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An Investigation of the Hydrologic Factors that Affect the Water Levels of Lake Poinsett

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AN INVESTIGATION OF THE HYDROLOGIC FACTORS THAT AFFECT THE WATER LEVELS OF LAKE POINSETT

BY

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AN INVESTIGATION OF THE HYDROLOGIC

FACTORS THAT AFFECT THE WATER LEVELS

OF LAKE POINSETT

This thesis is approved as a creditable and independent investigation by a candidate for the degree, Master of Science, and is acceptable as meeting the thesis requirements for this degree, but without implying that the conclusions reached by the candidate are necessarily the conclusions of the major department.

Thesis Adviser Date

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INTRODUCTION

With the continuance of the present trend of fewer working hours and more leisure time, the need for greater recreational opportunities is quite apparent. In recent years, natural and artificial lakes have been used more extensively for recreational purposes.

The citizens of eastern South Dakota are fortunate to live within driving distance of the largest natural lake in the state. This lake, known as Lake Poinsett, lies in the east central part of the state and has a surface area of approximately 8000 acres. It is connected to Dry Lake by a highway bridge and an underground aquifer, and it is connected to the Big Sioux River by a natural outlet channel. These lakes receive most of their water from natural resources such as runoff and direct precipitation. Water is also diverted from the Big Sioux River into the lakes by the manipulation of two sets of gates. Figure 1 shows Lake Poinsett and the connected waters.

The people of the surrounding area look to this lake for many different types of recreation and enjoyment. During the warm summer months people can be seen fishing, water skiing, boating, picnicing, or just enjoying the view that this lake offers. Many summer cottages, a few permanent homes, and a number of resorts line the shores of this fine lake. It is not difficult to realize that the increasing trend in outdoor activity will lead to more cottages, homes, and resorts. There will probably be an increase in the number

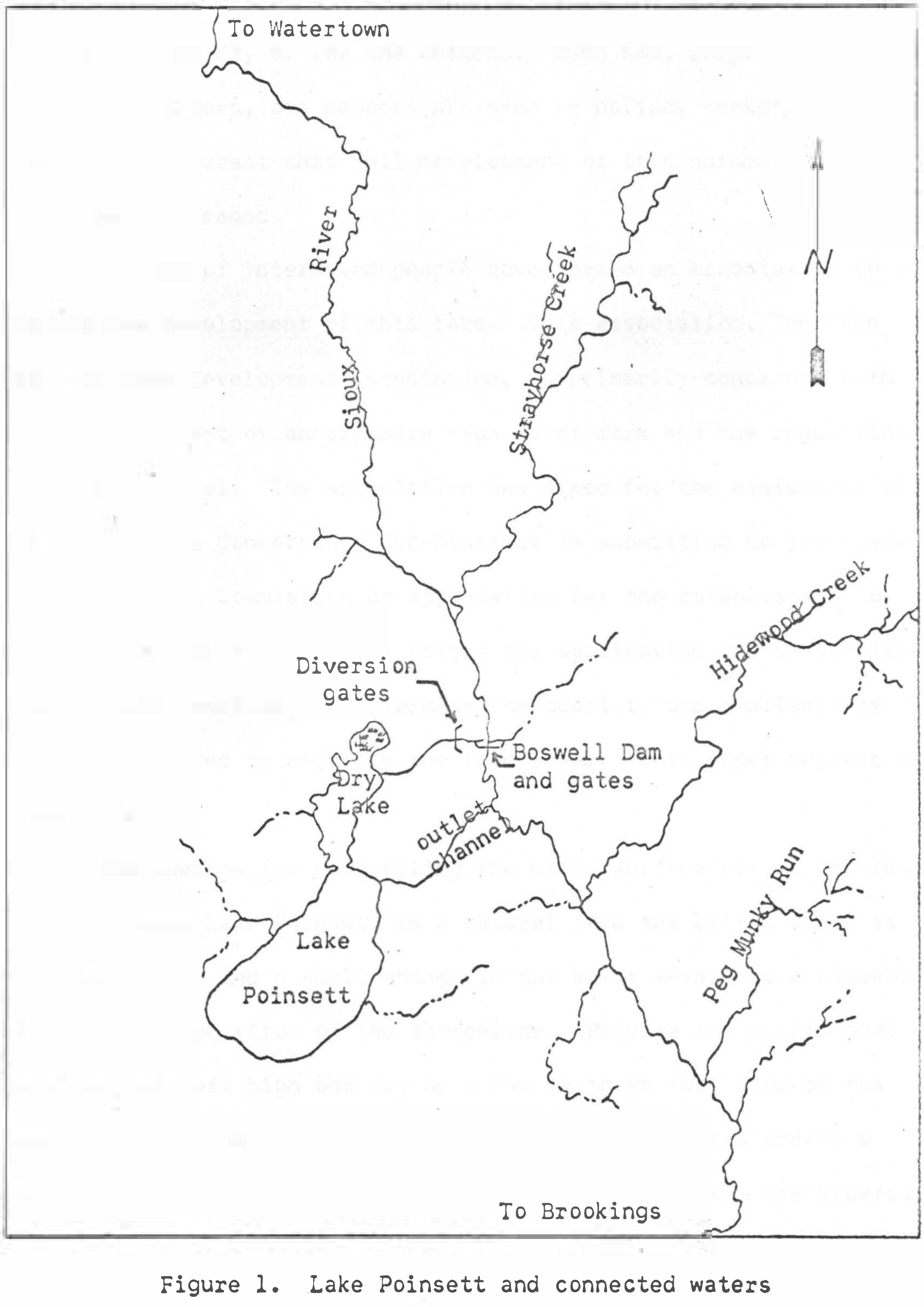


Figure 1. Lake Poinsett and connected waters

of people using the lake who do not own adjoining property but who do come for the day or for the weekend. Even now, large numbers of tents, trailers, and campers are seen on holiday weekends. Therefore, it is apparent that full development of this natural resource should be encouraged.

A group of interested people have formed an association to promote the development of this lake. This association, The Lake Poinsett Area Development Association, is primarily concerned with the establishment of an ordinary high water mark and the regulation of the lake level. The association has asked for the assistance of the East Dakota Conservancy Sub-District in submitting to the State Water Resources Commission an application for the establishment of an ordinary high water mark. Before the application can be submitted studies were necessary to determine the quantity and availability of water required to regulate the lake level. This paper reports on those studies.

The reasons for controlling the water surface elevation are many. Because Lake Poinsett is a natural lake the bottom slope is reasonably flat and a small change in the water level has a sizeable effect on the position of the shore-line. Private and public boat docks may be left high and dry by a two or three foot drop of the lake surface. Low lake levels during the winter months create a hazardous condition for fish life and, in addition, the ice creates a

pressure that has a tendency to force partially submerged boulders toward the shore. These boulders when later submerged after ice melt result in a dangerous condition for boaters, swimmers, and water skiers.

Purpose

In essence the purpose of this thesis is to determine the feasibility of controlling the Lake Poinsett surface elevation.

Objectives

The major objectives of the study were:

1. To weigh and evaluate the major factors that effect the water level of Lake Poinsett,

2. To determine a range of possible control elevations of the water surface, and

3. To investigate a series of structures that could be used to control the flow in the natural outlet channel.

LITERATURE REVIEW

A water resources project requires the application of certain principles in different phases of development such as: project planning and project studies.

In his discussion of project planning, Johnson (1) states that any water resources development, whether large or small, simple or complex, serving one purpose or many, should provide the facilities to accomplish the optimum development of related physical resources. The investigations and studies made for these projects must be considered in relation to the project as a whole. The project's objectives, purpose, and scope determine what must be investigated with respect to the desired results.

In many cases, the project may be of the dual or multi-purpose type. For this reason the investigation may be concerned with many factors which influence the selection of the project site, the magnitude of the project, and the purpose which it serves. Therefore, the entire project must be investigated as a unit before the design requirement for a single feature, such as an outlet structure or dam, can be firmly established. Each project purpose and each increment of its size must justify inclusion in the project by some appropriate measure of feasibility which is usually related to the benefits it produces, the needs it serves, or the investments it can repay.

Feasibility studies of water resources projects should always consider possible objections by the public. The exposed shoreline of a lake or reservoir may be unattractive and may make access to the water very difficult. An impoundment of fresh water when held at a constant level also makes an ideal breeding place for mosquitoes. Similar consideration must be incorporated during the phase of project planning.

Project planning has been defined (2) as the orderly consideration of a project from the original statement of the purpose through the evaluation of alternatives to the final decision on a course of action.

The most important objective in project planning is the determination of the project's feasibility. This involves project studies which will permit a sound analysis and conclusion with respect to the specific engineering-economic considerations. These are primarily (1):

1. That the project is responsive to an urgent present or anticipated social or economic need;

2. That the project as planned will adequately serve the intended purpose; and

3. That the services proposed to be performed through the project and the benefits it will produce will justify the cost.

The extent to which an investigation is carried out is not limited by any simple set of rules. Each project will have special situations that will limit the extent of the project studies. The maximum justifiable cost of investigation is limited by the magnitude

of the project. A project is generally considered unjustified if the cost of the necessary investigation would offset a large portion of the constructed project's value.

The following sections discuss various principles that are applied to some of the more important aspects of project planning and development.

Hydrologic Investigations

The hydrologic investigations which may be required for a study of a project include the determination of some of the following: streamflow yield, flood flows, ground water conditions, precipitation, evaporation, and water requirements for the project (1). Streamflow yield, precipitation, evaporation, and water requirements are determined by the following methods.

Streamflow Yield

The hydrologic data used in determining streamflow yield are long term streamflow records at the location of the proposed project. Because such records are rarely available, it is sometimes necessary to make actual streamflow measurements at the proposed site and to project these data to cover a sufficient length of time or to transfer runoff data from some similar location to the point of concern.

