Research Technical Completion Report

EFFECTS OF NON-CONTINUOUS TURNOUT OPERATION ON THE ABERDEEN-SPRINGFIELD CANAL SYSTEM

by

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ABSTRACT

The Aberdeen-Springfield Canal, a large open channel irrigation system in southeast Idaho, is experiencing fluctuating flow rates and water levels due to the non-continuous operation of some pumped turnouts. The fluctuations cause problems in system management and flow variations at other turnouts which require a constant water level to receive a steady flow of water. The canal system was simulated using the hydraulic simulation model CANAL, developed at Utah State University. The objectives were to determine the effects of the non-continuous turnout operation and to evaluate the effects of alterations to the canal system upon the fluctuating flow rates and water levels. Simulations evaluated increasing the number and size of spillways, modifications to channel cross sections, modification and addition of check structures, and restricting the timing and number of turnouts with non-continuous flow. Non-continuous turnout operation was found to cause little variation in water level in some areas of the canal system, and large variation in others. This fluctuation was largely dependent upon the volume of flow, canal cross section, and the presence (or absence) of spillways. Feasible alterations to the canal system had limited effects upon the fluctuations in flow rate and water levels caused by the non-continuous turnout operation. Limiting the number of turnouts operating non-continuously and the length of time for which the turnouts

operated non-continuously was effective in controlling the resulting fluctuations.

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CHAPTER 1

INTRODUCTION

(A) The Aberdeen-Springfield Canal

The Aberdeen-Springfield Canal is a large, unlined open channel system in southeast Idaho (figure 1). Water is diverted from the Snake River north of Blackfoot and flows southwest through the system, delivering water to nearly 500 farm turnouts. The main canal (Main Line) is 36 miles long, and branches into two main channels (High Line and Low Line) which are 32 and 19 miles in length, respectively. There are thirty-four laterals and sublaterals along the system, ranging in length from 4 miles to less than one half mile.

The Aberdeen-Springfield Canal was the first canal constructed in Idaho under the Carey Act (Lewis, 1924). The Carey Act, passed in 1894, allowed each of the western states to receive one million acres of land from the Federal Government, provided that; (a) the state contracted and supervised the construction of irrigation systems, and (b) that the land was placed under irrigation. The American Falls Canal and Power Company, as the Aberdeen-Springfield Canal Company was originally named, appropriated 1250 cubic feet per second from the Snake River in 1895, and proposed the project in April, 1896. By 1909 the company had spent nearly one million

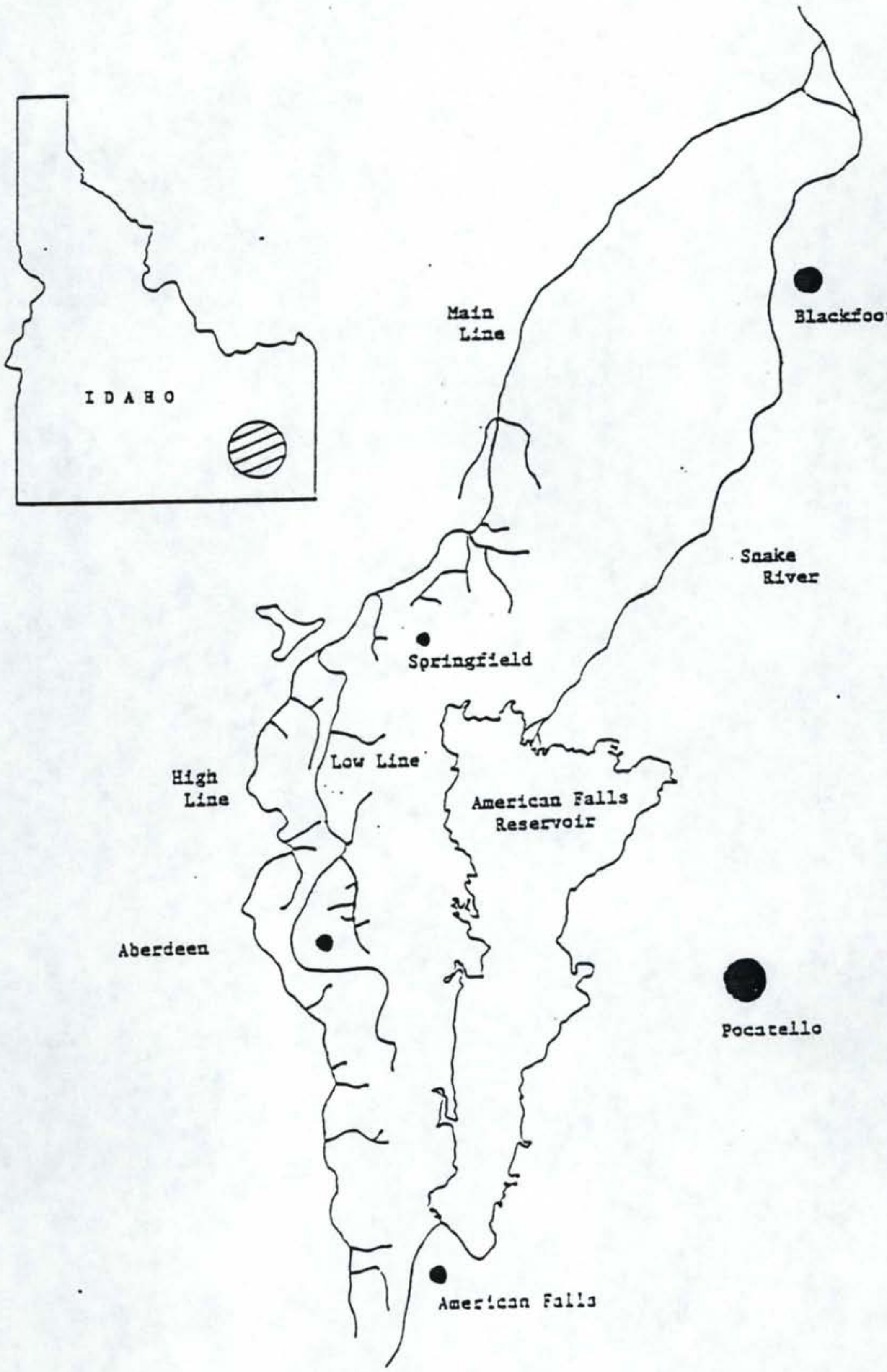


Figure 1. Map showing location and layout of the Aberdeen-Springfield canal system.

dollars constructing canals and had brought nearly 20,000 acres under irrigation (Lewis, 1924). The canal currently irrigates approximately 48,000 acres. The major crops are wheat, barley, potatoes, sugar beets, and alfalfa. There are also many livestock producers (mainly cattle) who use canal water to irrigate pasture and water stock.

The Aberdeen-Springfield Canal Company has 14 full time employees. The General Manager is responsible for coordinating the day to day operations of the canal system and the activities of the rest of the staff. The Secretary and a part time employee are responsible for all record keeping, correspondence, billing, and other office work. There is also a foreman who supervises the operations of 7 ditchriders and 4 equipment operators and mechanics. 3 employees are hired during the summer months to help with weed and rodent control on the canal banks. General Company policies are set by the Board of Directors. The 9 members on the board are elected to the position for 3 year terms.

To receive water from the Aberdeen-Springfield Canal Company irrigators must hold shares in the company. The amount of water each irrigator may use is related to the number of shares held, as well as the volume of "storage water" and "natural flow" which is available. The company owns 70,794 acre feet of storage water in Jackson Reservoir, 143,278 acre feet in Palisades Reservoir, and 53,784 acre feet in American

Falls Reservoir, although the actual amount available depends upon the volume of water actually in the reservoirs. When the natural flow in the Snake River becomes so low that it will not satisfy the water requirements of the canal system, the system must use its storage water. The water is turned into the river by the Department of Water Resources and diverted into the canals. Whether the canal is using natural flow or storage water, the instantaneous flow rate which may be diverted by each irrigator is limited to one miners inch (1/50 cfs) per share. When on natural flow, no upper limit on the total volume of water diverted is enforced. When the canal is using storage water, the total volume diverted is limited to the amount of storage water available divided by the total number of shares. Some allowance must also be made for the operational losses of the canal system (seepage, evaporation, and spills). To ensure enough water for crops, most irrigators have one share per acre.

In the day to day operations on the canal, irrigators must order water by 2:30 pm on the day before the water is to be delivered. The seven ditchriders are each responsible for setting the turnouts on a portion of the canal. Each ditchrider follows the same route every day, with the ride beginning at 8:00 am and ending about 1:00 pm. Each turnout is set at approximately the same time each day, so water is delivered in increments of 24 hours. The routine is slightly different on Sunday, when the ditchriders work only until noon. The General Manager talks with the ditchriders twice a day, at 8:00 am and 2:30 pm (again, Sunday slightly different). The orders for the next day are exchanged, and readings from staff gages at various locations are reported. The General Manager then orders the necessary adjustments in the flow rates, spillways, and check structures based on past years data and operating experience. The gage readings are depth measurements only, and except for inflow into the system, the flow rate at any location in the Main Line, High Line, and Low Line is merely estimated. Flow rate into some of the laterals is measured, and on others the flow rate is estimated based on ditchrider experience.

The standard turnout on the Aberdeen-Springfield Canal system is a box-submerged orifice (figure 2). The flow rate is determined by measuring the difference in water surface elevation across the known area of the rectangular orifice ($h_1 - h_2$). The water level inside the box is controlled by adjusting the gate on the circular culvert which delivers the water from the box. Constant flow rates are dependent upon stable water levels in the canal and downstream of the culvert. The flow generally exits the downstream end of the culvert into a ditch, a pond, or a pump with some type of spill structure to remove the water when the pump is shut off. The ditchrider sets the turnout to deliver the correct flow rate, and the flow is delivered for 24 hours (continuously), until the ditchrider returns the next day.

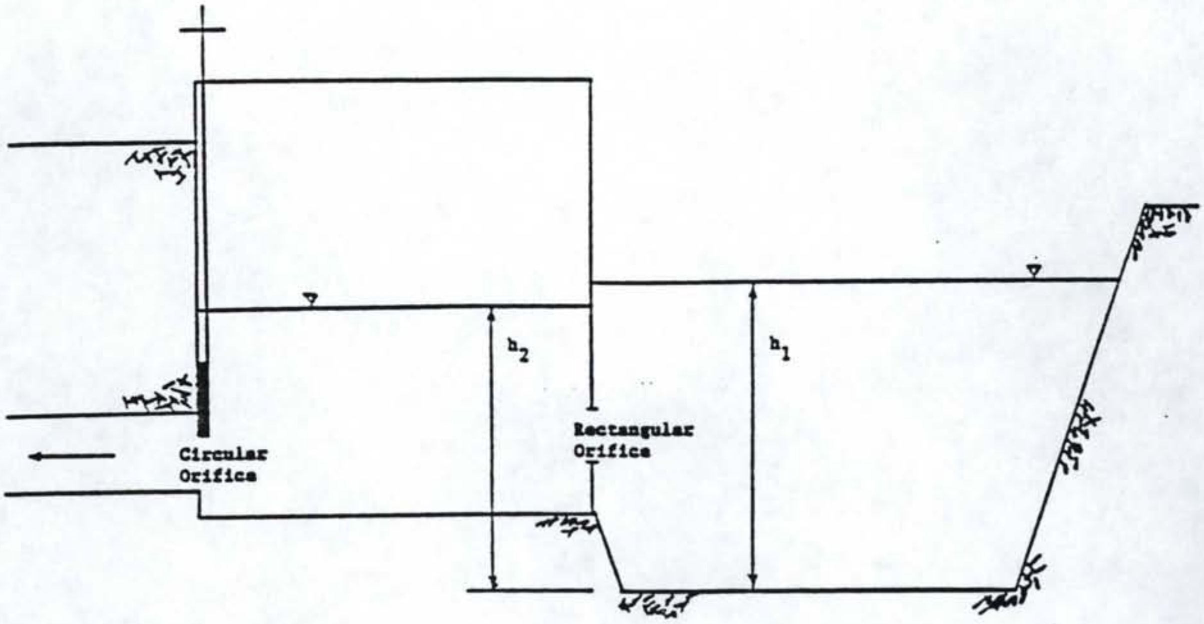


Figure 2. Box submerged orifice turnout.

