carried out on a 1:12 geometric scale hydraulic model of half of the structure.

II. MODEL DESCRIPTION

The Busse Woods Hydraulic Model was constructed to an undistorted 1:12 geometric scale using primarily timber construction (Photo 1). A subfloor was constructed in the region nearest the drop structure and stilling basin so that the disruptions caused by modifications to this area would be minimized. The model encompassed a reach approximately 270 ft upstream to 150 ft downstream of the structure. (Photo 2 and Figure 1) Due to the large degree of natural symmetry upstream of the drop structure, only the left bank and left hand (looking downstream) half of the structure was modelled. Modelling only a portion of such a structure is a typical procedure and normally has no measureable effect on the test results.

Modelling only a portion of the structure is common in hydraulic modelling and, provided that critical components are not omitted, results in test data unchanged from that obtained for a complete model. Modelling only a portion of the structure can be beneficial as it often allows an increase in scale ratio, thereby making both construction and data measurements more accurate.

In the case of the Busse Woods principal spillway structure, modelling one half of the structure still allowed all critical components to be incorporated into the model. While possible effects on flow separation at the center pier exist, these should be insignificant in their effect upon the performance of the structure. While it is true that a boundary layer will form along the centerline of the model causing velocities to be decreased, the raised portion of the essentially broadcrested weir upstream of the drop causes significant acceleration of the flow and a corresponding suppression of the boundary layer. Such boundary layer suppression, and the fact that flow separation at a pier is a function of both the longitudinal and the transverse velocities at the pier (along with the natural symmetry upstream of the structure), should lead to no measureable effect on the test results. We have chosen the adjective "should" only because of the fact that only one half of the structure was modelled and, therefore, no stronger definitive statement can be made without the ability to compare the results to those of a whole model.

Because gravitational and inertial forces are predominant, the Froude numbers in the model and the prototype were made identical. Typical scaling ratios are as follows:

Length	$L_r = \frac{L_m}{L_p} = 1{:}12$
Velocity	$V_r = (L_r)^{1/2} = 1{:}3.46$
Flow	$Q_r = (L_r)^{5/2} = 1{:}499$
Time	$T_r = (L_r)^{1/2} = 1{:}3.46$

These relationships are an integral part of the deriviation of the Froude number.

$$\text{Froude number} = \frac{V}{\sqrt{g\ L}}$$

where : V is a characteristic velocity
: L is a characteristic length
: g is the gravitational acceleration constant

All, or part, of the deriviation of the relationships given above is provided in most introductory Fluid Mechanics texts, as well as other references, such as Hydraulic Laboratory Techniques.[1]

The Busse Woods drop structure model consisted of a headbox upstream of the model, the model itself, and a tailbox downstream of the structure. Water entered the model through a manifold connected to a pair of parallel pipes of 6 and 12 inches in diameter. Each pipe utilized a previously calibrated orifice for flow measurement. The use of two supply pipes provided for better resolution at lower flows, while giving high flow capability. A hydraulically actuated gate was used at the downstream end of the model to control tailwater elevation.

The box beam used to simulate the 2.3 ft high beam was constructed of wood to the scaled structural dimensions provided by the Illinois Department of Transportion shown in Fig. A–1 of the appendix. To model roughness characteristics, the upstream slope and crest of the drop structure were constructed using multiple layers of rock scaled to match the 4" to 8" rock estimated to be in place by the Illinois Department of Transportation staff. The structure was constructed according to Busse Woods "as built" drawing sheet numbers 88, 28 of 79, and recent underwater topography information. To enable the conducting of scour studies in the reach downstream of the structure, the entire bed of the model, exclusive of the bed downstream of the structure, was constructed 10 inches (10 ft prototype) above the concrete floor of the modelling basin. After review of the soils analysis provided by the IDOT for the area near the drop structure (Table A–1 of the appendix), no sediment could be found to accurately model the sediment characteristics of the basin due to the extremely fine nature of the required sediment. Due to this, plus the fact that as it presently exists, the region just downstream of the basin is riprapped with 8 to 12 inch rock to prevent scour, the most suitable available fine sediment from two sources was deposited to the stilling basin floor elevation for a distance of 100 ft downstream of the apron. Section III of the report will detail the test results; however, both sediments that were tested exhibited similar scour patterns.

Photos 3 and 4 provide views of the nearly completed model prior to initial inundation.

III. TEST PROGRAM

A. Test Procedure

Two basic types of test runs were necessary to ascertain both the drop structure's rating curve and the potential for scour downstream of the structure's stilling basin. For any type of test run it was essential that the filling and drainage of the model prior to and after a series of runs be done cautiously to avoid the consequences of possible model shifting caused by a large imbalance in hydrostatic pressures on the structure.

To determine the rating curve once the model had filled to the weir crest level, the flow was gradually increased to the desired value, and the tailwater point gage located 127 ft downstream of the crest, was set to the proper tailwater elevation. Tailwater elevation was based on the tailwater rating curve simulated with the continuous hydrologic model LANDS and the dynamic hydraulic routing model FEQ. Resultant simulation data was supplied in disk format by the DuPage County Department of Environmental Concerns (Fig. A–2 of appendix). As mentioned earlier, due to modelling constraints, only a portion of the principal spillway structure was modelled. Because of this, any flows which normally pass through auxiliary or emergency spillways were not part of the model. Hence, the tailwater rating curve shown in Fig. A–2 was adjusted to correspond to the discharge passing through the principal spillway (denoted by dashed lines on the graph). The tailwater control gate at the downstream end of the model was adjusted to attain the correct tailwater elevation. Once the tailwater elevation had stabilized, the model inflow was verified to assure that no changes had occurred. The model was then allowed to run until headwater measurements indicated that the water surface elevation had stabilized. The tailwater elevation was again verified, and headwater readings were taken at two locations; 21.2 ft and 134.4 ft upstream of the vertical edge of the crest along the channel centerline. The flow was then adjusted to a new value and the process started over again. For sediment erosion testing the process was similar except that once the model had filled to the weir crest, the flow was increased and decreased as rapidly as prudent to minimize modification of the scour pattern during transitional flow periods.

B. Model Test Results

Original IDOT drawings indicated a crest elevation of 686.00 ft.; however, field survey verification of the actual conditions later identified that the structure is four inches lower, with the spillway crest being at 685.67 ft. The model was modified to represent the lower crest elevation, the test program was repeated, and all rating curve data given in this report are for the 685.67 ft. crest elevation. The resultant rating curve for the existing structure (without the beam) is shown in Figure 2 (tabulated in Table A–2). The effect upon the head discharge relationship caused by the installation of a 2.3 ft. high beam, 1.2 ft. above the existing crest, is shown in Figure 3 (tabulated in Table A–3).

The location for water surface elevation measurements chosen, 134.4 ft upstream of the crest, was a convenient location in the model and far enough upstream to avoid any noticeable localized effect near the structure crest, but it does not have any particular significance beyond this. The same is true of the tailwater reading location. A measurement of the water surface elevation was also taken at the upstream edge of the raised spillway approach area (21.2 ft from the crest). It was brought to SAFHL's attention during the study that the location of the water surface measurement in the field has been open to discussion. To better understand the water surface profile in the vicinity of the structure, SAFHL measured water surface profiles upstream of the structure for selected flows (Figs. 5 and 6 – tabulated in Tables A–6 and A–7). As anticipated, a noticeable drawdown occurs in the vicinity of the crest due to the increased velocity head as the water passes over the raised approach area.

After review of the test results, the Illinois Department of Transportation requested that the beam position be modified so that the recommended project stage–discharge relationship developed by the IDOT and shown in Table 5 of the appendix, be more closely represented. Trial runs were initiated using 0.6 ft as the orifice height (distance from crest to the bottom of the beam). Further runs testing a variety of orifice heights were performed in an effort to most closely represent the model study relationship to that recommended by the IDOT. After discussion with the IDOT, it was agreed that an orifice height of 0.68 ft was most representative, with the match of the experimental and recommended curves being remarkably close. Any differences seen between the two curves are within the experimental accuracy of the system. Testing to more fully develop a head discharge relationship with this orifice was completed, and the resultant rating curve is shown in Fig. 4. Tabular summaries of this rating curve test data and the recommended relationship can be seen in Tables A–4 and A–5 of the Appendix.

Little field information was available at the time of the model study regarding the sediment erosion downstream of the structure. It was only stated that a scour hole has formed, but the area is riprapped, and scour did not at that time seem to be of primary concern. For that reason, scour studies were done downstream of the structure primarily to compare the relative effects of the proposed modification. To accentuate the sediment erosion and thereby make the scour patterns more evident, most of the erosion studies were carried out without the use of riprap. As will be shown in the final erosion studies, Photos 5 and 6, the use of riprap presently installed downstream of the structure greatly diminishes the amount of sediment erosion.

Photos 7 and 8 of sediment erosion tests Nos. 4 and 5 indicate that if riprap were not in place, a relatively deep scour hole would form at higher flows. Table 1 summarizes the pertinent parameters of the sediment erosion tests and their resultant scour hole formation. The flow and duration used for the scour hole test runs were based on computer generated hydrographs supplied by IDOT for storms of intervals of 2, 10 and 100 years in Figs. A–3, A–4, and A–5 of the Appendix. Test data from run 7 indicated that no noticeable erosion occurred for a two–year storm. Run 6 did indicate substantial scour for the 10–year storm without riprap in place.

