UNITED STATES
DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY

STAGE-DISCHARGE RELATIONS AT DAMS ON THE ILLINOIS AND DES PLAINES RIVERS IN ILLINOIS

CHAMPAIGN, ILLINOIS

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By Dean M. Mades

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SYMBOLS AND UNITS

Symbol		Unit
A_n	Net cross-sectional area of butterfly valve or headgate	ft^2
B	Tainter gate width	ft
C	Tainter gate free orifice flow coefficient of discharge	
C_{fo}	Headgate or valve free orifice flow coefficient of discharge	
C_{gs}	Tainter gate submerged orifice flow coefficient of discharge	
C_{so}	Headgate or valve submerged orifice flow coefficient of discharge	
C_{sw}	Sharp crested weir flow coefficient of discharge	
C_w	Tainter gate free weir flow coefficient of discharge	
C_{wk}	Wicket-flow coefficient of discharge	
C_{ws}	Tainter gate submerged weir flow coefficient of discharge	
E	Water surface elevation	ft
g	Acceleration due to gravity (32.2)	ft/s^2
h_o	Static headwater depth above center of headgate or valve opening	ft
h_c	Static headwater depth above top of raised wicket or closed tainter gate	ft
HW	Water surface elevation of upstream pool at the dam	ft
h_g	Vertical height of tainter gate opening	ft
h_1	Static headwater depth above ogee spillway crest or wicket sill	ft
h_3	Static tailwater depth above ogee spillway crest or wicket sill	ft
L	Width of opening between wickets	ft
P	Number of needles in place	
Q	Discharge	ft^3/s
TW	Water surface elevation of downstream pool at the dam	ft
W	Total width of raised wickets or closed tainter gates	ft
WD	Number of lowered wickets	
Δh	Static head differential $(h_1 - h_3)$	ft
β	Angle a raised wicket forms from vertical	degree

CONVERSION FACTORS

The following factors may be used to convert the inch-pound units published herein to the International System of Metric Units (SI).

Multiply inch-pound unit	By	To obtain SI unit
inch (in)	25.4	millimeter (mm)
foot (ft)	.3048	meter (m)
mile (mi)	1.609	kilometer (km)
square foot (ft^2)	.0929	square meter (m^2)
square mile (mi^2)	2.590	square kilometer (km^2)
cubic foot (ft^3)	.02832	cubic meter (m^3)

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ABSTRACT

Stage-discharge relations were developed for the Brandon Road Dam on the Des Plaines River and the Dresden Island, Marseilles, Starved Rock, Peoria, and La Grange Dams on the Illinois River. At Brandon Road Dam, streamflow is regulated by the operation of tainter gates and headgates. Tainter gates are operated to regulate streamflow at the Dresden Island, Marseilles, and Starved Rock Dams. Peoria Dam and La Grange Dam comprise timber-Chanoine wickets, which are lowered to a horizontal position on the streambed when not used for streamflow regulation. Both dams have concrete abutments housing butterfly valves that are also used for regulation.

Discharge coefficients, in equations that express discharge as a function of tailwater depth, headwater depth, and vertical height of gate opening, were determined for conditions of free-orifice, submerged-orifice, and free-weir flow under a tainter gate. A free-orifice flow coefficient was determined for the headgates at Brandon Road Dam. Stage-discharge relations for flow over sections of lowered wickets and flow between raised wickets had been developed from hydraulic model studies by the U.S. Army Corps of Engineers in 1937 and 1938. These relations were verified by discharge measurements at U.S. Geological Survey stream-gaging stations on the Illinois River near the Peoria and La Grange Dams. Discharge coefficients in equations of free-orifice and submerged-orifice flow through a butterfly valve were computed from additional measurements at the stream-gaging stations.

A total of 50 measurements of discharge that ranged from 1,730 to 86,400 cubic feet per second were used to develop stage-discharge relations at the six dams. The computed relations compared favorably with published hydraulic design criteria. The stage-discharge relations derived from tainter gate control are applicable to all conditions except extremely high streamflow conditions when submerged weir flow exists. The stage-discharge relations for wicket control are applicable to flow over no more than 20 lowered wickets. The stage-discharge relations for butterfly valve and headgate control are applicable to all streamflow conditions.

INTRODUCTION

A cooperative effort between the U.S. Geological Survey and the U.S. Army Corps of Engineers, Chicago District (referred to as the Corps in this report), was begun in 1977 to establish stage-discharge relations at six dams on the Illinois and Des Plaines Rivers. The objective of this study was to verify or adjust theoretical stage-discharge relations that were developed when the dams were constructed. Ratings were to be developed and verified by field measurements at those dams lacking a stage-discharge relation. The ratings are needed to ensure that release requirements into the river below each dam are being met. The number of stream-gaging stations along the rivers are insufficient to monitor releases at each dam accurately.

The stage-fall-discharge rating at several stations is dependent on the slope of the water surface. As the water-surface slope between control structures on the rivers can approach zero during low flow, traditional methods of determining discharge that require slope are unsatisfactory. The equations for determining discharge documented in this report are independent of water-surface slope yet dependent on the operational variables at the dam. The equations were developed for the assumed condition of steady, uniform flow during the computation interval.

Purpose and Scope

The purpose of this report is to present the stage-discharge relations at six dams on the Illinois and Des Plaines Rivers in Illinois. The stage-discharge relations, referred to as ratings, are presented in tabular form for discrete combinations of headwater and tailwater elevations. Methods are described for calculating discharge during control conditions not represented by the conditions presented in the rating tables.

At several dams, headgates and sluice gates are not used for streamflow regulation. The report is limited in scope to the analysis of controls in use at each dam at present. No attempt was made to define the stage-discharge relation at the wicket dams when most or all wickets are lowered because with this configuration the structures no longer control streamflow.

Acknowledgement

The U.S. Army Corps of Engineers, Chicago District, provided some of the data used in this report. Discharge measurements were made for a variety of operating conditions which the Corps lockmasters readily arranged. Their cooperation was greatly appreciated.

DESCRIPTION OF THE STUDY AREA

The study area consists of five dams on the Illinois River and one dam on the Des Plaines River 13.3 river miles above its confluence with the Illinois River at river mile 273.0 (fig. 1). The drainage area and river-mile location of each dam are presented in table 1.

Table 1.–River-mile location and drainage area of stream-gaging stations and dams

Station No.	Station	River mile	Drainage area (mi^2)
05527500	Kankakee River near Wilmington	5.5	5,150
05532500	Des Plaines River at Riverside	44.3	630
–	Des Plaines River at Brandon Road Dam	13.3	1,506
–	Illinois River at Dresden Island Dam	271.5	7,279
–	Illinois River at Marseilles Dam	247.0	8,259
05543500	Illinois River at Marseilles	246.5	8,259
05552500	Fox River at Dayton	5.6	2,642
–	Illinois River at Starved Rock Dam	231.0	11,071
05555300	Vermilion River near Leonore	16.7	1,251
–	Illinois River at Peoria Dam	157.8	14,550
05567500	Mackinaw River near Congerville	58.7	767
05568500	Illinois River at Kingston Mines	144.4	15,819
05583000	Sangamon River near Oakford	25.7	5,094
05585000	La Moine River at Ripley	12.3	1,293
–	Illinois River at La Grange Dam	80.2	25,648
05585500	Illinois River at Meredosia	71.3	26,028

The U.S. Geological Survey (hereafter "the Survey") maintains several stream-gaging stations on the Illinois and Des Plaines Rivers and on several large tributaries to these rivers (fig. 1). The drainage area and river-mile location of these stations are presented in table 1.

The primary purpose of flow regulation by the dams is to provide sufficient upstream pool depth for a maximum barge draft of 9.0 feet. The Brandon Road, Dresden Island, Marseilles, and Starved Rock Dams are nonnavigable, concrete gravity dams. Various combinations of tainter gates and vertical lift gates are used to control headwater elevations at these dams. The Peoria and La Grange Dams are navigable, Chanoine wicket dams with adjoining concrete abutments that contain butterfly valves. Various combinations of wickets and valves are used to regulate the upstream pools at these structures.

Nonnavigable Dams

The nonnavigable dams are concrete structures with overflow and nonoverflow sections. Tainter gates are used to regulate the release of water between the ogee crest of the overflow section and the bottom of the gate (fig. 2). The gate may also be raised clear of the upstream water surface.

The nonoverflow section is a concrete structure with conduits that convey water from the upstream pool to downstream pool. Vertical lift slide gates, called headgates, are positioned across the entrance to each conduit. Headgates may be operated at different openings. A cross section of a typical headgate is illustrated in figure 3.

Navigable Dams

The Chanoine wickets at the Peoria and La Grange Dams are designed for lowering to permit unobstructed passage of high flows. Boat traffic is allowed to travel over the lowered wickets rather than through the navigation locks.

A Chanoine wicket consists of four oak timbers bolted together that total 3.75 feet in width and 1 foot in thickness. The wicket length may vary for different dams. A 3-inch space between adjacent wickets is provided to allow for lateral displacement (fig. 4).

A wicket is supported in an upright position at a 20-degree angle from vertical by an A-frame structural steel horse. The horse is pivoted at the wicket mid-length and at the concrete sill. A forged-steel prop pivoted at the upper end of the horse extends downstream, where it rests against a low, concrete step on the downstream portion of the sill. The hurter is a guide constructed so that as the top of the wicket is pulled upstream the prop slides away from its seat into a groove. The wicket may then be slowly lowered to a horizontal position on the sill.

A wicket is raised by attaching a hoist to the wicket bottom and raising the wicket and horse until the prop rests against the hurter seat. As tension on the wicket is released, the wicket rotates about the hinge at its midsection until its bottom rests against the sill step.

Various numbers of wickets are raised to maintain the upstream pool at or near the raised wicket crests. Long wooden planks called needles are placed on the upstream side of the spaces between raised wickets to provide low-flow regulation. Most or all of the wickets are lowered to pass high streamflows. Boat traffic is then allowed to travel over the lowered wickets rather than through the lock.

Flow at the wicket dams is also regulated by butterfly valves. The butterfly valve abutment at Peoria Dam contains a submerged concrete culvert on the right side of the stream channel, adjacent to the wickets. The abutment has a single inlet and six outlet ports (fig. 5). A butterfly valve is located in each outlet port. The ports are submerged at all times.

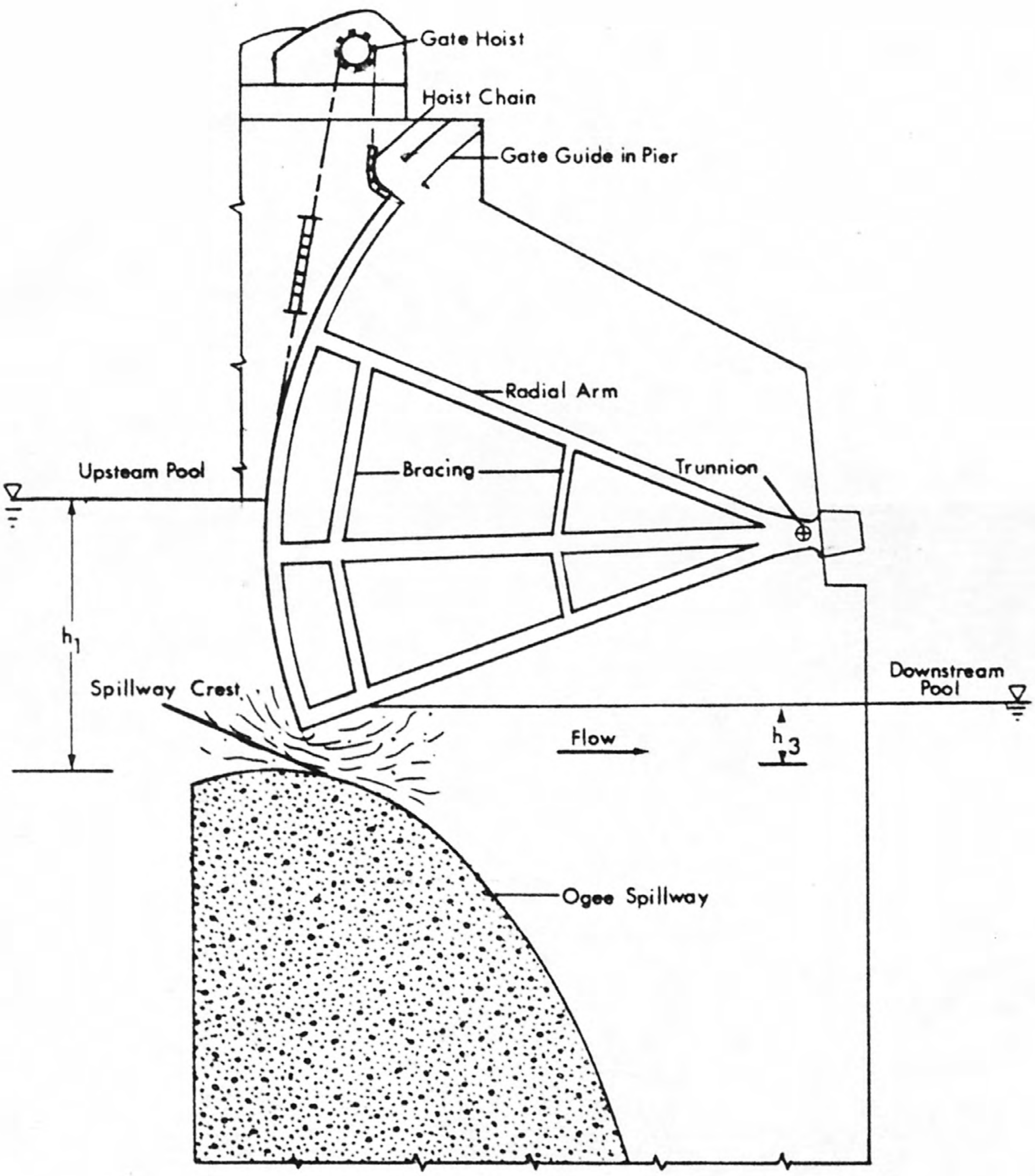
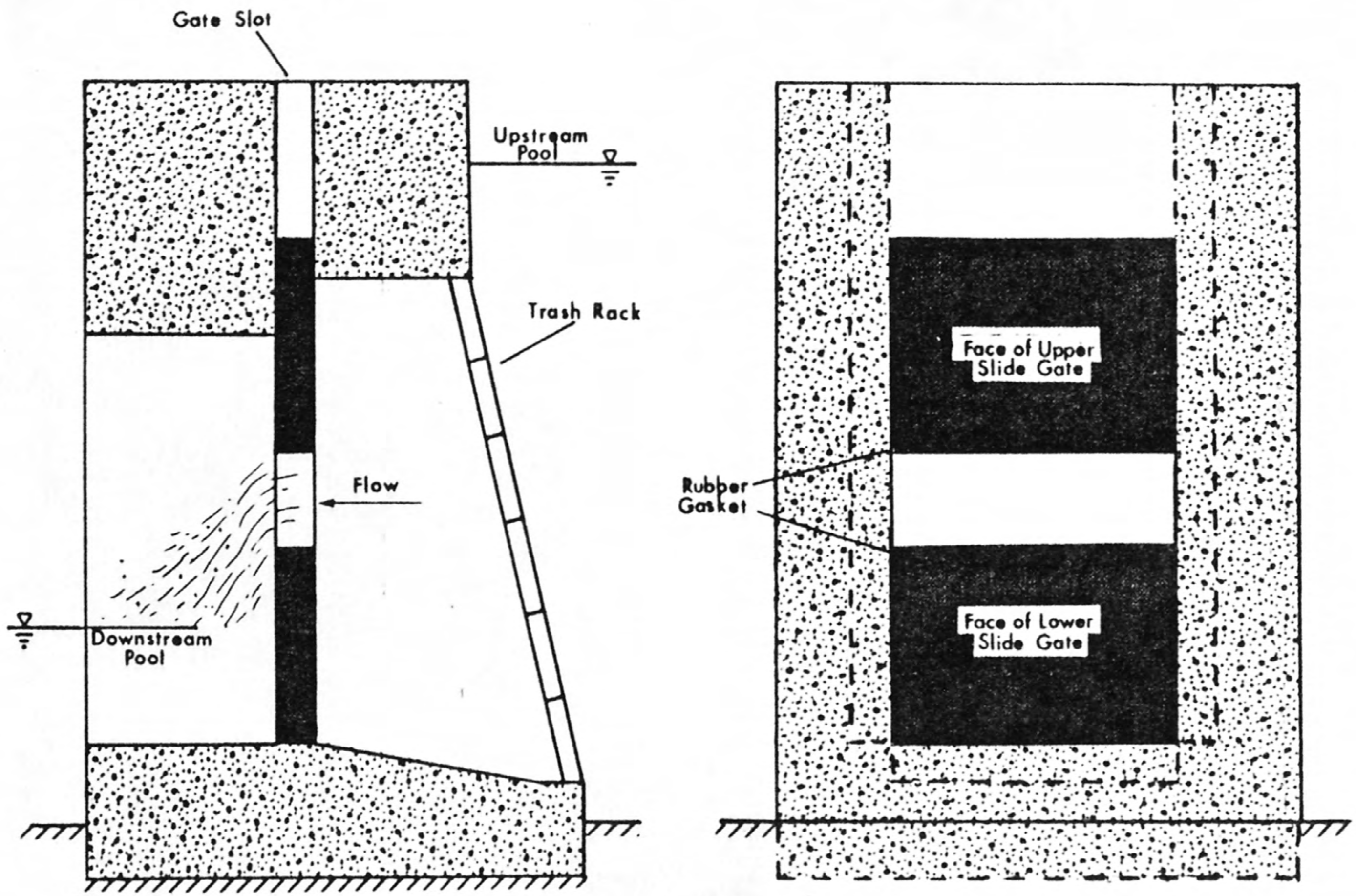


Figure 2.–Cross section showing construction of a typical tainter gate.



Cross Section of Headgate Abutment

Downstream Side of Headgate Abutment

Figure 3.–Diagrams showing construction of a typical headgate abutment.