In some instances streamflow records are available on the stream itself, but are at a considerable distance from the proposed site. Miller and Clark (1-20) state that such records can be

transferred to the project site by appropriate area and basin-characteristic relationships. The most useful tool for such an analysis is a duration curve (3). Such a curve (Figure 2) shows the per cent of the time that any discharge is equaled or exceeded. The shape of the curve presents a picture of the stream characteristics. A flat curve is derived from a river having a few floods and having large ground water discharges, whereas a steep curve indicates a flashy stream or river which is subject to very low flows. The most frequent use of the flow-duration curve is to determine the water supply potential of a stream or river. Once a duration curve is established for the stream, the area under that curve represents the volume of runoff for that specific location of the stream.

Chebotarev (4) suggests the use of the water-balance equation for all types of streamflow and runoff problems. This method, essentially one of conservation of matter, is based on the equality of the amount of water flowing into and out of a given portion of land. This equation is in the nature of an inventory as follows:

Inflows = Outflows + Change in Storage (Equation 1)

It must be kept in mind that this equation is based on the assumption that the inventory is for a specific area and a specific time period.

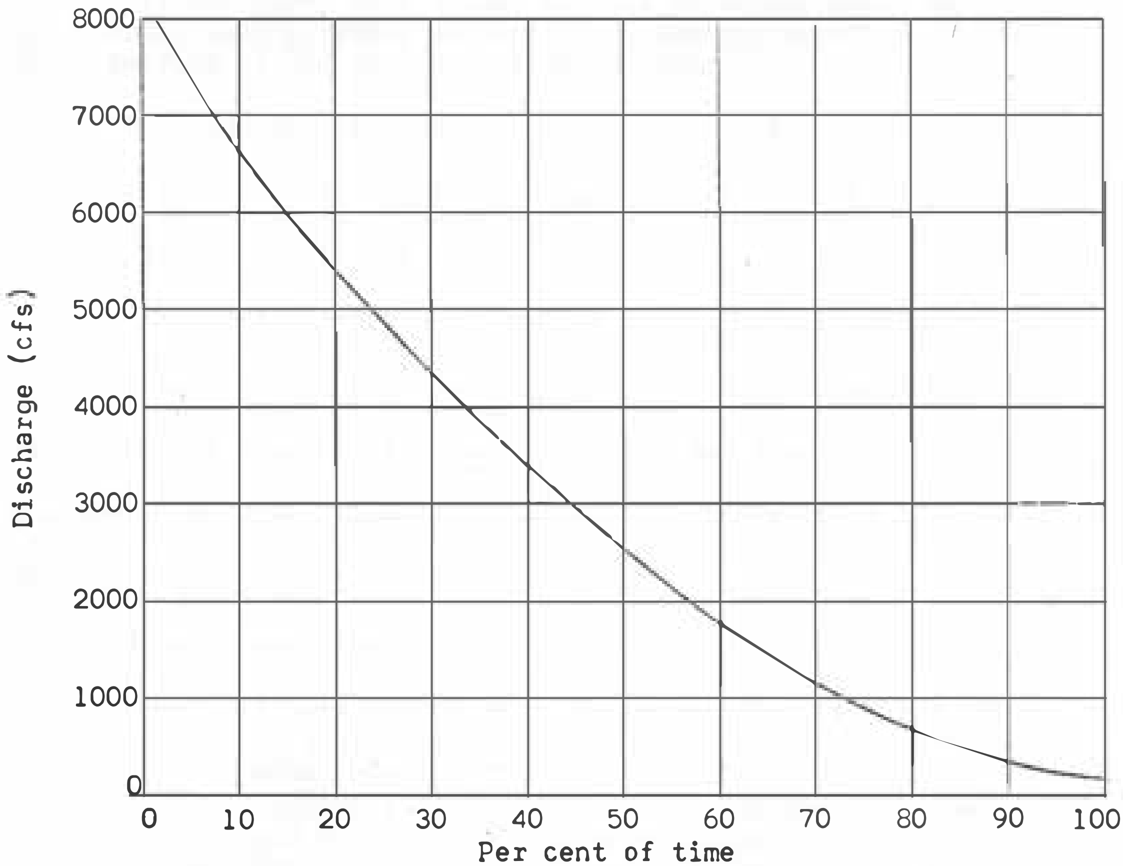


Figure 2. Flow-duration curve (3).

The items in the water-balance equation are as follows (4):

Inflows: precipitation, surface inflow, ground water inflow, and import (defined as water piped or channeled into the area).

Outflows: surface outflow, ground water outflow, export (defined as water piped or channeled out of the area), evaporation, transpiration, and interception (defined as precipitation intercepted by foliage and buildings and returned to the atmosphere without reaching the ground).

Change in Storage: this occurs as a change in ground water, soil moisture, snow cover, surface reservoir water and depression storage, and water temporarily existing on the surface of the ground as flowing water.

The items in the equation may be expressed in terms of either volume or rate of discharge, but volumetric terms will usually be found to be easier to work with.

The most important data used to determine streamflow yield are records obtained by stream gaging. As stated by Addison (5), measurement of streamflow usually involves two steps: first, the measurement of river stage; and second, the correlation of stage and discharge.

Stage is measured by:

1. Visual observation of the water surface elevation on a graduated scale or staff gage;
2. Measurement of the vertical distance from a reference point to the water surface by a chain and weight or a wire weight gage; or
3. Collection of a continuous autographic record using a float-actuated recorder housed in a shelter over a stilling well.

Determination of discharge or streamflow for correlation with river stage is most commonly obtained by:

1. Dividing the surface width of the stream (B) into a number of elements of uniform width (b) (Figure 3);

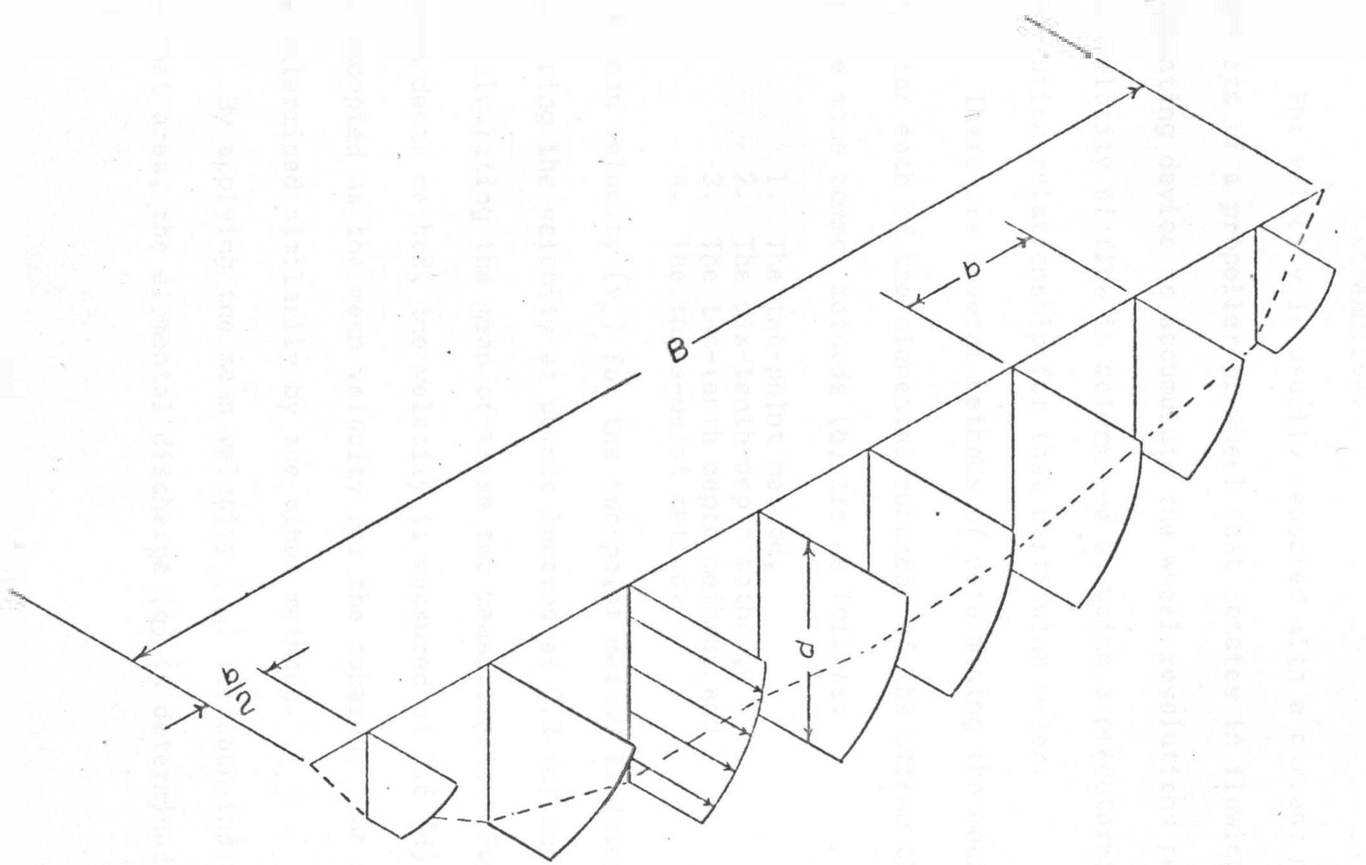


Figure 3. Subdivision of stream cross-section (5).

2. Measuring the depth (d) at each of these division points;
3. Measuring the mean velocity (v_m) at each of the division points;
4. Computing the discharge for each of the subareas; and
5. Summing the subarea discharges to determine the streamflow.

The velocity is usually measured with a current meter which consists of a propeller or wheel that rotates in flowing water, and a counting device to accumulate the wheel revolutions per minute. The velocity of flow is determined by using a predetermined velocity-revolution relationship for that particular meter.

There are several methods of determining the mean velocity (v_m) for each of the elemental subareas of the stream channel. Some of the more common methods (6) are as follows:

1. The two-point method,
2. The six-tenth depth method,
3. The two-tenth depth method, and
4. The three-point method.

The mean velocity (v_m) for the two-point method is determined by measuring the velocity at points located at 0.2 (d) and at 0.8 (d) and calculating the mean of these two measurements. For the six-tenth depth method, the velocity is measured at 0.6 (d); this value is accepted as the mean velocity for the subarea. The mean velocity is determined similarily by the other methods.