Several limitations involved in the modeling of the sediment erosion prevented complete confidence in the quantitative results of the testing. However, the qualitative results appear valid. These limitations arose from several sources. First, due to the fine nature of the soil in the vicinity of the structure, SAFHL was unable to procure sediment which exactly modeled that in the prototype, and of necessity chose the two most representative alternatives available. No direct testing between the two alternatives was done, however, observations indicated no noticeable difference in scouring between them. Secondly, it was only practical to model erosion for specific instances such as a single 100–year storm event and not over a long term hydrologic record. In modelling, the 2, 10, and 100–year storm events were tested using the peak synthetic hydrograph flow and basing the time duration of the run as the duration from when the rising flood wave exceeded 90% of peak until the receding portion of the hydrograph decreased below 90% of peak flow. A third limitation to completely ascertaining the cumulative scour pattern downstream of the structure was that little information was available on the sediment loading of the stream.

With regard to these limitations, in addition to choosing the most representative sediment practical, the testing of the structure was done in a manner as much as possible to accentuate short–term erosion. While sediment erosion potential downstream of a structure has been tested for generic designs, many questions arise as to its exact relationship to increased flow. Studies indicate that erosion is typically not linear with flow, but rather must reach some flow threshold before erosion can occur, with ever increasing flows greatly increasing the erosive potential. For this reason peak flows were chosen from the hydrographs for use as a constant flow rate throughout an erosion test. Sediment erosion rate decreases with time, eventually reaching some point of equilibrium. The test duration mentioned earlier extending from 90% of peak flow on the rise, through peak, to 90% peak flow on the recession curve, was chosen based on SAFHL's experience, as no data were available regarding the flow rate necessary to reach the erosion threshold rate. Often times during the recessionary portion of a flood hydrograph, sediment is redeposited in areas such as scour holes as the flows erosive potential decays. As mentioned above, since no information regarding the sediment loading of the stream was available, no sediment was added to the flow. Addition of such sediment may reduce the overall scour apparent downstream of the structure. The last condition which was considered during the development of the test program is that the reach downstream of the structure is riprapped to minimize scour. Data from runs 9 and 10 in Table 1, and shown in Photos 5 and 6, indicate the extent of scour encountered during model testing of an anticipated 100–year flood, both with and without the beam installed.

General trends indicate that sediment erosion is slightly reduced with the proposed beam installed.

TABLE 1

Summary of Sediment Erosion Tests

Sediment Erosion Test	Est. Storm Return Interval (yrs)	Q (cfs)	Duration (hrs/min)	T.W. Elev. (ft)	H.W. Elev. (ft)	Sediment Erosion Statistics Downstream of Stilling Basin: Max. Elev. of Riverbed (ft)	Min. Elev. of Riverbed (ft)	Length of Depression (ft)	Width near Stilling Basin (ft)	Width at end (ft)
w/o riprap; w/beam placed 1.2 ft above spillway crest										
4	100	2800	2 hrs, 10min	690.22	693.28	676.4	667.9	≈ 57	≈ 39	≈ 20
w/o riprap, w/o beam in place										
5	100	2810	2hrs, 3min	688.35	691.81	677.8	666.0	≈ 73	≈ 30	≈ 26
6	10	1010	7hrs, 0min	685.20	688.96	674.9	669.9	53.5	24	19
7	2	600	4hrs, 10min	683.75	688.11	674.9	674.9	N/A	N/A	N/A
With riprap, w/beam installed 1.2 ft above crest										
9*	100	2550	10hrs, 52min	689.85	693.11	674.9	672.0	25	31	16
With riprap, w/o beam in place										
10*	100	2710	15hrs, 18min	688.25	691.56	674.8	670.4	45	27	9

*100 year hydrograph peak flow – time computed for period for which flows were above 0.9 Q_{max}

Qualitative observations made during the study using dye injection and floating debris indicate that due to the relatively high tailwater, the stilling basin performs effectively only for lower flows. At higher flows, above approximately 1000 cfs, the basin blocks are submerged to the extent that the primary flow passes over all but the top 15 to 20% of the blocks, with a return flow toward the spillway crest at lower elevations. With this large submergence, the minimal effectiveness of the stilling basin at high flows, and the fact that the flows used in the testing were based on the recommended relationship and are better matched by the beam installed 0.68 ft above the crest, it was felt that retesting of the sediment erosion characteristics was unwarranted for the beam placed at 0.68 ft above the crest.

Due to the noticeable submergence of the structure under higher flow conditions, at higher flows the nappe did not ventilate. With no beam in place, the nappe was no longer ventilated for flows of 850 cfs or greater. With the beam placed 0.68 ft above the crest, nappe ventilation was visible in flows up to 1750 cfs. Ventilation, or lack thereof, of the nappe is an indication that submergence is having possible effects on the structure's head discharge relationship. The effects of tailwater elevation and subsequent submergence on the structure are provided in a supplement to this report. These observations are, of course, dependent on many other conditions of the flow and the structure. For example, if flow is increasing, ventilation of the nappe may not cease until the flow reaches 1800 cfs; but, if the flow is then reduced, the nappe may not begin to ventilate until flows of 1600 cfs or less.

With the decreased orifice height, we noticed that for flows as low as 250 cfs, there is already some interference with the beam and the flow must be treated as orifice flow, though it may not behave truly as orifice flow until slightly higher flow rates. This effect is even more pronounced at higher flows when a region of combined orifice and weir flow is encountered. There is a transitional regime between approximately 750–850 cfs where the flow may or may not behave as clearly orifice and weir flow. These effects were equally as apparent in previous runs with the beam in a different location.

Such areas of transition are found in many similar structures. A histeresis is often noted with rising or falling flows providing different flow regimes, and hence, slightly different head discharge relationships. This is often caused by the structural difference of the two pertinent flow equations; weir flow being proportional to $H^{3/2}$, while orifice flow is proportional to $H^{1/2}$. This leads to substantial changes in the stage discharge of the structure. Because of these phenomena, it was not possible to develop specific weir or orifice coefficients for the structure wth the beam installed at 0.68.

CONCLUSION

In conclusion, after an extended testing program, the placement of the flow retarding beam shown in Fig. 1 of the appendix at an elevation 0.68 ft, (approximately 8–1/4"), above the crest appears to closely represent the IDOT computed curve for flow retention. While cumulative studies on the scour downstream of the structure were not conducted, single storm events were tested with and without riprap and the basin appears to have minimal short term erosion with the riprap in place. Qualitative observations have indicated that the stilling basin blocks were only minimally effective at higher flow rates. A video tape supplement to this report provides further insight into the functioning of the structure under several flow conditions. A supplemental report regarding the effects of tailwater and subsequent submergence of the structure is part of a supplement "A" to this report.

REFERENCE

Hydraulic Laboratory Techniques, A Water Resources Technical Publication, U.S. Department of the Interior, Water and Power Resources Service, Denver, Colorado, 1980.

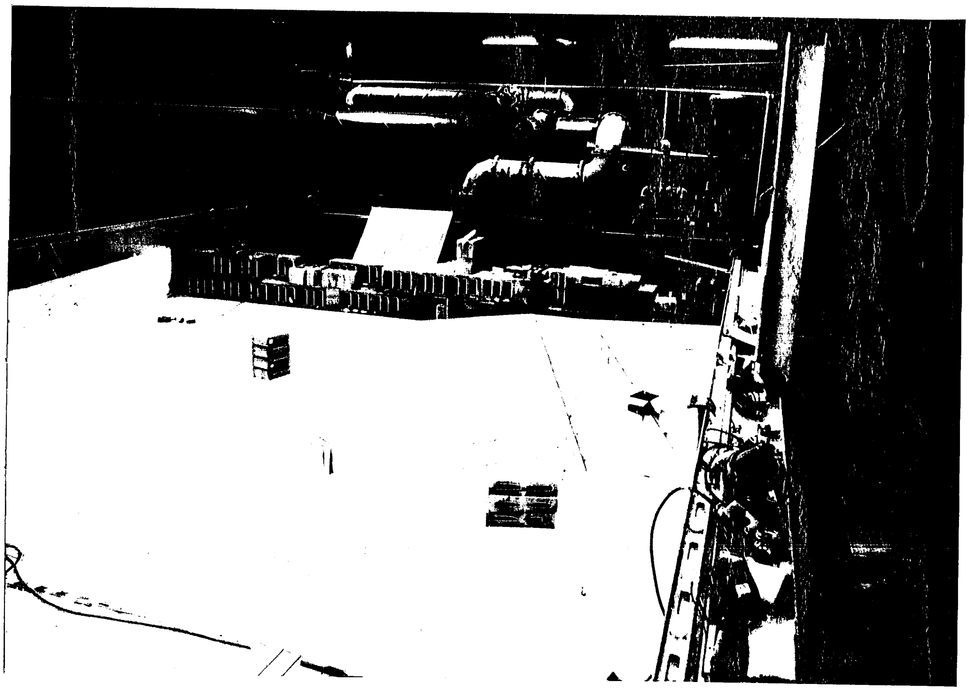

Photo 1. Photo showing typical model construction used.