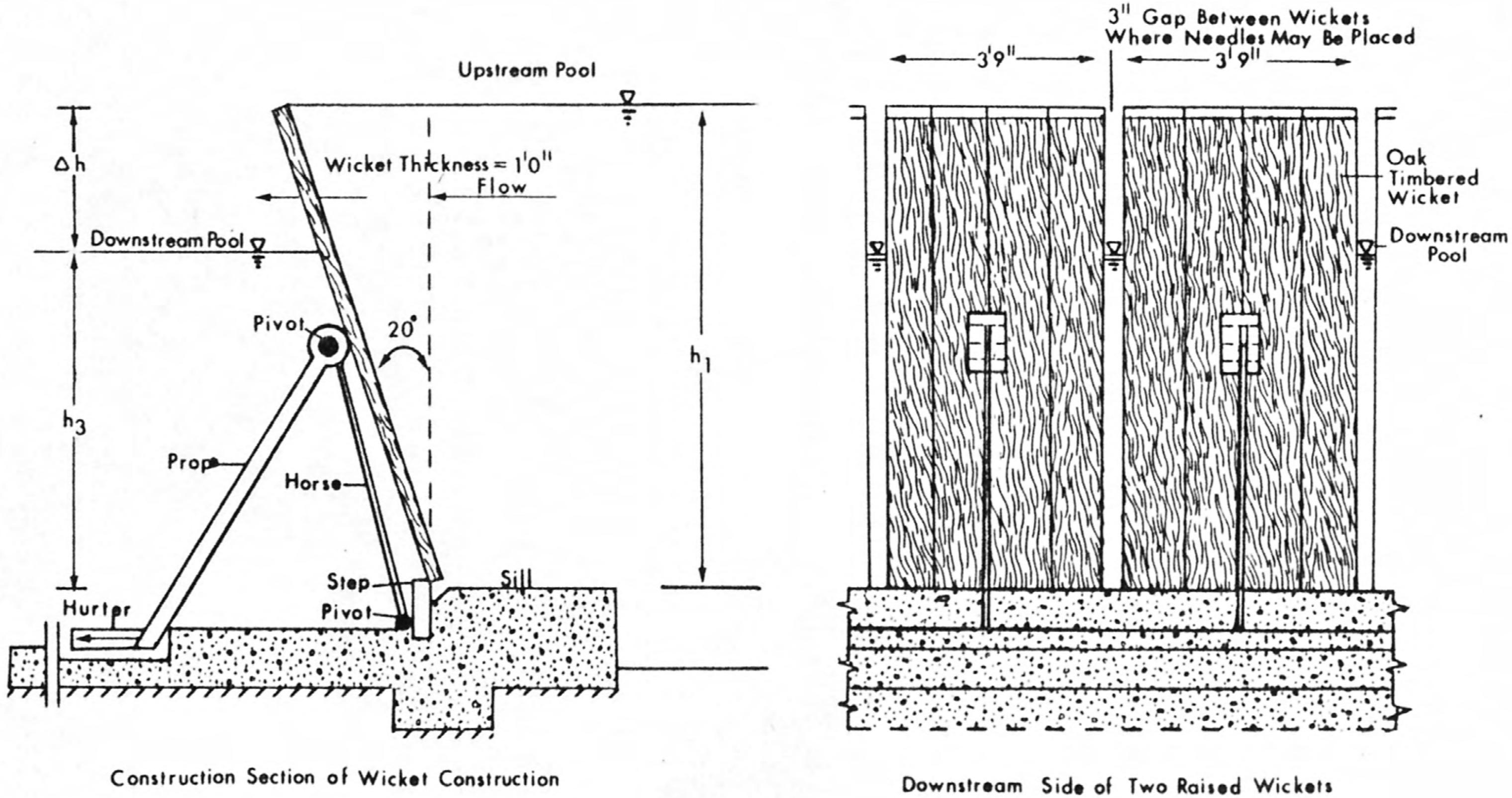
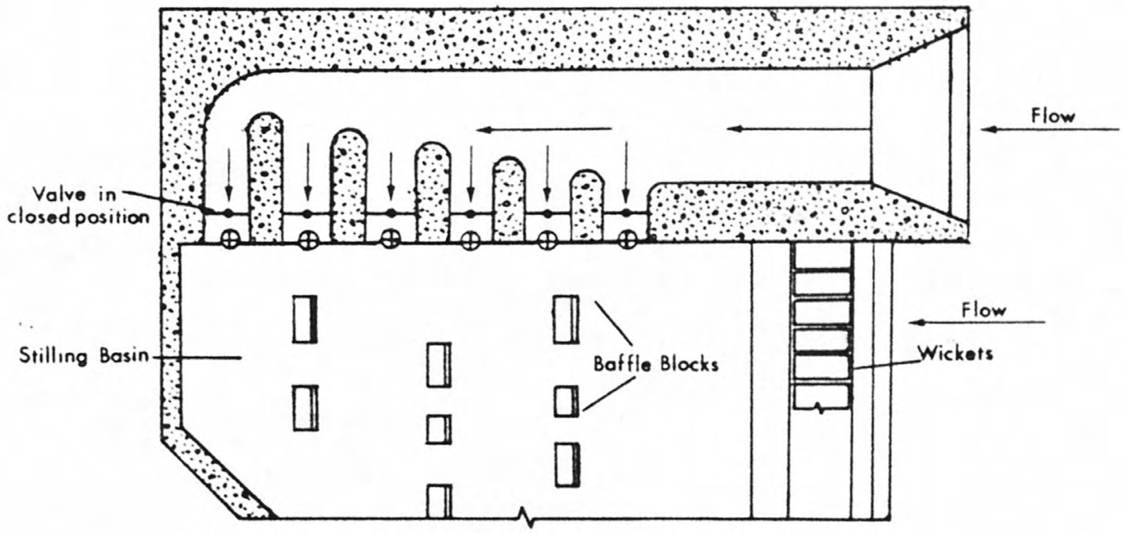
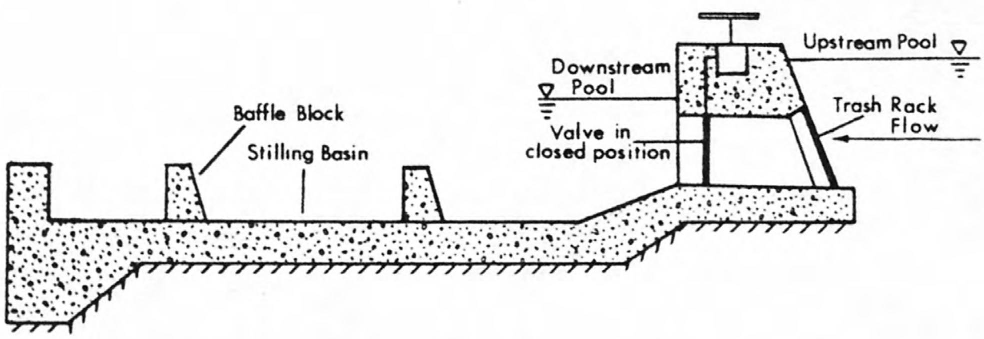


Figure 4.–Diagrams showing construction of typical Chanoine wickets.



Plan View of Peoria Butterfly Valve Abutment



Cross Section of La Grange Butterfly Valve Abutment

Figure 5.–Diagrams showing butterfly valve abutments at Peoria and La Grange Dams.

At La Grange Dam, the valve abutment is in line with the axis of the wicket dam. Twelve single-inlet, single-outlet conduits pass through the abutment. A butterfly valve is located near the inlet of each conduit (fig. 5). The outlets may be submerged at times.

THEORY

Discharge equations have been developed for flow control by a tainter gate, headgate, wicket, and butterfly valve. The hydraulic theory used in developing the following equations assumes steady, uniform flow during a computational interval. The theoretical equations are based on the energy and continuity equations between the approach section and a section downstream from the structure.

Tainter Gate Control

Collins (1977) discussed the possible flow regimes at a tainter gate having the geometry shown in figure 2. Table 2 summarizes the hydraulic conditions that define each regime and the corresponding steady-state discharge equations assuming the velocity head in the approach section is negligible.

Table 2.–Flow control by a tainter gate

Flow regime	Hydraulic conditions	Equation of flow	Equation number
Free-orifice	$h_g < 0.67\ h_1$ and $h_3 < h_g$	$Q = C\ [h_g B(2gh_1)^{0.5}]$	(1)
Submerged-orifice	$h_g < 0.67\ h_1$ and $h_3 \geqslant h_g$	$Q = C_{gs}\ [h_3 B(2g\Delta h)^{0.5}]$	(2)
Free-weir	$h_g \geqslant 0.67\ h_1$ and $h_3/h_1 < 0.6$	$Q = C_W\ [Bh_1{}^{1.5}]$	(3)
Submerged-weir	$h_g \geqslant 0.67\ h_1$ and $h_3/h_1 \geqslant 0.6$	$Q = C_W C_{WS}\ [Bh_1{}^{1.5}]$	(4)

The bracketed portions of equations 1 through 4 represent the theoretical expression for discharge under a tainter gate B units in width. The independent hydraulic control variables are static headwater depth (h_1), static tailwater depth (h_3), and gate opening (h_g). The variable, Δh, in equation 2 represents the difference between the headwater and tailwater depths. The gravitational constant, g, is equal to 32.2 ft/s^2. Headwater and tailwater depths are the vertical distances from the ogee spillway crest to upstream and downstream pool elevations, respectively. The velocity head at the approach section should be added to h_1 in equations 1-4 if the velocity head is appreciable.

The criteria used to separate orifice flow from weir flow is based on the fact that critical depth of flow in a rectangular channel is equal to two-thirds of the total head in the approach section. As the gate opening is increased above critical depth, the gate no longer acts as a control of discharge.

Small gate openings on high head dams can produce a hydraulic jump downstream. Free-orifice flow can exist when h_3/h_g is much greater than 1.0. Therefore, the criteria in table 2 for separating free-orifice flow from submerged-orifice flow is a general one. Free-orifice flow may occur during submergence ratios (h_3/h_g) as great as 2.0.

The criteria for separating free-weir flow from submerged-weir flow is also flexible. A value of h_3/h_1 equal to 0.6 is given as a minimum value necessary for submerged-weir flow to occur. Weir flow at a structure may actually become submerged at ratios as great as 0.85.

The "C" coefficients are discharge coefficients which are functions of h_1, h_3, and h_g. A discharge coefficient is defined as the ratio of measured discharge (Q) to theoretical discharge. Discharge coefficients are determined by measuring discharge during conditions when the hydraulic control variables are known and fixed. This procedure, referred to as calibration, may be performed on a hydraulic model under controlled laboratory conditions or in the field at the dam.

Headgate and Butterfly Valve Control

Headgates and butterfly valves are mechanisms for controlling the release of water through short, closed conduits passing through the dam. The conduits may have varying geometries. Discharge through these structures is either free- or submerged-orifice flow.

Equation 5 is the mathematical expression for free-orifice discharge controlled by a headgate or butterfly valve (figs. 3 and 5).

$$Q = C_{fo} [A_n (2gh_o)^{0.5}] \qquad (5)$$

The one hydraulic control variable is h_o, the static headwater depth above the center of the discharge section. The net cross-sectional area of flow is designated as A_n. Head loss coefficients for the individual outlet work components such as the trashrack, entrance, exit, gate, or valve are combined into the discharge coefficient C_{fo}. The discharge coefficient can be determined from field measurements of discharge or hydraulic model studies. Although C_{fo} varies directly with the ratio of headwater depth to conduit height, it is assumed constant for the small changes in h_o at each dam. The trashrack is assumed to be free of debris.

As the downstream pool elevation increases above the top of the conduit outlet, submerged-orifice flow occurs (equation 6).

$$Q = C_{so} [A_n (2g\Delta h)^{0.5}] \qquad (6)$$

The expression for submerged-orifice discharge controlled by a gate or valve is similar to equation 2; however, A_n is substituted for the product of h_3 and B.

Wicket Control

The U.S. Army Corps of Engineers (1938) presented several equations for flow controlled by a wicket dam (fig. 4). Discharge through the spaces between adjacent raised wickets is referred to as wicket-gap discharge to differentiate it from flow over lowered wickets. Wicket discharge refers to flow over any number of adjacent lowered wickets. Wicket-weir discharge refers to flow over the crest of raised wickets.

The equation of discharge through wicket spaces and over lowered wickets (equation 7) was calibrated by a hydraulic model study (U.S. Army Corps of Engineers, 1938).

$$Q = C_{wk} [\sec(\beta)(L)(0.67\ \Delta h + h_3)(2g\Delta h)^{0.5}] \qquad (7)$$

Discharge is dependent on tailwater (h_3) and headwater (h_1) depths. The discharge coefficient (C_{wk}) varies with the width of opening (L) and the degree of submergence at the wicket (h_3/h_1). The angle a raised wicket forms from vertical is β.

A sharp crested weir formula (equation 8) may be used to calculate discharge over the crest of raised wickets or closed tainter gates.

$$Q = C_{SW} \left[(W)(h_C)^{1.5} \right] \tag{8}$$

The depth of overflow referenced to the crest of the raised wicket or closed gate is h_C. The total width of the overflow section is W.

King and Brater (1963) reported that the discharge coefficient (C_{SW}) varies directly with the ratio of headwater depth (h_1) to the height of the impounding structure. The hydraulic model study of the U.S. Army Corps of Engineers (1938) indicated that C_{SW} was constant for the range of headwater depths expected at the wicket dams.

BRANDON ROAD DAM

Brandon Road Lock and Dam are on the Des Plaines River at river mile 13.3, approximately 15 miles upstream from Dresden Island Dam (fig. 6). Streamflow is regulated by raising various numbers of the 21 tainter gates clear of the water to maintain an upstream pool elevation of 538.5 feet. Each gate is 50 feet wide and 2.25 feet high. The ogee spillway crest on which a closed gate rests is at elevation 536.3 feet. Design charts indicate the streambed on the upstream side of the dam is at an elevation of about 506 feet. The lockmaster believes the present elevation is greater because of siltation.

The headgate structure adjacent to the tainter gate structure contains 16 sets of headgates. Each of the two vertical gates in a set is 10 feet high and 15 feet wide. The upper gate may be raised a maximum of 8.0 feet during regulation of high streamflows. The lower gates are never raised. Discharge through the headgate structure is never submerged.

Data Collection and Analysis

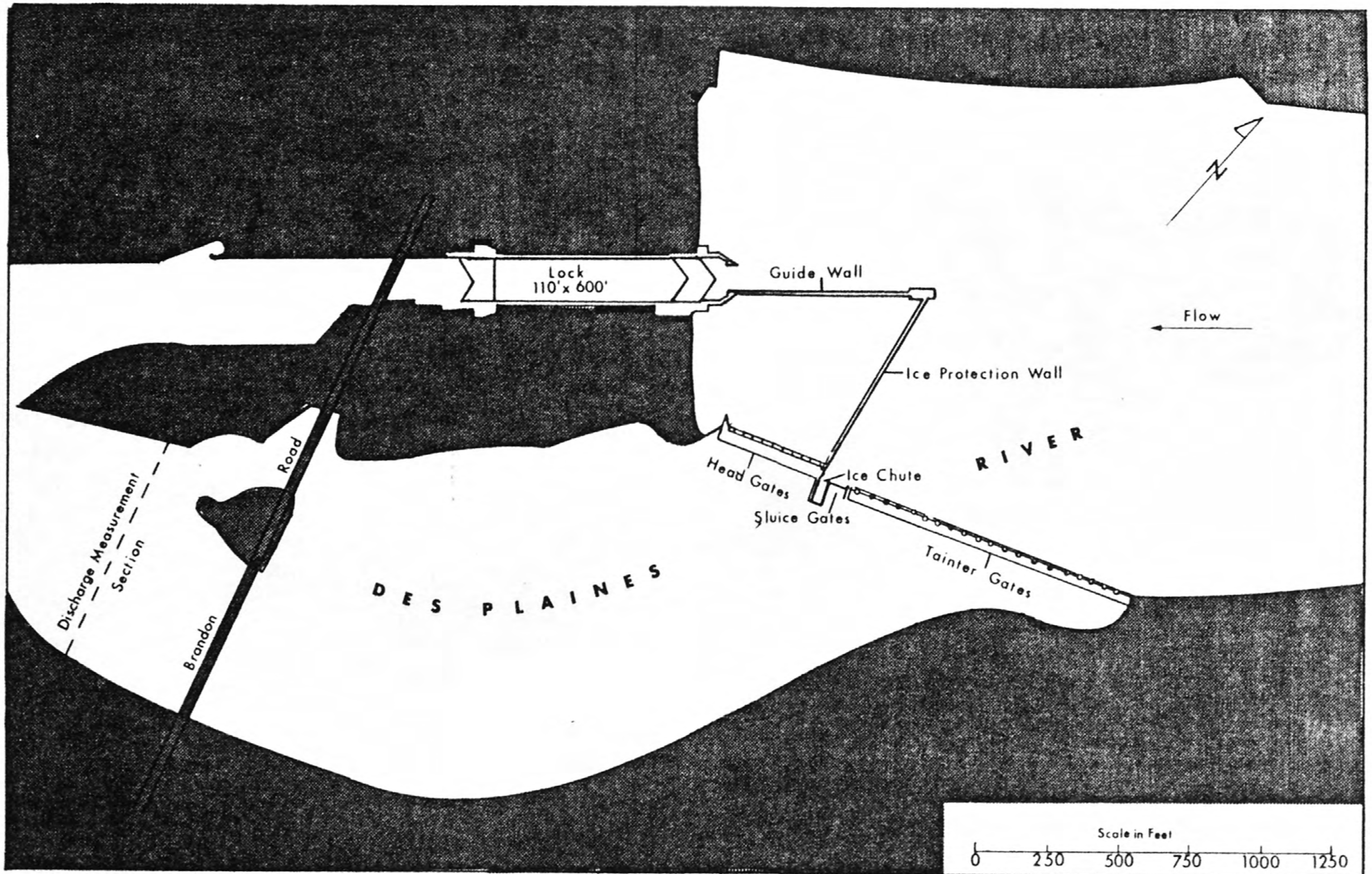
Five discharge measurements were made on the Des Plaines River at Brandon Road bridge, 0.25 miles downstream from the dam. Streamflow in Hickory Creek and Sugar Run, tributaries entering the Des Plaines River between the dam and Brandon Road bridge, was also measured.

Effluent from the East Side Joliet Municipal Treatment Plant enters Hickory Creek just upstream from its confluence with the Des Plaines River. Records of effluent release maintained by plant personnel were used to estimate intervening effluent discharge.

During each measurement, a combination of tainter gate and headgate settings was held constant to ensure steady flow. The elapsed time between consecutive measurements was sufficient for flow to stabilize at the measurement site. Pool elevations changed less than 0.3 foot during a measurement.

Measured discharges ranged from 1,730 to 15,200 ft^3/s. Table 3 summaries the hydraulic control conditions during each measurement. The combined discharges of Hickory Creek, Sugar Run, and the treatment plant are presented as intervening discharge.

Water flowed over the top of closed tainter gates during measurements 1, 2, 3, and 5 because the lockmaster could not risk lowering the upstream pool further by opening additional gates. The total flow over lowered gates was estimated by using a sharp crested weir formula (equation 8) and a discharge coefficient (C_{SW}) of 3.5. The height of water above the closed gates (h_C) was determined by subtracting the elevation of the top of the closed gates (538.55 feet) from the upstream pool elevation.



Base from U.S. Army Corps of Engineers,
Chicago, Illinois, September 30, 1978

Figure 6.–Brandon Road Lock and Dam

Table 3.—Discharge measurements and hydraulic control data at Brandon Road Dam

Measurement number	Date	Pool elevation, in feet[1]		Tainter gates open	Headgates		Discharge, in ft^3/s		
		upper	lower		number	opening (feet)	measured	intervening	adjusted[2]
1	04-06-78	539.00	507.6	16	1	8.0	15,200	1,660	13,300
2	05-24-78	538.60	505.3	2	1	8.0	3,590	144	3,410
3	06-18-80	538.65	505.6	5	0	–	3,950	68	3,800
4	06-18-80	538.50	505.6	3	1	8.0	4,170	68	4,100
5	06-18-80	538.70	505.5	0	0	–	1,730	68	1,470

[1] Elevations referenced to National Geodetic Vertical Datum of 1929.

[2] Adjusted discharge equals measured discharge less intervening flow and estimated flow over lowered gates.

Measurements 3 and 5 include substantial leakage from the headgate that was opened during measurement 4. The leakage is past an insecure rubber gasket on the bottom of the top vertical gate in headgate set number 1. This leakage did not exist in 1978.

Measurements 1, 2, and 4 were used to determine the discharge coefficient for free-weir flow under an open tainter gate (equation 3) and the coefficient for free-orifice flow through a headgate opening (equation 5). The determination of C_w and C_{fo} is complicated by the fact that free-weir and free-orifice flow occurred simultaneously during these three measurements.

The solution procedure was to first adjust each measurement by subtracting the corresponding intervening flow and flow over all lowered tainter gates (table 3). Equations 3 and 5 were added to form a mathematical expression for combined free-weir and free-orifice flow. For each measurement, the known headwater depths (h_1 and h_o) and gate opening geometries (B and A_n) were substituted for the bracketed portions of the combined equations. Each adjusted measurement was equated with its corresponding mathematical expression of combined flow resulting in a system of three equations (one for each measurement) having two unknowns (C_w and C_{fo}).

Since no unique values for C_w and C_{fo} will satisfy the three equations simultaneously, some criteria for determining the coefficients must be established. The criterion selected was to determine values for C_w and C_{fo} that minimized the sum of squared differences between adjusted streamflow and the mathematical expression for combined free-weir and free-orifice flow. Values for C_w and C_{fo} that minimized: $(13{,}300 - 3{,}549\ C_w - 3{,}667\ C_{fo})^2 + (3{,}410 - 348.8\ C_w - 3{,}616\ C_{fo})^2 + (4{,}100 - 489.5\ C_w - 3{,}603\ C_{fo})^2$ are 3.04 and 0.686, respectively. The combined discharge calculated from substituting these values into equations 3 and 5 are 0.1, 3.9, and -3.1 percent different from the adjusted discharges for measurements 1, 2, and 4, respectively.

The Corps at present assumes a discharge of 550 ft^3/s per open tainter gate for a normal upstream pool elevation of 538.55 feet. The corresponding discharge coefficient of 3.3 is 8.6 percent greater than the coefficient determined from measurements 1, 2, and 4. This difference is acceptable when considering (1) the discharge measurements were judged to be within 5 to 8 percent of the actual discharge, (2) the discharge of water flowing over closed gates cannot be measured directly, and (3) the present assumed coefficient of 3.3 is consistent with values published by the Corps (1952) and Bureau of Reclamation (1974). No modifications are suggested.

The Corps also assumes 2,800 ft^3/s of water will pass through one headgate opening of 8.0 feet. The corresponding free-orifice discharge coefficient for a normal upstream pool elevation of 538.55 feet is 0.78, a value that is consistent with those published by the Corps (1953) for a vertical sliding headgate raised upward from the bottom of a conduit. (This value is 13 percent greater than the value of 0.686 determined from measurements 1, 2, and 4). At Brandon Road Dam, only the top gate of the set of two gates is raised. Flow through such gate openings will contract more because it is not supported by the bottom of the conduit. Coefficients of discharge for rectangular orifices with partially suppressed contraction published by King and Brater (1963) vary between 0.6 and 0.7. A value of 0.7 is recommended for C_{fo} in equation 5. Coefficients published by the Corps (1953) and King and Brater (1963) indicate that the value for C_{fo} will decrease less than 10 percent as the gate opening is decreased. No attempt was made to define the relation of C_{fo} with gate opening.

Measurements 3 and 5 were used in an attempt to quantify the leakage through headgate set number 1. The free-weir discharge during measurement 3, based on equation 3 and a discharge coefficient of 3.3, was 2,970 ft^3/s. Subtracting this figure from the adjusted streamflow of 3,800 ft^3/s yields 830 ft^3/s of leakage.