By applying the mean velocity (v_m) for each individual subarea to that area, the elemental discharge (q) is determined. The total

stream discharge (Q) is equal to the summation of all the elemental discharges as shown by the following equation:

$$Q=\sum_{i=1}^{n} q = \sum_{i=1}^{n} bd_i v_{m_i} \qquad \text{(Equation 2)}$$

A series of discharge measurements made when the channel is flowing at various depths is used to establish a stage-discharge relationship for the stream at the gaging point. This relationship is used to determine the stream discharge from stage records. These discharges can be projected over a period of time to determine the yield of the stream (5).

Precipitation

In most instances the engineer is concerned with that portion of the precipitation that falls on the drainage basin and accumulates in the basin streams as runoff, and also with the precipitation that falls directly on the water surface.

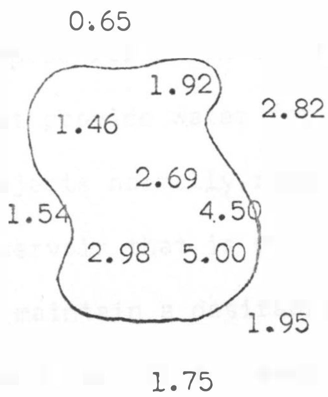
It is suggested (3) that for intermediate-sized areas (500 square miles to 2,000 square miles), if the precipitation stations are spaced with reasonable uniformity and the area is relatively free from topographic influences, an arithmetic average will suffice for determining the average depth of precipitation. Another acceptable method of analyzing the precipitation data in accordance with the individual recording stations is known as the "Thiessen Method." The precipitation stations are first plotted on a base map and perpendicular bisectors are erected at the midpoints of straight

lines joining adjacent stations. These bisectors form polygons around each station , and permit the proportioning of the precipitation on the basis of the relative areas contained in the respective polygons. Figure 4 presents a graphical representation of these two methods.

Evaporation

Evaporation is one of the major losses of water from a lake or reservoir. It is stated by McKay and Stichling (8) that evaporation directly from a water surface of a prairie lake is about three feet annually and generally exceeds the precipitation. Because this seriously affects reservoir regulation, it is desirable that the magnitude of this term be known with reasonable accuracy.

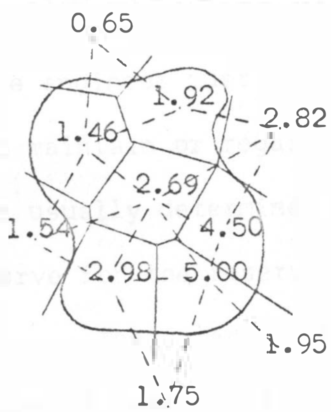
Some of the more sophisticated methods of determining evaporation, such as the energy budget and mass transfer method are not applicable to practical problems because of the lack of sufficient data. The method most commonly employed for evaporation determinations is the application of evaporation pan records to the reservoir in question. Reservoir evaporation is often estimated by multiplying the evaporation by a coefficient. The U. S. Weather Bureau Class "A" pan is now accepted as an international standard. Evaporation coefficients, for the Class "A" pan vary annually, monthly, and geographically. For annual evaporation the coefficient is generally accepted at about 0.70.



Arithmetic mean:

$$\frac{1.46+1.92.69+4.50+2.98+5.00}{6} = 3.09 \text{ in.}$$

(a)



Thiessen method:

Observed precip. (in.)	Area* (sq mi)	Per cent total area	Weighted precipitation (in.)
0.65	7	1	0.01
1.46	120	19	0.28
1.92	109	18	0.35
2.69	120	19	0.51
1.54	20	3	0.05
2.98	92	15	0.45
5.00	82	13	0.65
4.50	76	12	0.54
	626	100	2.84

Average = 2.84 in.

* Area of corresponding polygon within basin boundary

(b)

Figure 4. Areal averaging of precipitation by (a) arithmetical method, and (b) Thiessen method (7).

Water Requirements

The water requirements for reservoirs vary according to the purpose for which the reservoirs are intended. Storage reservoirs that provide water for municipalities, power plants, and irrigation projects normally require a greater water supply than does a reservoir that is used only for recreation and wildlife. In order to maintain a desirable pool elevation in the first type mentioned, there must be a steady flow of water into the reservoir to offset the demands. Whereas for the second type, when losses are due only to natural causes such as evaporation and seepage, the demand is normally not as great. Regardless of the magnitude of the demand, the engineer must have some knowledge as to how much water is required to maintain or regain a favorable pool elevation. This information is usually determined by the construction of an elevation-capacity curve for the reservoir being studied (2).

The capacity of a reservoir, whether natural or man-made, must be obtained from topographic surveys made within the reservoir site. An elevation-area curve is usually constructed by determining the area enclosed within each contour. The increment of storage between two elevations is usually computed by multiplying the average of the areas at the two elevations by the elevation difference. The summation of these increments below any level is the storage capacity below that elevation and when plotted is referred to as an elevation-capacity curve. Figure 5 is an example of an elevation-area and elevation-capacity curve.

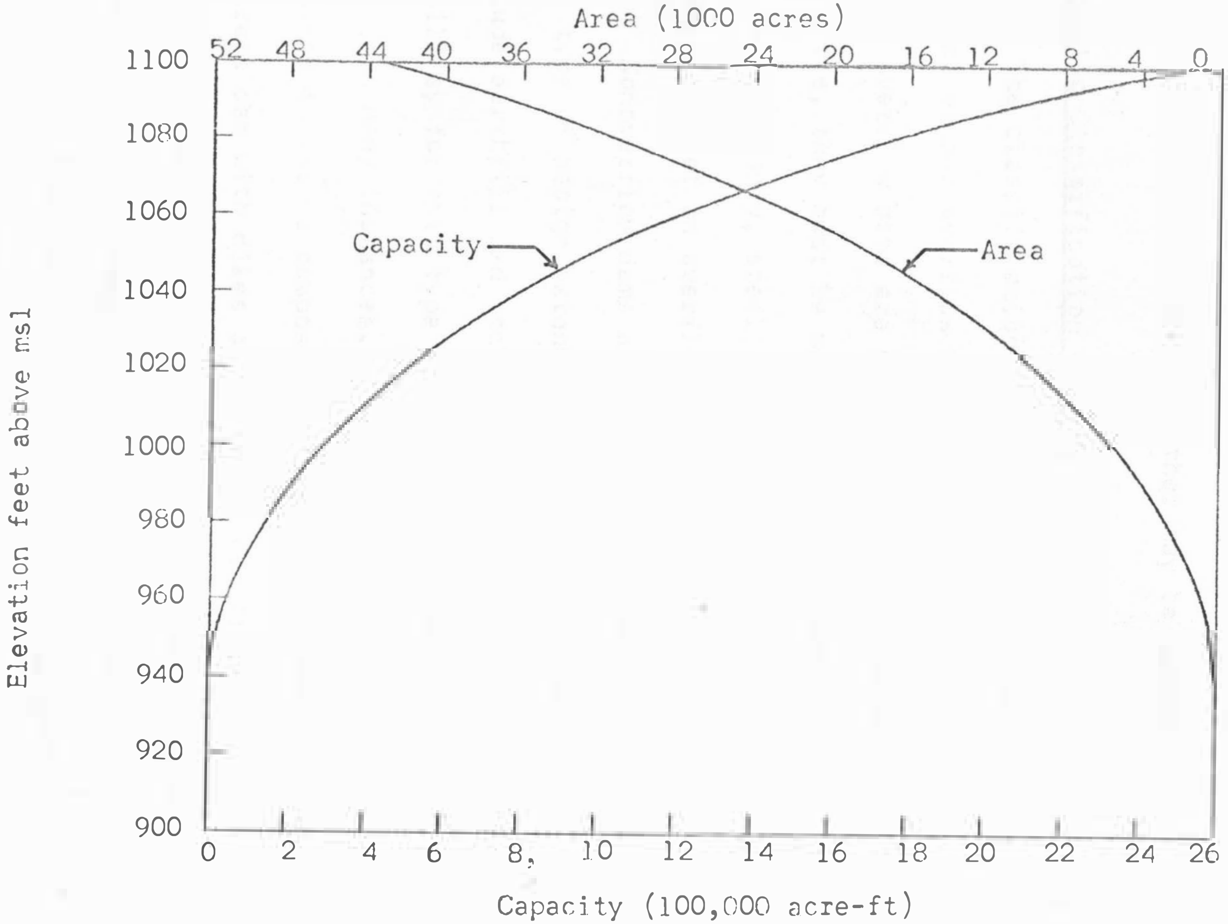


Figure 5. Elevation - area and elevation - capacity curves for the Cherokee Reservoir in Tennessee (2-148).

The elevation-capacity curve may be used by the engineer to provide specific information relative to a reservoir. The drop in water elevation during periods of zero inflow will reveal the total demand on the storage both from natural losses and consumer demands. In addition, the volume of water required to raise the surface elevation from one level to another may be determined (9).

Hydraulic Classification of Dams

The classification of a dam according to its hydraulic design would be either overflow or nonoverflow.

Overflow dams are designed to carry discharge over their crest. Therefore, they must be made of materials which will not erode easily. Concrete, masonry, steel, and wood are the materials used in the construction of an overflow dam.

Nonoverflow dams are designed so they will not be overtopped. This type of design extends the choice of building materials to include earthfill and rockfill dams. The necessity of constructing a spillway for this type of structure is one of the disadvantages.

In many instances, the overflow and nonoverflow dams are combined to form a composite structure, for example, an overflow concrete dam with dikes of earthfill construction. The overflow portion of the structure is most often of the concrete gravity type because it is well adapted for an overflow crest and it can be constructed on either sound rock or gravel foundations. In the latter case adequate cutoff devices must be provided to stop underground seepage (10).

<u>Factors Governing the Selection of the Type of Dam</u>

It is only in exceptional circumstances that an experienced engineer can say that only one type dam is suitable or most economical for a given situation. In some instances the selection of a type of dam is obvious, while in other situations preliminary designs and estimates for several types of dams may be required before one can be shown to be the most suitable.