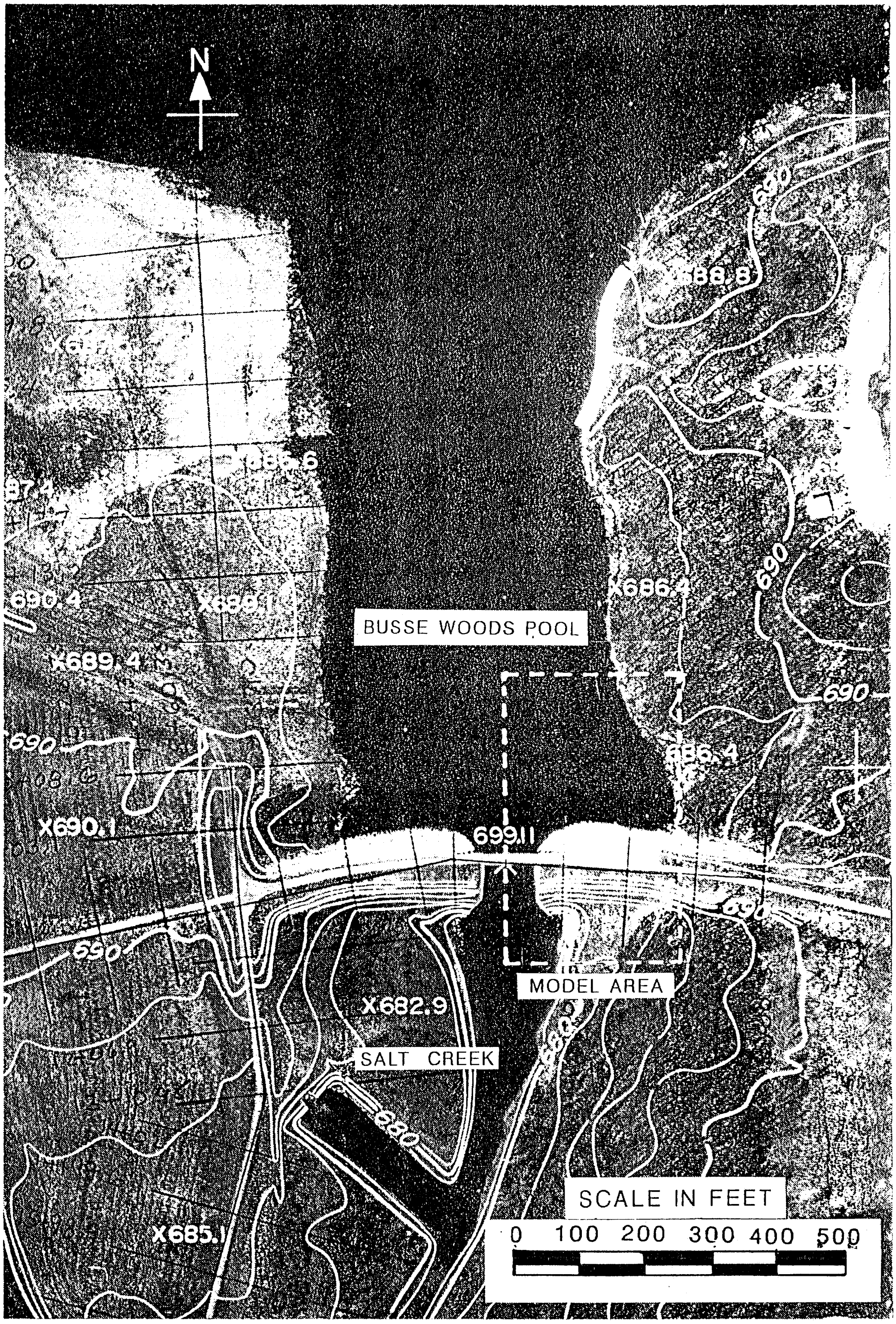

Photo 2. Aerial photo of Busse Woods Reservoir Drop Structure and vicinity (with the extent of the SAFHL Modeling area indicated).

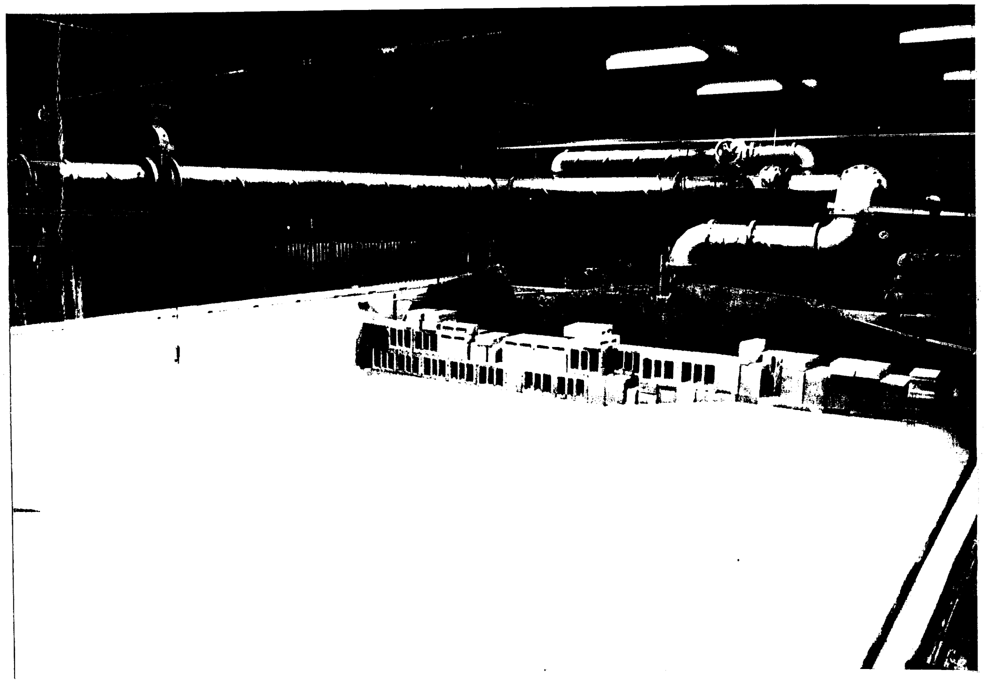

Photo 3. View of upstream portion of model.

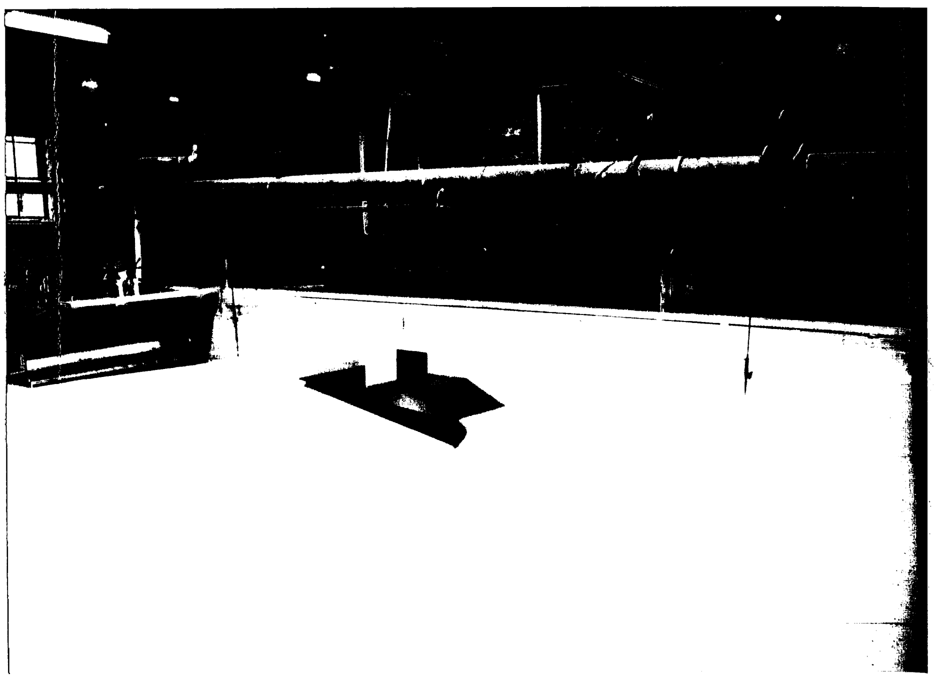

Photo 4. View showing drop structure prior to installation of rockface.

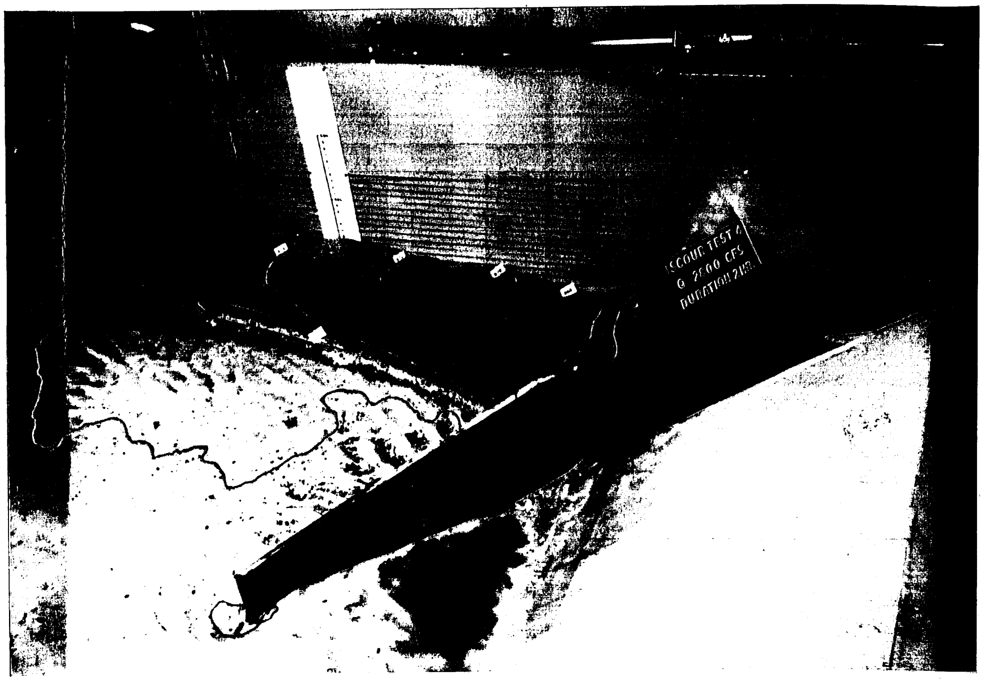

Photo 7. Sediment erosion test No. 4, Q = 2800 cfs. Duration 2 Hrs with beam, without riprap, 2 ft contour intervals.