Measurement 5 was an attempt to measure the leakage in a direct manner. The upstream pool was first lowered to elevation 583.3 feet, 0.25 foot lower than the top of the lowered tainter gates. All gates were then closed and the measurement was started 10 minutes later. Within 20 minutes after the start of measurement 5, the upstream pool had risen above the top of the closed gates. Although the measured discharge was adjusted for this extra discharge, it is probable that the downstream pool at Brandon Road bridge had not stabilized and that a large part of the measured streamflow was from a decrease in channel storage between the dam and the bridge. Measurement 5 is judged to be a very poor estimate of leakage past the dam.

After completing measurements 3, 4, and 5, a closer inspection of the tainter gate section revealed debris under several of the closed tainter gates that prevented a tight seal at the bottom of the gates. Therefore, the estimated leakage through headgate set number 1 is probably less than 850 ft^3/s.

Ratings and Discussion

Table 4 is a rating of free-weir flow under one tainter gate and through one headgate at Brandon Road Dam. The table was developed from equations 3 and 5 (pages 9 and 10) and discharge coefficients determined from five discharge measurements.

The tainter gate discharge under, in table 4, is the stage-discharge rating for flow under one open tainter gate. Discharges for headwater elevations not summarized in the table may be calculated by interpolation or from equation 3 with a free-weir coefficient of 3.3. Weir flow will never become submerged at this dam.

The tainter gate discharge over, in table 4, represents the sharp crested weir discharge rating of flow over the top of one closed tainter gate. A sharp crested weir coefficient of 3.5 and equation 8 were used to calculate this rating.

The remaining portion of table 4 is a rating of discharge through various headgate openings. The rating is for free-orifice control only, as the downstream pool never rises enough to submerge flow over the stationary lower gate in each set of headgates. The discharge coefficient of 0.7 is assumed applicable for all gate settings.

Table 4.—Stage-discharge rating for one tainter gate and one headgate at Brandon Road Dam

Upstream pool elevation (feet)	Tainter gate discharge (ft^3/s)		Discharge, in ft^3/s, through headgate openings of:								
	under	over	0.0	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0
536.0	0	0	0	326	642	946	1,240	1,520	1,790	2,040	2,290
536.5	15	0	0	332	652	963	1,260	1,550	1,820	2,090	2,340
537.0	97	0	0	337	663	979	1,280	1,580	1,860	2,130	2,380
537.5	217	0	0	342	674	995	1,310	1,600	1,890	2,170	2,430
538.0	366	0	0	347	685	1,010	1,330	1,630	1,930	2,210	2,480
538.5	538	0	0	352	695	1,030	1,350	1,660	1,960	2,250	2,520
539.0	732	52.8	0	357	705	1,040	1,370	1,690	1,990	2,280	2,570
539.5	945	162	0	362	715	1,060	1,390	1,710	2,020	2,320	2,610
540.0	1,170	306	0	367	725	1,070	1,410	1,740	2,050	2,360	2,650

DRESDEN ISLAND DAM

Dresden Island Lock and Dam are on the Illinois River at river mile 271.5, 1.5 miles downstream from the confluence of the Des Plaines and Kankakee Rivers. Streamflow is regulated by the operation of nine tainter gates (fig. 7) to maintain an upstream pool elevation of 504.5 feet. Each gate is 60 feet wide and 16 feet high. The ogee spillway crest elevation under the tainter gates is 490.5 feet, and the forebay elevation is 484.4 feet. The headgate structure adjacent to the tainter gate structure contains 16 headgates which are never used for regulation.

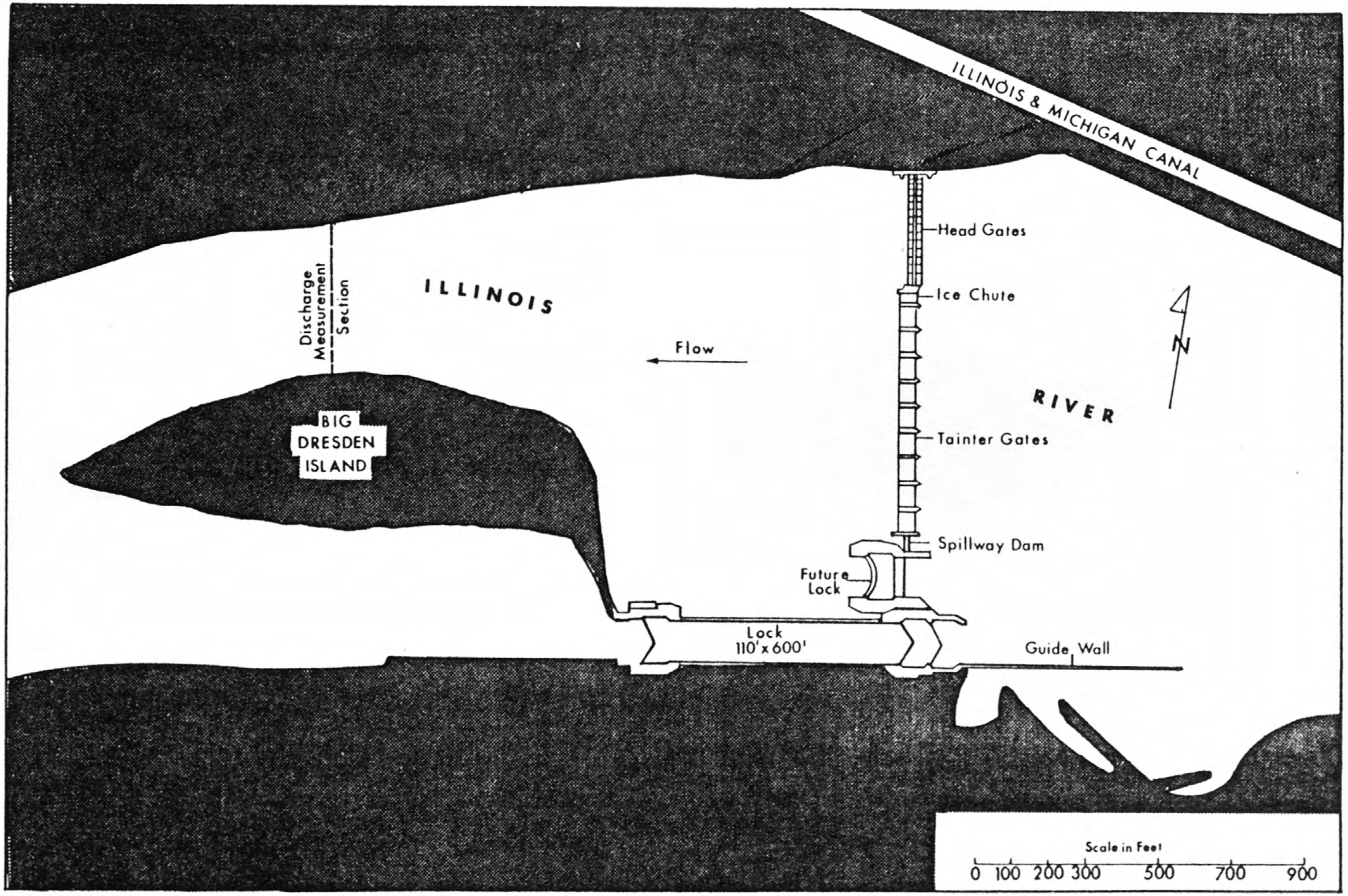
Data Collection and Analysis

Eleven discharge measurements were made on the Illinois River, 0.25 mile below Dresden Island Dam. The purpose of these measurements was to obtain data for determining discharge coefficients for equations 1, 2, and 3 (page 9).

During each discharge measurement, a combination of tainter gate settings was held constant to ensure steady flow. The elapsed time between consecutive measurements was sufficient for flow to stabilize at the measurement site. Pool elevations never changed more than 0.3 foot during any measurement.

Measured discharges ranged from 7,770 to 27,200 ft^3/s. The hydraulic control conditions during these measurements are summarized in table 5. Velocity heads at approach sections were calculated for each measurement.

Free-orifice flow was measured during measurements 1, 2, and 4 through 10. Measurements 3 and 11 were made during free-weir flow. During measurements 9 and 10, the tainter gates were set so that the submerged-orifice flow equation could be calibrated. Submerged-weir flow was not measured.



Base from U.S. Army Corps of Engineers,
Chicago, Illinois, September 30, 1978

Figure 7.–Dresden Island Lock and Dam

Table 5.–Discharge measurements and hydraulic control data at Dresden Island Dam

Measurement number	Date	Pool elevation, in feet[1] upstream	Pool elevation, in feet[1] downstream	Tainter gates number	Tainter gates opening (feet)	Flow regime[2]	Measured discharge (ft^3/s)	Discharge coefficient
1	04-06-76	504.6	(3)	2	4.5	FO	10,900	0.663
2	04-06-76	504.6	(3)	3	3.0	FO	11,000	.674
3	04-07-76	504.7	(3)	1	11.0	FW	10,400	2.90
4	04-07-76	504.7	(3)	1	8.0	FO	[4]9,280	.537
				1	1.0	FO		[5].711
5	04-07-76	504.7	(3)	1	6.0	FO	[4]9,150	.593
				2	1.0	FO		[5].711
6	04-07-76	504.8	(3)	6	1.0	FO	7,770	.711
7	04-18-78	504.5	(3)	2	6.0	FO	13,800	.627
8	04-18-78	504.5	(3)	3	4.0	FO	12,200	.560
9	06-04-80	504.6	494.8	3	3.5	FO	[4]27,200	[6].676
				5	2.5	SO		.439
10	06-04-80	504.6	494.4	5	3.5	FO	[4]25,900	[6].676
				3	1.5	SO		.245
11	06-04-80	504.9	494.0	2	11.0	FW	24,500	3.22

[1] Elevations referenced to National Geodetic Vertical Datum of 1929.
[2] FO designates free-orifice flow; SO designates submerged-orifice flow; and FW designates free-weir flow.
[3] Pool elevation is less than ogee spillway crest elevation.
[4] Combined flow through all gate openings.
[5] Coefficient for gates set at 1.0 foot is assumed equal to the coefficient determined from measurement number 6.
[6] Coefficient for gates set at 3.5 foot was determined from free-orifice discharge coefficient relation (fig. 8).

The free-orifice discharge coefficients (C) presented in table 5 are related to the vertical gate opening (h_g). Figure 8 illustrates the free-orifice flow coefficient relation defined by six of these coefficients. Measurement 8 was rejected as an outlier because too few velocity measurements were made at the measuring site.

A transition to free-weir flow occurs as the tainter gate opening is increased above 10 feet. The lower edges of the gates were out of the water at a gate opening of 11.5 feet during measurements 3 and 11. Based on these measurements, C_W (equation 3) is estimated to be 3.06.

A transition from free-orifice to submerged-orifice flow was assumed to occur at a submergence ratio (h_3/h_g) of 1.5 for measurements 9 and 10. The submerged-orifice discharge coefficient (C_{gs}) is indirectly proportional to the submergence ratio. Figure 9 illustrates the submerged orifice flow coefficient relation visually fitted to data derived from measurements 9 and 10. The relation is expressed mathematically as:

$$C_{gs} = 0.944\,(h_3/h_g)^{-1.412} \qquad (9)$$

Figure 9, or equation 9, may be used to determine C_{gs} for a known tailwater depth and gate opening.

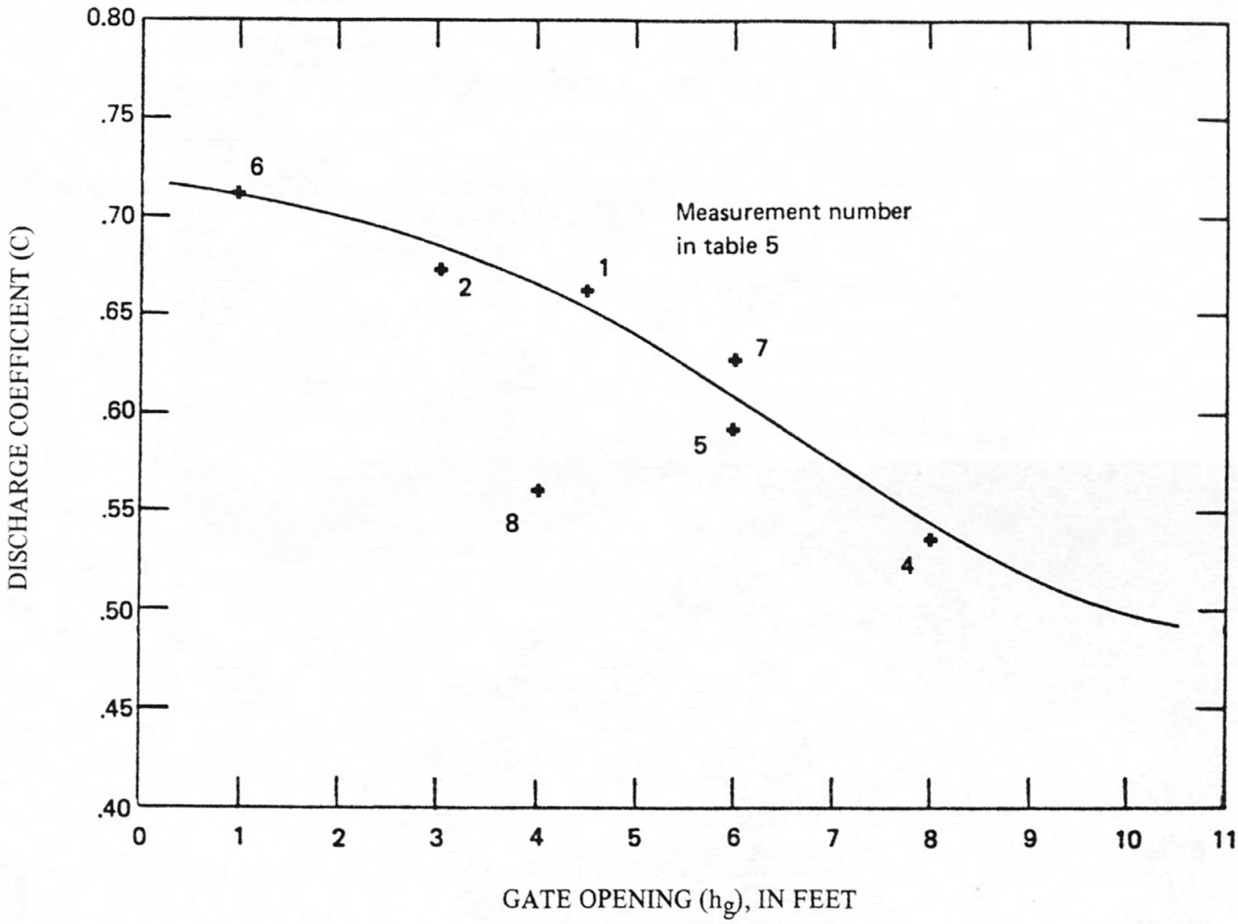


Figure 8.–Free orifice flow coefficient relation for Dresden Island Dam tainter gates.

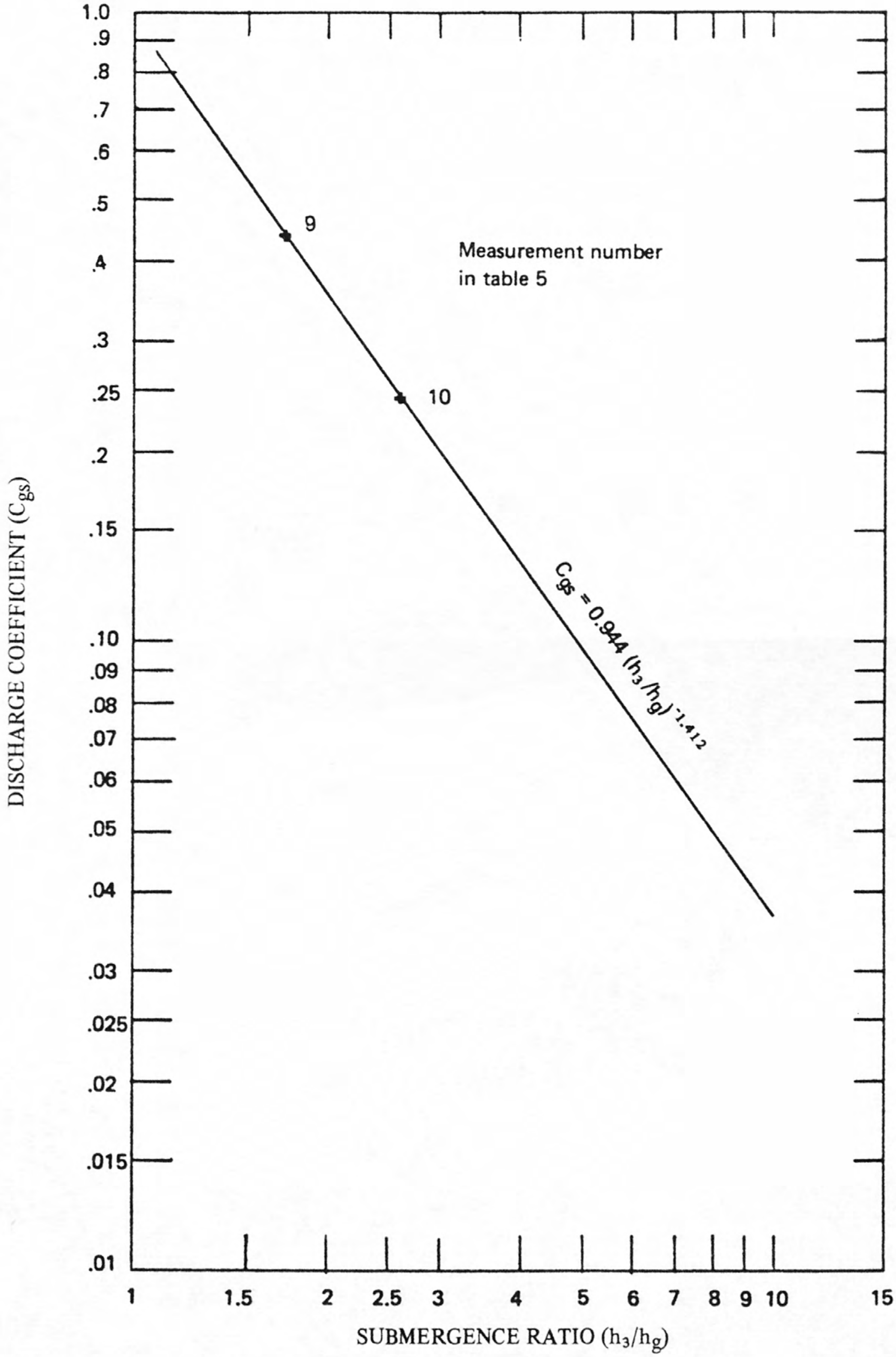


Figure 9.–Submerged orifice flow coefficient relation for Dresden Island Dam tainter gates.

Submerged-weir flow infrequently occurs at Dresden Island Dam. A review of the lockmaster's log for periods of abnormally high streamflow in 1966, 1968, and 1970 indicated that submerged weir flow conditions occurred infrequently for periods of 48 hours or less in duration. The calibration of equation 4 will be difficult because of the infrequent occurrence of submerged-weir control conditions and because streamflow will be unsteady during these periods.

Ratings and Discussion

Table 6 is a stage-discharge rating for one tainter gate at Dresden Island Dam. The table was developed from equations 1, 2, and 3 (page 9) and discharge-coefficient relations determined from 11 discharge measurements (figs. 8 and 9).

Table 6.–Stage-discharge rating for one tainter gate at Dresden Island Dam and upstream pool elevation of 504.5 feet

Gate opening (feet)	Discharge, in ft^3/s, for downstream pool elevations of:						
	491	493	495	497	499	501	503
0.0	0	0	0	0	0	0	0
0.5	644	397	283	216	166	121	74
1.0	1,280	1,060	753	576	441	322	197
1.5	1,910	1,870	1,340	1,020	783	572	349
2.0	2,530	2,530	2,010	1,530	1,170	859	523
2.5	3,130	3,130	2,760	2,100	1,610	1,180	717
3.0	3,720	3,720	3,580	2,730	2,080	1,520	928
3.5	4,290	4,290	4,290	3,400	2,610	1,890	1,150
4.0	4,830	4,830	4,830	4,120	3,160	2,300	1,390
4.5	5,340	5,340	5,340	4,890	3,740	2,730	1,640
5.0	5,830	5,830	5,830	5,710	4,360	3,180	1,910
5.5	6,290	6,290	6,290	6,290	5,020	3,650	2,220
6.0	6,710	6,710	6,710	6,710	5,700	4,150	2,520
6.5	7,090	7,090	7,090	7,090	6,430	4,680	2,840
7.0	7,440	7,440	7,440	7,440	7,200	5,230	3,170
7.5	7,740	7,740	7,740	7,740	7,740	5,810	3,520
8.0	8,020	8,020	8,020	8,020	8,020	6,420	3,880
8.5	8,340	8,340	8,340	8,340	8,340	7,110	4,260
9.0	8,650	8,650	8,650	8,650	8,650	7,810	4,660
9.5	8,990	8,990	8,990	8,990	8,990	8,580	5,160
10.0	9,380	9,380	9,380	9,380	9,380	9,380	5,620
10.5	11,000	11,000	11,000	11,000	11,000	submerged weir flow	
11.0	11,000	11,000	11,000	11,000	11,000		

Discharges from an upstream pool elevation of 504.5 feet may be read directly from the table. The table is partitioned into four sections, each of which represents a different flow regime. Tabulated discharges must be adjusted by a factor for each regime as headwater elevations deviate from 504.5 feet.