In selecting the type of dam for major structures, it is desirable to secure the advice of a competent engineering geologist in connection with the applicability of possible dam types to the foundations available.

The dam selected may also depend upon the availability of suitable materials, labor, equipment, and accessibility of the site.

In general, type selection will be dictated by: the physical characteristics of the site, by economic features, and by practical considerations (10). The factor that carries the most weight in type selection is economy. The following outline (11) indicates important factors that may influence the cost and therefore the selection of a dam.

A. Site Conditions
   1. Foundation materials
      a. allowable foundation stresses
      b. percolation values
      c. excavation requirements
      d. potential settlement

   2. Topography of the site
      a. arch dams usually limited to narrow canyons with high strong walls
      b. earth dams usually suitable for all conditions except very high or narrow sections.

3. Availability of materials
    a. convenient source of earth or aggregate may indicate an earth or masonry dam
    b. buttress dam requires smallest quantity of materials.

B. Hydraulic Factors
1. Spillway requirements
    a. overflow spillways are best for large capacities, most feasible on gravity or slab-buttress dams
    b. side-channel and tunnel spillways adaptable to any type of structure

2. Diversion Requirements
    a. costs are usually greater for an earth dam because of a greater base thickness

3. Outlet works and penstocks
    a. arch dams not adapted to large or numerous openings

C. Climatic Effects
1. Spalling of concrete in cold climates is a disadvantage of thin arch and buttress dams

D. Traffic Factors
1. crest highways are very costly for thin arch and buttress dams

E. Social Factors
1. Gravity dams provide the greatest safety against sudden destruction due to earthquake, bombing, etc. with resultant damage to affected communities

2. The benefits to be derived may control the cost of the dam; a temporary dam may have to suffice if benefits are small or short-range

3. Volume of employment of local labor

4. Esthetic considerations

Spillways

Usually the most important component of a dam is its spillway (11). The primary purpose of a spillway is to provide an efficient and safe means of conveying excess water past the dam to the downstream channel. There are many types of spillways, but approximately 90 per cent (11) are the overflow and overfall types (Figure 6). Both types are essentially large rectangular weirs, but the overflow type has a crest curve which conforms to the natural shape of an overflowing nappe of water.

The general equation (10-59) for computing the discharge capacity for a spillway of the overflow type is:

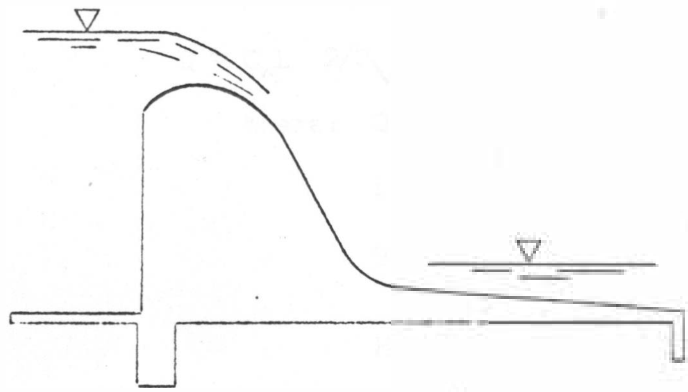
$$Q = CLH^{1.5} \qquad \text{(Equation 3)}$$

Where: Q = Total discharge in cfs

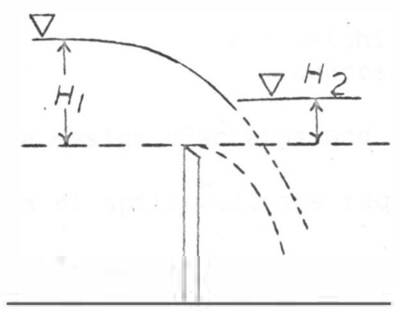
L = Crest length in feet

H = Total head on the crest in feet

C = Crest coefficient, ranging from 3.1 to 4.1 with 3.33 recommended for preliminary design purposes



(a)



(b)

Figure 6. Typical spillway profiles for (a) the overflow spillway, and (b) the overfall spillway (3)(12).

The general equation (12) for the capacity of the overfall spillway which has the characteristics of a submerged weir is as follows:

$$Q = C_w L \;\; 2/3\sqrt{2g}\; H^{1.5} \qquad \text{(Equation 4)}$$

Where: Q = Total discharge in cfs

L = Crest length in feet

g = Acceleration due to gravity in feet per second squared

H = Total head on the crest in feet

C_w= Coefficient of discharge

The discharge coefficient (C_w) of equation 4 can be calculated by the following formula:

$$C_w = 0.605 + 0.08\,\frac{H}{P} + \frac{H}{305} \qquad \text{(Equation 5)}$$

Where: C_w = Coefficient of discharge

H = Total head on the crest in feet

P = Height of the crest above the approach channel bed.

By knowing the design discharge and total head that might occur for these types of spillways, the required crest length could be calculated from the preceding formulas. The discharge capacity may also be calculated from these formulas provided that the head and crest length are known.

ANALYSIS OF DATA AND RESULTS

Introduction

The purpose of this investigation, as previously stated, was to study the feasibility of controlling the water surface elevation of Lake Poinsett. Because Lake Poinsett is connected to Dry Lake, both lakes were considered as a unit when analysing the data and presenting the results. Therefore, the term Lake Poinsett will hereafter refer to the Lake Poinsett-Dry Lake Complex.

According to Chow (9), the hydrologic data required for the investigation of a reservoir used specifically for recreation and wildlife preservation would include such information as: stream discharge; precipitation; evaporation from the water surface; and ground water flows. Therefore, the evaluation of these and other factors as they affect the water level of Lake Poinsett are of major concern. However, because records concerning ground water flow were not available, because an investigation to determine the ground water flow would be extensive, and because the influences of the ground water was not considered to be significant, the ground water flows were neglected during this investigation. how so

Factors Influencing the Lake Poinsett Water Levels

The factors that affect the water levels of Lake Poinsett would fall under the two main categories of water losses and water contributions. Included in these categories would be: losses to

consumptive use and evaporation; contributions from precipitation directly on the lake surface, direct runoff, and other miscellaneous inflows.

Water Losses

The magnitude of the water losses from a reservoir or storage area is the factor that determines what volume of inflow is necessary to maintain a desirable surface elevation. These losses may be either natural such as evaporation, or consumptive uses such as irrigation. The amount of water lost from Lake Poinsett due to use appeared to be insignificant; therefore, it was necessary to evaluate only the losses from natural causes.

An investigation of the natural losses from a lake would normally include the evaluation of both evaporation and ground water losses. However, as previously mentioned, ground water factors were not considered to be significant.

Evaporation from lake surfaces frequently constitutes the major loss and was evaluated as such. This loss could have been evaluated by a number of various methods, provided that sufficient data were available. However, the only evaporation data available for the immediate area were 13 years of pan evaporation records at Brookings (13). These records were used to calculate a mean pan evaporation value which was determined to be 48 inches per year. By applying an evaporation pan coefficient of 0.70 to the mean pan evaporation value, the average evaporation loss from the lake

surface was determined. The resulting value of 33.8 inches per year compared favorably with the accepted value of 36 inches per year for most prairie reservoirs (4).

Contributions

Precipitation deposited directly on a lake surface is one of the major contributions. Data in the form of long-term mean annual precipitation were obtained from the records of the Environmental Science Services Administration, U. S. Department of Commerce (13). Mean annual precipitation for the stations in the Lake Poinsett area were as follows: Watertown (20.52 inches), Castlewood (20.82 inches), Arlington (21.63 inches), and Bryant (23.20 inches). The mean of these values (21.54 inches per year) was accepted as a representative value for precipitation deposited directly on the lake surface. This contribution was equivalent to approximately 1.79 acre-ft per year per acre of surface area.

Direct runoff is that portion of the inflow that runs directly into the lake system from the immediate drainage area. To estimate this volume, it was necessary to know the size of the drainage area and the runoff per unit of area. The drainage area was determined by defining the area on a contour map and measuring it with a [illegible] 37,500 acres contributed direct runoff to the lake system. Because the total drainage area contained many potholes and sloughs that were not considered typical of the direct drainage area, the available runoff data per unit area were considered of little value for determining the direct runoff.

The direct runoff was determined indirectly by transferring data to the direct drainage area from an adjacent or nearby drainage basin possessing similar precipitation and drainage characteristics. Data (14) (15) were obtained for two isolated drainage basins for which runoff data was available. These basins, the Skunk Creek basin near Sioux Falls, and the Whetstone basin in the northeast corner of the state were considered to be hydrologically similar to the direct drainage area around the lake. Because these basins receive slightly higher average precipitation than the Lake Poinsett basin, the runoff data were adjusted on a porportional precipitation basis before being applied to the drainage area around Lake Poinsett. It was determined that the runoff per acre of the direct drainage area was 0.0985 acre-ft per year. This would contribute 3,700 acre-ft of water to the lake annually.

Another source of inflow that was observed was the reverse flow that occurred in the outlet channel (Figure 1) during the period when the elevation of the Big Sioux River stage exceeded that of the Lake Poinsett water level. This condition normally exists for only a short time (2 to 3 days a year) during the spring runoff. This inflow is restricted by a small bridge and measurements revealed that the maximum flow would range from 60 to 100 cfs. Because the magnitude of the flow was small and the flow period short, the total inflow from this source was not considered significant.

Available Flow for Diversion to Lake Poinsett

Although the lake system is a natural phenomenon, some man-made diversionary structures are in use to maintain a desirable

surface elevation. These structures (Figure 1) consist of two sets of gates and a channel which are used to divert and transport water to the lake area from Boswell Dam. This water is diverted from the Big Sioux River into the diversion channel by the manipulation of the two sets of gates.

Because diverted water is one of the major sources of inflow, it was necessary to determine by appropriate methods the average yearly volume of water that could be diverted from the Big Sioux River and added to the lakes. This volume could be determined by the analysis of daily flow records for the river at the point of diversion (Boswell Dam). Because records were not available for this specific location, it was necessary to estimate this volume indirectly.