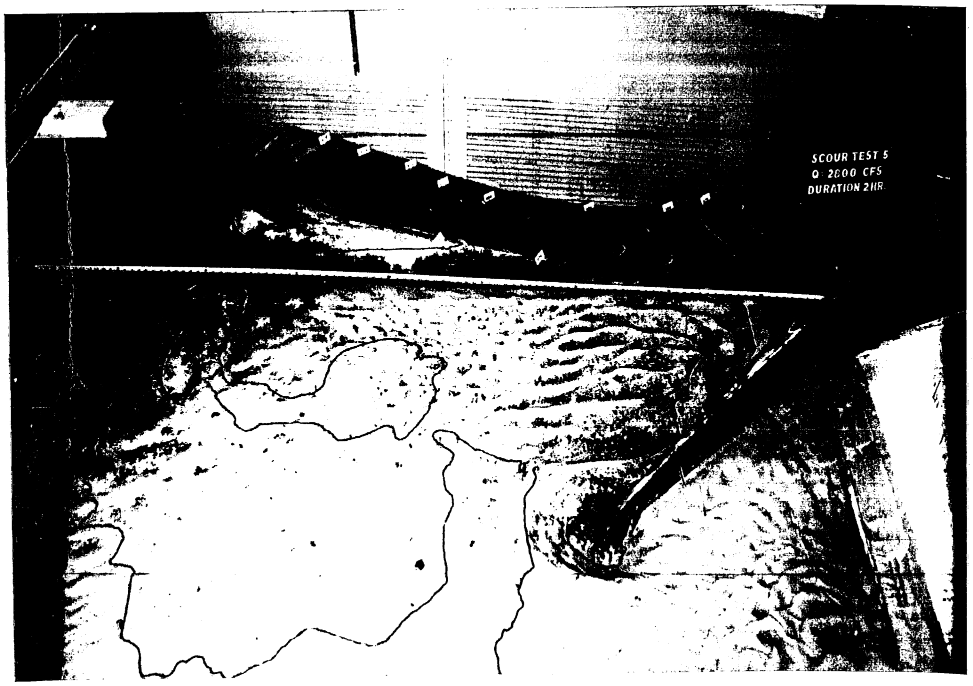

Photo 8. Sediment erosion test No. 5, Q = 2810 cfs. Duration 2 Hrs without beam, without riprap, 2 ft contour intervals.

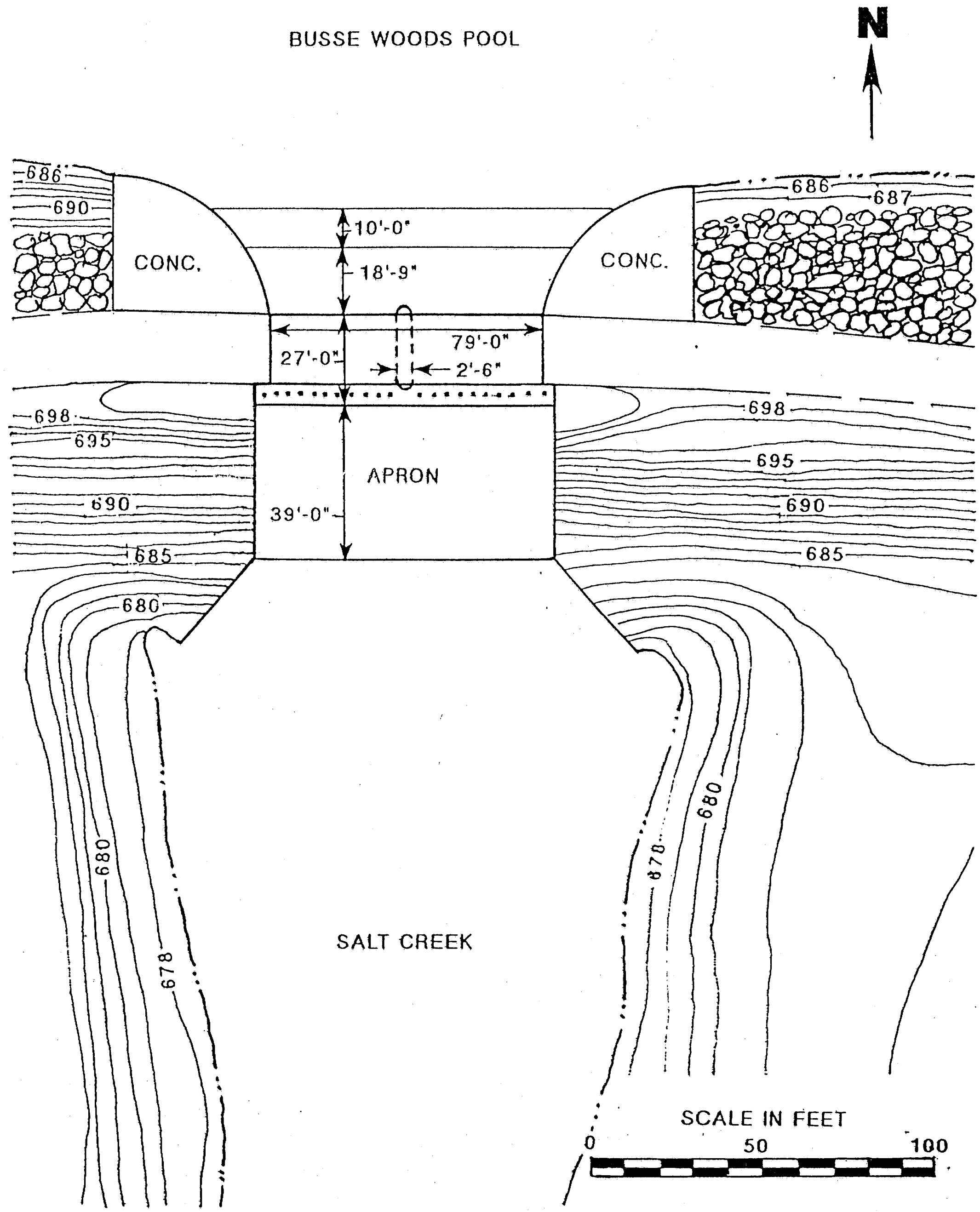

Figure 1. Detail sketch of Busse Woods Reservoir Drop Structure and Vicinity

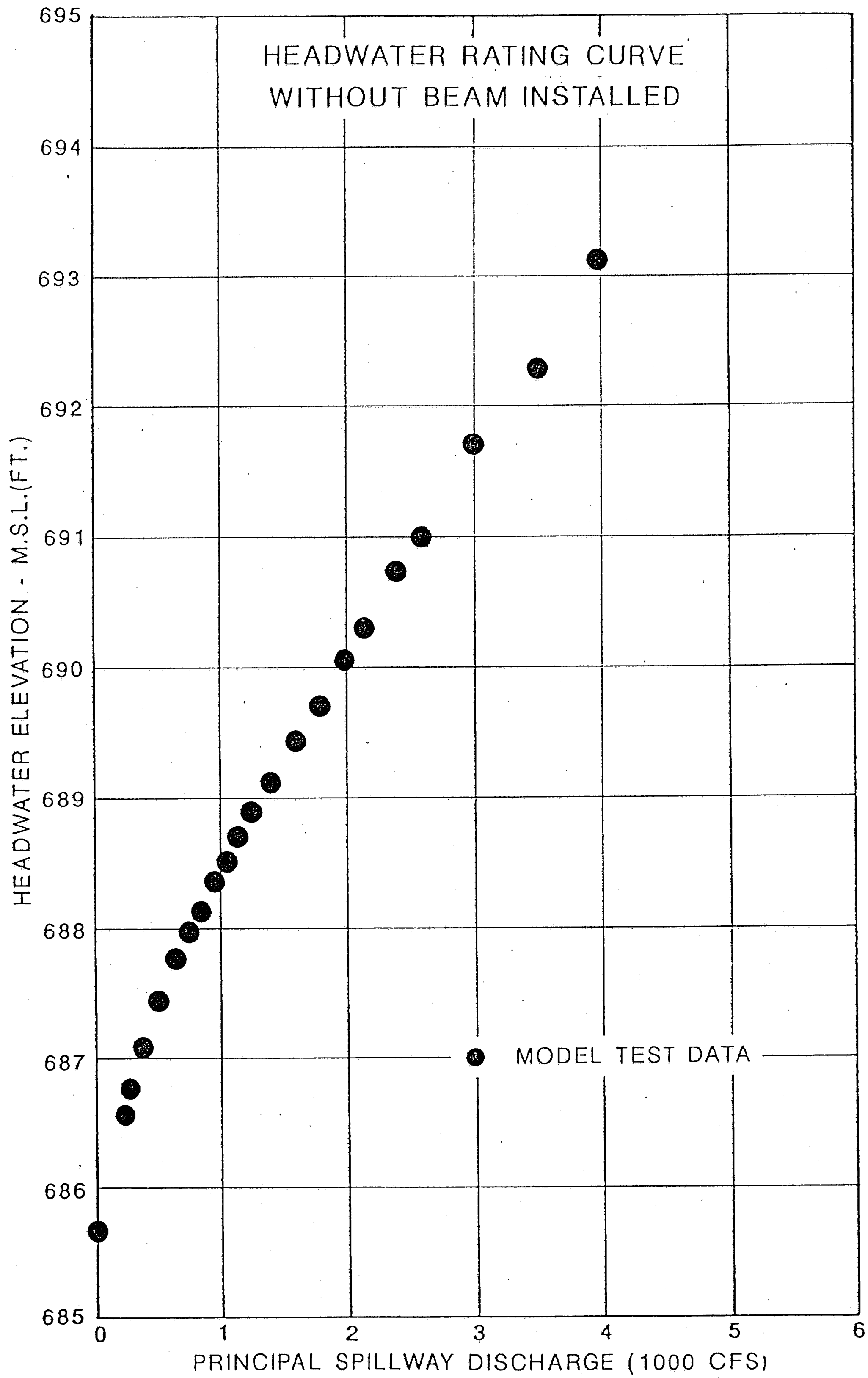

Figure 2. Model generated headwater rating curve without beam installed.