Some turnouts along the system have had pumps attached directly to the canal. The pump controls the flow through the turnout instead of gravity. With this type of turnout, when the pump is turned on, it removes the required amount of water from the canal, and when the pump is turned off, the water remains in the canal. This type of turnout is controlled by the irrigator, and is a non-continuous flow turnout.

In the mid 1970's many farmers along the canal switched from gravity irrigation to sprinkler systems. Sprinkler irrigation provides a number of benefits over traditional gravity irrigation methods. It can allow for more precise application of water, and may reduce problems associated with over-irrigation, such as leaching soil nutrients and nitrates into the groundwater. Sprinkler irrigation is also more labor efficient than gravity irrigation. Land which is too uneven or too permeable for gravity irrigation can be efficiently irrigated with sprinklers. Sprinklers also provide crop and soil cooling, frost protection, and a means to apply chemicals (pesticides, herbicides, fungicides, and fertilizers) to the crops. Over this period of time the amount of water delivered by the canal decreased. This reduction was due to; 1) more efficient methods of irrigation, 2) a shift in crop production from alfalfa, which uses a large quantity of water, to grain, which needs much less, 3) the conversion of farmers to well water, and 4) slightly less irrigated area. Irrigators converted to wells because they could use the water more efficiently with their pumps, and the necessary volume of water was always available when needed. After the correct amount of water is applied to the crop, the pump may be shut off and the water remains in the ground. In this way the irrigators avoided having to use the water for twenty-four hours as they did on the canal.

In response to irrigators demands, the company began allowing some direct hookups to the canal. Water was not diverted through these turnouts when pumps were shut off. The method was extremely convenient for the irrigators. They had all the benefits of a well, without the additional cost of drilling the well and pumping the water from the ground. When enough water had been applied, or irrigation sprinkler lines needed to be moved, they simply shut off the pump and the water remained in the canal. The turnout was charged for the water for 24 hours, even though it was only taken for part of the time. While the use of the water was lost to the irrigator, water was cheap and plentiful and the loss was worth the added convenience and efficiency on the farm.

Over the years, more and more irrigators were allowed to divert canal water non-continuously by hooking their pumps directly to the canal. As the number of farmers doing this was initially small, only minor and localized problems were caused by the fluctuating flow in the canal that resulted from rejection of flow by the turnouts. Eventually the fluctuation in the canal flow began to cause problems in the management of the canal system. Turnout flow rates, which are dependent upon stable water levels in the canals, became difficult to measure and control. Fluctuating water levels infringed upon canal freeboard, resulting in excessive spills and in some cases, overtopping of canal banks.

In 1988 the Aberdeen-Springfield Canal Board of Directors voted to institute a continuous flow policy. Under this policy, which was to be implemented by 1991, all irrigators would have to take continuous flow. Irrigators using sprinkler irrigation systems would have to make provisions for storage or spill. This policy was rescinded in 1989 pending an evaluation of the canal system to determine any alterations to the system which would accommodate non-continuous flow. The University of Idaho was contracted to do a study of the canal system, using a computer simulation model to predict the effects of various alterations to the system. This paper is a direct result of this study.

(B) Canal Simulation Models

Unsteady, non-uniform flow in open channels can be accurately predicted using the one dimensional equations (parallel to flow direction) based on the conservation of mass and the conservation of momentum (Chow, 1959). These equations are commonly called the Saint-Venant equations. Gichuki, Walker, and Merkley (1990) present the equations as follows:

Conservation of Mass (Continuity Equation)

$$\frac{dQ}{dx} + \frac{dA}{dt} + I = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

Conservation of Momentum (Equation of Motion)

$$\frac{1}{A\,g}\,\frac{dQ}{dt} + \frac{2\,Q}{A^2 g}\,\frac{dQ}{dx} + (1-F^2)\,\frac{dy}{dx} - S_0 - S_f = 0 \quad \ldots\ldots \quad (2)$$

Where Q = flow rate (L^3T^{-1})

A = flow cross-sectional area (L^2)

y = flow depth (L)

I = net seepage outflow (L^2T^{-1})

x = distance (L)

t = time (T)

g = ratio of weight to mass (LT^{-2})

S_0 = channel slope

S_f = friction slope

F = Froude number.

Several types of mathematical models for determining the solution of the Saint-Venant equations have been used, including empirical, linearization, hydrologic, and hydraulic models (Fread 1978 - in Husain et al. 1988). The hydraulic model is best suited for solving unsteady flow problems in canals. Husain and others (1988) list the following simulation programs which have been applied on various projects around the world.

> Gradually Varied Unsteady Flow Profiles (GVUFP) - developed by the Tennessee Valley Authority and applied to single river systems. Program limitations include the selection of small and equal length reaches.

> Stream Hydraulic Package (SHP) - developed by Resource Management Associates for Hydraulic Engineering Center (HEC). The program is applicable to single river systems.

> Branched Network Flow Package (BNFP) - developed by the United States Geological Survey, this program is also applicable to single river systems.

> Dynamic Wave Operational Model (DWOPER) - developed by the National Weather Service, the program is applicable to single and branched network systems.

DWOPER is the only model that has been widely used for simulation of unsteady flow conditions in canals because it has more options for boundary conditions, is applicable to branching systems, and has a special computational algorithm which provides numerical stability. Husain and others (1988) modified DWOPER in order to simulate flow through the Al-Hassa irrigation system in Saudi-Arabia, adding the hydraulic equations for features such as submerged and free-flow control gates, bottom falls, and junctions with distributaries. The modified version is called the Channel Network Model.

Several researchers have developed and used simulation models to study canal system operations. Zimbelman and Bedworth (1983) developed a computer program which would allow an open channel system to be operated on a demand basis rather than a scheduled basis. Test results concluded that the algorithm was capable of controlling a system at a higher level of efficiency than manual control, where efficiency is measured in terms of the stability of water surface elevations within the system. The program requires extensive monitoring of conditions throughout the system and a centralized control.

Clemmens (1986) used a simulation model to develop demand patterns for hypothetical surface irrigation conditions. These demand patterns were used to determine the canal capacity required to meet various levels of demand. Results showed that canals operated on a demand basis required 2-3 times greater capacities than systems operated on a rotation or continuous flow basis.

Hamilton and Devries (1986) developed a computer model for simulating operation of non-branching canal systems composed of a series of channels separated by control structures. The model yielded good results when used to simulate flow through the Delta Field Division Canal of the California Aqueduct.

Manz and Westhoff (1987) used a modified version of the Irrigation Conveyance System Simulation (ICSS) model, developed at the University of Calgary, to determine the effects of aquatic weeds growing in canal systems. They found that the canal roughness changes substantially as the amount and type of aquatic plants in the canal changes.

Merkley (1987) developed the hydraulic simulation model CANAL to simulate unsteady flow conditions in branching canal systems. CANAL was developed at Utah State University as part of the Water Management Synthesis II Project (WMS II), a program funded by the United States Agency for International Development (US AID). CANAL was developed because other simulation models were limited by one or more of the following constraints: (1) research oriented; (2) available primarily for main-frame computers; (3) do not handle canal filling, turnouts, or branching canal networks; (4) are not readily transferable from one canal system to another; (5) have a large computation time (Gichuki, Walker, and Merkley, 1987). Rogers also evaluated CANAL (Merkley and Rogers, 1991). Some of his findings are presented below.

Computational Accuracy - CANAL uses an implicit solution technique to solve the complete Saint Venant equations. Evaluation has proven the numerical solution to be technically correct within the limitations of the program's 5 minute calculation increment. However, numerical results are not accurate for rapid flow changes and events of duration shorter than 5 minutes. Operations which yield sudden depth and flow changes are not modeled accurately, instead yielding slower and smoother changes. Depth or flow "spikes" can be missed, resulting in inaccurate maximum and minimum values.

Numerical Solution Criteria - CANAL's numerical solution satisfies the criteria of conservation of mass, stability, and convergence. Direct testing for conservation of mass yielded perfect results, with no loss or gain of water. The implicit solution is unconditionally stable. As with other models, CANAL's solution will diverge when flow conditions are beyond the intended scope of the model (e.g. dewatering a canal reach).

Robustness - CANAL is very robust, seldom encountering execution errors. Input data restrictions and automatic error checking prevent most situations that would create problems with the solution.

Internal and External Boundary Conditions - Boundary condition analysis in CANAL is of average accuracy. Mathematical representation of structures is relatively simple and straightforward, yielding a robust and dependable solution. In some cases, accuracy is compromised by ignoring details such as transitions and variable gate coefficients. This inaccuracy may influence some studies, but usually is not important.

Turnouts - The turnout modeling capability of program CANAL is excellent.

Operations Duplication - CANAL's three different modes of operation offer the user a great deal of flexibility. Structure settings and discharges can be prescribed through user-created schedules entered prior to program execution, or they can be specified during program execution by the user or by the automatic gate setting mode. The program is an excellent tool for duplicating day to day operations of real canals, allowing an operator to test different control structure settings and observe the hydraulic consequences. Duplication of abnormal or emergency operations is more difficult.

Conclusions - CANAL is an excellent tool for modeling daily canal operations to help calculate structure settings, determine control strategies, and provide operator training. It is easy to use and understand, runs quickly, can accurately simulate most canals, and has minimal hardware requirements. CANAL is not as well suited to studying canal system design, where worst case flow scenarios involving rapid flow change are involved.

The hydraulic simulation model CANAL was eventually chosen for the study of the Aberdeen-Springfield canal system. Information about the operation of CANAL is provided in Appendix A.

CHAPTER 2

PURPOSE AND OBJECTIVES

(A) Project Justification

The problems with fluctuating water surface and flow levels which the Aberdeen-Springfield Canal Company is experiencing have the potential to occur in other irrigation systems. Irrigation with sprinkler systems has become more prevalent, and now accounts for over half of the total cropland in Idaho (SCS, 1984), and an estimated 90% of the cropland irrigated by the Aberdeen-Springfield Canal. Sprinkler irrigation provides a number of benefits over traditional gravity irrigation methods. It can allow for more precise application of water, and may reduce problems associated with over-irrigation, such as leaching soil nutrients and nitrates into the groundwater. Sprinkler irrigation is also more labor efficient than gravity irrigation. Land which is too uneven or too permeable for gravity irrigation can be efficiently irrigated with sprinklers. Sprinklers also provide crop and soil cooling, frost protection, and a means to apply chemicals (pesticides, herbicides, fungicides, and fertilizers) to the crops.

Hooking a pump directly to the canal is an easy, convenient method to draw water for sprinkler irrigation. The water is there when needed, and no provisions need to be made to store or remove the water when the pump is shut off. The irrigator essentially has the convenience of drawing water from a well without the added pumping expense. This project's goals were; 1) determining the effects of these non-continuous flow turnouts on the flow rates and water levels in the canals, and 2) evaluate methods to minimize any adverse effects.

The objective of the project is not to test the hydraulic simulation model CANAL. The model has been tested extensively by the author and others. It has been verified using data from the South Gila Canal near Yuma, Arizona, and on the Abraham Canal near Delta, Utah, and also by comparisons with the U.S. Bureau of Reclamation Gate Stroking Model (GSM) (Gichuki, Walker, and Merkley, 1990). Merkley also applied CANAL to the Right Main Canal of the Lam Nam Oon Irrigation Project in Thailand to determine appropriate gate scheduling for reduced fluctuations in the canal (Merkley, Walker, and Skogerboe, 1989, and Merkley, Walker, and Gichuki, 1990) .