The leftmost column of discharges in table 6 represents the free-orifice flow rating. These discharges must be multiplied by $[(HW - 490.5)/14.0]^{0.5}$ to determine the appropriate discharge under each gate from upstream pool elevations (HW) other than 504.5 feet. For example, the free-orifice discharge from an upstream pool of 505.0 feet (HW) under a gate set at 6.0 feet is determined by multiplying the rating discharge of 6,710 ft^3/s by 1.018, which equals 6,830 ft^3/s.

As the downstream pool rises above elevation 490.5, orifice discharge under the lower gate openings will become submerged. The right section of table 6 represents the submerged orifice rating for various downstream pool elevations. At upstream pool elevations (HW) other than 504.5 feet, these discharges must be multiplied by $[(HW - TW)/(504.5 - TW)]^{0.5}$ to determine the appropriate discharge under each gate for a downstream pool elevation of TW feet. For example, the discharge from an upstream pool of 506.0 feet (HW) to a downstream pool of 490.0 feet (TW) under a gate set at 6.0 feet is determined by multiplying the rating discharge of 5,700 ft^3/s by 1.128, which equals 6,430 ft^3/s.

The large change in discharge at the transition from free-orifice to submerged-orifice discharge shown in table 6 results from a change in equations and coefficients used to calculate discharge for each regime. The transition from free-orifice flow to submerged-orifice flow was assumed to occur at a submergence ratio of 1.5.

The free-weir discharge from an upstream pool of 504.5 feet under one gate raised clear of the water will be 11,000 ft^3/s. This discharge must be multiplied by $[(HW - 490.5)/14.0]^{1.5}$ to adjust for upstream pool elevations different than 504.5 feet. The velocity head in one forebay with 11,000 ft^3/s of water flowing in it is 1.3 feet. The total head at the approach section for elevation 504.5 feet is equal to the depth of the flow, 14.0 feet, plus the velocity head of 1.3 feet. Therefore, weir flow conditions exist at gate openings greater than 10.2 feet (0.67 x 15.3). Submerged-weir flow exists when the tailwater depth exceeds 9.2 feet (0.6 x 15.3).

Weir flow becomes submerged as the downstream pool rises above an elevation of 499.7 feet. The lower right section of table 6 is not filled in because submerged-weir flow was not measured and values of C_{WS} (equation 4) could not be determined.

MARSEILLES DAM

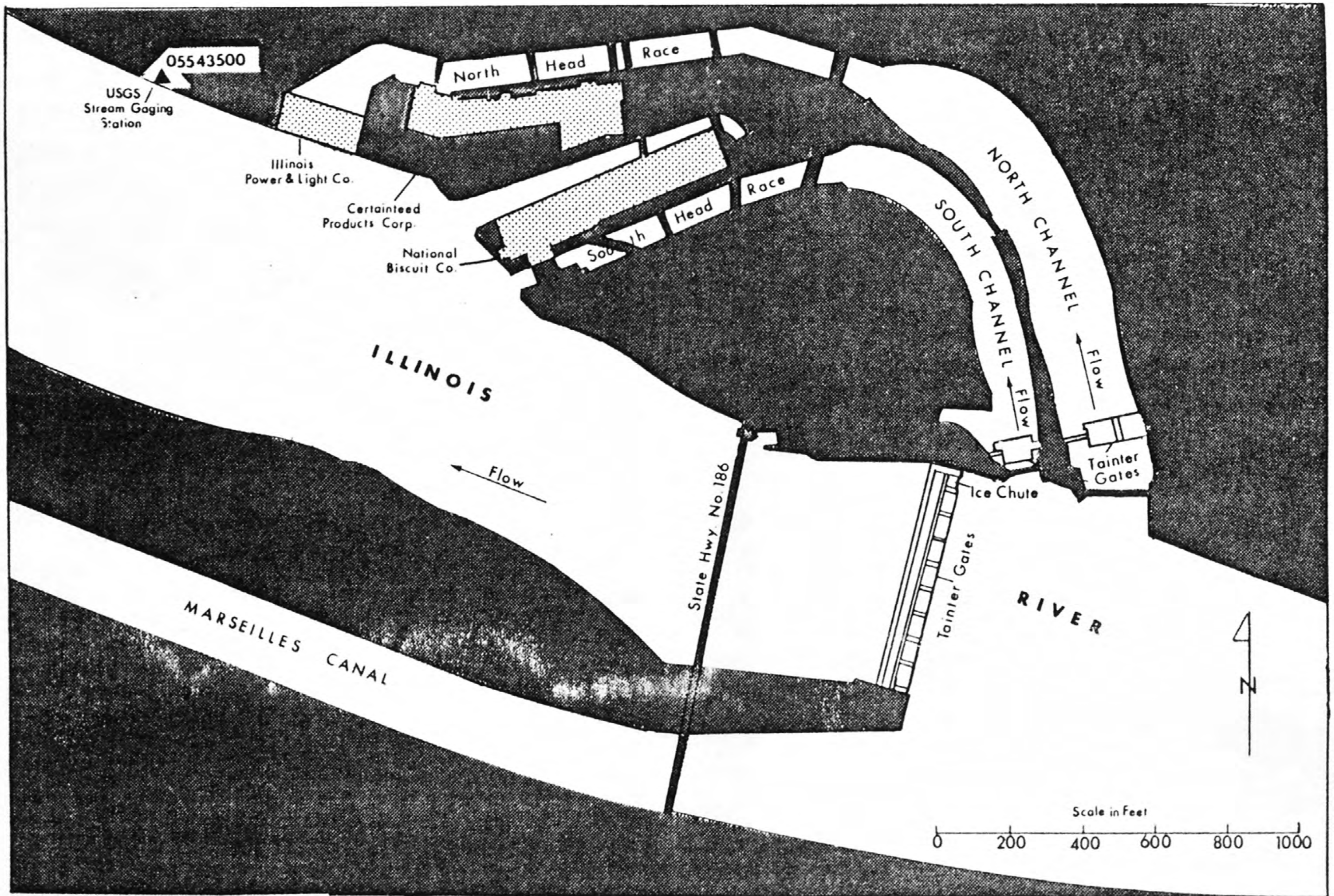
Marseilles Dam is on the Illinois River at river mile 247.0 (fig. 10). The lock is 2.5 miles downstream on the Marseilles Canal. Streamflow is regulated by the operation of eight tainter gates to maintain an upstream pool elevation of 482.8 feet. Each gate is 60 feet wide and 17 feet high. The ogee spillway crest elevation under the tainter gate is 469.8 feet, and the forebay elevation is 468.6 feet.

Illinois Power and Light Company operates a hydroelectric powerplant just downstream from the dam. Water is diverted from the upstream pool through the north channel to turbines on the right edge of the downstream pool.

Data Collection and Analysis

A gaging station (05543500) is maintained on the Illinois River about one-quarter of a mile downstream from the dam. Discharge at the gaging station includes the discharge past the dam and through the powerplant diversion canal.

Discharge at the gaging station was determined from a well-defined stage-discharge relation. The diversion canal discharge was measured during periods when it appeared that the canal discharge was a significant portion of the total streamflow past the dam and was subtracted from the discharge at the gaging station. Discharge measurements are routinely made to verify the gaging-station rating. Several of these measurements were also used in the dam rating analysis.



Base from U.S. Army Corps of Engineers, Chicago, Illinois, September 30, 1978

Figure 10.–Marseilles Dam

Equations 1, 2, and 3 were calibrated from the 10 different combinations of gate settings summarized in table 7. Discharges were determined from direct measurements or the stage-discharge relation at the gaging station. The powerplant diversion canal discharge was measured and deducted from total streamflow when necessary. Measurement 11 was used to verify the free-orifice and submerged-orifice flow coefficient relations.

Each combination of gate settings was held constant to ensure steady flow. The elapsed time between consecutive measurements was sufficient to allow flow to stabilize. A velocity head was calculated at an approach section to each gate that was open.

Discharge through the 11 combinations of gate settings ranged from 3,610 to 86,400 ft^3/s. Table 7 summarizes the hydraulic controls and upstream and downstream pool elevations for each setting.

Free-orifice flow was measured during measurements 3 through 11. Submerged-orifice flow was assumed to exist at submergence ratios (h_3/h_g) greater than 1.5. Measurements 1, 2, and 7 through 10 were made when free-weir flow existed at one or more gates. Submerged-weir flow may have existed during measurements 1 and 7. However, the degree of submergence was so little that free weir flow was assumed to have existed.

Multiple flow regimes existed at the dam during several measurements. The coefficients for these cases were determined in a step-wise manner by using information determined from measurements where a single control existed.

The free-orifice discharge coefficients (C) in table 7 are related to the vertical gate opening (h_g). Figure 11 illustrates the free-orifice flow coefficient relation defined by these values. Coefficients for gate openings greater than 6.0 feet were estimated from data obtained during free-weir flow. Measurement 3 was eliminated from the analyses as an outlier because the free-orifice flow coefficients calculated for gate openings of 4.0, 5.0, and 6.0 feet are much greater than the coefficients determined from measurements 4 through 6. Apparently, canal discharge was much greater than was thought and should have been measured.

A transition to free-weir flow occurs as the tainter gate opening is increased above 9 feet. Measurements 1 and 2 were made when the gates were clear of the water surface. Based on these measurements, the free-weir-flow coefficient (C_w) is estimated to be 2.75. This value was used to determine the portion of free-weir flow in measurements 7 through 10.

A transition from free-orifice to submerged-orifice flow was assumed to occur at a submergence ratio (h_3/h_g) of 1.5. The submerged-orifice coefficient is inversely proportional to the submergence ratio (h_3/h_g) as illustrated in figure 12. This relation is expressed mathematically as:

$$C_{gs} = 0.943\,(h_3/h_g)^{-1.169} \qquad (10)$$

Equation 10 was determined by trial and error. Equation 2 and variations of equation 10 were used to estimate discharges for the hydraulic conditions during measurements 7 through 11. Equation 10 is the final form that minimized the differences between estimated and measured discharge. Figure 12 or equation 10 may be used to determine C_{gs} for a known tailwater depth (h_3) and gate opening.

The free-orifice and submerged-orifice discharge coefficient relations illustrated in figure 11 and 12 were used with equations 1 and 2 (page 9) to estimate the discharge for the hydraulic conditions present during measurement 11. The estimated discharge was 6 percent less than the measured discharge.

A review of the lockmaster's log for periods of abnormally high streamflow indicated that the downstream pool infrequently rises enough to significantly reduce free-weir discharge because of excessive tailwater submergence. Discharge measurements are needed at the dam when downstream pool elevations are greater than 480.0 feet, so that submerged-weir discharge coefficients can be accurately determined.

Table 7.–Discharge measurements and hydraulic control data at Marseilles Dam

Measurement number	Date	Pool elevation, in feet[1]		Tainter gates		Flow regime[2]	Measured discharge[3]			Discharge coefficient
		upstream	downstream	number	opening (feet)		river (ft^3/s)	canal (ft^3/s)	dam (ft^3/s)	
1	05-15-70	483.7	480.1	8	9+	FW	86,400*	(5)	86,400	2.79
2	04-07-78	483.1	475.8	4	9+	FW	40,100	1,630*	38,500	2.71
3	04-12-78	483.2	474.4	2	6.0	FO	[4] 25,600	(5)	25,600	.69
				1	5.0	FO				.63
				1	4.0	FO				.65
4	04-20-78	483.2	473.2	4	4.0	FO	20,400	1,860*	18,500	.65
5	04-20-78	483.1	473.2	3	5.0	FO	19,000	1,860*	17,100	.63
6	08-09-78	483.1	471.2	1	3.0	FO	5,780	2,170*	3,610	.68
7	03-07-79	482.8	479.1	2	3.0	SO	[4] 62,000*	(5)	62,000	.26
				6	9+	FW				2.75
8	06-03-80	483.0	476.4	1	0.5	SO	[4] 49,000	(5)	49,000	.05
				1	4.0	SO				.52
				2	5.0	FO				.63
				2	6.0	FO				.61
				2	9+	FW				2.75
9	06-03-80	483.0	476.2	1	0.5	SO	[4] 47,500	(5)	47,500	.05
				1	2.5	SO				.36
				2	5.0	FO				.63
				2	6.0	FO				.61
				2	9+	FW				2.75
10	06-03-80	483.0	476.2	1	0.5	SO	[4] 45,700	(5)	45,700	.05
				1	1.5	SO				.14
				2	5.0	FO				.63
				2	6.0	FO				.61
				2	9+	FW				2.75
11	06-05-80	483.2	475.4	1	0.5	SO	[4,6] 33,800	(5)	33,800	
				2	2.0	SO				
				2	4.0	SO				
				3	5.0	FO				

[1] Elevations referenced to National Geodetic Vertical Datum of 1929.
[2] FO designates free-weir flow; FO designates free-orifice flow; and SO designates submerged-orifice flow.
[3] Discharges determined from gaging station stage-discharge relation. Asterisk (*) indicates that discharge was measured.
[4] Combined flow through all gate openings.
[5] Canal discharge not measured because it appeared to be a small percentage of total discharge.
[6] Measurement used to verify submerged-orifice and free-orifice flow coefficient relations.

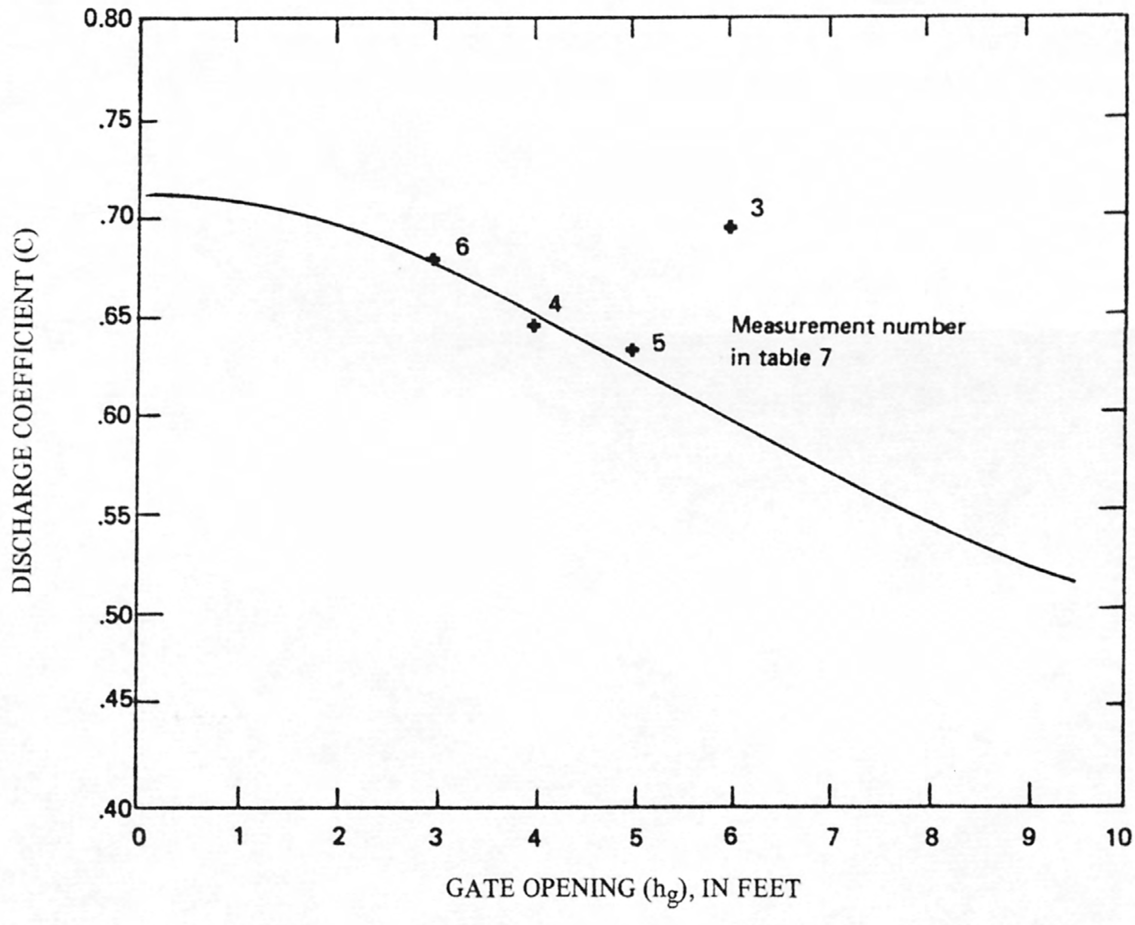


Figure 11.–Free orifice flow coefficient relation for
Marseilles Dam tainter gates.

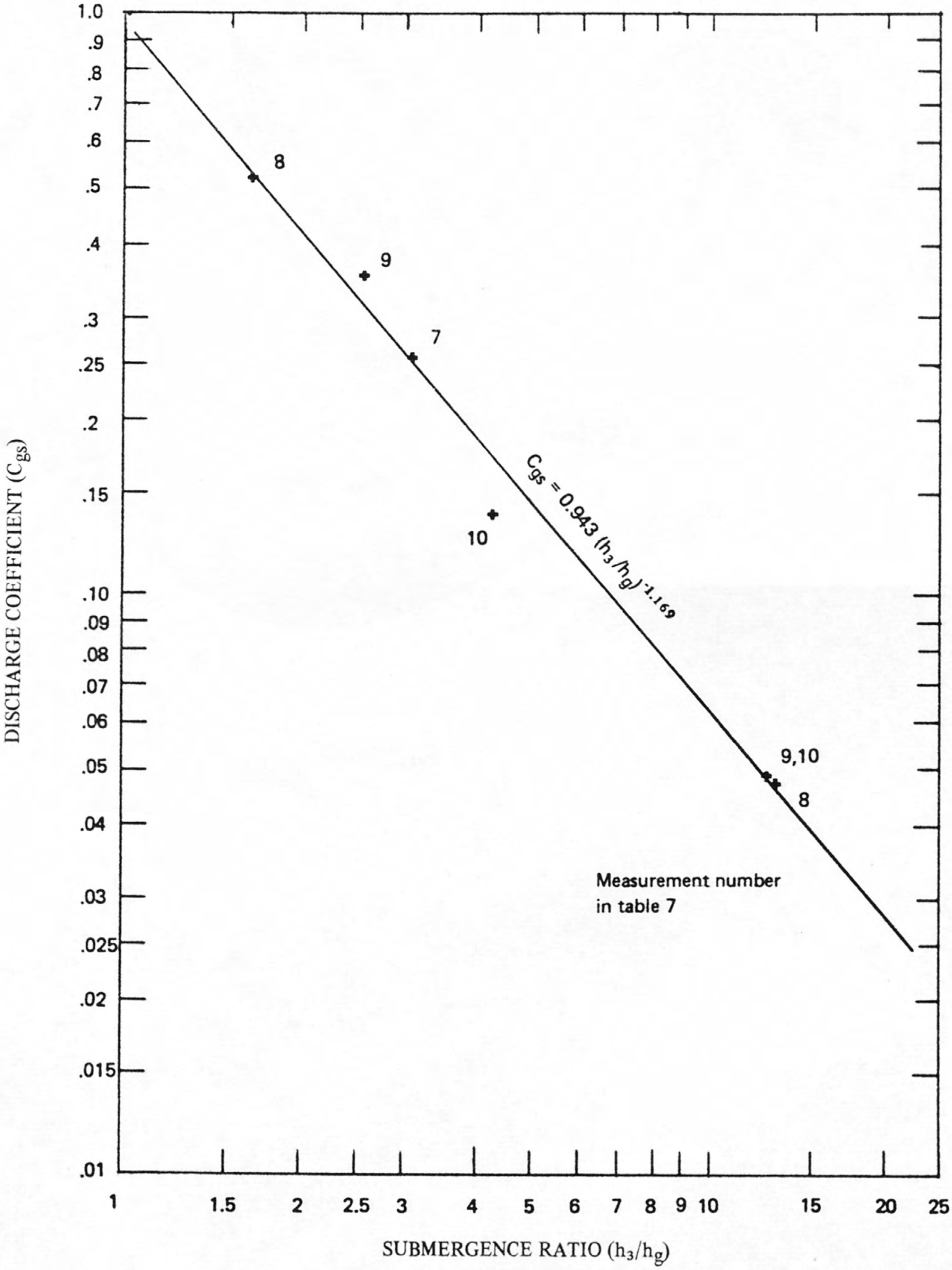


Figure 12.–Submerged orifice flow coefficient relation for Marseilles Dam tainter gates.

Table 8 is a stage-discharge rating for one tainter gate at Marseilles Dam. The table was developed from equations 1, 2, and 3 and discharge coefficient relations determined from 10 discharge measurements.