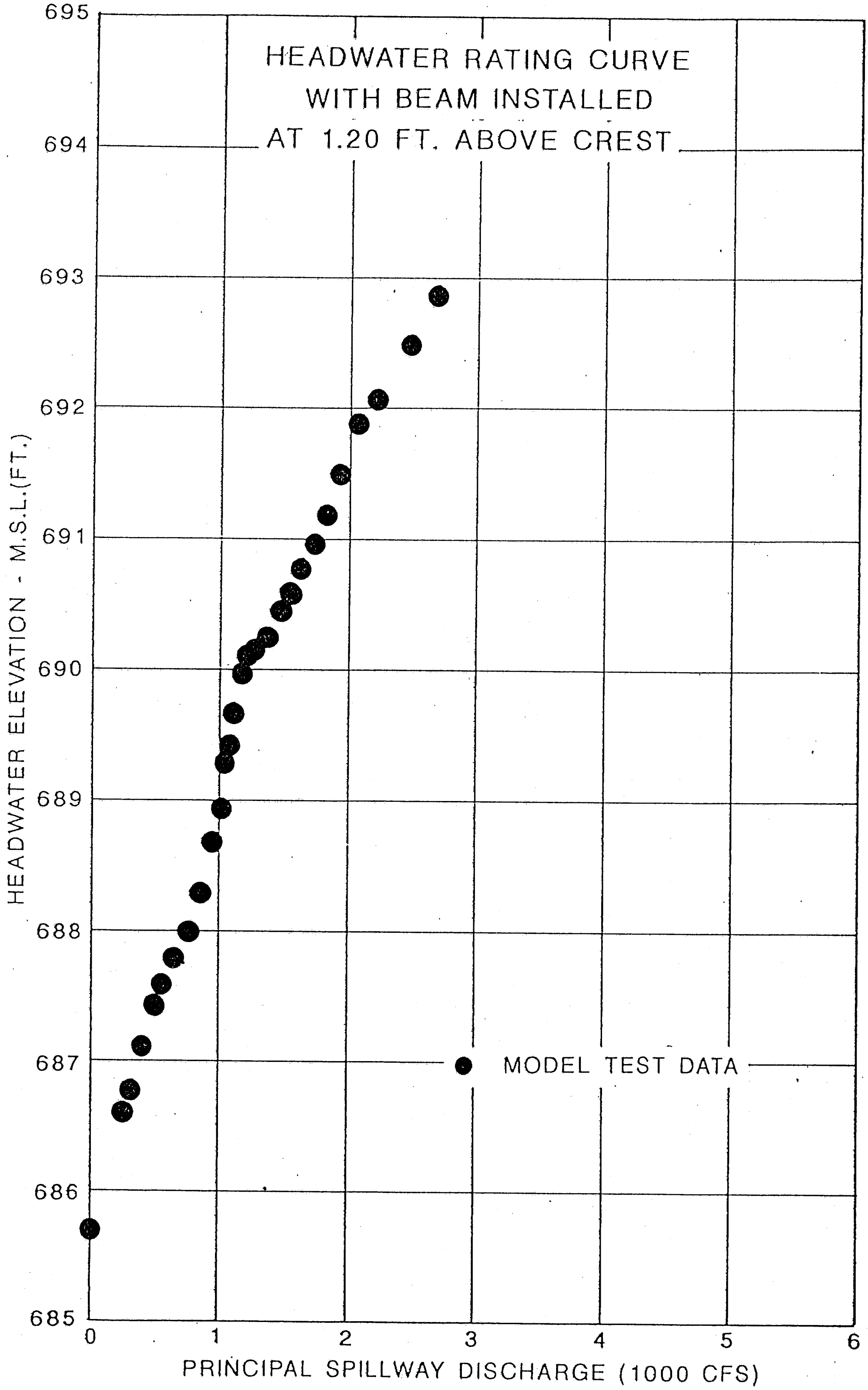

Figure 3. Model generated headwater rating curve with beam installed 1.2 ft above spillway crest.

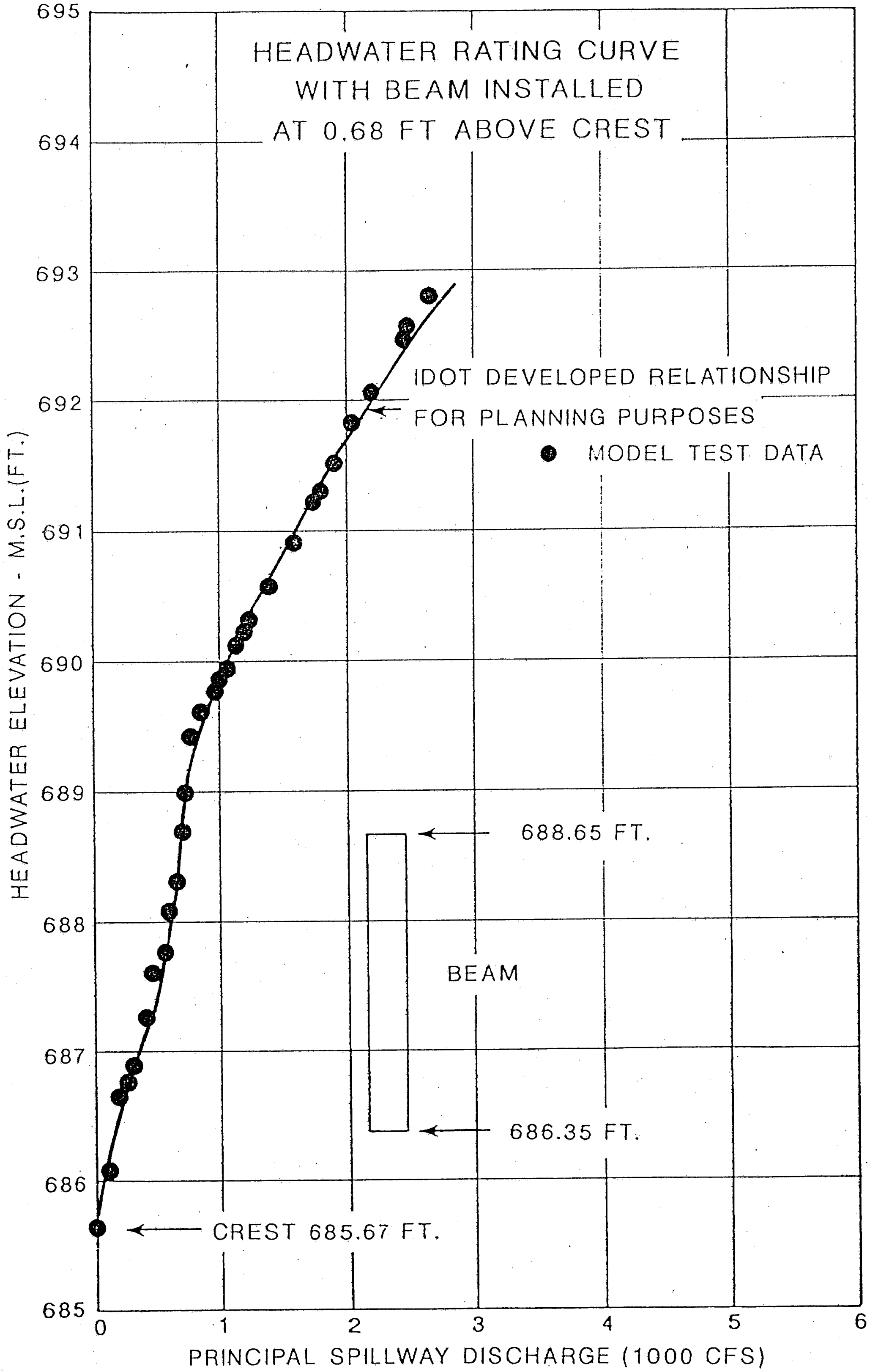

Figure 4. Comparison of headwater rating curves for IDOT recommended project stage–discharge relationship and SAFHL Model generated project stage–discharge relationship.