While this project does not address the accuracy of the model, it does address the procedures for calibrating the model to the Aberdeen-Springfield canal system, and the benefits which could be obtained by using the model as an operational tool.

(B) Canal System Simulation Concepts

There are four general requirements for using a computer model to evaluate various flow scenarios through a canal system; (1) a simulation program which will accurately simulate the system, (2) detailed measurements of the physical parameters of the system, (3) calibration, or "fitting" to the system, and (4) several alternative methods of control which can be evaluated. These requirements are explained in more detail in the following sections.

(1) Computer Model

The computer model must be capable of accurately simulating the canal system. For any simulation, it is important that control structures on the system are available within the model. Channel configurations, cross-sections, and lengths must be acceptable for the model. For this project, it was especially critical that flow through turnouts could be accurately represented, reflecting the variation in flow rate with canal depth. Also important is that the model can be run on readily available computer equipment.

(2) Physical Parameters

Physical properties of the canals and canal structures must be known for an accurate hydraulic simulation. Flow through the canal system is dependent upon many parameters, including; length, cross-section, seepage rate, longitudinal slope, roughness, control structures, turnouts, spillways, and storage reservoirs.

(3) Calibration

Measured values of the canal system parameters do not provide enough information to accurately simulate canal flow. The measured values provide a base to work from, and this must be manipulated until the simulation results match the actual flow in the canals. Adjustments must be made to compensate for spatial variability in parameters, inaccurate or incomplete measurement, and for parameters whose values cannot be measured.

(4) Alternative Flow Scenarios

Using a computer model that simulates hydraulic flow, a number of alternatives can be evaluated to determine their effects on fluctuating water levels in the canal system. Specific items to be examined for this study include:

1) Variation of channel cross-section to provide additional storage for water that remains in the canal during non-continuous flow periods.

2) Modification and/or addition of spillways to remove excess water and reduce the fluctuation in the canals.

3) Modification and/or addition of storage within the system, where excess water could be stored when needed, but reclaimed for later use.

4) Modification and/or addition of check structures to better control water levels in the canals.

5) Controlling non-continuous users so that the number of users shutting off at any one time remains manageable.

6) Limiting the number of non-continuous users so that the potential for rejection into canal is within acceptable levels.

CHAPTER 3

PROCEDURES

(A) Data Collection

(1) Canal Length and Structure Location

Maps of the Aberdeen-Springfield Canal system were used to determine the general layout of the canals and laterals. The system is covered by 11 US Geological Survey 7.5 minute topographic maps. Maps of the system were also available from the Aberdeen-Springfield Canal Company. These maps showed the canal system with regard to township, range, and section numbers. Both the USGS maps and the company maps were incomplete and somewhat outdated. The USGS maps were made or revised between 1955 and 1979, while the company maps were last updated in 1970. Several laterals had been shortened or removed in the intervening years, and the maps did not show the locations of canal structures or turnouts.

A three-wheeled motorcycle was ridden along the canal banks of nearly every stretch of canal bank to determine the lengths of various canals and laterals and the location of control structures and turnouts. The cycle was equipped with a counter that recorded the revolutions of the front wheel. The wheel was calibrated over one mile stretches of road to reflect 5.7 feet/revolution. Locations of canal structures and turnouts were recorded, and used in the construction of the simulation model. The data is displayed in Appendix B. Some reaches of the system could not be travelled with the cycle due to fences, sprinkler systems which ran over the bank of the canal, and otherwise impassible banks. Length of these reaches was determined from aerial photographs. Ditchriders located turnouts and control structures on the aerial photos, and a numerical value was assigned to the position.

A new map was constructed which shows the present canal system. The map was constructed by the University of Idaho Cartography Department using aerial photos, USGS topographic maps, and canal company maps as a reference. The map shows turnouts, control structures, spillways, irrigated land, roads, and towns on the system. Information from the map was used in the construction of the simulation model.

(2) Flow Measurement and Control

Data on flow measurement and control was obtained from Company records and actual measurements and observations on the canal system. As there are 34 laterals and sublaterals, only flow measurement and control devices on the Main Line, High Line, and Low Line are given here. Structures on the laterals may be determined by examination of the canal configuration files, located in the appendix.

(3) Main Line

Flow is diverted into the Aberdeen-Springfield Canal system from the Snake River north of Blackfoot. Water diverted from the river flows through a large channel for approximately 3 miles to the head of the main line. There the water is held in a pool and the necessary flow of water diverted into the Main Line. An automatic control structure keeps the pool at a constant level, spilling the extra water back to the Snake River. The flow rate entering the Main Line is measured at a U.S. Bureau of Reclamation (USBR) gage located 750 feet downstream. The gage measures the depth of flow through a section of canal with concrete sidewalls. The section is flow-metered monthly by the USBR to determine necessary shifts in the flow curve. The gage records the flow depth every 15 minutes, and this data is transmitted to the USBR. The canal company reads the gage manually several times every day, and records the readings from approximately 8:00 a.m. and 2:00 p.m. daily.

The Main Line is 34.3 miles long. The water flows 18.4 miles to the first control structure. Water level from a staff gage located upstream of the control structure is recorded every morning and afternoon by the ditchrider. The structure consists of nine rectangular weirs 5.2 ft wide and one rectangular sluice gate 7.2 ft wide. The sluice gate is automatically controlled to keep the water surface on the upstream side of the control structure constant. The weirs consist of wooden boards (2" X 4", 2" X 6", etc.) inserted into grooves in the concrete structure, and are adjusted manually by adding or removing boards. Although the width varies, this is the standard configuration of weirs throughout the system. Not all sluice gates are automatically controlled.

There are four control structures on the Main Line. The second is located 1.9 miles below the first. There are staff gages upstream and downstream of the control structure. A rectangular sluice gate is automatically adjusted to keep the water surface on the upstream side constant. Laterals A and H begin above the check. These laterals have no control structures on the canal, and are dependent upon a constant water level in the main canal to keep flow into the laterals constant. Both laterals do have control structures located some distance from the Main Line which control flow into the tail ends of the laterals.

The third control structure is located 6.1 miles below the second, 26.5 miles from the head. The structure consists of 7 rectangular weirs and 1 rectangular sluice gate, each approximately 5.7 feet wide. The sluice gate is automatically adjusted to maintain a constant water level upstream of the structure. An emergency spillway (Hilton Spill Area) is located immediately above this control structure. The entire Main Canal can be diverted into the spill area in case of an emergency. The spill area is a natural low spot with very high permeability, which allows the diverted water to seep into the ground within a few days. Laterals C and C1 originate from the Main Canal 0.75 miles upstream of the structure. A staff gage is located downstream of the structure, and is read twice daily.

The last control structure on the Main Line is also the head of the High Line. The structure consists of two weirs 3.75 feet wide and four manually controlled sluice gates 4.0 feet wide. The water level is maintained in the pool above the control structure by an automatic sluice gate on the spill into the Big Fill Reservoir. The constant water level allows flow into the High Line and Low Line canals to remain fairly constant. Water may be drawn from the reservoir into the Low Line canal, so spilled water is not lost. Flow into the reservoir is measured twice daily as it flows over a Cippoletti weir.

(4) High Line

A staff gage is located near the head of the High Line, and is read twice daily. The gage is not a rated section, and the exact flow rate into the High Line is unknown. Adjustments are made on a day to day basis using operator experience and past years data.

The High Line is 30.5 miles long, and has 14 control structures. The location, type, and special characteristics of these are presented below.

1) Location

from Head:	3.4 miles
Rectangular Weirs:	7 * 5.0 ft
Rectangular Sluice:	1 * 5.0 ft

The sluice gate is automatically controlled to maintain constant water level upstream of the structure. F Lateral originates immediately upstream of the check, and E Lateral originates 1.0 miles upstream.

2) Location

from Head:	11.6 miles
from US Structure:	8.3 miles
Rectangular Weirs:	4 * 4.6 ft
Rectangular Sluice:	1 * 5.0 ft

The sluice gate is automatically controlled to keep the water level upstream constant. J and I laterals originate 1.2 miles upstream, G lateral 1.6 miles upstream. A staff gage located 2.4 miles above this structure is read and recorded once daily.

3) Location

from Head:	12.5 miles
from US Structure:	0.8 miles
Rectangular Weirs:	5 * 5.1 ft
Rectangular Sluice:	1 * 5.1 ft

The sluice gate is automatically controlled to keep the water level upstream constant. No staff gage readings are recorded at this control structure.

4) Location

from Head:	13.9 miles
from US Structure:	1.4 miles
Rectangular Weirs:	6 * 4.6 ft
Rectangular Sluice:	0

Manually controlled structure, no staff gage readings recorded.

5) Location

from Head:	15.9 miles
from US Structure:	2.1 miles
Rectangular Weirs:	6 * 4.6 ft
Rectangular Sluice:	0

The High Line spill is located 1.2 miles upstream of the structure. The spill is used for emergencies only. K lateral originates immediately upstream of the check. No staff gage readings are recorded at this control structure.

6) Location

from Head:	17.3 miles
from US Structure:	1.4 miles
Rectangular Weirs:	4 * 4.5 ft
Rectangular Sluice:	1 * 4.7 ft

The sluice gate is automatically controlled to keep the water level upstream constant. No staff gage readings are recorded at this control structure.

7)	Location	
	from Head:	18.0 miles
	from US Structure:	0.6 miles
	Rectangular Weirs:	5 * 4.6 ft
	Rectangular Sluice:	0

Manually controlled structure, no staff gage reading recorded.

8)	Location	
	from Head:	24.3 miles
	from US Structure:	6.4 miles
	Rectangular Weirs:	2 * 10.2 ft
	Rectangular Sluice:	0

Nash spill is located 1.7 miles above this control structure. The reading from the staff gage at this spill is recorded once daily. The spillway has 51.5 ft of surface area split up among ten adjustable weir bays.

9)	Location	
	from Head:	25.6 miles
	from US Structure:	1.3 miles
	Rectangular Weirs:	2 * 5.0 ft
	Rectangular Sluice:	1 * 5.0 ft

The sluice gate is automatically controlled to keep the water level upstream constant. Q lateral originates 0.6

miles above this structure. No staff gage readings are recorded at this control structure.

10) Location

from Head:	26.0 miles
from US Structure:	0.3 miles
Rectangular Weirs:	0
Rectangular Sluice:	3 * 3.9 ft

Cedar Spill is located directly above the structure. The water level upstream of the control structure is maintained with an automatic sluice gate on the spill structure. There is also 18.3 ft of surface area split among five weir bays. Water levels on staff gages above and below the control structure are recorded twice daily.

11) Location

from Head:	27.2 miles
from US Structure:	1.2 miles
Rectangular Weirs:	0
Rectangular Sluice:	2 * 4.0 ft

This is a manually controlled structure. There is a spillway into a pond immediately above this check. Water from the pond can be recovered into the High Line 525 feet below the check, so the spilled water is not lost. A gage inside the pond is read and recorded once daily.

12) Location
from Head: 28.2 miles
from US Structure: 1.0 miles
Rectangular Weirs: 2 * 5.8 ft
Rectangular Sluice: 0

Manually controlled structure, no staff gage reading recorded.

13) Location
from Head: 30.1 miles
from US Structure: 1.9 miles
Rectangular Weirs: 2 * 4.7 ft
Rectangular Sluice: 0

This is a manually controlled structure, no staff gage reading recorded. P Lateral originates 0.3 miles upstream of the structure.

14) Location
from Head: 30.4 miles
from US Structure: 0.3 miles
Rectangular Weirs: 2 * 4.1 ft
Rectangular Sluice: 0

S Lateral originates directly above this control structure. The bulk of the flow goes down S lateral as there are only 3 turnouts further down the High Line. A small pond takes some spill water here. This is a private pond and this water cannot be recovered by the canal company. For modeling purposes this is treated as the end of the High Line and the flow to the remaining three turnouts is treated as one turnout.