Table 8.–Stage-discharge rating for one tainter gate at Marseilles Dam and upstream pool elevation of 482.8 feet

Gate opening (feet)	Discharge, in ft^3/s, for downstream pool elevations of: 470	472	474	476	478	480	482
0.0	0	0	0	0	0	0	0
0.5	618	580	470	387	310	228	118
1.0	1,230	1,230	1,060	870	697	513	266
1.5	1,840	1,840	1,700	1,400	1,120	824	427
2.0	2,430	2,430	2,390	1,970	1,580	1,150	598
2.5	3,010	3,010	3,010	2,560	2,050	1,500	777
3.0	3,560	3,560	3,560	3,190	2,550	1,880	961
3.5	4,080	4,080	4,080	3,850	3,080	2,260	1,150
4.0	4,600	4,600	4,600	4,530	3,620	2,660	1,350
4.5	5,090	5,090	5,090	5,090	4,190	3,080	1,590
5.0	5,550	5,550	5,550	5,550	4,780	3,510	1,820
5.5	6,010	6,010	6,010	6,010	5,440	3,960	2,050
6.0	6,440	6,440	6,440	6,440	6,110	4,440	2,290
6.5	6,850	6,850	6,850	6,850	6,820	4,980	2,550
7.0	7,230	7,230	7,230	7,230	7,230	5,530	2,810
7.5	7,600	7,600	7,600	7,600	7,600	6,110	3,090
8.0	7,900	7,900	7,900	7,900	7,900	6,720	3,380
8.5	8,350	8,350	8,350	8,350	8,350	7,450	3,790
9.0	8,690	8,690	8,690	8,690	8,690	8,270	4,220
9.5	9,080	9,080	9,080	9,080	9,080	9,080	4,640
10.0	9,490	9,490	9,490	9,490	9,490	submerged weir flow	
10.5	9,490	9,490	9,490	9,490	9,490		

Discharges from an upstream pool elevation of 482.8 feet may be read directly from the table. The table is partitioned into four sections, each of which represents a different flow regime. Tabulated discharges must be adjusted by a factor for each regime as headwater elevations deviate from 482.8 feet.

The leftmost column of discharges on table 8 represents the free-orifice flow rating. These discharges must be multiplied by $[(HW - 469.8)/13.0]^{0.5}$ to determine the appropriate discharge under each gate from upstream pool elevations (HW) other than 482.8 feet. For example, the free-orifice discharge from an upstream pool elevation of 483.5 feet (HW) under a gate set at 6.0 feet is determined by multiplying the rating discharge of 6,440 ft^3/s by 1.027, which equals 6,610 ft^3/s.

As the downstream pool elevation rises above 469.8 feet, orifice discharge under the lower gate openings will become submerged. The right section of table 8 represents the submerged-orifice rating for various downstream pool elevations. At upstream pool elevations (HW) other than 482.8 feet, these discharges must be multiplied by $[(HW - TW)/(482.8 - TW)]^{0.5}$ to determine the appropriate discharge under each gate for a lower pool elevation of

TW feet. For example, the discharge from an upstream pool elevation of 483.2 feet (HW) to a downstream pool elevation of 476.0 feet (TW) under a gate set at 4.0 feet is determined by multiplying the rating discharge of 4,530 ft^3/s by 1.029, which equals 4,660 ft^3/s.

The large change in discharge at the transition from free-orifice to submerged-orifice discharge shown in table 8 results from a change in equations and coefficients used to calculate discharge for each regime. The transition from free-orifice flow to submerged-orifice flow was assumed to occur at a submergence ratio of 1.5.

The discharge from an upstream pool elevation of 482.8 feet under one gate raised clear of the water will be 9,490 ft^3/s. This discharge must be multiplied by $[(HW - 469.8)/13.0]^{1.5}$ to adjust for upstream pool elevations different than 482.8 feet. The velocity head in one forebay with 9,490 ft^3/s of water flowing in it is 1.9 feet. The total head at the approach section for elevation 482.8 feet is equal to the depth of flow, 13.0 feet, plus the velocity head of 1.9 feet. Therefore, weir flow conditions exist at gate openings greater than 10.0 feet (0.67 x 14.9). Submerged-weir flow exists when the tailwater depth exceeds 8.9 feet (0.6 x 14.9).

Weir flow becomes submerged as the downstream pool rises above elevation 478.7 feet. The lower right section of table 8 is not filled in because submerged-weir flow coefficients (C_{WS}) could not be accurately determined.

STARVED ROCK DAM

Starved Rock Lock and Dam are on the Illinois River at river mile 231.0. Streamflow is regulated by the operation of 10 tainter gates (fig. 13) to maintain an upstream pool elevation of 458.5 feet. Each gate is 60 feet wide and 19 feet high. The ogee spillway crest on which a closed gate rests is at elevation 441.5 feet, and the forebay elevation is 438.4 feet. The headgate structure adjacent to the tainter gate structure contains 28 headgates, which are never used for regulation.

Data Collection and Analysis

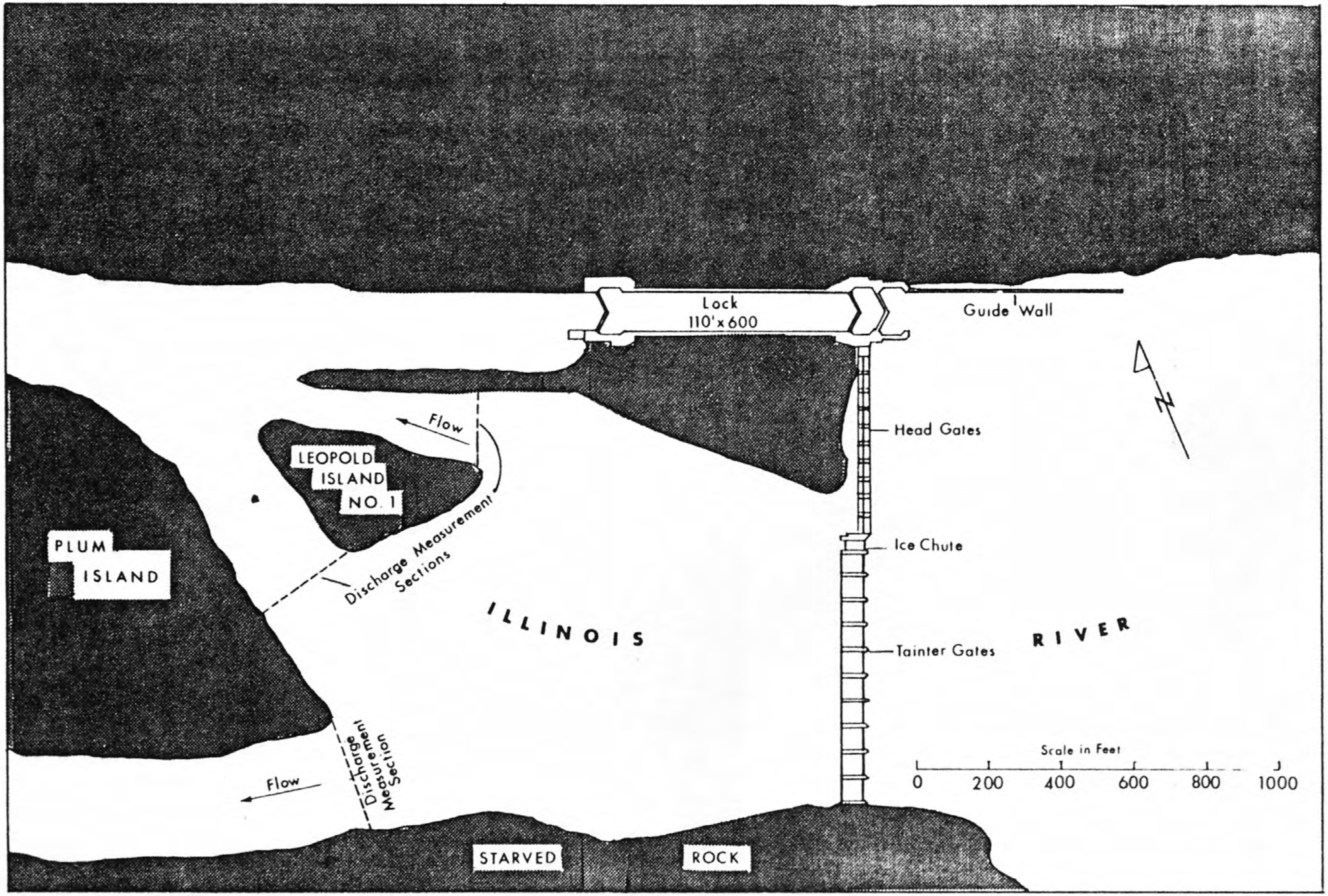
Nine discharge measurements were made on the Illinois River to obtain data for determining discharge coefficients for equations 1, 2, and 3 (page 9). Measurements 1 through 6 (table 9) were made about one-fourth mile downstream from the dam. The remaining measurements were made at Starved Rock State Park, about three-fourths mile downstream from the dam.

During each discharge measurement, a combination of tainter gate settings was held constant to ensure steady flow. The elapsed time between measurements was sufficient to allow flow to stabilize at the measurement site. Upstream and downstream pool elevations never changed more than 0.3 foot during any measurement. A velocity head was calculated at an approach section to each gate that was open.

Measured discharges ranged from 6,220 to 26,400 ft^3/s. Table 9 summarizes the hydraulic control conditions during each measurement.

Free-orifice flow was measured during measurements 5, 6, 8, and 9. Measurements 1, 2, 3, 4, and 9 were made when submerged-orifice flow existed at one or more gates. Free-weir flow was measured during measurement 7. Submerged-weir flow was not measured.

The free-orifice discharge coefficients (C) in table 9 vary with the vertical gate opening (h_g). Figure 14 illustrates the relation defined by these values.



Base from U.S. Army Corps of Engineers,
Chicago, Illinois, September 30, 1978

Figure 13.--Starved Rock Lock and Dam

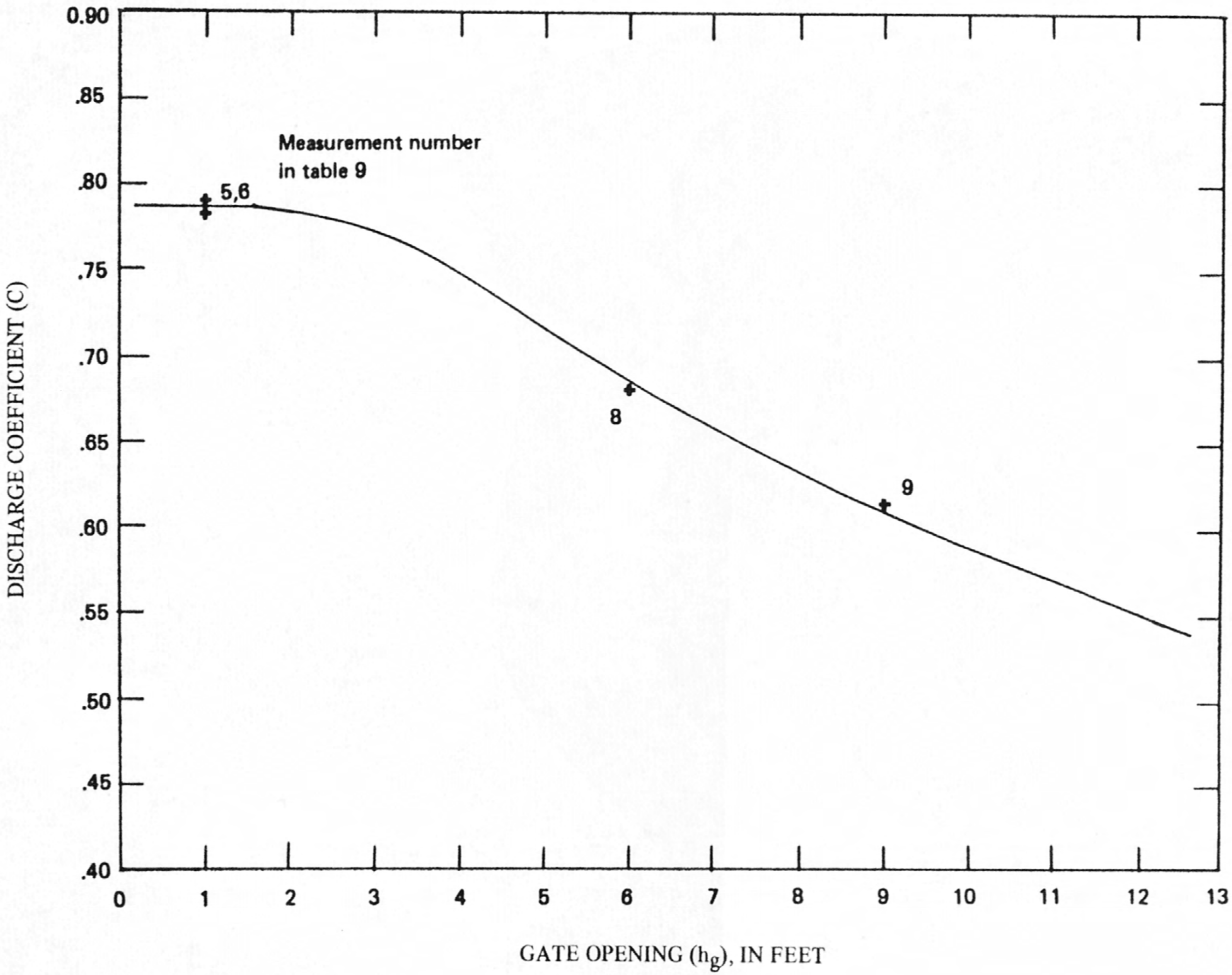


Figure 14.–Free orifice flow coefficient relation for Starved Rock Dam tainter gates.

Table 9.—Discharge measurements and hydraulic control data at Starved Rock Dam

Measurement number	Date	Pool elevation, in feet[1]		Tainter gates		Flow regime[2]	Measured discharge (ft^3/s)	Discharge coefficient
		upstream	downstream	number	opening (feet)			
1	04-19-78	459.2	450.8	7	3.0	SO	23,900	0.262
2	04-19-78	459.1	451.0	3	7.0	SO	26,400	.645
3	05-24-78	458.8	448.2	4	3.0	SO	15,700	.370
4	05-24-78	458.6	448.2	6	2.0	SO	14,400	.230
5	08-30-78	458.4	442.0	4	1.0	FO	6,220	.786
6	08-30-78	458.4	442.0	5	1.0	FO	7,810	.789
7	06-17-80	458.9	447.8	1	12.0	FW	15,200	2.88
8	06-17-80	458.8	448.0	2	6.0	FO	16,700	.681
9	06-17-80	458.9	447.7	1	9.0	FO	[3]12,900	.615
				1	1.0	SO		[4].118

[1] Elevations referenced to National Geodetic Vertical Datum of 1929.

[2] FO designates free-orifice flow; SO designates submerged-orifice flow; and FW designates free-weir flow.

[3] Combined flow through all gate openings.

[4] Coefficient for 1.0 foot is obtained from submerged-orifice flow coefficient relation.

A transition from free-orifice to submerged-orifice control was assumed to occur at a submergence ratio (h_3/h_g) of 1.5. The submerged-orifice discharge coefficient is indirectly proportional to the submergence ratio (h_3/h_g), as illustrated in figure 15. This relation is expressed mathematically as:

$$C_{gs} = 0.909 (h_3/h_g)^{-1.120} \qquad (11)$$

Figure 15, or equation 11, may be used to determine C_{gs} for a known tailwater depth and gate opening.

A transition to free-weir control occurs as the tainter gate opening is increased above 12 feet. The lower edge of the tainter gate was out of the water at a gate opening of 12.0 feet during measurement 7. Based on this measurement, the free-weir discharge coefficient (C_w) was estimated to be 2.88.

A review of the lockmaster's log for periods of abnormally high streamflow indicated that submerged-weir flow occurs periodically at Starved Rock Dam. Discharge measurements are needed at the dam when downstream pool elevations are greater than 452.0 feet so that submerged-weir discharge coefficients can be accurately determined.

Rating and Discussion

Table 10 is a stage-discharge rating for one tainter gate at Starved Rock Dam. The table was developed from equations 1, 2, and 3 (page 9) and discharge-coefficient relations determined from nine discharge measurements.

Discharges for an upstream pool elevation of 458.5 feet may be read directly from table 10. The table is partitioned into four sections, each of which represents a different flow regime. Tabulated discharges must be adjusted by a factor for each regime as headwater elevations deviate from 458.5 feet.

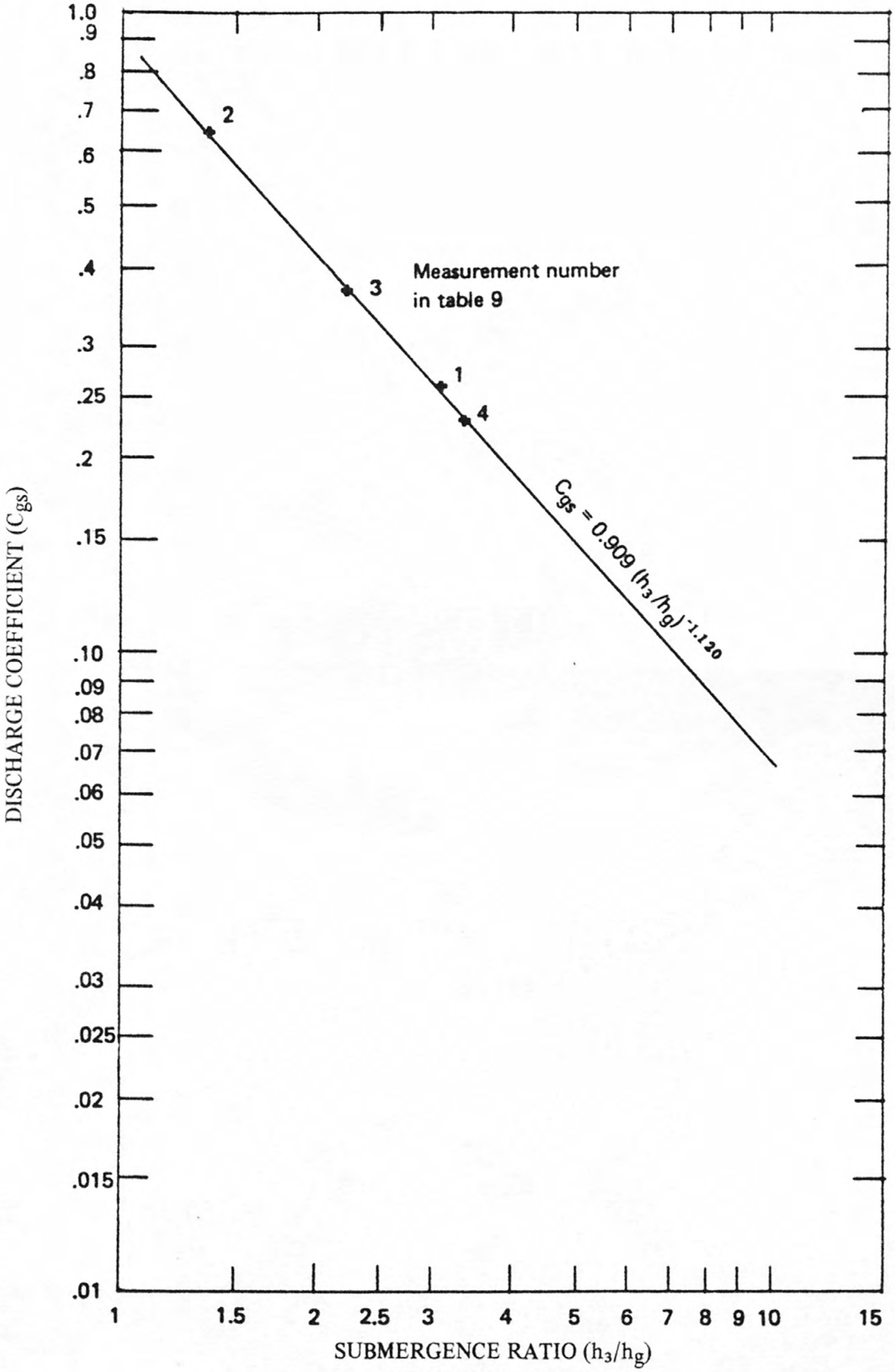


Figure 15.–Submerged orifice flow coefficient relation for Starved Rock Dam tainter gates.