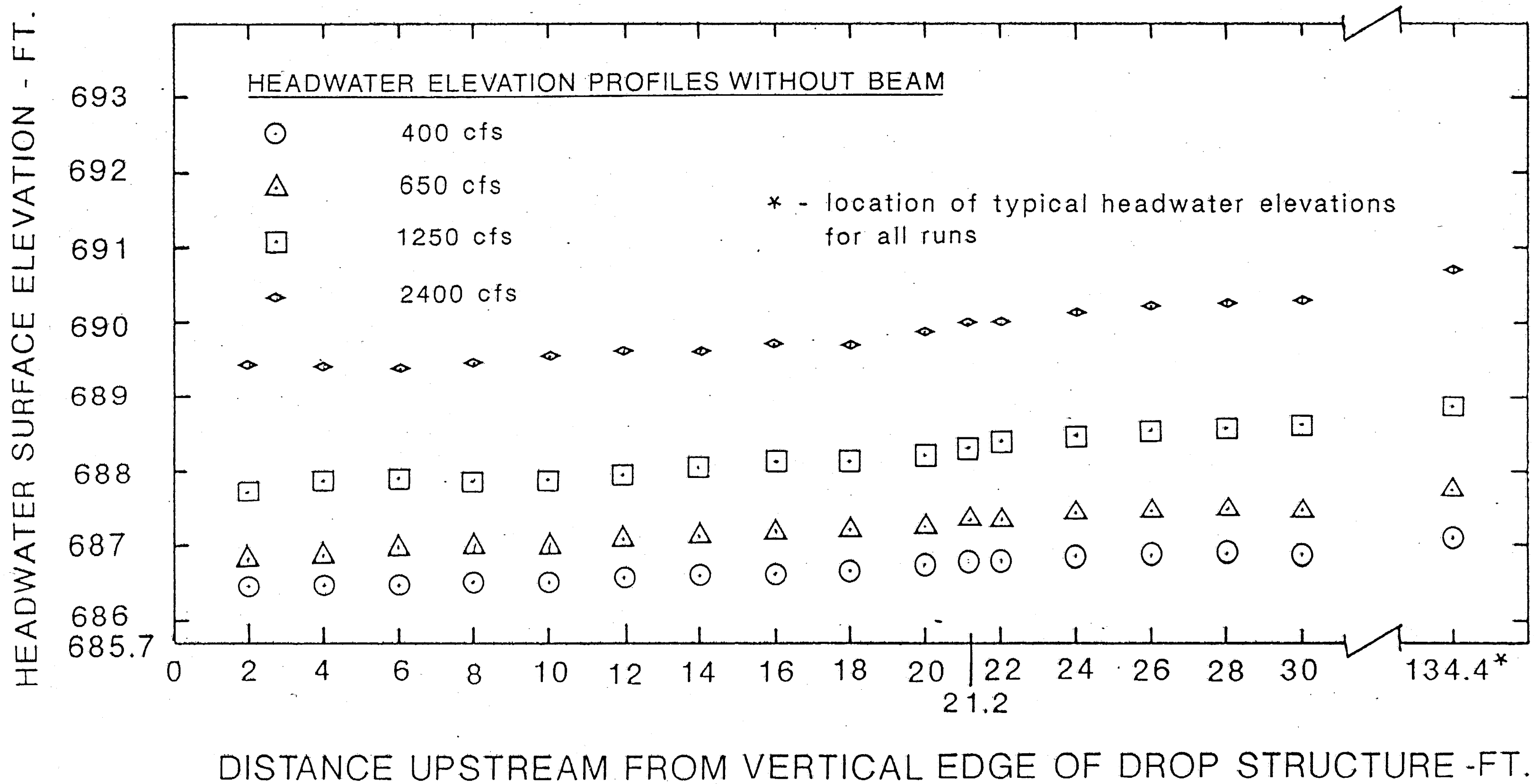

Figure 5. Model generated water surface profile with distance upstream from vertical edge of drop structure – without beam.

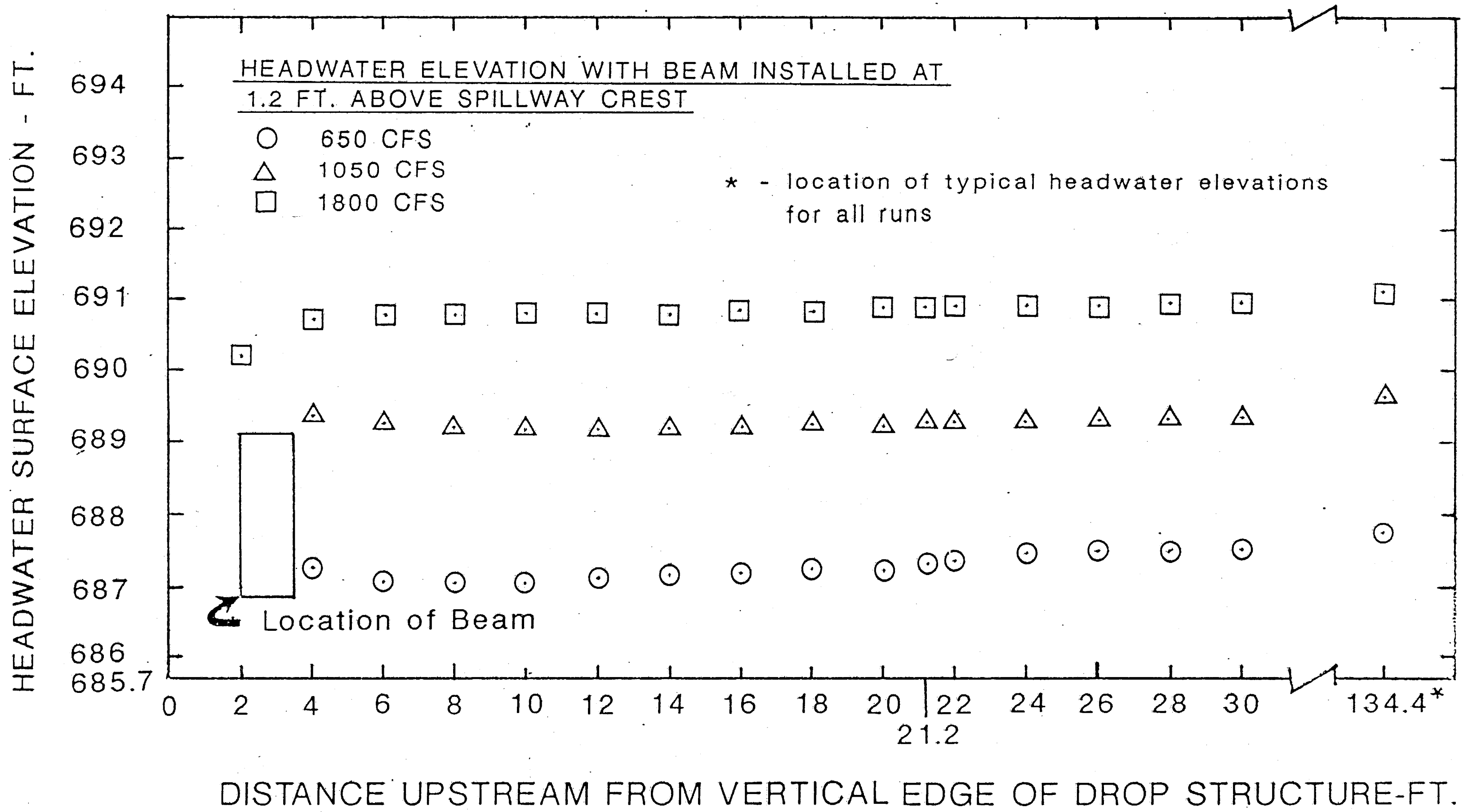

Figure 6. Model generated water surface profile with distance upstream from vertical edge of drop structure – with beam installed 1.2 ft above crest.

APPENDIX

TABLE A–1.
Properties of Materials Downstream from
Busse Woods Principal Spillway

100% pass the #4 sieve
100% pass the #10 sieve
95–100% pass the #40 sieve
80–100% pass the #200 sieve
Liquid limit ranges from 30 to 50
P.I. ranges from 15 to 30

TABLE A–2
Model Generated Test Data Without Beam Installed (Total Weir Flow)

Q (cfs)	H.W. El. (ft)	T.W. El (ft)	Comment
0	685.67	674.9	No flow
250	686.56	681.80	Total weir flow
300	686.76	682.20	Total nappe ventilation
400	687.09	862.80	"
505	687.42	683.33	"
650	687.73	684.00	"
760	687.97	684.37	"
850	688.12	684.70	No nappe ventilation
950	688.34	685.00	"
1050	688.51	685.32	"
1150	688.70	685.60	"
1250	688.88	685.90	"
1400	689.12	686.25	"
1600	689.43	686.62	"
1800	689.70	687.00	"
1990	690.08	687.30	"
2150	690.30	687.55	"
2400	690.73	687.87	"
2600	691.00	688.12	"
3000	691.71	688.95	"
3400	692.29	689.65	"
4000	693.13	690.26	"

Note: H.W. Elevation taken 134.4 ft upstream of structure.

TABLE A–3
Model Generated Test data with beam installed 1.2 ft above spillway crest

Q (cfs)	H.W. El. (ft)	T.W. El. (ft)	Comment
0	685.67	674.9	No flow
250	686.56	681.80	Weir flow
300	686.76	682.20	"
400	687.09	682.80	"
500	687.40	683.33	"
550	687.55	683.50	"
650	687.74	684.00	Orifice flow
750	687.98	684.37	"
850	688.27	684.70	"
950	688.66	685.00	"
1010	688.90	685.20	"
1030	689.26	685.27	"
1050	689.41	685.32	Transition to orifice and weir flow
1100	689.63	685.48	Clearly orifice and weir flow
1150	689.95	685.60	"
1200	690.10	685.75	"
1250	690.12	685.90	"
1350	690.22	686.13	"
1450	690.42	686.35	"
1520	690.57	686.50	"
1600	690.75	687.15	"
1710	690.94	687.70	"
1800	691.17	688.07	"
1900	691.47	688.40	"
2050	691.84	688.85	"
2200	692.05	689.38	"
2450	692.47	689.70	"
2680	692.82	690.20	"

Note: H.W. Elevation taken 134.4 ft upstream of structure.

TABLE A-4

Model Generated Test data with beam installed 0.68 ft above spillway crest

Q (cfs)	H.W. El. (ft)	T.W. El. (ft)	Comment
0	685.67	674.9	No flow
100*	686.11	680.30	Weir flow
200*	686.65	681.45	"
250	686.77	681.80	Orifice flow
300	686.91	682.20	"
410	687.29	682.80	"
500	687.62	683.33	"
550	687.79	683.50	"
600	688.11	683.75	"
650	688.34	684.00	"
700	688.71	684.15	"
750	688.99	684.37	"
800	689.46	684.55	Transition to orifice and weir flow
850	689.61	684.70	Clearly orifice & weir flow
950	689.79	685.00	"
1000	689.90	685.20	"
1050	689.97	685.32	Air cavity fluttering along D.S. side of beam
1150	690.16	685.60	
1200	690.28	685.75	"
1250	690.36	685.90	"
1400	690.63	686.25	"
1600	690.96	687.15	"
1750	691.24	687.85	"
1800	691.33	688.07	Air cavity has disappeared
1900	691.56	688.40	"
2050	691.86	688.85	"
2200	692.10	689.38	"
2450	692.49	689.70	"
2500	692.58	689.85	"
2680	692.85	690.20	"

*Very small manometer deflection reducing accuracy of these points.