(5) Low Line

Flow is diverted into the Low Line over a weir structure. The flow goes into a concrete lined drop structure with three separate drops before it enters the Low Line canal. This flow is augmented with flow from the Big Fill Reservoir at the bottom of the drop structures. A staff gage measures the depth of the pool here and is recorded twice daily. The Low Line is 20.1 miles long, and has six control structures. Location, type, and special characteristics of these control structures are presented below.

1) Location

from Head:	2.2 miles
Rectangular Weirs:	5 * 7.0 ft
Rectangular Sluice:	0

Manually controlled structure, no staff gage readings recorded.

2) Location

from Head:	8.9 miles
from US Structure:	6.7 miles
Rectangular Weirs:	4 * 4.5 ft
Rectangular Sluice:	0

U Lateral originates 425 feet above this control structure, and V lateral originates 0.8 miles below it. This is a manually controlled structure, with no staff gage readings recorded.

3) Location

from Head:	11.6 miles
from US Structure:	2.7 miles
Rectangular Weirs:	4 * 5.5 ft
Rectangular Sluice:	0

There is a spillway 1.2 miles below the control structure. This is a manually controlled structure, with no staff gage readings recorded.

4) Location

from Head:	15.6 miles
from US Structure:	4.0 miles
Rectangular Weirs:	4 * 4.0 ft
Rectangular Sluice:	0

W lateral originates 425 feet above the control structure. There is also an emergency spillway located 1320 feet above the check. This is a manually controlled structure, with no staff gage readings recorded.

5) Location

from Head:	18.8 miles
from US Structure:	3.2 miles
Rectangular Weirs:	1 * 4.8 ft
Rectangular Sluice:	1 * 4.8 ft

There is a spillway located 1.0 mile above the control structure. The spillway has an automatic sluice gate to maintain a constant water surface in the canal. Water level at a staff gage near the spill is read and recorded twice daily.

6) Location
from Head: 20.1 miles
from US Structure: 1.3 miles
Rectangular Weirs: 2 * 4.6 ft
Rectangular Sluice: 0

This is essentially the end of the Low Line. Flow that passes over this structure continues to a spillway where it is spilled into American Falls Reservoir. There are no turnouts past this point. This is a manually controlled structure, with no staff gage readings recorded.

(6) **Cross-sections**

Canal cross-sections were surveyed using a construction level. 187 cross sections were measured, with a minimum of one every mile on the Main Line, High Line and Low Line canals, one every half mile on the laterals and locations where the cross-section varied rapidly. Relative elevations at four to seven points from bank to bank were taken and the position of each point measured with regard to one bank. CANAL requires a trapezoidal cross-section, so the data were plotted and the best trapezoidal cross section fit to the data by hand. Later, when the simulation model was being built, the cross-sections for each reach were combined, and the best fit for the reach determined, again by hand. The cross-sections used for the simulation model are given in the CANAL configuration files, located in Appendix B.

(7) **Longitudinal Slope**

Measurements of longitudinal slope of the canal system were taken along with the cross-section measurements. Several USGS benchmarks were located along the canal system. The bottom of the canal was referenced to this benchmark elevation whenever possible. These benchmarks were more numerous along the higher end of the system, where the canal runs near the railroad tracks. Many benchmarks that were shown on the USGS topographic maps at the lower end of the system had been removed or could not otherwise be located. Several laterals were surveyed for longitudinal slope for distances varying from 400 yards to over a mile. All measurements of slope were used in the initial configuration files for the simulation model CANAL. As these were not extensive measurements over the entire length of the canals, some of these values were adjusted during calibration to more accurately reflect actual conditions.

(8) **Roughness**

Roughness coefficients were determined by comparing photos of the dry canals with photos presented by Chow (1959) for estimating the Manning roughness coefficient. These values were generally between 0.03 and 0.05. Values were determined by evaluating the roughness of the canal bottom and sides, the degree of winding present in the reach, the growth of grasses, weeds, and moss, and the amount of weeds which had been blown into the canals by the wind. Laterals generally had smoother bottoms and sides, but were more winding and clogged with weeds. The initial values for roughness were adjusted during calibration to better reflect actual conditions. Values used in the simulation are listed in the CANAL configuration files in Appendix B.

(9) Seepage Rates

Ditchrider estimates of water loss were used to estimate seepage in canal laterals. CANAL calculates seepage using a given seepage rate and the calculated wetted perimeter. If a lateral lost 4 cfs (200 inches) the seepage rate was adjusted on CANAL until the lateral showed the corresponding 4 cfs seepage loss. Evaporation loss is included in this value. The seepage varies over the course of the season, being higher in spring when the water is initially turned in. It was difficult to verify ditchrider estimates even on laterals where inflow is measured, due to the operation of non-continuous turnouts.

Seepage rates found to be representative on the laterals were all in the range of 275-350 mm/day (10.8-13.8 inches/day). These rates are slightly below the lower end of soil permeability predicted by USDA soil surveys. Surveys of Bingham and Power counties (1973,1981) predicted soil permeability in the range of 365-1220 mm/day (14.4- 48 inches/day). Some sealing of the canal over the course of the irrigation season could account for this slightly lower rate.

The seepage rates from the laterals were also transferred to the Main Line, High Line, and Low Line canals. Two stretches along the Main Line canal are known to have significantly higher seepage rates. One stretch approximately 7 miles long near the downstream end of the Main Line, runs over a lava field. Over the course of the irrigation season, sinkholes form in the bottom of the canal, some as large as 5 feet in diameter. Water flows through the sinkholes into the lava field at much higher rates than normal seepage along other canal reaches. Losses are also high near the town of Moreland, where the canal flows through soil with a high gravel content.

(10) <u>Historical Water Use</u>

Data for historical use of water on the Aberdeen-Springfield Canal came from the annual reports (1973-1990). The available data included total annual diversion into the canal system, total deliveries to headgates, and the percentage of water lost in transmission, as well as data on the acreage and type of crops grown. The data show that water use on the system has changed significantly since the mid 1970's. Average annual diversion into the canal system from 1954-75 was 342,000 acre feet. Diversion from 1976-1990 was only 282,000 acre feet, a decrease of 17.7%. Total volume of water delivered to the headgates decreased 38.2% over this period, from 199,000 acre feet to 123 thousand acre feet. Total volume of water lost in transmission increased 9.8%, from 143,000 to 159,000 acre feet. This drop in water use is illustrated in figure 3.

The decrease in water use is attributed to irrigators switching from low efficiency gravity irrigation to higher efficiency sprinkler systems, and growing crops which require less water. Figure 4 illustrates the latter point, showing the decrease in alfalfa acreage over the years and the increased grain acreage. There has also been a slight decrease (6.9%) in the average irrigated acreage since 1976, from 48,300 acres to 45,000 acres.

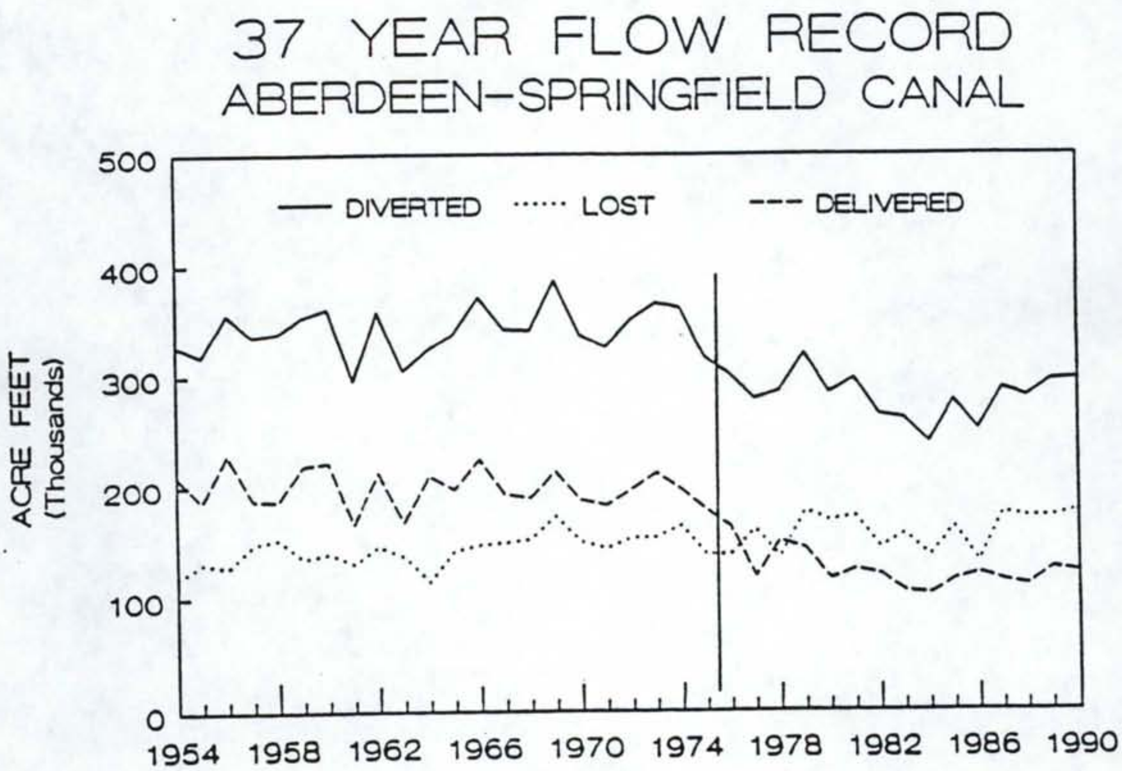


Figure 3. 37 year flow record into the Aberdeen-Springfield Canal.

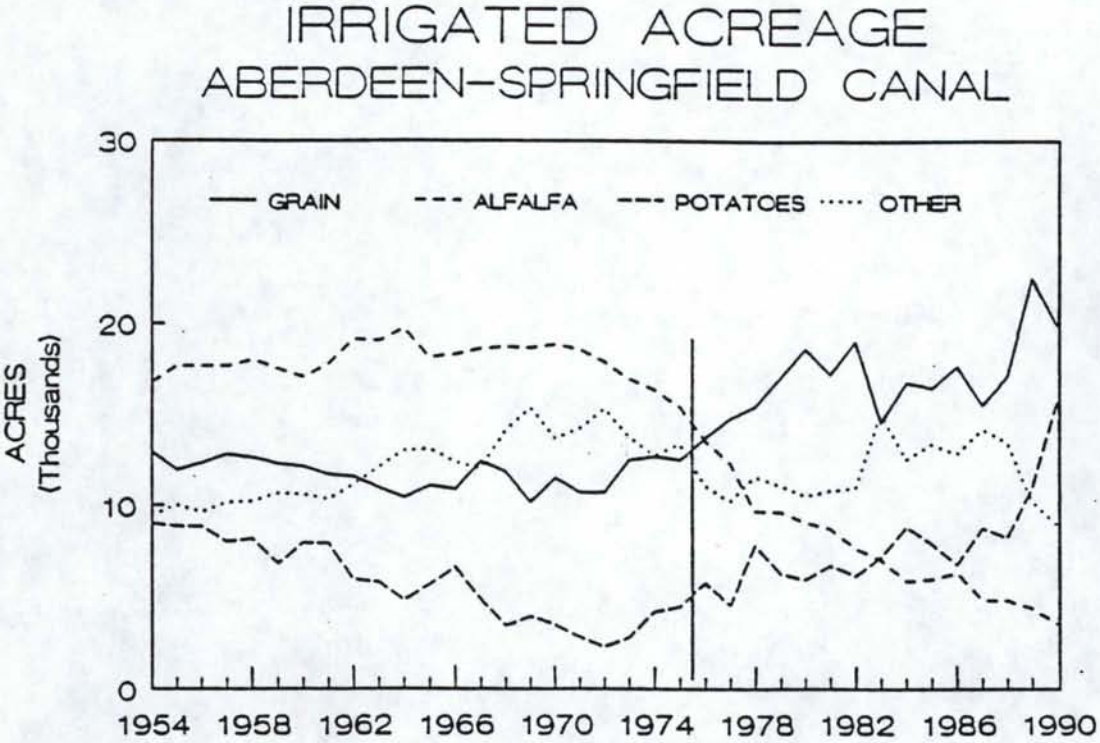


Figure 4. Irrigated acreage on the Aberdeen-Springfield Canal.