Because flow records for the Big Sioux River were available, both upstream and downstream from Boswell Dam, it was decided to use the method of extending data from one or several stations in a stream basin to another location in the same basin as suggested by Chow (9). Records of runoff data (14) (15) for the area involved were those recorded at the Watertown and Brookings gaging stations.

The flow characteristics of the Big Sioux River, principally those of high spring runoff and low summer and winter flows, required that the duration of high river flows be determined. To accomplish this, all of the mean monthly flow data (Table 1) were determined for the Watertown and Brookings gaging stations and plotted in the form of a histogram (Figure 7). The period of high

river flows, from March 1 to July 31, was selected for further evaluation to determine the availability of river water for diversion to Lake Poinsett.

Table 1. Mean monthly flow (cfs) of the Big Sioux River at the Watertown and Brookings gaging stations.

Month	Watertown* (cfs) 1945-1964	Brookings** (cfs) 1953-1964
October	3.05	32.60
November	4.20	27.10
December	2.20	16.50
January	0.60	5.70
February	1.40	17.20
March	36.40	277.60
April	142.75	350.30
May	80.70	199.90
June	63.50	247.00
July	43.00	214.00
August	23.50	100.00
September	3.46	47.20
Mean Annual Runoff (acre-ft)	24,985	92,800

* 1 mile downstream from inlet-outlet to Lake Kampeska, $2\frac{1}{2}$ miles northwest of Watertown, on right bank 20 ft. downstream from highway bridge. SW$\frac{1}{4}$ SW$\frac{1}{4}$ NW$\frac{1}{4}$ Sec. 13, T. 117 N., 2.53 W.

** $1\frac{1}{2}$ miles downstream from Deer Creek, and $9\frac{1}{2}$ miles southeast of Brookings, on right bank 3 ft. downstream from highway bridge, NW$\frac{1}{4}$ NW$\frac{1}{4}$ Sec. 8, T. 108 N., R. 49 W.

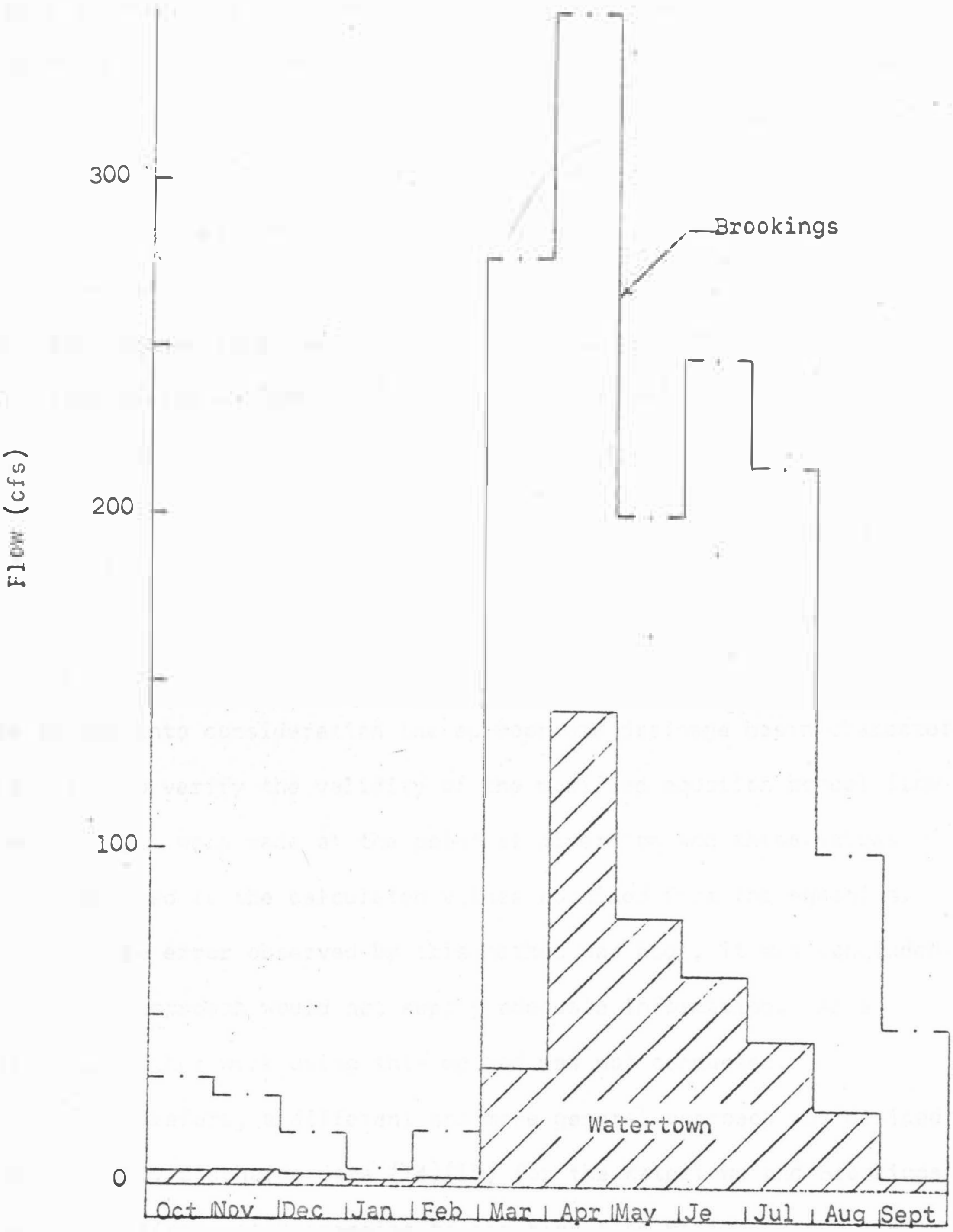


Figure 7. Mean monthly flow (cfs) of the Big Sioux River at the Watertown and Brookings gaging stations.

The first attempt to determine the stream yield at the point of diversion was made by the use of the water-balance equation (Equation 1). This method considers the inflow, outflow, and change in storage within a basin or a portion of a basin. This hydrologic equation is an inventory for a specific area and a specific time period. The area involved was the total drainage area between the Watertown and Brookings gaging stations. This area as defined by the Soil Conservation Service (16) was equal to 1205 square miles. The time period was defined as the high flow period (March 1 - July 31) for each year that records were available at both the Watertown and Brookings gaging stations.

To use this water-balance method of investigation to determine the flow at the point of diversion from previous records, an equation of the water-balance form was developed for this specific problem by taking into consideration the appropriate drainage basin characteristics. To verify the validity of the modified equation actual flow measurements were made at the point of diversion and these values were compared to the calculated values obtained from the equation. Because the error observed by this method was high, it was concluded that this approach would not supply adequate information. As a result, further work using this method was not conducted.

Therefore, a different and more general approach was decided upon. Daily discharge data (14)(15) for the Watertown and Brookings gaging stations, corresponding to the high flow period (March 1 - July 31), were collected and tabulated in the form of a frequency

table (Table 2). This table is an accumulation of information concerning the flow interval and the number of days that the flow magnitude was within each flow interval. This information was used to plot the flow duration curves for the two gaging stations (Figure 8). These curves, plotted as flow rate (cfs) versus the percent of the time that the flow equaled or exceeded the indicated value, show the time-discharge relationship for the gaging stations. It was decided that by applying the appropriate drainage area-discharge relationship to these curves a reasonable duration curve for the flow at Boswell Dam could be established.

The hydrologic factors of the total drainage area were considered to be nearly constant. Therefore, it was decided that the runoff would be porportional to the contributing area. By knowing the Watertown flow, the inflow between the Watertown and Brookings gaging stations, and the percentage of the total drainage area that lies between the Watertown gaging station and Boswell Dam, the magnitude of the Boswell flow could be calculated.

The area under a duration curve for a specific time interval represents the volume of flow for that time interval. Therefore, the difference in the areas under the Watertown and Brookings curves would represent the inflow from the total drainage area between the two stations.

The drainage areas were determined from data supplied by the Soil Conservation Service (16) as follows: area between Watertown

Table 2. Frequency of daily flow magnitudes at the Watertown and Brookings gaging stations for the period of March 1 - July 31.

Flow Interval (cfs)	Number of days and accumulative percentages			
	Watertown 1945-1964 (days)	Accumulative Percentages	Brookings 1953-1964 (days)	Accumulative Percentages
0	513	17.62	156	9.3
1- 9	491	34.49	128	16.94
10- 19	274	43.91	88	22.19
20- 29	242	52.23	156	31.49
30- 39	172	58.14	95	37.16
40- 49	142	63.02	61	40.79
50- 59	141	67.86	56	44.13
60- 69	91	70.98	38	46.39
70- 79	96	74.62	33	48.36
80- 89	73	77.12	39	50.68
90- 99	56	79.04	47	53.48
100- 149	223	86.70	212	66.11
150- 199	105	90.30	138	74.33
200- 249	87	93.28	75	78.80
250- 299	51	95.02	57	82.20
300- 349	28	95.97	62	85.90
350- 399	42	97.40	50	88.88
400- 449	19	98.04	33	90.85
450- 499	13	98.48	32	92.76
500- 599	19	99.12	47	95.56
600- 699	6	99.24	22	96.88
700- 799	4	99.45	10	97.48
800- 899	3	99.54	5	97.78
900- 999	2	99.60	3	97.96
1000-1199	3	99.69	7	98.38
1200-1399	5	99.85	5	98.68
1400-1599	2	99.91	5	98.97
1600-1799	3	100.00	4	99.20
1800-1999			2	99.31
2000-2999			6	99.66
3000-3999			4	99.89
4000-4999			1	99.94
5000-5999			0	---
6000-6999			0	---
7000-7999			1	100.00
8000-8999				
9000-9999				
10,000				