Note: H.W. Elevation taken 134.4 ft upstream of structure.

Table A–5

RECOMMENDED PROJECT STAGE DISCHARGE RELATIONSHIP
Developed by the Illinois Department of Transportation
For Planning Purposes

Q (cfs)	H.W. El. (ft)
0	685.67
21	685.87
60	686.07
111	686.27
171	686.47
239	686.67
314	686.87
422	687.17
526	687.67
613	688.17
689	688.67
758	689.17
904	689.67
1118	690.17
1372	690.67
1483	690.87
1660	691.17
1978	691.67
2321	692.17
2689	692.67

TABLE A–6
Model Generated Water Surface Profile Test Data Without Beam Installed

Distance upstream from vertical edge of drop structure (ft)	Q=400 cfs headwater elevation	Q=650 cfs headwater elevation	Q=1250 cfs headwater elevation	Q=2400 cfs headwater elevation
2	686.47	686.81	687.69	689.40
4	686.49	686.88	687.85	689.39
6	686.49	686.97	687.90	689.38
8	686.53	686.97	687.85	689.45
10	686.53	686.97	687.88	689.55
12	686.58	687.05	687.95	689.61
14	686.61	687.09	688.05	689.63
16	686.63	687.15	688.12	689.68
18	686.68	687.18	688.13	689.71
20	686.73	687.22	688.20	689.87
21.2	686.77	687.30	688.30	689.99
22	686.79	687.33	688.37	690.01
24	686.85	687.40	688.45	690.13
26	686.86	687.43	688.53	690.21
28	686.86	687.45	688.55	690.28
30	686.87	687.45	688.59	690.31
134.4	687.09	687.74	688.84	690.69
Tailwater Elevation	682.08	684.00	685.90	687.90
Comment	nappe ventilated	nappe ventilated	no nappe ventilation	submerged

TABLE A-7
Model Generated Water Surface Profile Data With Beam Installed 1.2 ft Above Spillway Crest

Distance Upstream from vertical edge of drop structure (ft)	Q=650 cfs headwater elevation	Q=1050 cfs headwater elevation	Q=1800 cfs headwater elevation
2	X	X	690.18
4	687.23	689.35	690.69
6	687.06	689.26	690.77
8	687.05	689.14	690.77
10	687.06	689.16	690.76
12	687.11	689.17	690.77
14	687.16	689.19	690.78
16	687.18	689.19	690.18
18	687.23	689.22	690.82
20	687.23	689.22	690.84
21.2	687.33	689.26	690.87
22	687.35	689.27	690.87
24	687.45	689.29	690.91
26	687.47	689.31	690.91
28	687.49	689.32	690.94
30	687.49	689.33	690.96
134.4	687.74	689.61	691.20
Tailwater Elevation	684.00	685.32	688.07
Comment	Transition to orifice flow	Transition to orifice and weir flow	Clearly orifice & weir flow

X = Location of beam – no reading possible.

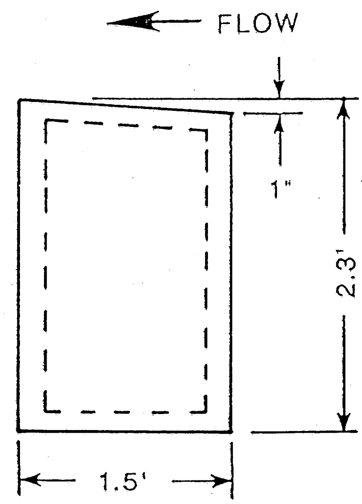

Figure A–1 Dimensions of proposed flow retarding beam.

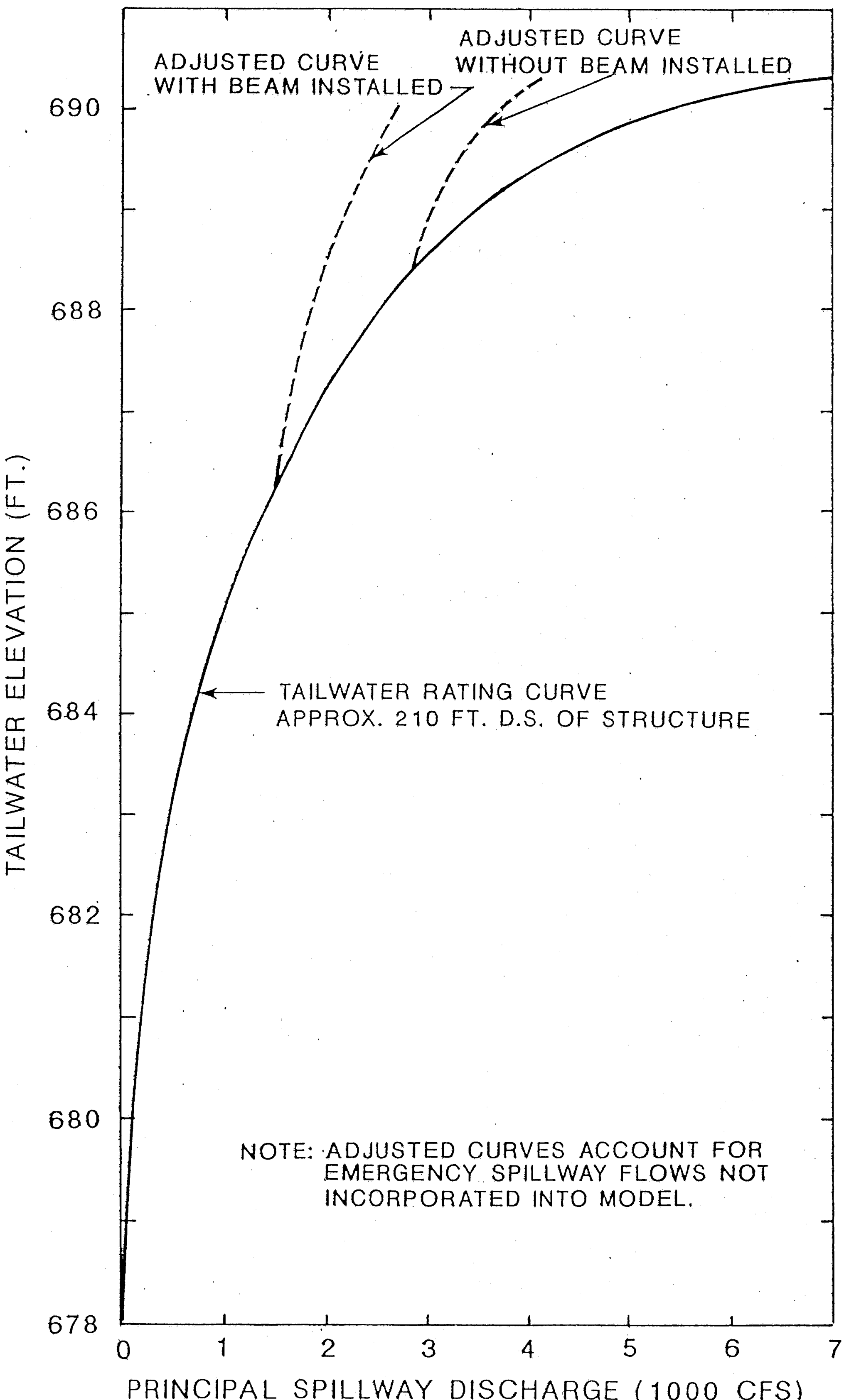

Figure A–2 Tailwater rating curve supplied by DuPage County Dept. of Environmental Concerns. (Note: Tailwater elevation vs. total discharge or principal spillway discharge for the curves labeled adjusted.)