(B) SELECTION OF THE HYDRAULIC SIMULATION MODEL

In selecting a hydraulic simulation model for the Aberdeen-Springfield Canal system, several factors were considered:

1) The model must be able to accurately simulate canal system structures, turnouts, and flow conditions.
2) The model must be easy to learn and operate.
3) The model must run on computers which are available at the University of Idaho, and preferably at the Research and Extension Center in Aberdeen.
4) The model must be affordable.

The USBR recommended the Utah State University Main System Hydraulic Model (CANAL). The model met all the necessary requirements. 1) It had been tested against other simulation models and actually applied on canal systems in Utah, Arizona, and Thailand with favorable results (Gichuki, Walker and Merkley, 1990). Rogers (Merkley and Rogers, 1991) also evaluated CANAL, and his review indicated the model would be feasible for the study. Parts of Rogers' evaluation were presented earlier. 2) The model was easy to learn and operate. The Users Manual (Merkley, 1987) provides a clear and straightforward guide to the program. In addition, the program developer, Dr Gary Merkley, was available for consultations to clear up problems. Dr. Merkley made several requested adjustments to the program which increased the utility of the model. 3) The program was compatible with microcomputer equipment available at the University of Idaho Research and Experiment station in Aberdeen. 4) The program was very affordable, costing only forty dollars. In addition, Dr. Merkley is currently constructing a new version of the model which will be even more flexible and user friendly, making a future upgrade possible for the Canal Company.

(C) Calibration

(1) V Lateral Calibration

Calibration of the hydraulic simulation model CANAL began with V lateral. Four Stevens Type A continuous water level recorders were placed along V Lateral at the upstream and downstream ends of Reach 2 and 3 as shown in Figure 5.

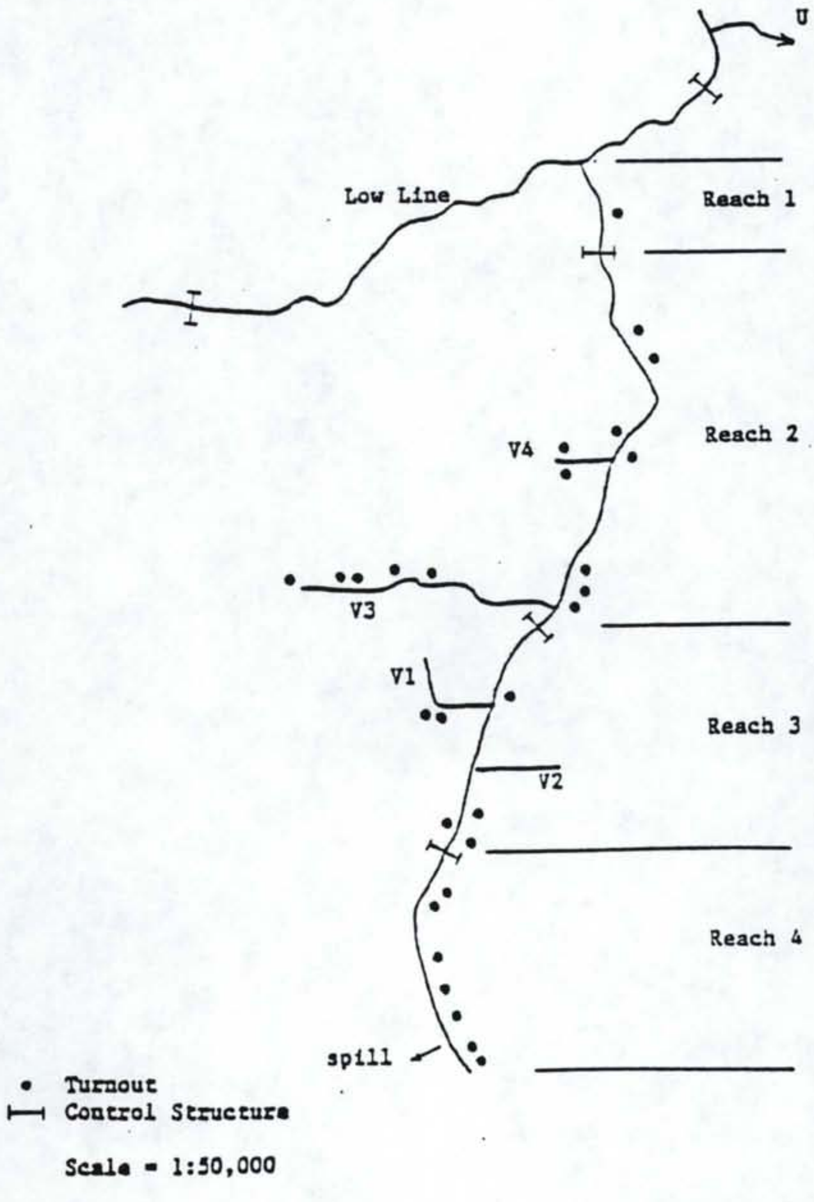


Figure 5. Reach configuration used for V Lateral simulations.

Data on water surface elevation was collected continuously throughout the 1990 irrigation season. Water levels in the lateral showed distinct cyclical variations due to non-continuous turnout operations. The water surface fluctuations for a four day period, July 11-14, 1990 is shown in figure 6.

A simulation model of V Lateral was constructed using the measured values for control structure dimensions, lateral length, cross-sections, longitudinal slope, and turnout location, and the estimated roughness parameters. Using the

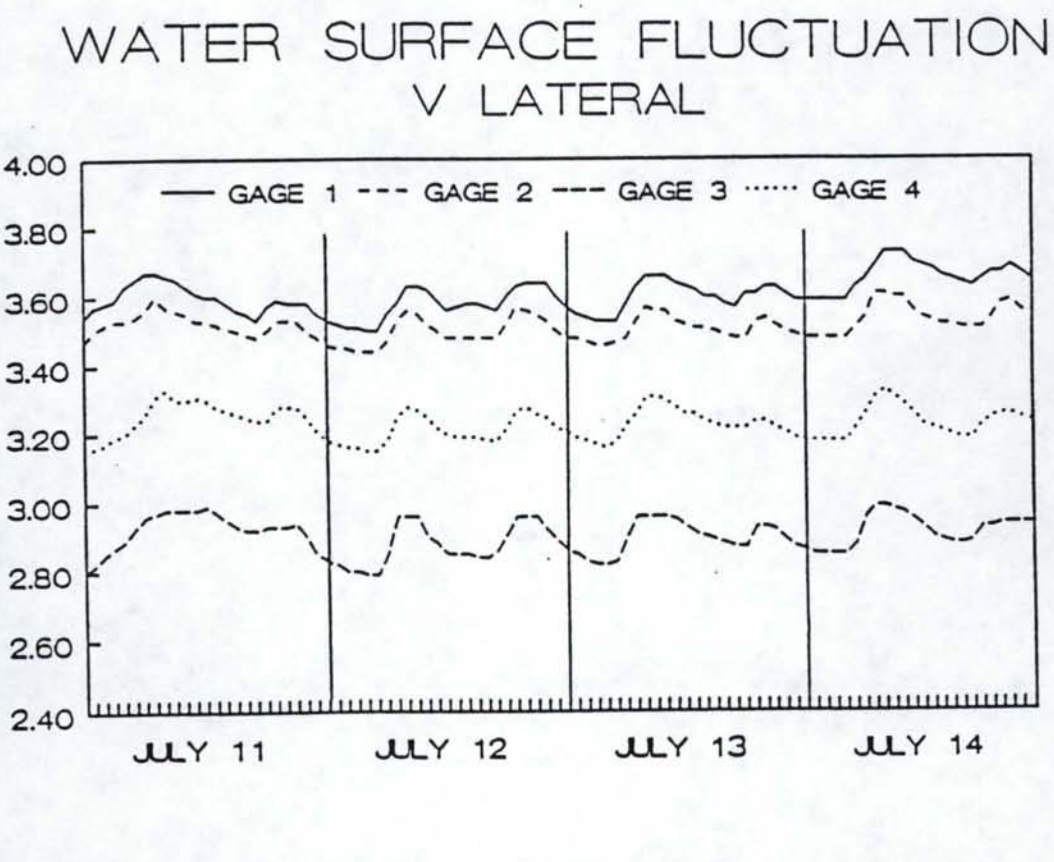


Figure 6. Measured water surface fluctuations on V Lateral, July 11-14, 1990.

inflow measured at the head and the outflow measured at the turnouts and the spillway, the seepage rates were then adjusted until the total seepage agreed with the ditchriders estimate. It was difficult to do an exact water balance because of the cyclical nature of the canal and the changing volume of storage water in the canal as a result. When the seepage rates were set, the canal model was operated at steady state, with all turnouts operating continuously. Control structure settings were changed, and the roughness and longitudinal slope parameters were adjusted until the calculated depth in the model was close to the measured depth in the lateral. Exact agreement was not necessary, as local variations in the lateral cross-section and slope make the accuracy of point measurements questionable. The calibrated system dimensions, configuration data, control structure data and turnout data used for simulation of V Lateral is in Appendix B.

(2) <u>Turnout Calibration</u>

For simulation under steady state flow conditions, the turnout parameters are not important. All that matters is having the correct flow exit the canal at the correct location. When unsteady flow conditions exist, however, the flow through the turnouts varies, which in turn changes the flow in the canals. It is therefore necessary for turnout flow rates in the model to vary like actual turnout flow rates as canal water levels fluctuate. As there are over 500 turnouts on the system, no attempt was made to predict the fluctuating flow through each turnout. Instead the goal was to get the model turnouts to act like an "average" turnout on the actual system. In this way the fluctuation is overestimated at some turnouts and underestimated at others, but the entire fluctuation is approximately the same.

The box-submerged orifice turnouts used on the Aberdeen-Springfield canal system are not one of the available turnout options in CANAL. Flow through a box-submerged orifice is measured through the rectangular orifice, but is actually controlled by the gate on the circular pipe leaving the box. Therefore the turnouts were modeled using circular orifices. Figure 7 shows a schematic diagram of a circular orifice.

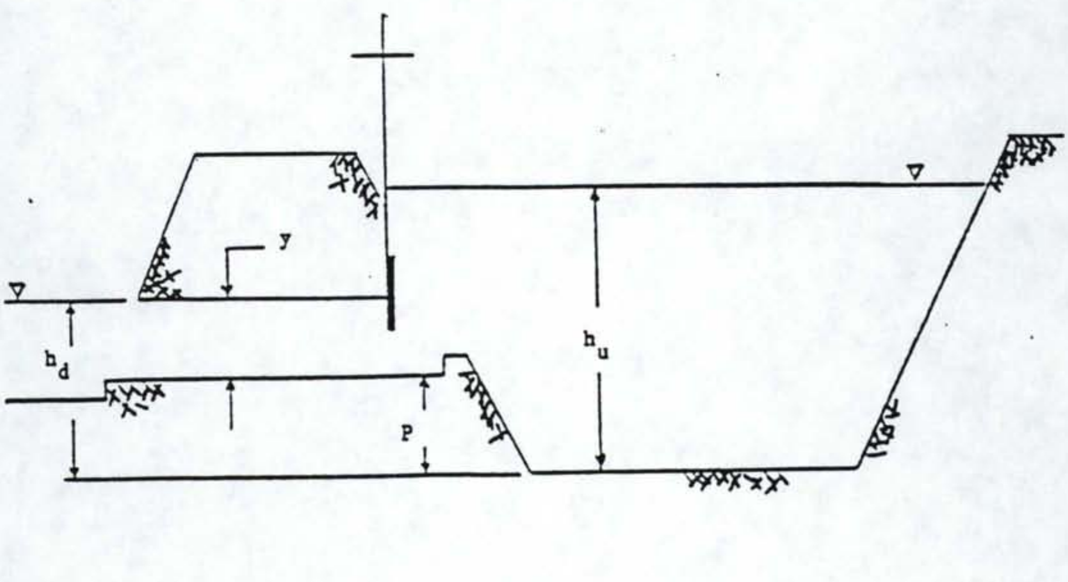


Figure 7. Schematic diagram of a circular orifice.