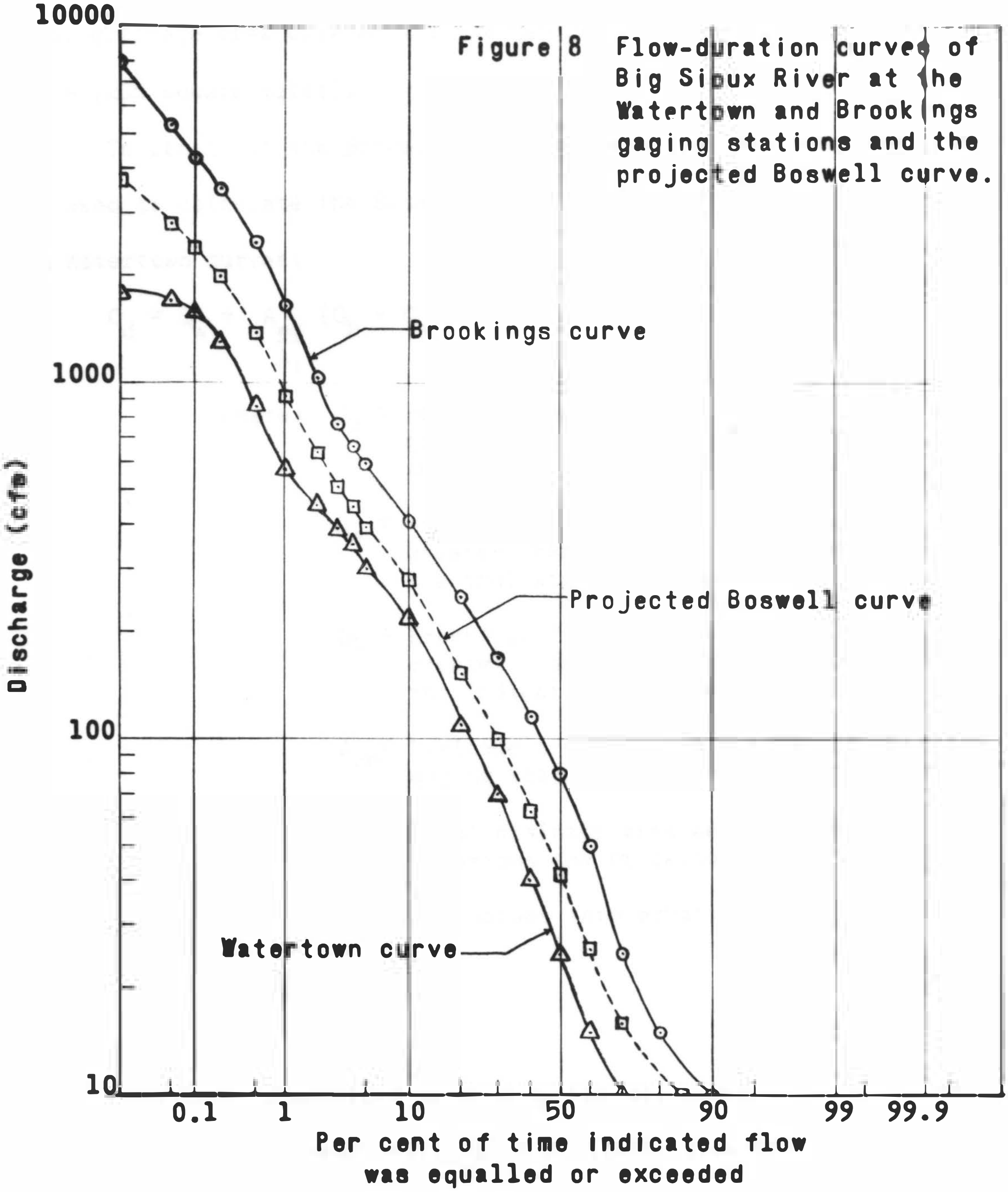


Figure 8 Flow-duration curves of Big Sioux River at the Watertown and Brookings gaging stations and the projected Boswell curve.

and Boswell Dam (A_{wd} = 375 square miles), area between Boswell Dam and the Brookings gaging station (A_{db} = 830 square miles), and the total drainage area between the two gaging stations ($A_t = A_{wd} + A_{db}$ = 1205 square miles).

To construct the Boswell duration curve the following equation was used to calculate the Boswell curve with respect to the Brookings and Watertown curves:

$$Q_d = Q_w + \frac{A_{wd}}{A_t} (Q_b - Q_w) \qquad \text{(Equation 6)}$$

Where: Q_d = Boswell flow which is equal to or greater than the indicated value for the indicated percentage of the time

Q_w = Watertown flow which is equal to or greater than the indicated value for the indicated percentage of the time

Q_b = Brookings flow which is equal to or greater than the indicated value for the indicated percentage of the time

A_{wd} = Drainage area between the Watertown gaging station and Boswell Dam

A_t = Total drainage area between the Watertown and Brookings gaging station

By substituting the area values into equation 6, the following equation was obtained:

$$Q_d = Q_w + 0.31 (Q_b - Q_w) \qquad \text{(Equation 7)}$$

The results obtained by this method are shown in a tabulated form in Table 3 and graphically as the Boswell duration curve in Figure 8.

Assuming that this was the best method to determine a flow-duration curve at Boswell Dam, it was decided to use this curve to estimate the mean annual flow volume. This was accomplished by dividing the area under the Boswell curve into trapezoids and determining the volume of flow represented by each individual area.

Before determining what volume of water could be diverted annually, it was necessary to make some estimate by observation as to how much flow would pass through the Boswell Dam when the gates were closed. By measuring small stream flows below the dam when the gates were open and visually comparing these flows with the amount of water that passed through the gates when they were closed, it was estimated that 100 cfs would flow down stream at all times. Providing that the capacity of the diversion channel was sufficient to carry all of the flow exceeding 100 cfs, then the volume that could be diverted would be represented by the area bounded by the duration curve and the ordinate representing 100 cfs.

The area of a trapezoid is equal to the width multiplied by the average height. Therefore, the volume of flow for each small trapezoidal area under the duration curve could be calculated by the following formula.

Table 3. Percentage of time and flow magnitude from the duration curves for the Big Sioux River at the Brookings and Watertown gaging stations and the projected flow magnitude at Boswell dam.

Percent of the time flow exceeded the indicated flow magnitude	Brookings 1953-1964 (cfs)	Watertown 1945-1964 (cfs)	Projected flow magnitude for the Boswell duration curve* (cfs)
70	25	10	16.2
60	50	15	25.8
50	80	25	42.0
40	115	40	63.2
30	170	70	101.0
20	250	110	153.4
10	410	220	279.0
5	590	300	390.0
4	660	350	446.0
3	760	390	511.0
2	1050	450	636.0
1	1650	575	908.0
0.5	2500	860	1368.0
0.2	3550	1310	2004.0
0.1	4300	1600	2436.0
0.05	5350	1710	2840.0
0.01	8000	1800	3720.0

* Calculated by Equation 7

$$v_i = \frac{(q_1 + q_2)}{2} (P_1 - P_2)\ T\ 86{,}400 \qquad \text{(Equation 8)}$$

Where: v_i = Volume of flow for each incremental trapezoid $(ft)^3$

q_1 = Flow exceeded P_1 percent of the time (cfs)

q_2 = Flow exceeded P_2 percent of the time (cfs)

T = Total time of possible diversion (days)

86,400 is a unit conversion factor

The total volume (V) was then expressed as follows:

$$V = \sum_{i=1}^{n} v_i \qquad \text{(Equation 9)}$$

Where: V = Total volume of flow in $(ft)^3$

v_i = Volume of flow for each individual trapezoid $(ft)^3$

n = Number of trapezoidal areas

The results obtained by this method showed that an annual mean volume of approximately 18,133 acre-ft would be available for diversion.

Volume of Water Possible to Divert with the Present Diversion System

The control of the lake level is not only dependent upon the available water supply at Boswell Dam, but also upon the capacity of the diversion system to transport that water to the lake. Therefore, it was necessary to determine what portion of the water could be diverted into the lakes by the present system.

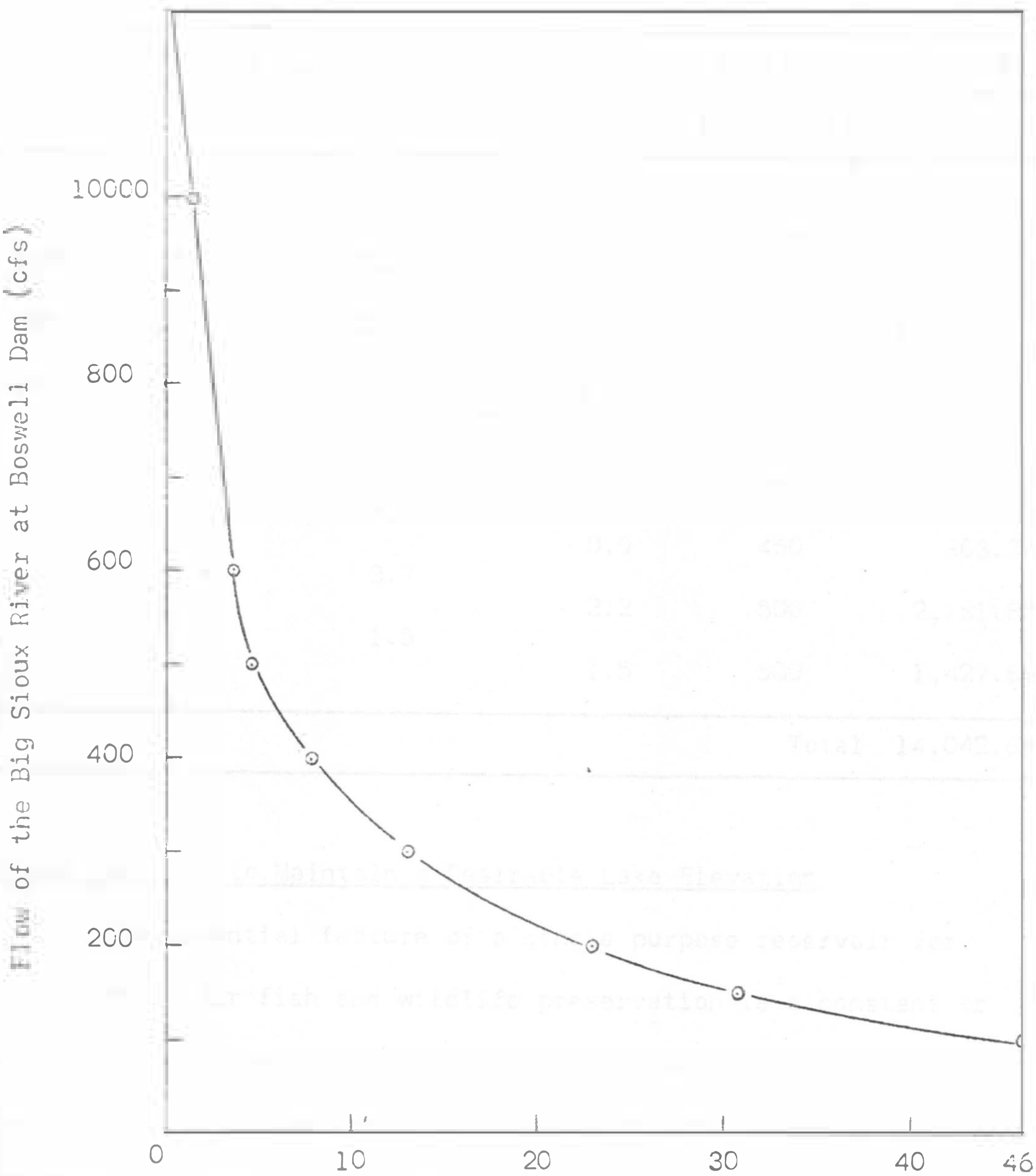
As previously stated, flows up to 100 cfs would pass the Boswell gates. Therefore, the only time that flow could be diverted would be when the flow exceeded 100 cfs. From Figure 8, it was observed that the flow exceeds 100 cfs only 30 percent of the 153 days