Table 10.—Stage-discharge rating for one tainter gate at Starved Rock Dam and upstream pool elevation of 458.5 feet

Gate opening (feet)	Discharge, in ft^3/s, for downstream pool elevations of:								
	442	444	446	448	450	452	454	456	458
0.0	0	0	0	0	0	0	0	0	0
0.5	781	686	594	521	454	387	315	231	102
1.0	1,560	1,490	1,290	1,130	987	841	686	502	221
1.5	2,340	2,340	2,030	1,780	1,550	1,330	1,080	791	348
2.0	3,110	3,110	2,820	2,470	2,150	1,830	1,490	1,090	481
2.5	3,880	3,880	3,630	3,180	2,770	2,360	1,910	1,400	617
3.0	4,620	4,620	4,460	3,910	3,400	2,900	2,360	1,720	757
3.5	5,340	5,340	5,340	4,660	4,060	3,460	2,810	2,040	899
4.0	6,020	6,020	6,020	5,430	4,730	4,030	3,280	2,400	1,040
4.5	6,540	6,540	6,540	6,220	5,410	4,610	3,760	2,750	1,190
5.0	7,210	7,210	7,210	7,030	6,120	5,210	4,240	3,100	1,340
5.5	7,780	7,780	7,780	7,780	6,840	5,830	4,740	3,470	1,490
6.0	8,320	8,320	8,320	8,320	7,590	6,460	5,260	3,840	1,640
6.5	8,860	8,860	8,860	8,860	8,380	7,110	5,780	4,230	1,800
7.0	9,380	9,380	9,380	9,380	9,170	7,800	6,320	4,620	1,950
7.5	9,880	9,880	9,880	9,880	9,880	8,500	6,870	5,030	2,110
8.0	10,400	10,400	10,400	10,400	10,400	9,220	7,490	5,440	2,390
8.5	10,800	10,800	10,800	10,800	10,800	9,960	8,090	5,860	2,580
9.0	11,300	11,300	11,300	11,300	11,300	10,700	8,710	6,360	2,770
9.5	11,800	11,800	11,800	11,800	11,800	11,500	9,360	6,830	2,970
10.0	12,200	12,200	12,200	12,200	12,200	12,200	10,000	7,320	3,170
10.5	12,700	12,700	12,700	12,700	12,700	12,700	10,700	7,820	3,380
11.0	13,100	13,100	13,100	13,100	13,100	13,100	11,500	8,350	3,590
11.5	13,500	13,500	13,500	13,500	13,500	13,500	12,300	8,900	3,810
12.0	13,900	13,900	13,900	13,900	13,900	13,900	13,100	9,550	4,030
12.5	14,300	14,300	14,300	14,300	14,300	14,300	14,300	10,200	4,260
13.0	14,500	14,500	14,500	14,500	14,500	14,500	submerged weir		
13.5	14,500	14,500	14,500	14,500	14,500	14,500	flow		

The leftmost column of discharges in table 10 represents the free-orifice flow rating. These discharges must be multiplied by $[(HW - 441.5)/17.0]^{0.5}$ to determine the appropriate discharge under each gate from upstream pool elevations (HW) other than 458.5 feet. For example, the free-orifice discharge from an upstream pool of 459.5 feet under a gate set at 7.0 feet is determined by multiplying the rating discharge of 9,380 ft^3/s by 1.029, which equals 9,650 ft^3/s.

As the downstream pool elevation rises above the lower edge of the gate, orifice discharge under the gate will become submerged. The right section of table 10 represents the submerged orifice rating for various lower pool elevations (TW). At upstream pool elevations (HW) other than 458.5 feet, these discharges must be multiplied by $[(HW - TW)/(458.5 - TW)]^{0.5}$ to determine the appropriate discharge under each gate for a downstream pool elevation of TW feet. For example, the discharge from an upstream pool of 460.0 feet (HW) to a downstream pool elevation of 450.0 feet (TW) under a gate set at 5.0 feet is determined by multiplying the rating discharge of 6,120 ft^3/s by 1.085, which equals 6,640 ft^3/s.

The large change in discharge at the transition from free-orifice to submerged-orifice discharge shown in table 10 results from a change in equations and coefficients used to calculate discharge for each regime. The transition from free-orifice flow to submerged-orifice flow was assumed to occur at a submergence ratio of 1.5.

The discharge from an upstream pool elevation of 458.5 feet (HW) under one gate raised clear of the water will be 14,500 ft^3/s. The free-weir headwater adjustment factor is $[(HW - 441.5)/17.0]^{1.5}$. The velocity head in one forebay with 14,500 ft^3/s of water flowing in it is 2.2 feet. The total head at the approach section for elevation 458.5 feet is equal to the depth of flow, 17.0 feet, plus the velocity head of 2.2 feet. Therefore, weir flow exists at gate openings greater than 12.8 feet (0.67 x 19.2). Submerged-weir flow exists when the tailwater depth exceeds 11.5 feet (0.6 x 19.2).

Weir flow becomes submerged as the downstream pool rises above elevation 453.0 feet. The lower right section of table 10 is left blank because submerged-weir flow was not measured, and C_{WS} (equation 4) could not be determined.

PEORIA DAM

Peoria Dam is the second navigation dam on the Illinois River, 158 miles upstream from the mouth of the Illinois River. The lock and dam are about 4 river miles downstream from Peoria.

Peoria Dam is a navigable-Chanoine wicket dam (fig. 16). The dam is operated to maintain an upstream pool elevation of approximately 440.0 feet. This elevation corresponds to the raised wicket crest elevation and provides sufficient depth for a maximum barge draft of 9.0 feet in the channel upstream from the dam.

Streamflow is regulated by completely raising various numbers of the 134 wickets that compose the dam. Each wicket is 3.75 feet wide, 16.42 feet high, 1 foot thick, and forms a 20 degree angle from vertical when in an upright position.

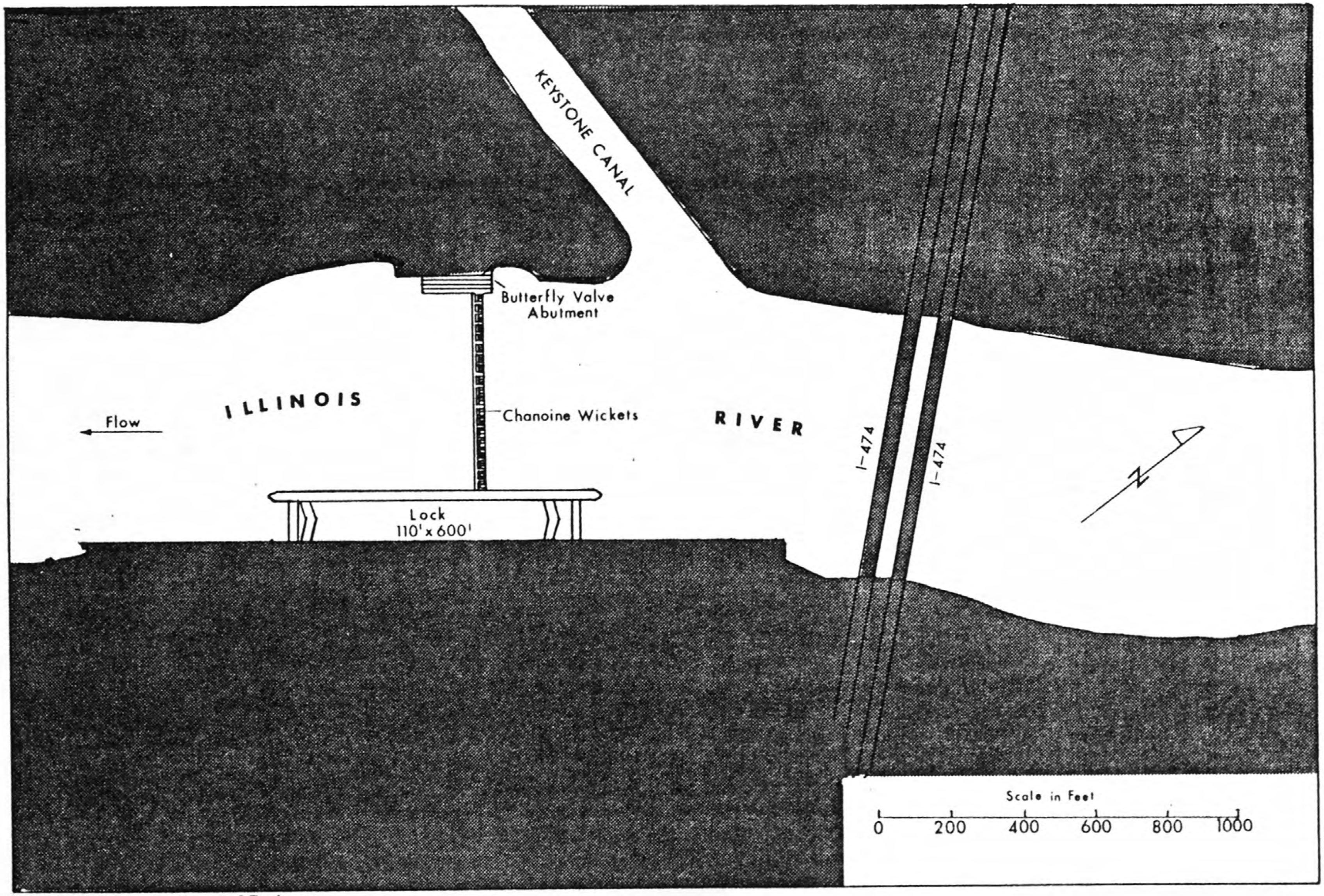
Any number of six butterfly valves may also be completely opened to provide additional regulation. The net cross sectional area of each valve when fully open is 29.4 square feet (ft^2), and the net area of the trashrack across the single inlet is 157 ft^2. Therefore, the trashrack is the control when six valves are open.

Data Collection and Analysis

A gaging station (05568500) is maintained on the Illinois River near Kingston Mines. The gaging station is 13.4 miles downstream from Peoria Dam (fig. 1). The drainage area of the Illinois River at Peoria Dam is 14,550 mi^2, 92 percent of the 15,819 mi^2 drainage area at the Kingston Mines gage. A gaging station (05567500) is also maintained on the Mackinaw River near Congerville. The Mackinaw River contributes 90 percent of the intervening drainage area between the Peoria Dam and the Kingston Mines gaging station.

Discharge measurements are routinely made on the Illinois River at the Kingston Mines gage to verify the stage-fall-discharge relation. Rather than introduce additional uncertainty into the rating analysis by attempting to transfer gage data to the dam site, the analysis was based on measurements at the gaging station.

Eight discharge measurements were used in the Peoria Dam rating analysis. The purpose of these measurements was to verify the discharge-coefficient relation for flow between raised wickets that was determined by a hydraulic model study (U.S. Army Corps of Engineers, 1938). The measurements were also used to determine discharge coefficients for submerged-orifice flow through a butterfly valve (equation 6, page 10). A combination of lowered wickets and (or) open valves was fixed during each discharge measurement, so that steady flow prevailed.



Base from U.S. Army Corps of Engineers,
Chicago, Illinois, June 30, 1970

Figure 16.–Peoria Lock and Dam

Hydraulic control data such as the numbers of lowered wickets, open valves, and needles in place were obtained from the lockmaster's log. The lockmaster also logs daily staff gage readings (at 8:00 a.m.) of upstream and downstream pool elevations at the dam.

Average headwater and tailwater elevations for a particular day were calculated from equation 12, a weighting of readings for three consecutive days

$$\overline{E}_i = 0.125\ E_{i-1} + 0.500\ E_i + 0.375\ E_{i+1} \tag{12}$$

where

$\overline{E}_i$ = average headwater (tailwater) elevation during day i.

E_i = headwater (tailwater) elevation at 8:00 a.m. on day i.

E_{i-1} = headwater (tailwater) elevation at 8:00 a.m. on preceding day.

E_{i+1} = headwater (tailwater) elevation at 8:00 a.m. on following day.

Headwater and tailwater depths at the dam were calculated by subtracting the sill elevation of 424.6 feet from the average headwater and tailwater elevation.

Measured discharges ranged from 3,570 to 12,500 ft^3/s. Hydraulic control data and average pool elevations for each measurement are summarized in table 11. The mean daily discharges shown for comparison in table 11 were reported in the water resources data reports for Illinois, water years 1975-78 (U.S. Geological Survey, 1976-79).

Measurements 2, 4, and 7 indicated the wicket discharge coefficients for equation 7 (page 10) determined by the Corps hydraulic model study were reasonable. The discharge-coefficient relations for wicket-gap discharge and flow over lowered wickets are illustrated in figure 17. The relations are applicable for openings between 0.25 and 80.0 feet.

Flow through openings wider than 80.0 feet is contracted less as the head differential (Δh) decreases. Equations of flow through a contracted opening, presented by Matthai (1976), may be used to study these conditions.

A review of the lockmaster's log for the years 1975 through 1978 showed that the dam is infrequently operated with more than 15 wickets lowered. Streamflow through these intermediate openings is deep enough to allow navigation over the lowered dam. Historical data for the Kingston Mines gage indicate that when all wickets are lowered and the water surface elevation at the dam is 439 feet the discharge is about 14,000 ft^3/s. For a water surface elevation of 440 feet, the discharge is about 16,000 ft^3/s.

The submerged-orifice valve coefficient was estimated by deducting wicket related discharge, calculated from equations 7 and 8 (pages 10 and 11), from the measured discharge. The submerged-orifice coefficient (C_{so}) is estimated to be 0.4, based on measurements 1, 3, 5, 6, and 8.

The laboratory determined value of the wicket weir discharge coefficient (C_{sw}) was impossible to verify because all the gaps between wickets have never been sealed off by needles at one time. The laboratory determined value of 3.5 is consistent with coefficients presented by King and Brater (1963) and was used in the calculation of wicket weir flow for measurements 1 through 6.

The total discharges calculated from equations 6, 7, and 8 using the coefficients presented above are shown in the last column of table 11. Differences between calculated and measured discharges ranged from ±15 percent for measurements 2 and 4 to less than 3 percent for the six remaining measurements. A large part of these differences may be because of the method used to compute headwater and tailwater depths and to the distance between the dam and the discharge measuring site.

Table 11.–Discharge measurements and hydraulic control data at Peoria Dam

Measurement number	Date	Pool elevation, in feet[1]		Wickets lowered	Valves open	Needles placed	Illinois River discharge (ft^3/s)		
							Kingston Mines gage		Peoria Dam
		upstream	downstream				Instantaneous	Mean daily	Calculated
1	09-28-76	440.6	431.2	0	6	95	5,000	5,200	4,990
2	11-18-76	440.4	430.8	0	0	74	3,570	3,800	4,120
3	03-16-77	440.4	436.1	5	6	0	11,200	11,300	11,300
4	06-06-77	440.3	430.6	0	0	70	4,880	4,900	4,160
5	06-15-77	440.4	432.6	0	5	56	6,310	6,370	6,370
6	07-27-78	440.6	436.4	7	6	0	12,500	13,200	12,800
7	09-26-78	439.9	435.6	7	0	0	10,300	10,800	10,400
8	01-18-80	440.0	433.5	0	6	0	8,600	9,240	8,700

[1] Elevations referenced to National Geodetic Vertical Datum of 1929.

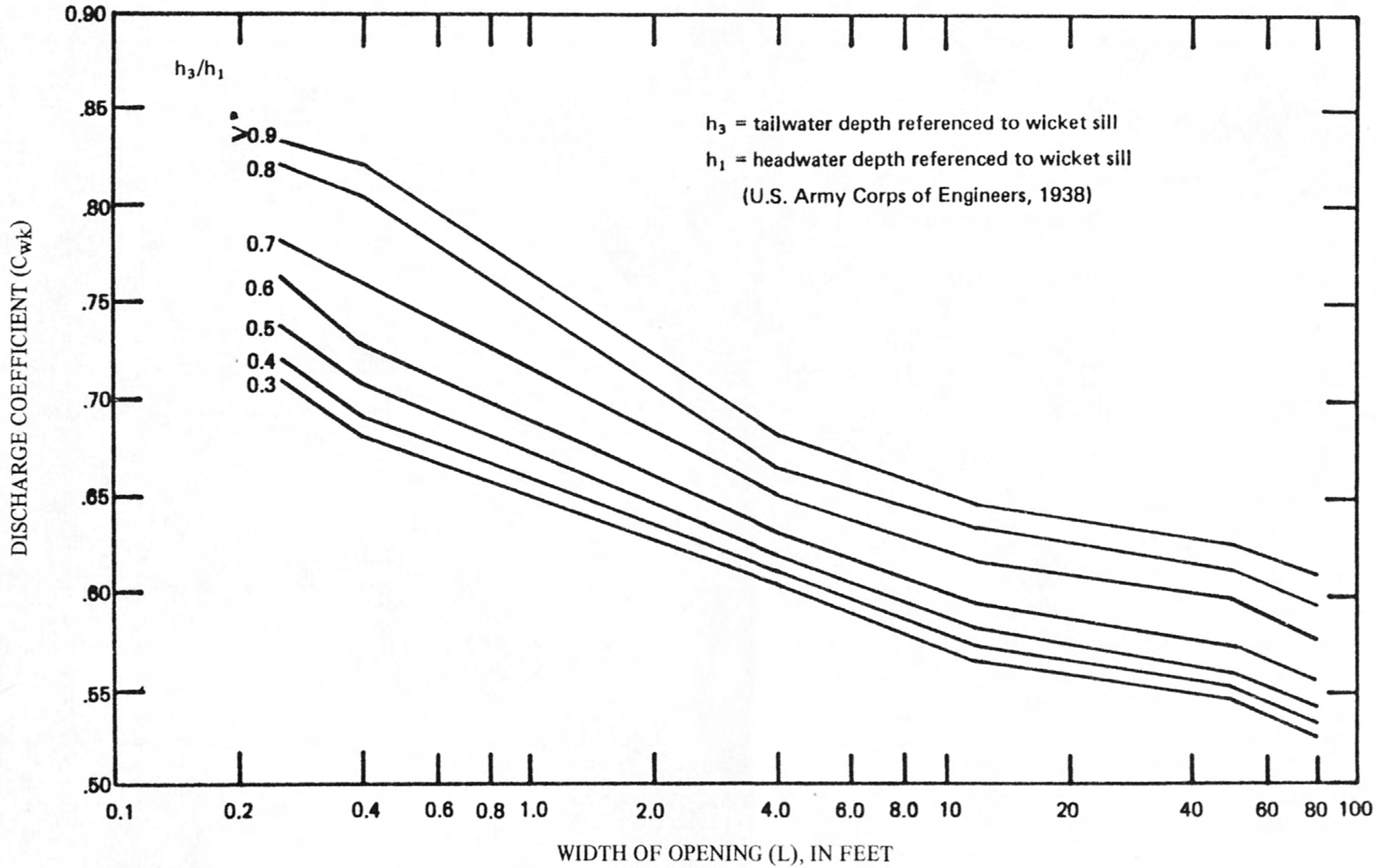


Figure 17.–Discharge coefficient relations for wicket control at Peoria Dam.

Table 12 is a stage-discharge rating over the crest of 134 raised wickets, each 3.75 feet wide. These discharges were calculated from equation 8 (page 11) through the use of a discharge coefficient (C_{SW}) of 3.5. The tabulated discharges must be decreased by the factor WD/134, where WD is the number of lowered wickets.

Table 12.—Weir discharge rating for 134 wickets at Peoria Dam

Headwater elevation (feet)	Discharge, in ft³/s, at incremental elevations of: 0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
440	0	56	157	289	445	622	817	1,030	1,260	1,500
441	1,760	2,030	2,310	2,610	2,910	3,230	–	–	–	–

Table 13 is a discharge rating for one butterfly valve. Discharge is presented as a function of tailwater elevation and head differential (Δh). Head differential should be used to find the tabulated submerged-orifice discharge for upstream pool elevations other than 440.0 feet. Equations 6 and a submerged-orifice coefficient (C_{SO}) of 0.4 were used to calculate the rating discharges.

Table 13.—Stage-discharge rating for one fully open butterfly valve at Peoria Dam

Tailwater[1] elevation (feet)	Δh[2] (feet)	Discharge (ft³/s)	Tailwater[1] elevation (feet)	Δh[2] (feet)	Discharge (ft³/s)
430.0	10.0	298	435.0	5.0	211
430.5	9.5	291	435.5	4.5	200
431.0	9.0	283	436.0	4.0	189
431.5	8.5	275	436.5	3.5	177
432.0	8.0	267	437.0	3.0	163
432.5	7.5	258	437.5	2.5	149
433.0	7.0	250	438.0	2.0	133
433.5	6.5	241	438.5	1.5	116
434.0	6.0	231	439.0	1.0	94.4
434.5	5.5	221	439.5	0.5	66.7

[1] Elevations referenced to National Geodetic Vertical Datum of 1929.

[2] Use head differential (Δh) for upper pool elevations different than 440.0 feet.

Tables 14, 15, and 16 are stage-discharge ratings of combined discharge through wicket gaps, over raised wickets, and over lowered wickets for upper pool elevations of 439.5, 440.0, and 440.5 feet. The first column of each table represents the wicket-gap discharge rating with no needles in place.