The controlling equation for the circular orifice with submerged flow is (Merkley, 1987):

$$Q = C_d * A * (2 * g * (h_u - h_d))^{.5} \quad (3)$$

where,
- Q = turnout discharge rate
- C_d = discharge coefficient
- A = Open area of circular orifice
- g = gravitational acceleration
- h_u = upstream flow depth
- h_d = downstream flow depth

As the downstream flow depth (h_d) is outside the canal system, the simulation model cannot calculate it and a value must be assigned. Two parameters are necessary, P_1 and P_2. The parameter P_1 represents the downstream water surface when there is no flow through the turnout. The parameter P_2 represents the rate at which this water surface increases as flow through the turnout increases. Values for pipe diameter (y) and the position (P) must also be input. Thus there are four variables which can be adjusted to control turnout flow rate fluctuation; P_1, P_2, P, and y.

Using V Lateral as a test lateral, the effects of varying these parameters was evaluated. The canal model was run to steady state, and several turnouts were shut off as the inflow into the canal was increased. This caused the water surface in the canal to fluctuate. Varying P_2 resulted in no change in the resulting fluctuation up to a value of $P_2 = 2$, and at that point the turnout flow rate began oscillating from the initial value to zero as the model calculated backflow conditions for the turnout. Figure 8 shows that changing the downstream depth (P_1) from 0 to 1.5 feet had no effect on the resulting fluctuation in flow rate at various turnouts along the lateral. Figure 9 shows that varying pipe diameter (y) from 1.0 to 1.5 feet had no effect either. Figure 10 shows that fluctuation in turnout flow rate increases as turnout position (P) is increased from 0.5 to 1.0 to 1.5 feet. Thus the amount of turnout fluctuation may be controlled simply by adjusting the turnout position. By matching the model turnout position with the approximate position of the turnouts on the

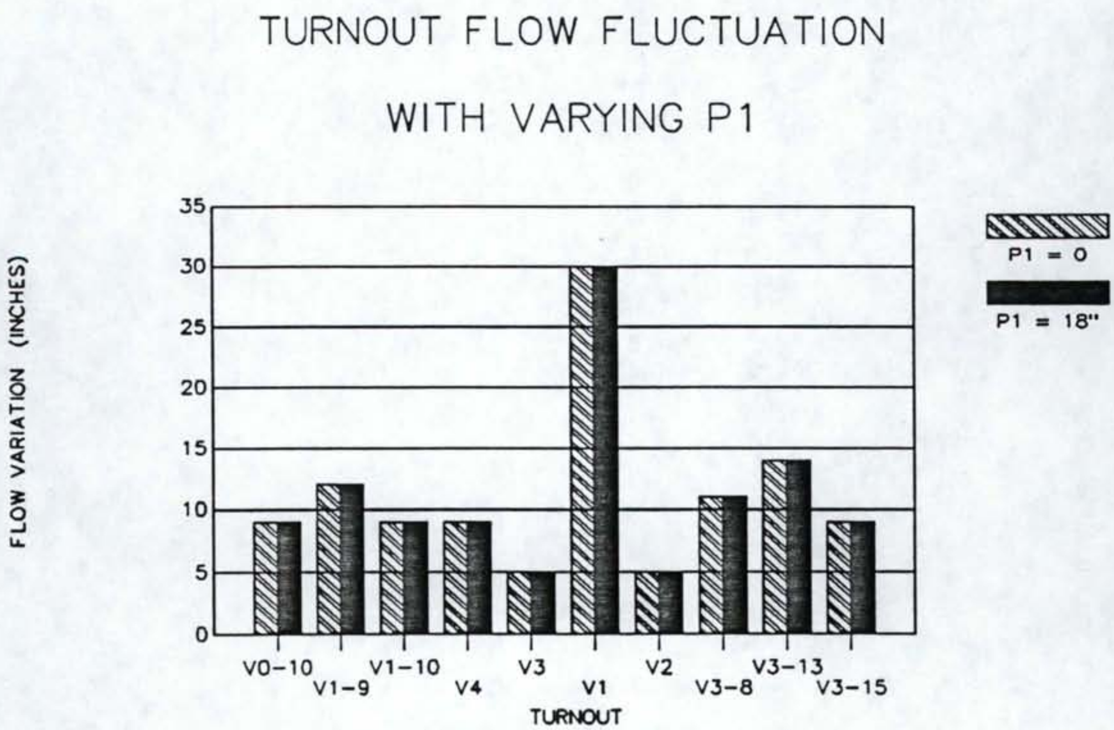


Figure 8. Effects of varying P_1 on turnout flow rates.

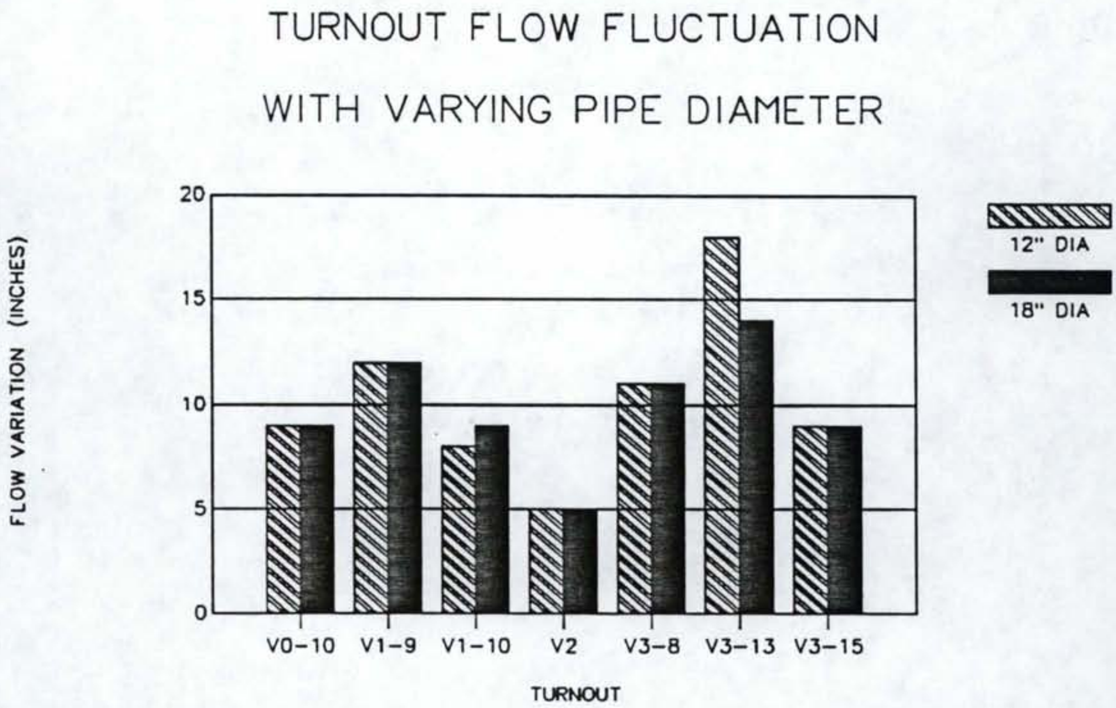


Figure 9. Effects of varying pipe diameter on turnout flow rate.

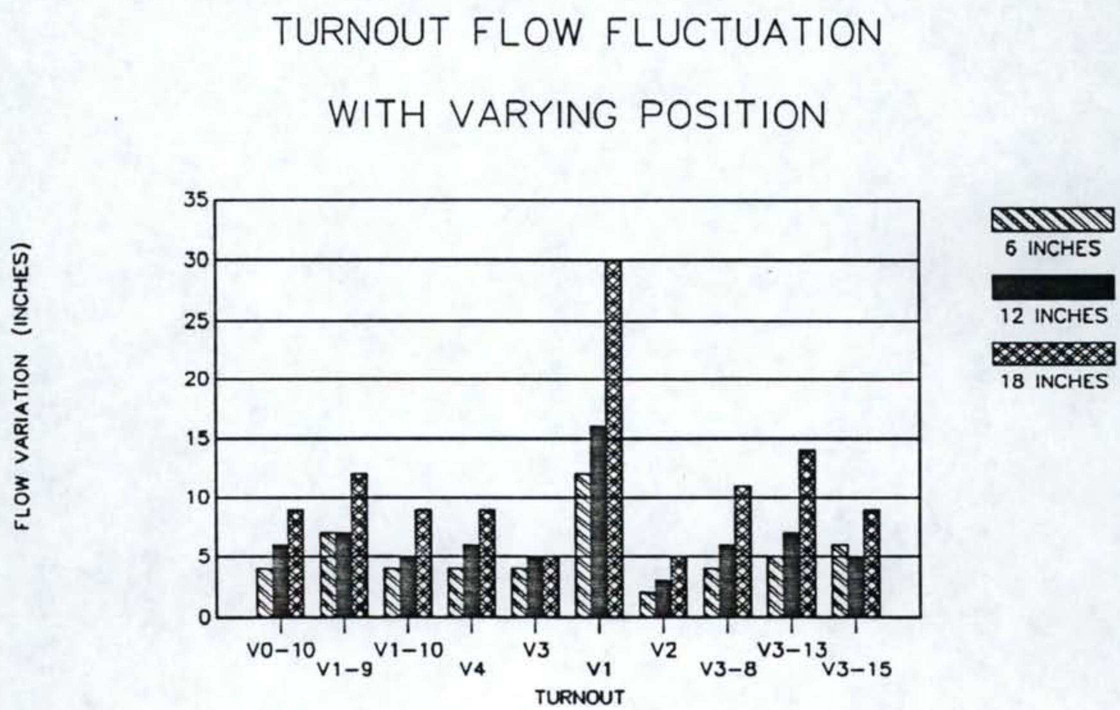


Figure 10. Effects of varying position on turnout flow rate.

canal, the magnitude of flow rate fluctuations should coincide.

(3) <u>Control Structure Calibration</u>

As exact flow measurements across the control structures were not available, the exact coefficients could not be calculated. Variation of the control structure flow coefficients had little effect on the cyclical aspect of the flow. It was decided to leave all flow coefficients as standardized. In actuality, many control structures would have different coefficients. If the purpose of this study were to predict gate settings to control flow rates, this discrepancy would have serious effects on the accuracy of the simulation model. For the purpose of studying canal fluctuation, however, the effects are negligible.

(4) <u>System Calibration</u>

The responses of the V Lateral simulation model to changes in longitudinal slope and roughness parameters aided the calibration of other parts of the canal system. Simulation models were constructed of the High Line, Low Line, Main Line, and several laterals using measured data. Several models had to be constructed because CANAL has limits on the number of branches, reaches, and turnouts allowed. By breaking the canal system into individual canals and laterals, the run timefor a simulation was greatly decreased and modifications to the canal models could be made easier. Staff gage readings provided a general record of water level in the canals, and periodic measurements were made at various locations along the entire system. The simulation models were adjusted to reflect the periodic measurements until it was judged the models could accurately predict flow conditions in the canals. The calibrated data for these simulation models (system dimensions, configuration data, control structure data, and turnout data) are located in Appendix B.