from March 1 to July 31. As a result, it would be theoretically possible to divert flow from the river to the lake for only 46 days a year. However, on many of these days the river flow exceeds 100 cfs by only a small amount. Similarly, very high flows such as 1000 cfs occur rather infrequently (averaging about two days a year).

To determine the quantity of flow that could be diverted, it was necessary to relate the number of days of occurrence to the magnitude of the various flows that occurred during the high flow period. This relationship is shown in Figure 9.

Flow measurements made in the diversion channel during a period of high river stage and normal lake elevation determined that the maximum capacity of the present system was approximately 500 cfs. The total volume of flow possible to divert is represented by the area under the curve and between the flow magnitudes of 100 and 600 cfs (Figure 9). The volume represented by this area was determined by dividing the area into small trapezoidal areas and calculating the flow represented by each. This information is shown by Table 4.

The results from this part of the investigation determined that by intensive gate management 14,043 acre-ft of water could be diverted to the lake during an average year.



Number of days the flow was equal to or greater than the indicated value during the period of high flow (March 1 - July 31)

Figure 9. Relationship to the flows at Boswell Dam in excess of 100 cfs to the number of days of occurrence on an average year.

Table 4. Relationship of flow, number of days of occurrence, and total diverted volume

Flow (cfs)	Percent Time	No. of Days	Difference of (Days)	Average Diverted Flow (cfs)	Total Diverted Flow (acre-ft)
100	30	45.9			
			15.3	25	758.69
150	20	30.6			
			7.7	75	1,145.47
200	15	22.9			
			9.9	150	2,945.50
300	8.5	13.0			
			5.3	250	2,628.14
400	5.0	7.7			
			3.1	350	2,152.10
500	3.0	4.6			
			0.9	450	803.32
600	2.4	3.7			
			2.2	500	2,181.85
1000	1.0	1.5			
			1.5	500	1,427.62
				Total	14,042.69

Inflow Required to Maintain a Desirable Lake Elevation

The essential feature of a single purpose reservoir for recreation or for fish and wildlife preservation is a constant or nearly constant pool elevation. The only flucuation of the lake elevation would be the result of inflows or water losses. The magnitude of these factors, as related to Lake Poinsett, was presented previously. Therefore, it was necessary to determine what effect they would have on the lake elevation. To evaluate these effects, a relationship between lake elevation, surface area, and storage capacity was determined. Separate elevation-surface area and

elevation-capacity curves for Lake Poinsett and Dry Lake were constructed. These separate curves were combined to obtain a single curve for the lake complex. The data for plotting the combined curves and the graphical presentation of these data are shown in Table 5 and Figure 10 respectively. These curves can be used to determine what elevation and surface area changes will result from a specific volume of inflow or outflow.

Because a legal lake elevation had not been established, a range of probably control elevations was investigated. This range, from elevation 1649 to 1652 was considered because interested parties[1] indicated that the desirable elevation would be in this range. It was observed from the elevation-capacity curve (Figure 10) that the relationship between the change in lake elevation and the change in capacity was nearly linear in the desirable control range. As a result, 9,350 acre-ft of water would be required to change the lake elevation one foot.

The elevation-capacity information for the Lake Poinsett-Dry Lake Complex was combined with the direct runoff contribution to determine the quantity of water that must be diverted from the Big Sioux River to change the lake elevations. During the diversion period, it was assumed that evaporation losses would be approximately

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[1]Private conversation with Mr. Vern Butler, Engineer-Manager East Dakota Conservancy Subdistrict.

Table 5. Surface area and capacity data for the Lake Poinsett-Dry Lake Complex.*

Elevation in feet above MSL**	Lake Poinsett		Dry Lake		Complex	
	Capacity (acre-ft)	Area (acres)	Capacity (acre-ft)	Area (acres)	Capacity (acre-ft)	Area (acres)
1634.5	3	12	0	0	3	12
1637.0	5,290	4,215	0	0	5,290	4,215
1639.5	16,893	5,060	0	0	16,893	5,060
1642.0	30,793	6,045	0	0	30,793	6,045
1644.5	47,193	7,040	0	0	47,193	7,040
1646.0			Bottom of Dry Lake			
1647.0	65,493	7,605	400	850	65,893	8,455
1649.0	84,893	7,900	3,300	1,500	88,193	9,200
1652.0	104,693	7,980	7,400	1,820	112,093	9,800

* From maps supplied by East Dakota Conservancy Subdistrict.
** Mean Sea Level.

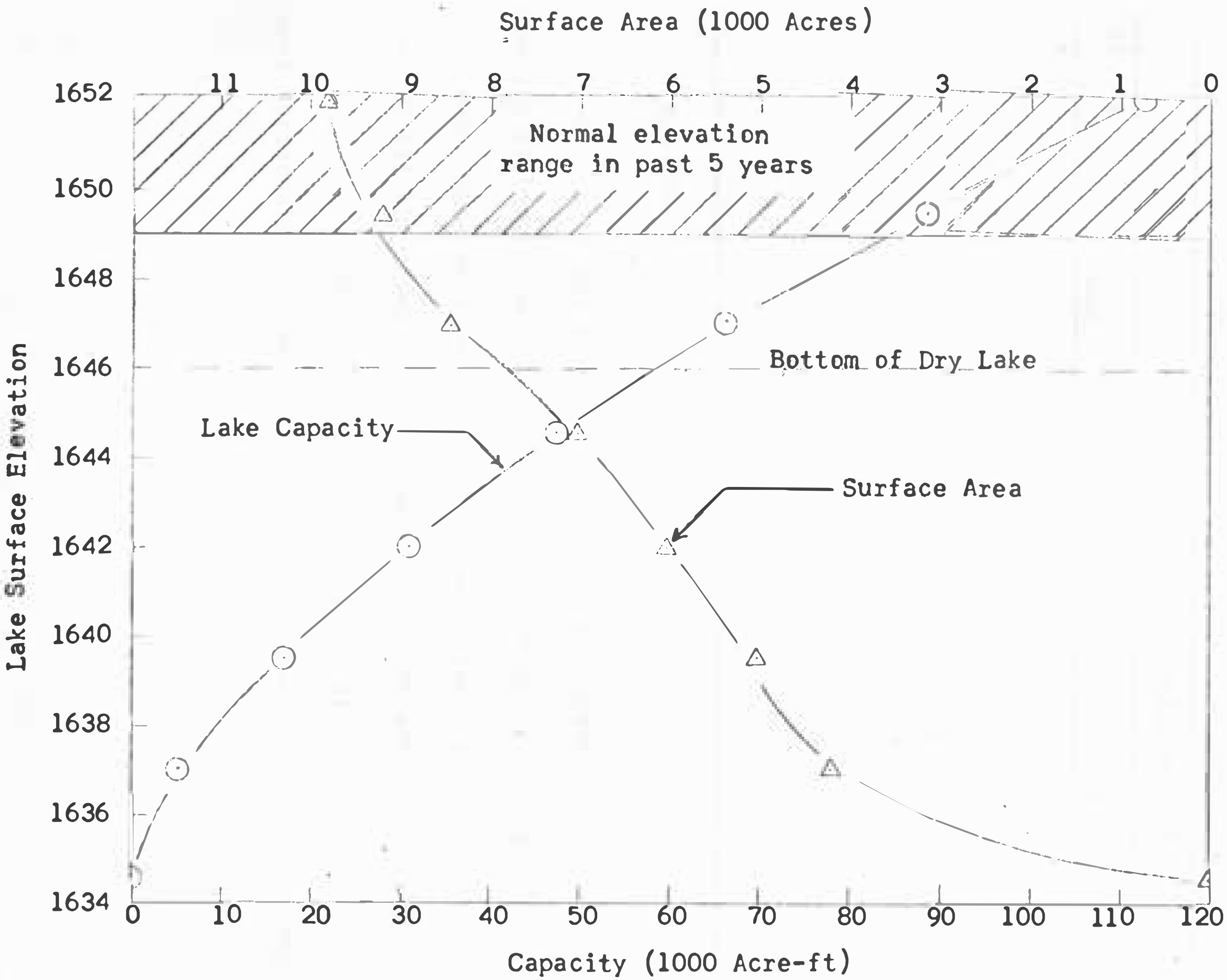

Figure 10. Elevation-surface area and elevation-capacity relationship of the Lake Poinsett-Dry Lake Complex

equal to direct precipitation and therefore the two volumes could be neglected. This information is presented in Table 6 and Figure 11.