Table 14.—Stage-discharge rating of wicket control at Peoria Dam for an upstream pool elevation of 439.5 feet with no needles in place

Downstream pool elevation (feet)	Number of lowered wickets								
	0	1	2	3	4	5	7	10	15
	Discharge, in cubic feet per second								
430.0	7,530	8,200	8,830	9,430	10,100	10,700	11,900	13,700	16,700
430.5	7,500	8,170	8,800	9,400	10,000	10,600	11,900	13,700	16,700
431.0	7,430	8,100	8,720	9,310	9,930	10,500	11,800	13,600	16,400
431.5	7,340	8,000	8,620	9,200	9,810	10,400	11,600	13,400	16,300
432.0	7,240	7,890	8,500	9,080	9,680	10,300	11,500	13,200	16,000
432.5	7,150	7,790	8,380	8,960	9,540	10,100	11,200	13,100	15,900
433.0	7,050	7,670	8,260	8,820	9,400	9,980	11,100	12,700	15,700
433.5	6,910	7,530	8,100	8,660	9,220	9,770	10,900	12,500	15,400
434.0	6,770	7,370	7,930	8,470	9,030	9,570	10,600	12,300	15,100
434.5	6,590	7,180	7,730	8,260	8,800	9,340	10,400	12,000	14,800
435.0	6,390	6,960	7,500	8,010	8,530	9,060	10,100	11,600	14,400
435.5	6,200	6,750	7,270	7,760	8,260	8,780	9,780	11,300	13,900
436.0	5,970	6,490	6,990	7,460	7,950	8,430	9,400	10,800	13,200
436.5	5,680	6,180	6,650	7,100	7,550	8,020	8,940	10,300	12,600
437.0	5,280	5,740	6,170	6,590	7,010	7,440	8,290	9,560	11,700
437.5	4,800	5,220	5,610	5,990	6,390	6,770	7,550	8,690	10,700
438.0	4,220	4,580	4,940	5,270	5,610	5,950	6,640	7,660	9,390
438.5	3,480	3,790	4,080	4,350	4,640	4,930	5,480	6,320	7,760
439.0	2,490	2,710	2,920	3,120	3,320	3,520	3,920	4,520	5,550

Table 15.—Stage-discharge rating of wicket control at Peoria Dam for an upstream pool elevation of 440.0 feet with no needles in place

Downstream pool elevation (feet)	Number of lowered wickets								
	0	1	2	3	4	5	7	10	15
	Discharge, in cubic feet per second								
430.0	7,920	8,630	9,290	9,930	10,600	11,200	12,500	14,400	17,600
430.5	7,900	8,610	9,270	9,900	10,600	11,200	12,500	14,400	17,500
431.0	7,850	8,550	9,200	9,830	10,500	11,100	12,400	14,300	17,400
431.5	7,760	8,450	9,100	9,730	10,400	11,000	12,300	14,100	17,300
432.0	7,660	8,340	8,990	9,620	10,300	10,900	12,100	14,000	17,100
432.5	7,570	8,240	8,880	9,490	10,100	10,700	12,000	13,800	16,900
433.0	7,460	8,130	8,750	9,340	9,960	10,600	11,800	13,600	16,700
433.5	7,350	8,000	8,620	9,200	9,800	10,400	11,600	13,400	16,300
434.0	7,210	7,850	8,440	9,020	9,620	10,200	11,300	13,100	16,100
434.5	7,050	7,680	8,260	8,830	9,400	9,980	11,100	12,800	15,700
435.0	6,860	7,470	8,050	8,590	9,160	9,710	10,800	12,400	15,400
435.5	6,650	7,250	7,810	8,340	8,880	9,430	10,500	12,100	14,900
436.0	6,450	7,020	7,570	8,080	8,610	9,130	10,200	11,700	14,400
436.5	6,210	6,750	7,260	7,760	8,260	8,770	9,760	11,300	13,800
437.0	5,900	6,410	6,900	7,370	7,850	8,330	9,270	10,700	13,000
437.5	5,470	5,950	6,400	6,830	7,280	7,720	8,600	9,910	12,100
438.0	4,970	5,400	5,810	6,200	6,610	7,010	7,810	9,000	11,100
438.5	4,370	4,740	5,110	5,460	5,810	6,170	6,870	7,920	9,720
439.0	3,600	3,920	4,220	4,500	4,800	5,090	5,680	6,540	8,030

Table 16.—Stage-discharge rating of wicket control at Peoria Dam for an upstream pool elevation of 440.5 feet with no needles in place

Downstream pool elevation (feet)	Number of lowered wickets								
	0	1	2	3	4	5	7	10	15
	Discharge, in cubic feet per second								
430.0	8,940	9,670	10,300	11,000	11,700	12,400	13,700	15,800	19,000
430.5	8,920	9,650	10,300	11,000	11,700	12,400	13,600	15,700	18,900
431.0	8,880	9,610	10,200	11,000	11,600	12,300	13,600	15,600	18,900
431.5	8,800	9,520	10,200	10,900	11,500	12,200	13,500	15,500	18,700
432.0	8,720	9,430	10,100	10,800	11,400	12,100	13,300	15,300	18,400
432.5	8,610	9,310	9,970	10,600	11,200	12,000	13,200	15,200	18,200
433.0	8,510	9,200	9,860	10,500	11,100	11,800	13,000	14,900	18,000
433.5	8,400	9,080	9,730	10,400	11,000	11,700	12,800	14,700	17,900
434.0	8,270	8,940	9,570	10,200	10,800	11,400	12,600	14,400	17,600
434.5	8,130	8,780	9,410	9,990	10,600	11,200	12,400	14,200	17,300
435.0	7,950	8,600	9,200	9,780	10,400	10,900	12,100	13,900	17,000
435.5	7,750	8,380	8,970	9,540	10,100	10,700	11,800	13,500	16,500
436.0	7,550	8,150	8,730	9,270	9,850	10,400	11,500	13,200	16,100
436.5	7,330	7,910	8,470	9,000	9,540	10,100	11,200	12,800	15,400
437.0	7,070	7,620	8,150	8,660	9,180	9,700	10,700	12,300	14,800
437.5	6,730	7,250	7,750	8,230	8,720	9,210	10,200	11,700	14,000
438.0	6,280	6,760	7,230	7,670	8,120	8,580	9,480	10,800	13,100
438.5	5,760	6,190	6,620	7,010	7,430	7,840	8,660	9,880	11,900
439.0	5,140	5,510	5,880	6,230	6,610	6,960	7,680	8,750	10,600

Wicket gap discharge is indirectly proportional to the number of needles in place. The discharge found in the first column of tables 14 and 15 must be multiplied by P/135, where P is the number of needles in place, to calculate the amount of flow reduction achieved by placing the needles. Wicket-weir discharge, the flow over the crest of the raised wickets, must first be deducted from the discharges in table 16 before multiplying by P/135. For a fully raised dam, 622 ft^3/s (table 12) must be subtracted. If 10 wickets are lowered, the wicket-weir flow portion of the eighth column of discharges in table 16 is 576 ft^3/s [622 x (1 - WD/134)].

Tables 14, 15, and 16 were developed from equations 7 and 8 (pages 10 and 11) and the discharge-coefficient relations illustrated in figure 17. Discharges for combinations of upstream and downstream pool elevations not presented in these tables may be calculated from interpolation or from equations 7 and 8.

LA GRANGE DAM

La Grange Dam is the first navigation dam on the Illinois River, 80.2 miles upstream from the mouth. The lock and dam are about 8 miles downstream from Beardstown.

La Grange Dam is a navigable Chanoine wicket dam (fig. 18). The dam is operated to maintain an upstream pool elevation of 429.0 feet. This elevation corresponds to the raised wicket crest elevation and provides sufficient depth for a maximum barge draft of 9.0 feet in the channel upstream from the dam.

Streamflow is regulated by completely raising various numbers of the 135 wickets that compose the dam. Each wicket is 3.75 feet wide, 14.92 feet high, 1 foot thick, and forms a 20 degree angle from vertical when in an upright position.

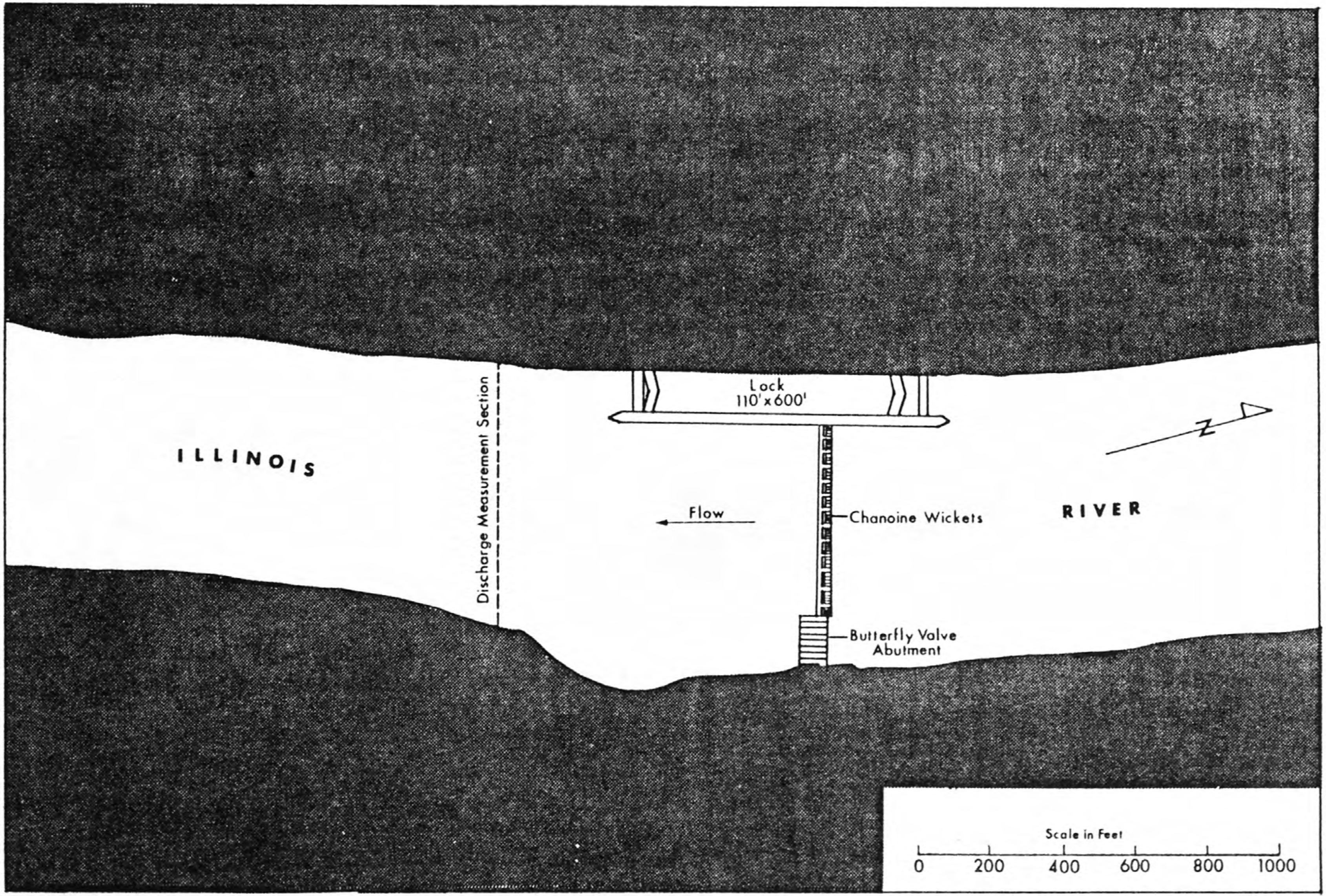
Any number of 12 butterfly valves may also be completely opened to provide additional regulation. The net cross-sectional area of each valve is 29.4 ft^2. The outlet of each valve becomes submerged as the downstream pool rises above an elevation of 423.0 feet.

Data Collection and Analysis

A gaging station (05585500) is maintained on the Illinois River at Meredosia. The gaging station is 8.9 miles downstream from La Grange Dam. The drainage area of the river at La Grange Dam is 25,648 mi^2, 99 percent of the 26,028 mi^2 drainage area at Meredosia.

Discharge measurements are routinely made on the Illinois River at Meredosia to verify the stage-fall-discharge relation at the gaging station. Rather than introduce additional uncertainty into the rating analysis by attempting to transfer Meredosia gage data to the dam site, the analysis for this study was based on measurements made at the gaging station. One additional measurement was made at the dam.

Six discharge measurements were used in the rating analysis. The purpose of these measurements was to verify the discharge-coefficient relation for flow between raised wickets that was determined by a hydraulic model study (U.S. Army Corps of Engineers, 1938). The measurements were also used to determine discharge coefficients for free- and submerged-orifice flow through a butterfly valve (equations 5 and 6, page 10). A combination of lowered wickets and (or) open valves was fixed during each discharge measurement to ensure steady flow.



Base from U.S. Army Corps of Engineers,
Chicago, Illinois, June 30, 1965

Figure 18.–La Grange Lock and Dam

Hydraulic control data such as the numbers of lowered wickets, open valves, and needles in place were obtained from the lockmaster's log. The lockmaster also logs daily staff gage readings (at 8:00 a.m.) of upstream and downstream pool elevations at the dam. Average headwater and tailwater elevations for a particular day were calculated from equation 12 (page 36), a temporal weighting of readings for 3 consecutive days. Headwater and tailwater depths at the dam were calculated by subtracting the sill elevation of 415.0 feet from the average headwater and tailwater elevation.

Measured discharges ranged from 4,160 to 16,200 ft^3/s. Hydraulic control data and average pool elevations for each measurement are summarized in table 17. Measurement 6 was made about a quarter of a mile below the dam. The remaining measurements were made at the Meredosia gage. The mean daily discharges shown for comparison on table 17 were reported in the water resources data reports for Illinois, water years 1975-78 (U.S. Geological Survey, 1976-79).

Measurements 4 and 6 indicated the wicket-discharge coefficients determined by the Corps hydraulic model study were reasonable. Part of the 10 percent difference between the measured discharge for measurement 4 and the discharge calculated from equation 8 is probably due to leakage around the 62 needles that were in place.

The discharge-coefficient relations for wicket-gap discharge and flow over lowered wickets are illustrated in figure 19. The relations are applicable for openings between 0.25 and 80.0 feet. Flow through openings wider than 80.0 feet is contracted less as the head differential (Δh) decreases. Equations of flow through a contracted opening, presented by Matthai (1976), may be used to study these conditions.

A review of the lockmaster's log for the years 1975 through 1978 showed that the dam is infrequently operated with more than 20 wickets lowered. Streamflow through these intermediate openings is deep enough to allow navigation over the lowered dam. Historical data for the Meredosia gage indicate that when all wickets are lowered and the water-surface elevation at the dam is 428 feet, the discharge is about 22,000 ft^3/s. For a water-surface elevation of 429 feet, the discharge is about 25,000 ft^3/s.

The free-orifice and submerged-orifice valve coefficients were estimated by deducting wicket related discharge, calculated from equations 7 and 8 (pages 10 and 11) from the measured discharge. The free-orifice coefficient (C_{fo}) is estimated to be 0.5, based on measurements 1 and 3. Based on measurement 5, the submerged-orifice coefficient (C_{so}) is estimated to be 0.6. Equations 5 and 6 (page 10) were used in estimating these coefficients.

The laboratory determined value of the wicket weir discharge coefficient (C_{sw}) was impossible to verify because all the gaps between wickets have never been sealed off by needles at one time. The laboratory determined value of 3.5 is consistent with coefficients presented by King and Brater (1963) and was used in the calculation of wicket weir flow for measurements 3, 5, and 6.

Ratings and Discussion

Table 18 is a stage-discharge rating over the crest of 135 raised wickets, each 3.75 feet wide. These discharges were calculated from equation 8 for a discharge coefficiet (C_{sw}) of 3.5. The tabulated discharges must be decreased by the factor WD/135, where WD is the number of lowered wickets.

Table 17.—Discharge measurements and hydraulic control data at La Grange Dam

Measurement number	Date	Pool elevation, in feet[1]		Wickets lowered	Valves open	Needles placed	Illinois River discharge (ft^3/s)		
							Meredosia gage		La Grange Dam
		upstream	downstream				Instantaneous	Mean daily	Calculated
1	08-21-75	429.0	422.0	0	10	0	9,680	9,570	10,200
2	10-22-75	429.0	420.8	0	0	0	6,540	6,540	6,790
3	07-13-76	429.1	422.7	4	6	0	11,100	11,000	11,000
4	09-23-76	429.0	419.5	0	0	62	4,160	3,930	3,770
5	08-03-78	429.1	425.1	20	6	15	16,200	16,000	16,200
6	01-23-80	429.2	421.7	5	0	0	9,860	10,300	9,940

[1] Elevations referenced to National Geodetic Vertical Datum of 1929.

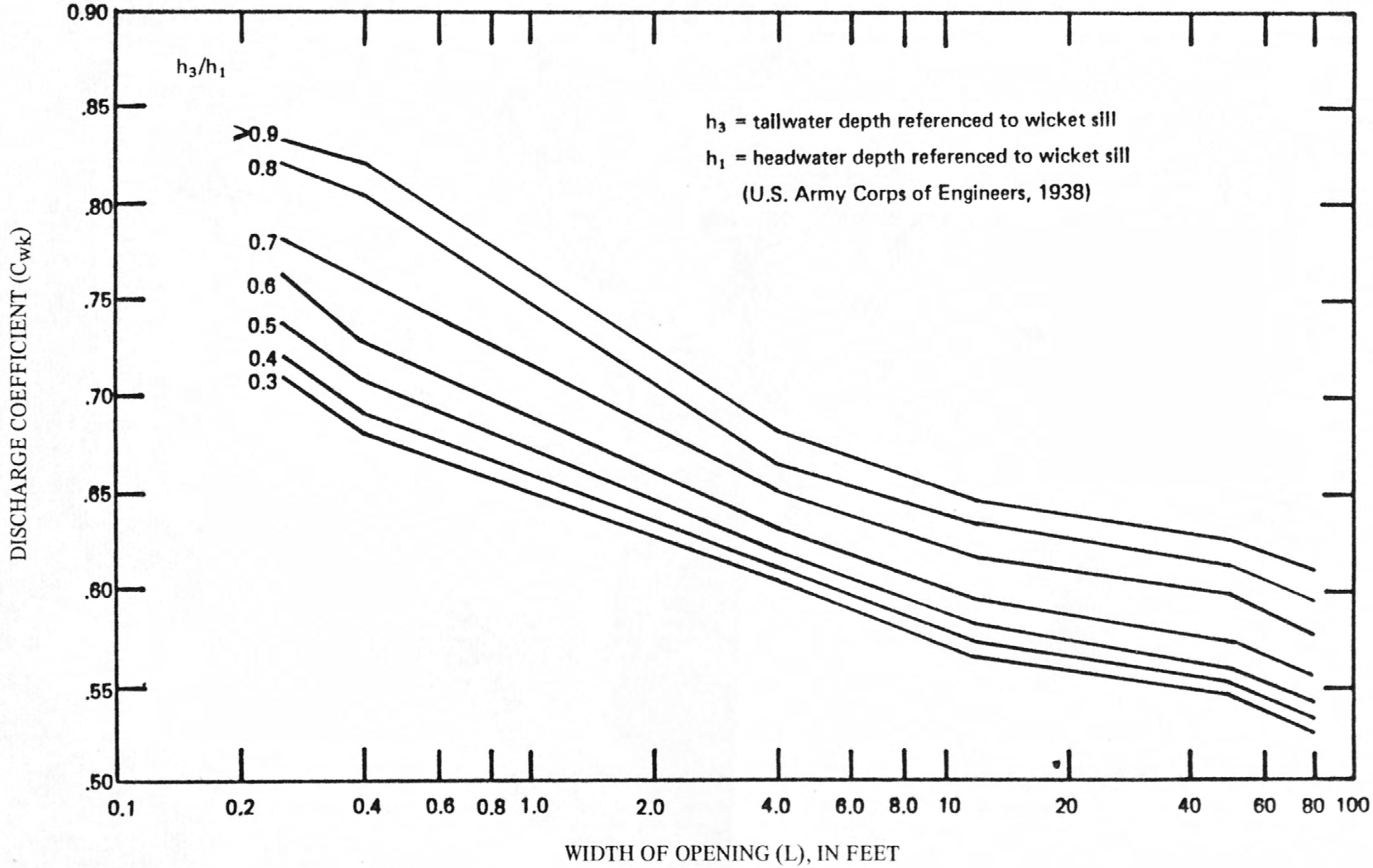


Figure 19.–Discharge coefficient relations for wicket control at La Grange Dam.

Table 18.—Weir discharge rating for 135 wickets at La Grange Dam

Headwater elevation (feet)	Discharge, in ft^3/s, at incremental elevations of: 0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
429	0	56	158	291	448	626	823	1,040	1,270	1,510
430	1,770	2,040	2,330	2,630	2,940	3,260	–	–	–	–

Table 19 is a discharge rating for one butterfly valve. Discharge is presented as a function of tailwater elevation and head differential (Δh). Head differential should be used to find the tabulated submerged-orifice discharge for upstream pool elevations other than 429.0 feet. A free-orifice valve coefficient (C_{fo}) of 0.5 and equation 5 (page 10) were used to compute the rating for tailwater elevations less than or equal to 423.0 feet.