(D) Model Application

After the model was calibrated for the entire system, it was used to evaluate the effects of non-continuous flow. The canal system was evaluated during the peak of the irrigation season, when the maximum flow rate was being delivered. The simulation models were run using actual inflow and outflow values from July 7, 1990. By varying the number of noncontinuous turnouts and the length of time the turnouts were shut off, and by staggering the time periods when they were shut off, canal fluctuation for several different non-continuous scenarios were generated. Alterations to the simulation models were made, such as changing the cross-section, varying control structures, and modifying spills. The various non-continuous flow scenarios were run again, and the effects of the canal modifications evaluated. Simulations were performed on the Main Line, High Line, Low Line and C1, C, E, G, Q, and V Laterals. These laterals are representative of other laterals in the system, and it was assumed that the effects of modifications to these laterals would be transferrable to other laterals.

The effect of automatic control structures on the canal were simulated using the "gate stroking" concept (Falvey and Luning, 1979) incorporated in the simulation model. This is not a precise simulation of the control structure operation, and no attempt was made to calibrate the model to reflect actual automatic control structure operation. The model generally controls the depth immediately upstream of an automatic check within an inch of the specified depth. It is assumed that the actual control structures operate at approximately the same level of efficiency.

No attempt was made to determine or simulate the effects of wind on water levels in the canals, since the fetch of most canal reaches is small. Also no attempt was made to determine the effects of diurnal fluctuations in seepage and evaporation rates.

U.S. Bureau of Reclamation records of inflow into the canal system (at 15 minute intervals) were used to determine the effects of inflow variation into the canal. Inflow variation was negligible except for July, when inflow into the canal system was near the maximum (1300 cfs). Variation in the inflow rate during this period was as high as 5%. Simulations predicted fluctuations of this size were absorbed into the canal system without causing significant water level fluctuations downstream.

CHAPTER 4

RESULTS

The computer model was used to generate a large number of simulated operations on the Aberdeen-Springfield canal system. Because V Lateral was more accurately calibrated than the rest of the system, a large number of simulations were run on V Lateral. The effectiveness of control measures on V Lateral was used as a gage to indicate which measures to evaluate on other parts of the system. The results from V Lateral will be presented first, followed by the results from the Main Line, High Line, Low Line and several laterals. It is believed that the results are transferrable to some extent among the laterals.

Data will be presented to show the effects of non-continuous turnout flow on canal flow rate and water levels in the canal, and the effects of various control strategies on the fluctuations. Figures are presented to visually illustrate the Branch and Reach configuration used in the simulation. Graphs are presented to show the maximum flow rate variation at the downstream end of each reach, and the predicted water surface fluctuation. Water surface fluctuation is the average of the maximum fluctuation at the upstream and downstream end of each reach.

The results shown demonstrate the upward fluctuation in canal water surface and flow rate caused by non-continuous turnout operation. Increased water level causes increased flow through turnouts and infringes on canal freeboard. Control of the canal system becomes more difficult as flow rate may appear artificially high when control structures are set. Of equal importance in the canal system operations is the downward fluctuation caused when the non-continuous turnouts begin operation. This is particularly a problem where the actual flow rate is unknown, as there may be insufficient water in the canal to satisfy all turnout demands. If a significant number of non-continuous turnouts begin drawing water after the flow rate into a reach or lateral has been set, and the flow rate is not high enough, the canal water level drops significantly, and turnouts do not receive the requested flow rate. This situation can cause serious damage to pumps as they begin to draw air.

Preliminary simulations of canal laterals showed that the dimensions of the downward fluctuation are close to those of the upward fluctuation, provided enough water is flowing through the canal to satisfy all reaches. When there is not enough water, the canal fluctuations become more severe. It was decided to run simulations which measured the upward fluctuation caused by non-continuous turnout operation as opposed to this downward fluctuation. The results are not given as upward fluctuation, however, but merely as fluctuation. It is assumed that the downward fluctuation is approximately the same, provided that the flow rate is high enough to satisfy all turnout demands.

(A) V Lateral

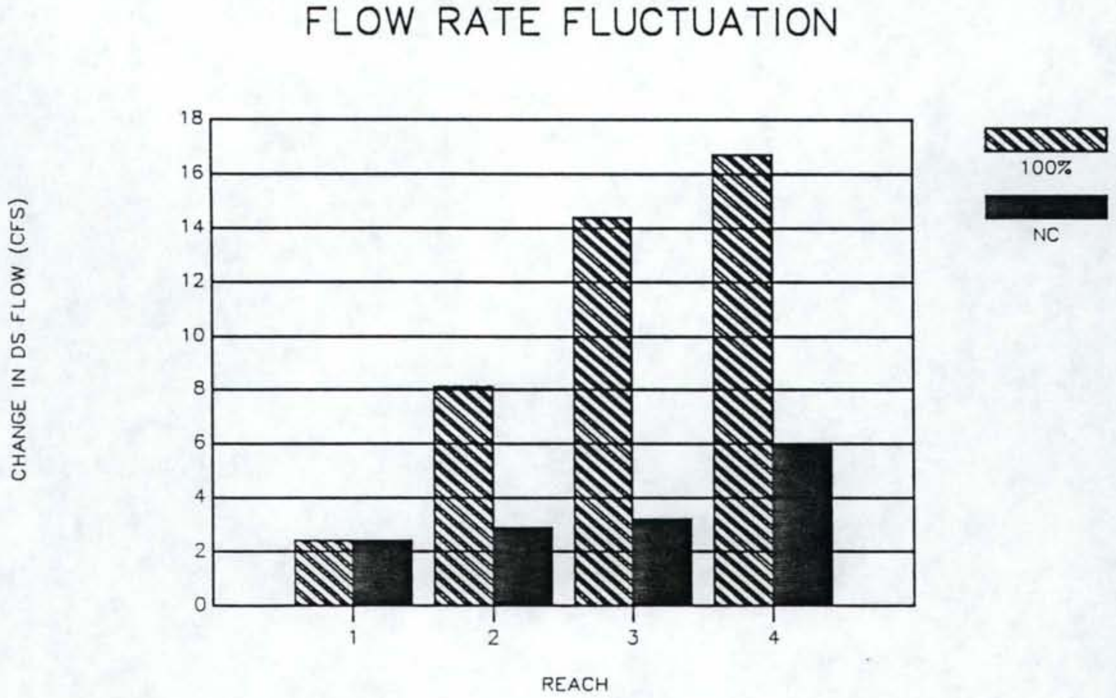
The configuration files for V Lateral appear in Appendix B. Table 1 shows the turnout flows used in the simulation, from July 7, 1990. The turnouts labeled "NC" under "Outlet" are currently non-continuous turnouts, while those labeled "C" are currently continuous turnouts. There is a spill near the downstream end of Reach 4, and flow rate variation reported in this reach is over the spill. Inflow into V Lateral for the simulations was 42.0 cfs, based on 36.8 cfs for turnouts, 1.8 cfs for the spill, and 3.4 cfs for seepage. The inflow into V Lateral is measured at a 12 foot Cipoletti weir near the head.

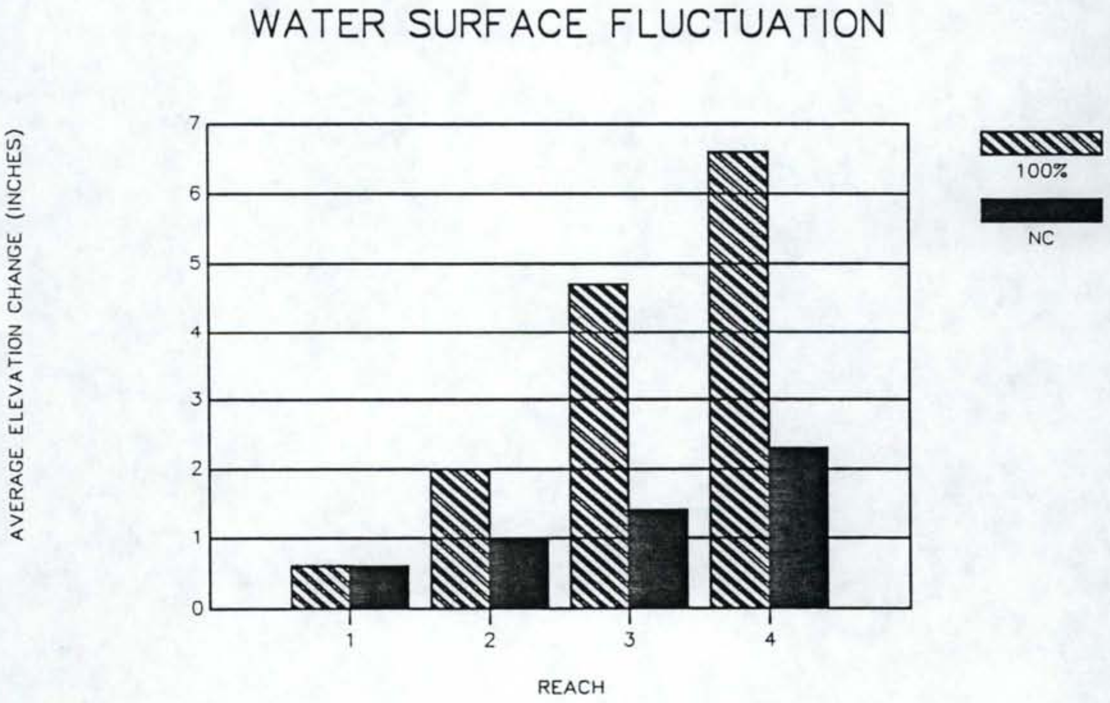
The effect of shutting off turnout flows for 2 hours is shown in figures 11(a) and (b). There is a large difference between fluctuations caused by current non-continuous turnouts (NC) and that which would occur if all turnouts (100%) on the lateral were to shut off. Fluctuations increase from the upstream to the downstream end of the lateral. Figure 11(b) shows that if all turnouts were non-continuous and shut off for two hours, the average water surface elevation in reach 4 would fluctuate 6.6 inches, even with a spillway.

	Turnout	Flow Rate (cfs)	Outlet
Reach 1	V 0-2	2.4	NC
Reach 2	V 0-10	2.5	C
	V 1-4	0	NC
	V 1-9	3.3	C
	V 1-10	2.4	C
	V4 LATERAL	2.9	C
	V 1-17	2.0	NC
	V 1-18	0	C
	V 1-19	0	C
	V3 LATERAL	3.3	C
Reach 3	V 2-7	1.2	NC
	V1 LATERAL	5.7	C
	V2 LATERAL	2.0	C
	V 2-13	1.4	NC
	V 3-5	0	C
	V 3-6	0	C
Reach 4	V 3-8	0.7	C
	V 3-9	1.4	NC
	V 3-12	0	C
	V 3-13	1.4	C
	V 3-15	1.2	C
	SPILL	2.1	
	V 4-6	1.6	NC
	V 4-7	1.4	NC

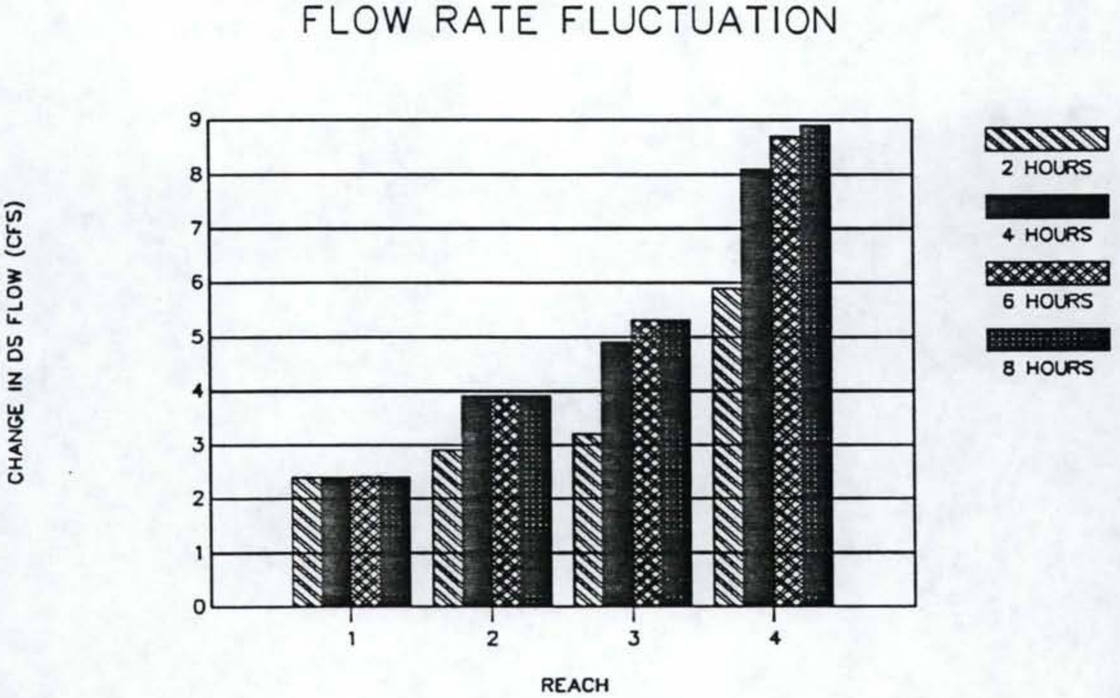
Table 1. Turnout flows used in simulations of V Lateral.