Table 6. Volume of diverted water (acre-ft) required to raise the water level of Lake Poinsett from the initial to the control elevation.

	Control Elevation MSL*				
Initial Elevation MSL*	1948	1949	1950	1951	1952
1652					0
1651				0	5,650
1650			0	5,650	15,000
1649		0	5,650	15,000	24,350
1648	0	5,650	15,000	24,350	33,700

* Mean Sea Level

Figure 11 may be used by locating the intersection of the control elevation and the initial elevation curve and reading the required volume of diversion from the abscissa. As an example, assume the initial elevation of Lake Poinsett is 1648 and it is desired to raise the elevation to 1650. From elevation 1650 on the ordinate move horizonally to the curve representing elevation 1648. From this

intersection, read downward to the abscissa to determine what volume of water must be diverted at Boswell Dam to produce the desired lake elevation change. In this case, the required volume would be 15,000 acre-ft.

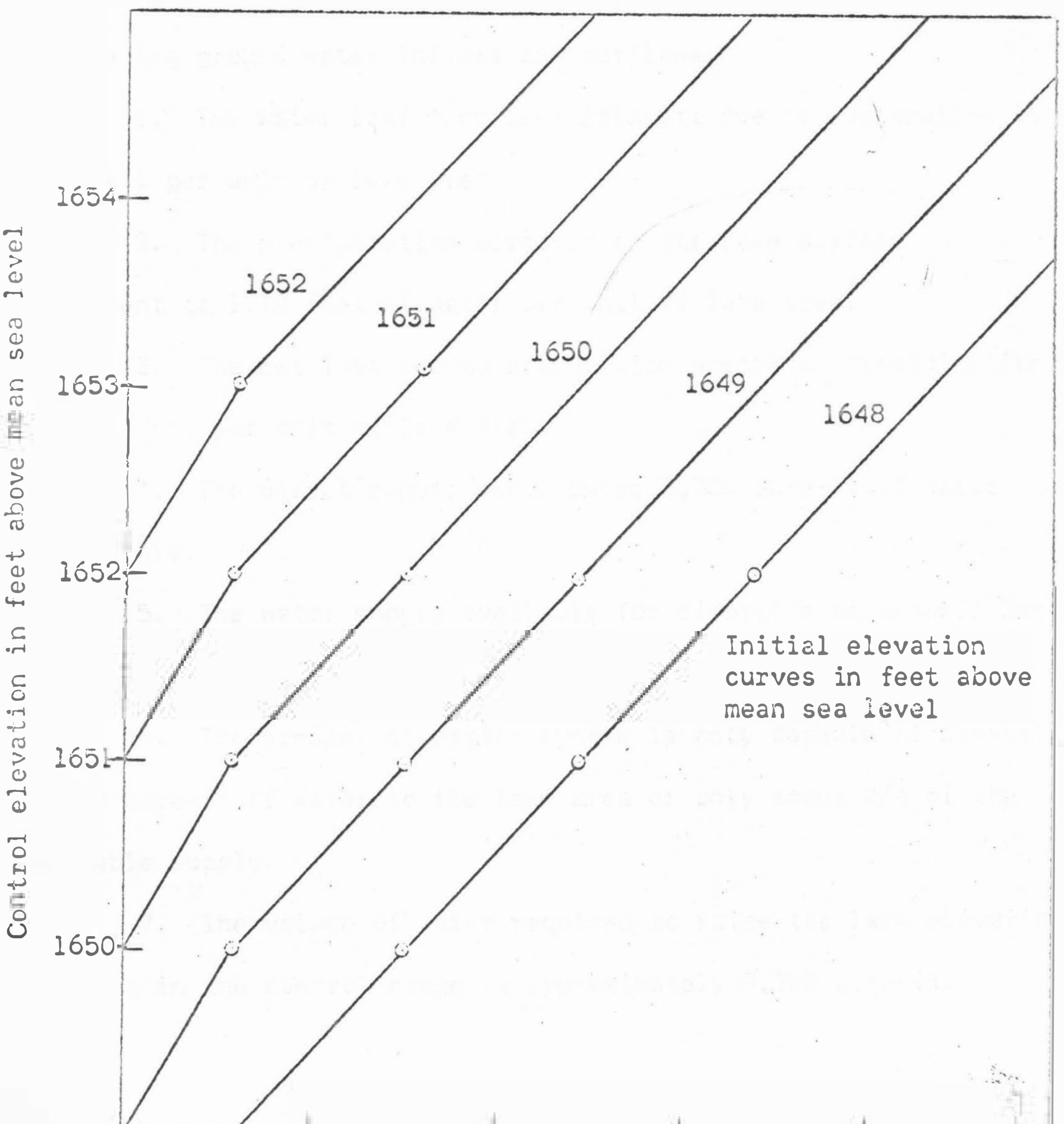

Figure 11. Volume of diverted water required to raise the water level of Lake Poinsett from initial to control elevation

Summary

The following information summarizes the conditions relative to Lake Poinsett that would exist during an average year without considering ground water inflows and outflows.

1. The water lost from Lake Poinsett due to evaporation is 2.82 feet per unit of lake area.

2. The precipitation directly on the lake surface is equivalent to 1.79 feet of water per unit of lake area.

3. The net loss due to evaporation exceeding precipitation is 1.03 feet per unit of lake area.

4. The direct runoff contributes 3,700 acre-ft of water to the lake.

5. The water supply available for diversion at Boswell Dam is 18,133 acre-ft.

6. The present diversion system is only capable of diverting 14,043 acre-ft of water to the lake area or only about 3/4 of the available supply.

7. The volume of water required to raise the lake elevation one foot in the control range is approximately 9,350 acre-ft.

INVESTIGATION OF THE OUTLET STRUCTURE

By taking into consideration the control elevation previously mentioned, it was possible to investigate the types of structures that might be constructed in the outlet of Lake Poinsett. The type selected would be determined by the availability of suitable construction materials, foundation conditions, height of the structure, and safety to property and life downstream.

The size of any type of structure for this project would be relatively small. Therefore, the availability of building materials would not be a significant factor. However, from an economical standpoint the type selected should be the one for which materials are found within a reasonable distance from the construction site.

Foundation investigations are necessary to predict what type of structure might be built at the selected site. Inspection of the materials and information from soil maps (17) indicate that the foundation material in this area would be classed as sands and coarse gravels. This type of material is ideally suited for gravity type masonry or concrete dams of less than fifty feet in height and is also well suited for earthfill and rockfill dams of any desirable height. However, a foundation material of this type exhibit considerable seepage of flow under the dam and provisions should be included to prevent damage to the structure.

The height of the structure would be governed by the desired lake elevation and depth to suitable foundation materials. The proposed lake elevation in the range of 1649 to 1652 and the depth to suitable foundation materials at elevations 1640 to 1646 (17) indicate that an outlet structure for Lake Poinsett would be from 6 to 10 feet in height plus freeboard.

In case of the failure of the structure the topographic features of the outlet channel and the small discharge that would be released would eliminate the possibility of loss of life or destruction of property. Therefore, safety against failure is not considered to be significant.

The most logical selection of an outlet structure would be of the composite type. The overflow portion of the dam would be either a concrete gravity or a sheet piling type of structure. The nonoverflow section would consist of earthfill. It would be necessary to provide for seepage control regardless of the type selected.

The design discharge of the overflow portion of the outlet structure would be limited by the capacity of the outlet channel. It was previously stated that the maximum capacity of this channel is approximately 100 cfs. Therefore, the design discharge for the overflow structure should not exceed this value unless steps [illegible] to improve the discharge capacity of the channel. These steps would include moving or enlarging the bridges that span the channel and deepening the channel.

Considering that the maximum design discharge under present channel conditions would be 100 cfs and that the maximum head on the overflow crest would be approximately one foot, the crest length of the overflow portion of the outlet structure was determined. By substituting these values into equation 3 and 4, and solving for the necessary crest lengths of the overflow structures, it was calculated that a crest length of approximately 30 feet would be sufficient.

In summary, an outlet structure to control the elevation of Lake Poinsett would be of the composite type. The overflow section would be of either the concrete gravity or sheet piling type. The nonoverflow would be constructed of earthfill. The dam would be from 6 to 10 feet in height plus freeboard and would have an overflow crest approximately 30 feet in length.

CONCLUSIONS

The results of this investigation are based on long term records of mean runoff, mean precipitation, and mean evaporation. However, in some instances these factors will be expected to vary from the mean by a large amount. There may be years of drought when little or no precipitation or runoff occurs. Similarly, there may be years when precipitation and runoff are excessive and the water supply may be many times the average amount.

The average annual evaporation from Lake Poinsett exceeds the average annual precipitation on the lake surface by 1.03 feet. To overcome this loss, there must be 9,350 acre-ft of inflow to the lake that may be contributed by direct runoff or diverted from the Big Sioux River. The average direct runoff of 3,700 acre-ft per year plus 5,650 acre-ft diverted from the Big Sioux River would offset this loss. However, the remainder of the water available during an average year for diversion (8,393 acre-ft) is sufficient to raise the elevation of the lake another 0.90 feet. As a result, it was concluded that except during periods of extended drought, the water supply and the present diversion system are adequate with regular management to maintain a desirable controlled surface elevation on Lake Poinsett.

It was also concluded that a suitable outlet structure to control the lake elevation would be of the composite type. The nonoverflow section of this structure could be of earthfill construction whereas the overflow section could be either a concrete

gravity type or sheet piling type. A crest length of approximately 30 feet would be required for the overflow section. Provisions for eliminating seepage under this structure should also be incorporated into the design. The height of the dam, as measured from the elevation of suitable foundation materials to the control elevation, would be from 6 to 10 feet.

AREAS OF FUTURE STUDY

1. Future investigations should be conducted to determine the effect of groundwater flows on the Lake Poinsett water elevation.

2. An investigation should be made to determine what economic impact the stabilization of the water level would have on the surrounding area.

3. Further studies should be made to determine the capacities of the diversion channel and the outlet channel under various conditions.

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