Table 19.—Stage-discharge rating for one fully open butterfly valve at La Grange Dam

Tailwater[1] elevation (feet)	Δh[2] (feet)	Discharge (ft^3/s)	Tailwater[1] elevation (feet)	Δh[2] (feet)	Discharge (ft^3/s)
418.0	–	354	424.0	5.0	317
418.5	–	354	424.5	4.5	300
419.0	–	354	425.0	4.0	283
419.5	–	354	425.5	3.5	265
420.0	–	354	426.0	3.0	245
420.5	–	354	426.5	2.5	224
421.0	–	354	427.0	2.0	200
421.5	–	354	427.5	1.5	173
422.0	–	354	428.0	1.0	142
422.5	–	354	428.5	0.5	100
423.0	6.0	354	429.0	0.0	0
423.5	5.5	332			

[1] Elevations referenced to National Geodetic Vertical Datum of 1929.
[2] Use head differential (Δh) for upper pool elevations different than 429.0 feet.

Tables 20, 21, and 22 are stage-discharge ratings of combined discharge through spaces between raised wickets, over raised wickets, and over lowered wickets for upstream pool elevations of 428.5, 429.0, and 429.5 feet. The first column of each table represents the wicket-gap discharge rating.

Wicket-gap discharge is indirectly proportional to the number of needles in place. As P needles are placed, the discharge found in the first column of tables 20 and 21 must be multiplied by P/136 to calculate the amount of flow reduction achieved by placing the needles. Wicket-weir discharge, the flow over the crest of the raised wickets, must

Table 20.—Stage-discharge rating of wicket control at La Grange Dam for an upstream pool elevation of 428.5 feet with no needles in place

Downstream pool elevation (feet)	Number of lowered wickets									
	0	1	2	3	4	5	7	10	15	20
	Discharge, in cubic feet per second									
418.0	6,600	7,210	7,770	8,300	8,860	9,400	10,400	12,200	14,700	17,100
418.5	6,590	7,190	7,740	8,270	8,820	9,360	10,400	12,000	14,600	17,000
419.0	6,570	7,160	7,710	8,230	8,770	9,300	10,300	11,900	14,500	16,900
419.5	6,560	7,150	7,690	8,210	8,750	9,280	10,300	11,900	14,500	16,900
420.0	6,540	7,130	7,670	8,180	8,720	9,240	10,300	11,800	14,400	16,800
420.5	6,500	7,090	7,620	8,140	8,660	9,190	10,200	11,800	14,400	16,700
421.0	6,420	6,990	7,530	8,040	8,560	9,080	10,100	11,600	14,200	16,600
421.5	6,330	6,890	7,420	7,930	8,450	8,960	9,980	11,500	14,000	16,300
422.0	6,240	6,790	7,310	7,810	8,320	8,830	9,840	11,300	13,900	16,100
422.5	6,140	6,680	7,190	7,680	8,180	8,680	9,660	11,100	13,700	15,900
423.0	6,030	6,560	7,050	7,530	8,020	8,510	9,470	10,900	13,400	15,700
423.5	5,890	6,400	6,890	7,360	7,840	8,310	9,250	10,700	13,100	15,400
424.0	5,710	6,220	6,770	7,150	7,610	8,080	9,000	10,400	12,700	15,000
424.5	5,520	6,010	6,470	6,900	7,350	7,810	8,690	10,000	12,300	14,400
425.0	5,330	5,800	6,250	6,660	7,100	7,530	8,380	9,660	11,900	13,800
425.5	5,100	5,540	5,960	6,360	6,770	7,190	8,010	9,220	11,300	13,200
426.0	4,770	5,190	5,580	5,950	6,330	6,720	7,480	8,620	10,500	12,300
426.5	4,350	4,720	5,080	5,420	5,780	6,130	6,820	7,850	9,610	11,200
427.0	3,830	4,160	4,480	4,780	5,100	5,400	6,010	6,930	8,490	9,930
427.5	3,170	3,450	3,710	3,960	4,210	4,470	4,980	5,730	7,030	8,220
428.0	2,270	2,470	2,650	2,830	3,010	3,210	3,570	4,110	5,040	5,890

Table 21.–Stage-discharge rating of wicket control at La Grange Dam for an upstream pool elevation of 429.0 feet with no needles in place

Downstream pool elevation (feet)	Number of lowered wickets									
	0	1	2	3	4	5	7	10	15	20
	Discharge, in cubic feet per second									
418.0	6,980	7,610	8,200	8,770	9,350	9,930	11,000	12,700	15,500	18,100
418.5	6,960	7,600	8,180	8,750	9,320	9,900	11,000	12,700	15,400	18,000
419.0	6,940	7,570	8,150	8,710	9,280	9,850	10,900	12,600	15,300	17,900
419.5	6,920	7,550	8,120	8,680	9,250	9,800	10,900	12,600	15,300	17,800
420.0	6,910	7,530	8,100	8,660	9,220	9,780	10,900	12,500	15,200	17,800
420.5	6,890	7,510	8,070	8,620	9,190	9,740	10,800	12,400	15,200	17,800
421.0	6,820	7,430	7,990	8,530	9,090	9,650	10,700	12,300	15,100	17,500
421.5	6,730	7,330	7,890	8,420	8,980	9,530	10,600	12,200	14,900	17,300
422.0	6,630	7,230	7,770	8,300	8,850	9,390	10,400	12,000	14,700	17,100
422.5	6,540	7,130	7,670	8,190	8,720	9,250	10,300	11,800	14,500	16,900
423.0	6,440	7,000	7,540	8,050	8,570	9,090	10,100	11,600	14,300	16,700
423.5	6,300	6,860	7,370	7,870	8,390	8,900	9,910	11,400	14,100	16,400
424.0	6,160	6,700	7,210	7,690	8,190	8,690	9,680	11,100	13,800	16,100
424.5	5,970	6,500	7,000	7,470	7,960	8,450	9,410	10,800	13,300	15,600
425.0	5,780	6,290	6,780	7,230	7,710	8,170	9,100	10,500	12,900	15,100
425.5	5,580	6,060	6,530	6,960	7,420	7,870	8,760	10,100	12,400	14,400
426.0	5,320	5,780	6,210	6,640	7,070	7,500	8,340	9,620	11,800	13,700
426.5	4,970	5,400	5,800	6,190	6,600	6,990	7,790	8,970	11,000	12,800
427.0	4,520	4,910	5,290	5,640	6,010	6,370	7,100	8,170	9,990	11,700
427.5	3,980	4,320	4,650	4,970	5,290	5,610	6,250	7,190	8,820	10,300
428.0	3,290	3,580	3,850	4,110	4,380	4,640	5,160	5,960	7,300	8,540

Table 22.–Stage-discharge rating of wicket control at La Grange Dam for an upstream pool elevation of 429.5 feet with no needles in place

Downstream pool elevation (feet)	Number of lowered wickets									
	0	1	2	3	4	5	7	10	15	20
	Discharge, in cubic feet per second									
418.0	7,990	8,650	9,280	9,860	10,500	11,000	12,300	14,000	16,900	19,700
418.5	7,970	8,630	9,250	9,830	10,400	11,000	12,200	14,000	16,800	19,600
419.0	7,960	8,610	9,210	9,800	10,400	11,000	12,200	13,900	16,800	19,500
419.5	7,930	8,580	9,170	9,760	10,300	10,900	12,100	13,800	16,700	19,300
420.0	7,920	8,560	9,160	9,740	10,300	10,900	12,100	13,800	16,600	19,300
420.5	7,900	8,540	9,130	9,710	10,300	10,800	12,000	13,700	16,600	19,200
421.0	7,850	8,480	9,070	9,640	10,200	10,800	11,900	13,600	16,500	19,100
421.5	7,770	8,400	8,980	9,550	10,100	10,700	11,800	13,500	16,400	18,900
422.0	7,670	8,290	8,870	9,420	10,000	10,500	11,700	13,400	16,200	18,700
422.5	7,570	8,170	8,750	9,300	9,860	10,400	11,500	13,200	16,000	18,500
423.0	7,480	8,060	8,630	9,170	9,720	10,200	11,400	13,000	15,800	18,300
423.5	7,360	7,930	8,490	9,020	9,560	10,100	11,200	12,800	15,600	18,000
424.0	7,220	7,780	8,320	8,850	9,370	9,900	11,000	12,500	15,200	17,700
424.5	7,050	7,610	8,130	8,630	9,160	9,670	10,700	12,200	14,800	17,300
425.0	6,860	7,400	7,910	8,400	8,900	9,400	10,400	11,900	14,400	16,800
425.5	6,670	7,190	7,690	8,160	8,650	9,130	10,100	11,600	14,000	16,200
426.0	6,440	6,940	7,420	7,860	8,330	8,800	9,720	11,100	13,400	15,600
426.5	6,180	6,640	7,090	7,520	7,970	8,400	9,290	10,600	12,800	14,800
427.0	5,790	6,220	6,640	7,040	7,460	7,860	8,680	9,900	11,900	13,800
427.5	5,320	5,710	6,090	6,460	6,830	7,200	7,950	9,040	10,900	12,600
428.0	4,760	5,110	5,440	5,760	6,090	6,420	7,070	8,040	9,700	11,200

first be deducted from the discharges in table 22 before multiplying by P/136. For a fully raised dam, 626 ft^3/s (table 18) must be subtracted. If 10 wickets are lowered, the wicket-weir flow portion of the eighth column of discharges in table 22 is 580 ft^3/s [626 x (1 - 10/135)].

Tables 20, 21, and 22 were developed from equations 7 and 8 (pages 10 and 11) and the discharge-coefficient relations illustrated in figure 19. Discharges for combinations of upstream and downstream elevations not presented on these tables may be calculated from interpolation or from equations 7 and 8.

EXAMPLES

The following five examples illustrate the use of rating tables, discharge coefficients, and equations of flow for tainter gate, headgate, butterfly valve, and wicket control. Table 23 summarizes the single-valued discharge coefficients determined for each dam. Within table 23, reference is made to the appropriate figures for coefficients related to gate opening, headwater and tailwater depth, or width of lowered wicket opening. The Brandon Road Dam rating is used in the first example. The ratings for the Marseilles and La Grange Dams are discussed in the four remaining examples.

Table 23.–Summary of discharge coefficients to use in equations 1-8

Control structure	Discharge coefficient	Value of coefficient or appropriate reference for the following dams			
		Brandon Road	Dresden Island	Marseilles	Starved Rock
Tainter gate	C	(1)	fig. 8	fig. 11	fig. 14
	C_{gs}	(1)	fig. 9	fig. 12	fig. 15
	C_w	3.3	3.06	2.75	2.88
	C_{ws}	(1)	(2)	(2)	(2)
	C_{sw}	3.5	(1)	(1)	(1)
Headgate	C_{fo}	0.7	(1)	(1)	(1)
		Peoria	La Grange		
Wicket	C_w	fig. 17	fig. 19		
	C_{sw}	3.5	3.5		
Butterfly valve	C_{fo}	(1)	0.5		
	C_{so}	0.4	0.6		

[1] Particular type of flow will never occur.

[2] Discharge coefficient was not determined because of insufficient data.

EXAMPLE I

The upstream pool elevation at Brandon Road Dam is 538.5 feet. Ten tainter gates are open, and one headgate is open 4.0 feet.

1. What is the discharge at the dam? From table 4 (page 15), the discharge under one open tainter gate is 538 ft^3/s, and the flow through one headgate set at 4.0 feet is 1,350 ft^3/s. Therefore, the total combined discharge under 10 tainter gates and through 1 headgate is 6,730 ft^3/s. Leakages are not included in this value.

2. How much will the present discharge change if the upstream pool rises to elevation 539.2 feet? As table 4 does not include discharges for elevation 539.2 feet, equations 3, 5, and 8 (pages 9-11) must be used or an estimate may be made by interpolating the values in table 4. A static headwater depth (h_1) of 2.9 feet is determined by subtracting the ogee spillway crest elevation of 536.3 feet from 539.2 feet. The headwater depth (h_O) above the center of the headgate opening is determined by subtracting 520.5 plus half the gate opening from the upstream pool elevation. (The elevation of the top of the lower headgate, shown in figure 3, is 520.5 feet.) The value of h_O for a 4.0-foot gate opening is 16.7 feet. A headwater depth (h_O) above the crest of the closed tainter gates of 0.65 feet is determined by subtracting the closed gate crest elevation of 538.55 feet from 539.2 feet. The free-weir and sharp crested weir discharge coefficients (C_W and C_{SW}) for flow under the open gates and over the closed gates are 3.3 and 3.5, respectively (table 23). The headgate free orifice flow coefficient (C_{fO}) is 0.7 (table 23).

   Through the use of equation 3 (page 9), the flow under 10, 50-foot-wide tainter gates is 8,150 ft^3/s. The discharge under one 15-foot-wide headgate set at 4.0 feet is 1,380 ft^3/s (equation 5). The sharp crested weir flow over the top of the remaining 11 gates is computed from equation 8 to be 1,010 ft^3/s. The total flow will increase 3,810 ft^3/s to 10,540 ft^3/s. The approach velocity heads for each component of total discharge are negligible.

EXAMPLE II

At Marseilles Dam, the upstream pool elevation is 482.8 feet, and two tainter gates are open 1.5 feet each. The downstream pool elevation is 470.0 feet.

1. What is the total discharge of the river downstream from the dam? From table 8 (page 27), the discharge under each gate is 1,840 ft^3/s; therefore, the discharge under two gates is 3,680 ft^3/s. Any diversion canal discharge is not included in this figure.

2. How much will the tainter gate controlled flow increase if the upstream pool elevation increases to 483.3 feet? As the downstream elevation is lower than the bottom of the two open gates, the free-orifice adjustment factor $[(HW - 469.8)/13.0]^{0.5}$ may be used to compute the increase in tainter-gate flow. The adjustment factor for headwater elevation 483.3 feet is 1.019; therefore, tainter-gate controlled flow will increase from 3,680 ft^3/s to 3,750 ft^3/s.

EXAMPLE III

During a recent storm, the upstream and downstream elevations at Marseilles Dam were 483.0 and 476.4 feet, respectively. The gate settings (h_g) were: one gate at 0.5 feet, one at 4.0 feet, two at 5.0 feet, two at 6.0 feet, and two raised above the upstream pool water surface.

1. What was the flow regime at the gates? The headwater and tailwater depths must first be determined by subtracting the ogee spillway crest elevation of 469.8 feet from each pool elevation. The headwater depth (h_1) was 13.2 feet, and the tailwater depth (h_3) was 6.6 feet. As the forebay elevation is 468.6 feet, the approach depth is 14.4 feet. Submerged-orifice flow is assumed at submergence ratios (h_3/h_g) greater than 1.5. Therefore, submerged-orifice flow existed at gates set at 0.5 and 4.0 feet, and free-orifice flow existed at the gates set at 5.0 and 6.0 feet. Free-weir flow existed at the two gates raised out of the water.

2. What was the total discharge under all eight gates? Table 8 (page 27) may be used to estimate this discharge. The submerged-orifice, free-orifice, and free-weir adjustment factors will be used to adjust for the 0.2 foot difference in actual upper pool elevation and the elevation table 8 was developed for. The submerged-orifice adjustment factor is 1.016, the free-orifice adjustment factor is 1.008, and the free-weir adjustment factor is 1.023. The submerged-orifice flow under gate settings of 0.5 and 4.0 feet is estimated by interpolating between the discharges in table 8 for lower pool elevations 476 and 478 feet. The free-orifice flow under gates set at 5.0 and 6.0 feet is found in the leftmost column of discharges in table 8.

1 gate at 0.5 foot	= 1 x 372 x 1.016 =	378
1 gate at 4.0 feet	= 1 x 4,350 x 1.016 =	4,420
2 gates at 5.0 feet	= 2 x 5,550 x 1.008 =	11,200
2 gates at 6.0 feet	= 2 x 6,440 x 1.008 =	13,000
2 gates raised clear of water surface	= 2 x 9,490 x 1.023 =	19,400
		48,400 ft^3/s

The estimated total gate-controlled flow of 48,400 ft^3/s is 1.8 percent less than the discharge calculated from equations 1, 2, and 3 with the appropriate discharge coefficients. The approach velocity heads in the forebays of gates set at 0.5, 4.0, 5.0, and 6.0 feet are 0.003, 0.041, 0.65, and 0.88 feet, respectively. The velocity head in the forebay at gates raised clear of the water surface was 2.00 feet. Such velocity heads must be determined and added to the headwater depth (h_1) before applying equations 1, 2, and 3.

EXAMPLE IV

The upstream pool elevation at La Grange Dam is 429.0 feet, and the downstream elevation is 421.5 feet. There are no lowered wickets, needles in place, or valves open.

1. What is the discharge at the dam? From table 21 (page 50), the total discharge is 6,730 ft^3/s.

2. How many needles should be placed to reduce discharge by 1,000 ft^3/s? The desired flow reduction is 14.9 percent. As all of the original flow is through the spaces between wickets, 20 needles should be placed (0.149 x 136).

3. As the downstream elevation increases to 423.8 feet, how much will the original discharge change? From table 21, the discharge at a downstream pool elevation of 423.5 feet is 6,300 ft^3/s, and at 424.0 feet it is 6,160 ft^3/s. By interpolation, the flow is estimated to be 6,220 ft^3/s. Therefore, flow will decrease 510 ft^3/s for the 2.3-foot increase in tailwater depth.

4. How should the dam be operated to maintain the original pool elevations and pass an additional 2,000 ft^3/s? The total desired release is 8,730 ft^3/s. Discharge over three lowered wickets is 8,240 ft^3/s (table 21). An additional 345 ft^3/s can be released from one valve (table 19), bringing the total release to 8,770 ft^3/s. A less time consuming alternative is to open five or six valves and not lower any wickets. The discharge from six valves is 2,120 ft^3/s, and the discharge past the dam with no lowered wickets is 6,730 ft^3/s. The total release with six valves open would be 8,850 ft^3/s. With five valves open, the total release would be 354 ft^3/s less or 8,500 ft^3/s.

EXAMPLE V

The upstream and downstream elevations are 429.4 feet and 424.8 feet, respectively. Twelve valves are open, but there are no lowered wickets or needles in place.

1. What is the discharge at the dam? The valve discharge for a head differential of 4.6 feet may be calculated from equation 6 or estimated by interpolating between the discharges on table 19 (page 48) for tailwater elevation 424.0 and 424.5 feet. The valve discharge is 3,640 ft^3/s.

   The discharge past the dam can be estimated by interpolating between values found in tables 21 and 22. For a tailwater elevation of 424.5 feet, the interpolated discharge is 6,830 ft^3/s. The discharge to a downstream pool elevation of 425.0 feet is calculated by interpolation to be 6,640 ft^3/s. The estimated discharge from elevation 429.4 to 424.8 is 6,720 ft^3/s. (This value is within 1.0 percent of the value calculated from equations 7 and 8 with the appropriate discharge coefficients.) The total discharge at the dam is 10,360 ft^3/s.

SUMMARY AND CONCLUSIONS

Stage-discharge rating tables have been developed for six dams on the Illinois and Des Plaines Rivers. These tables facilitate estimation of discharge at locations having no nearby stream-gaging stations. The ratings will also be useful for estimating low flows when the water-surface slope between control structures on the river can approach zero and traditional methods of determining discharge based on slope are unsatisfactory.

Hydraulic equations based on the assumption of steady flow were used to compute the stage-discharge relations at four different control structures. Discharge measurements were made to determine coefficients of discharge for the free-orifice, submerged-orifice, and free-weir flow regimes at the tainter gate dams. Data were insufficient to calibrate the submerged-weir flow relation. Present ratings were verified by these measurements and revised if needed. Equations of free- and submerged-orifice flow through a headgate at Brandon Road Dam and butterfly valves at Peoria and La Grange Dams were calibrated from discharge measurements.

A discharge relation for flow controlled by a Chanoine wicket dam was verified by discharge measurements. The coefficients of discharge for this relation were originally determined from a hydraulic model study by the U.S. Army Corps of Engineers in 1937 and 1938. Recent discharge measurements below Peoria and La Grange Dams indicate that these coefficients and the equation of flow may be used to estimate the discharge past the dams accurately.

The rating tables for each dam have been developed for discrete combinations of upstream and downstream pool elevations. Adjustment factors are presented so that discharge for pool elevations not tabulated can readily be computed. Steady flow equations are presented for each flow regime that can occur at the dams. Discharge coefficients are also presented for each dam so that discharges for conditions not summarized in the rating tables can be computed from the steady flow equations.

Discharge coefficients determined from discharge measurements were comparable to coefficients published in hydraulic design texts. However, extrapolations based on these measurements may be subject to error until additional measurements are made for the hydraulic conditions extrapolated to.

Additional discharge measurements are needed to calibrate the submerged-weir flow regime at Dresden Island, Marseilles, and Starved Rock Dams. The conditions for this regime exist only at very high streamflows, which historically have infrequently occurred at these structures. Arrangements should be made to measure from a bridge because at these stages conditions would be very dangerous for a boat measurement.

Measurements used to develop rating tables for wicket and butterfly valve control at Peoria and La Grange Dams were made at gaging stations downstream from the dams. Additional measurements are needed at these dams to more accurately verify discharge coefficients for each control.

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