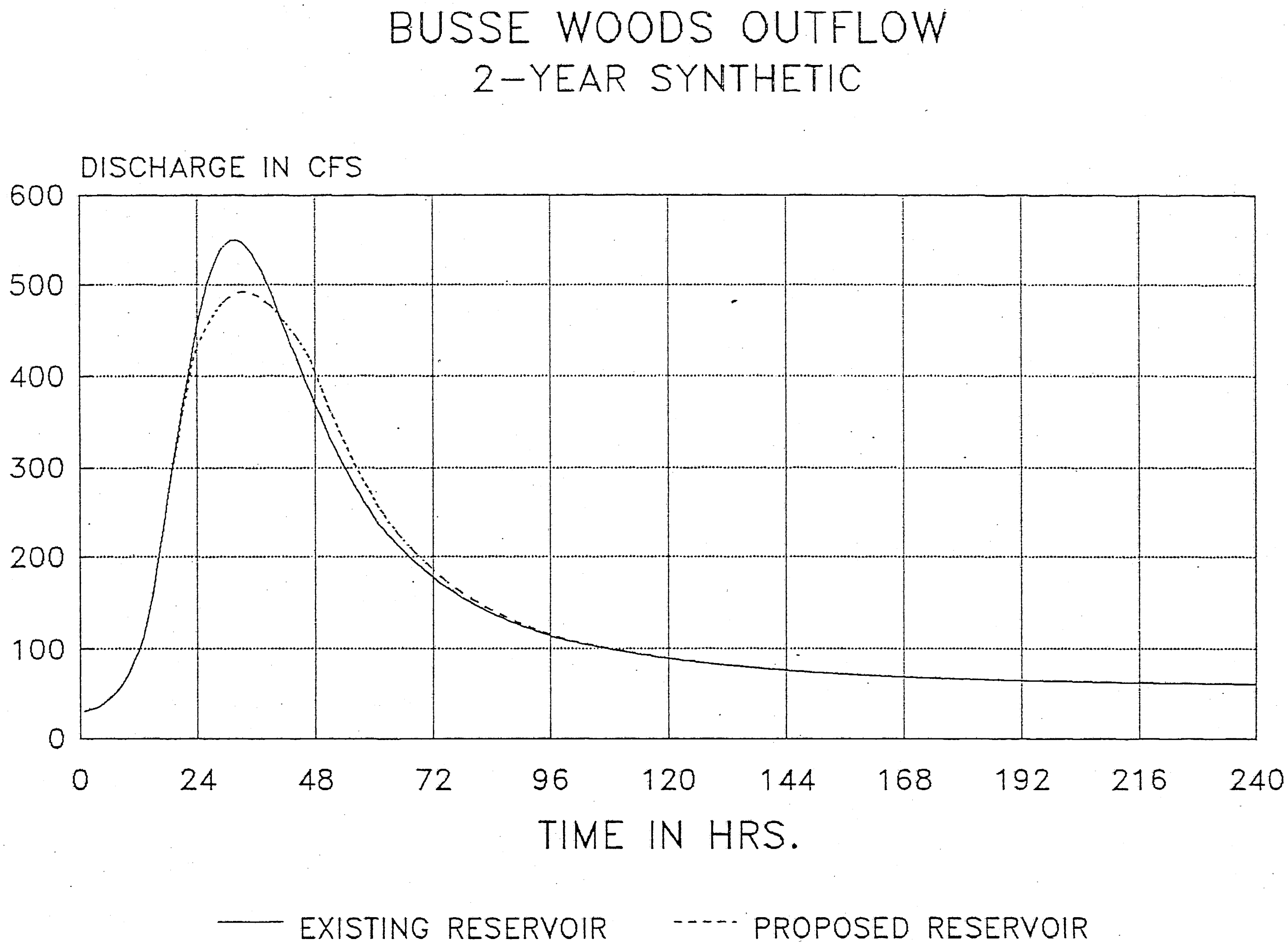

Figure A–3 Busse Woods outflow rate – 2 year hydrograph.

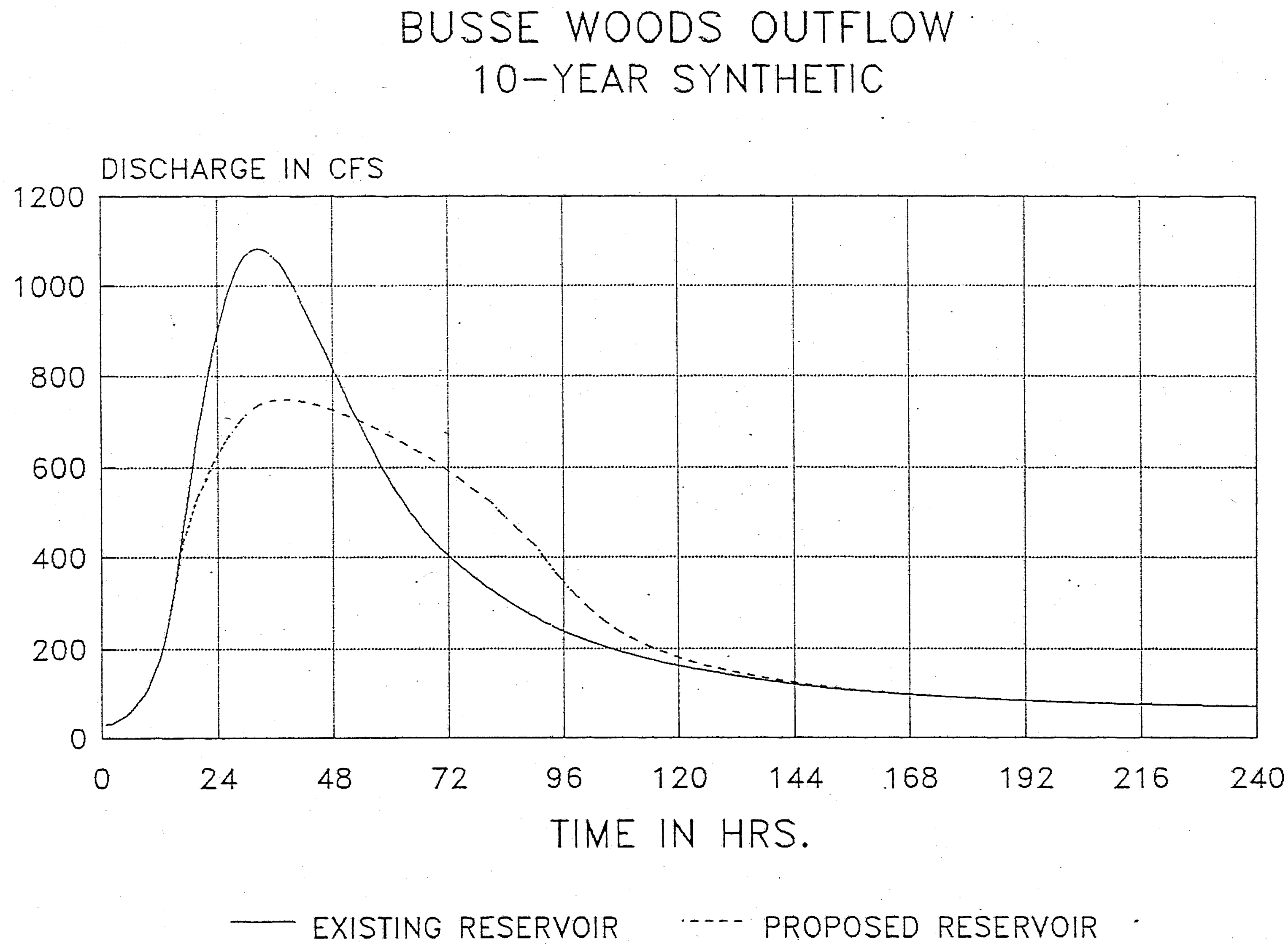

Figure A–4 Busse Woods outflow rate – 10 year hydrograph.

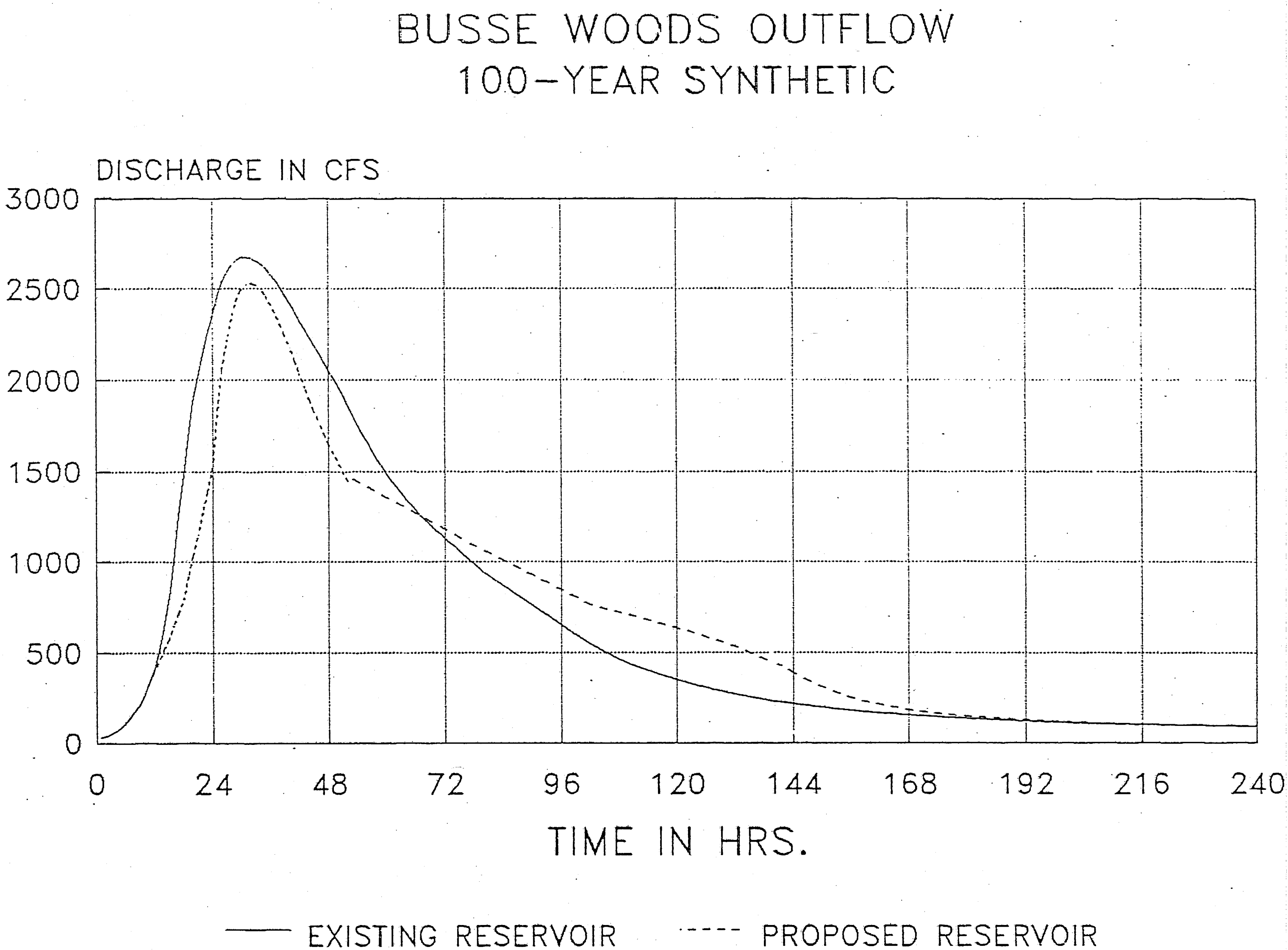

Figure A–5 Busse Woods outflow rate – 100 year hydrograph.