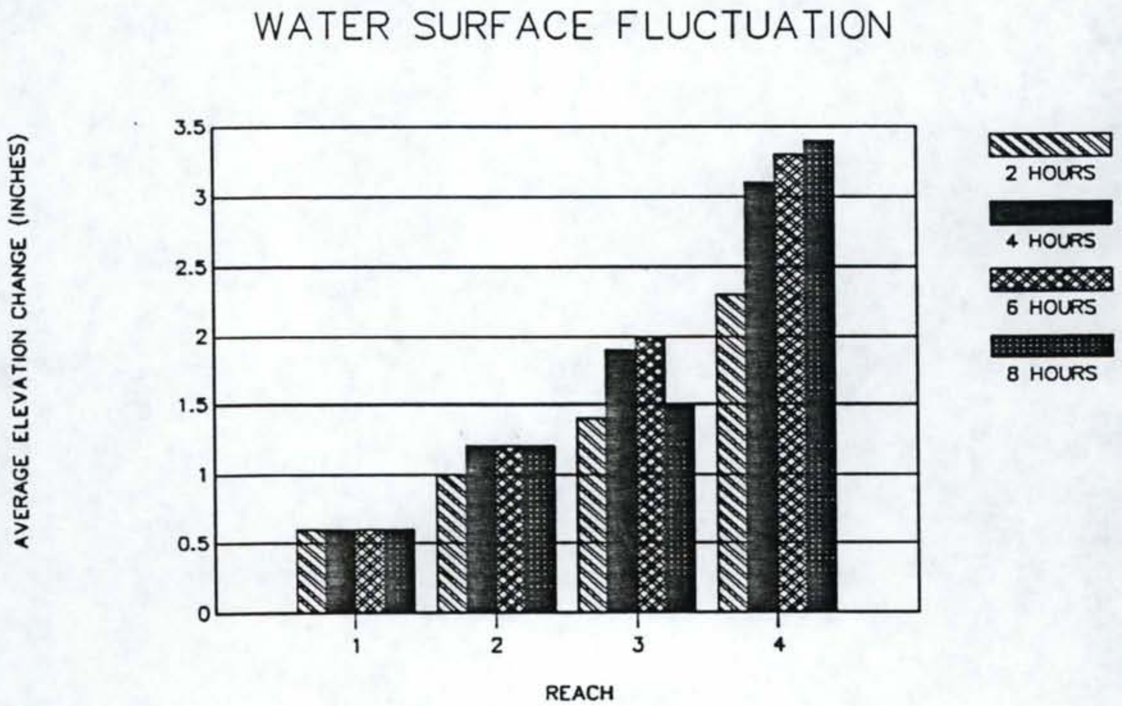
Figures 12(a) and (b) show the effects of shutoff times of longer than 2 hours for current non-continuous users. The non-continuous turnouts were shut off for 2, 4, 6, and 8 hours in the simulations. While variation appears to have reached a peak before 4 hours in the first two reaches, variation continues to increase for the two downstream reaches.





Figures 11(a) and (b). V Lateral flow rate and water surface fluctuation from 2 hour turnout shutoff.





Figures 12(a) and (b). V Lateral flow rate and water surface fluctuation with increasing shutoff time for non-continuous turnouts.

Figures 13(a) and (b) show the effects of increasing the base of the canal by 3, 7, and 10 feet. All current non-continuous turnouts were shut off for 2 hours. Flow rate and water surface variation decrease slightly as the base width increases. This decrease is within 1.5 cfs and 1.0 inches of the fluctuations which occur with the original base width in all reaches.

The effect of replacing existing, manually operated weir control structures with manually operated sluice gates is shown in figures 14(a) and (b), and 15(a) and (b). For figures 14(a) and (b), all non-continuous turnouts were shut off for 2 hours. 100% of the turnouts were shut off to produce the data illustrated in figures 15(a) and (b). While the sluice gate control structures substantially decrease the flow rate fluctuation being transmitted from one reach to the next, the resulting water level fluctuation in the upstream reaches is much higher. The sluice gates are much less sensitive to water fluctuation with regard to the flow rate passing through the structure. Values for 100% turnout shutoff are again substantially higher than current levels of non-continuous use.

The effect of converting the existing manually controlled structures to automatic control structures and varying the width of the spillway is shown in figures 16(a) and (b). All non-continuous turnouts were shut off for 2 hours, and the

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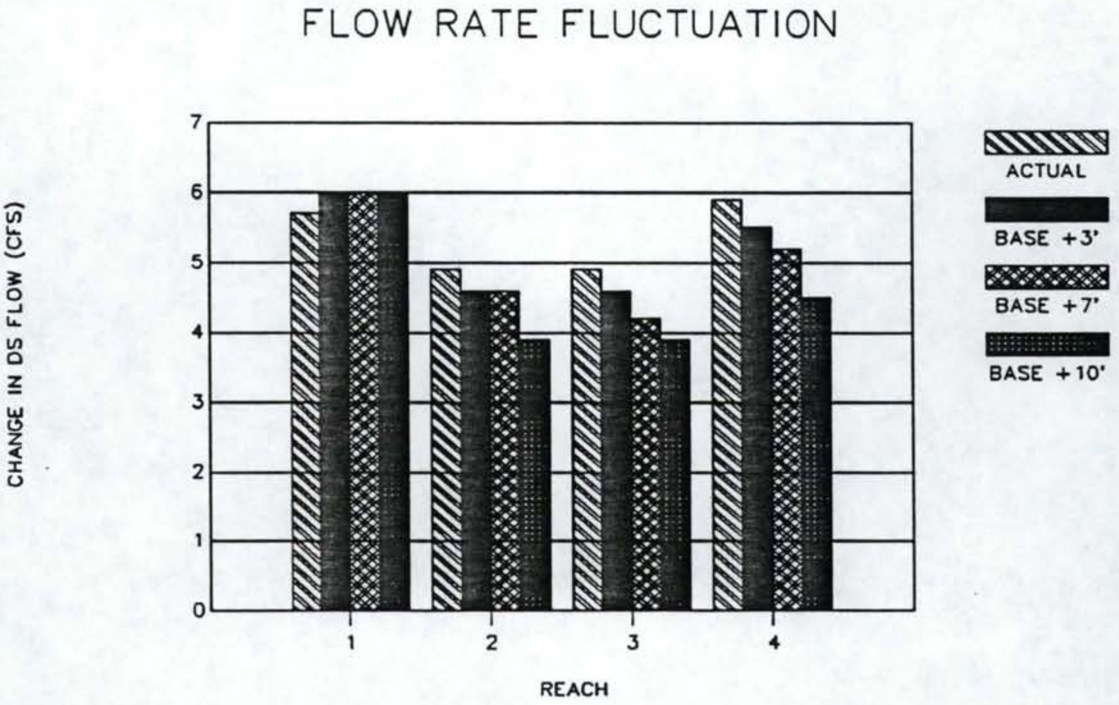
The effect of converting the existing manually controlled structures to automatic control structures and varying the width of the spillway is shown in figures 16(a) and (b). All non-continuous turnouts were shut off for 2 hours, and the simulations were run in automatic mode. Flow rate variation is much higher as the control structures adjust to keep water surface on the upstream side of the control structure (downstream end of the reach) constant. Flow not being diverted through the turnouts is passed downstream more efficiently with little being stored. Resulting fluctuation in the last reach is much higher, as the water has no place to go except over the spill. The actual overflow weir spillway width is 12 feet. Decreasing the spill width to 6 feet decreases the ability to remove the additional water and results in increased water surface fluctuation. Increasing the spillway width to 18 and 24 feet decreases the water surface variation only slightly, indicating that a spillway width of 12 feet in this location is adequate.



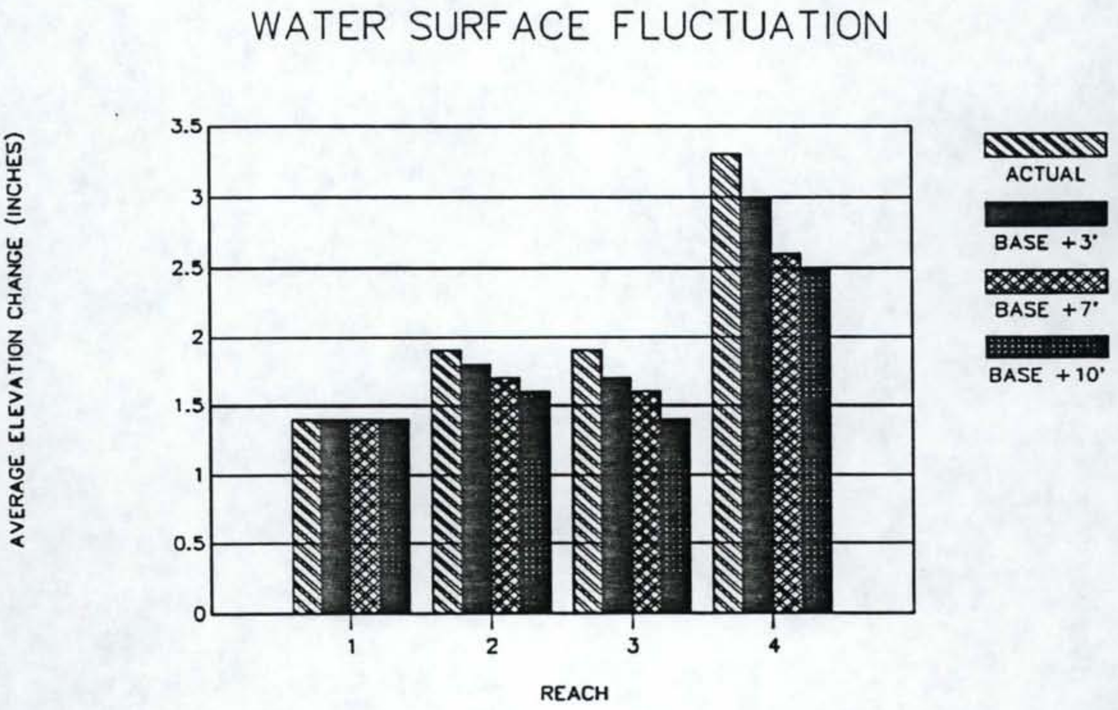
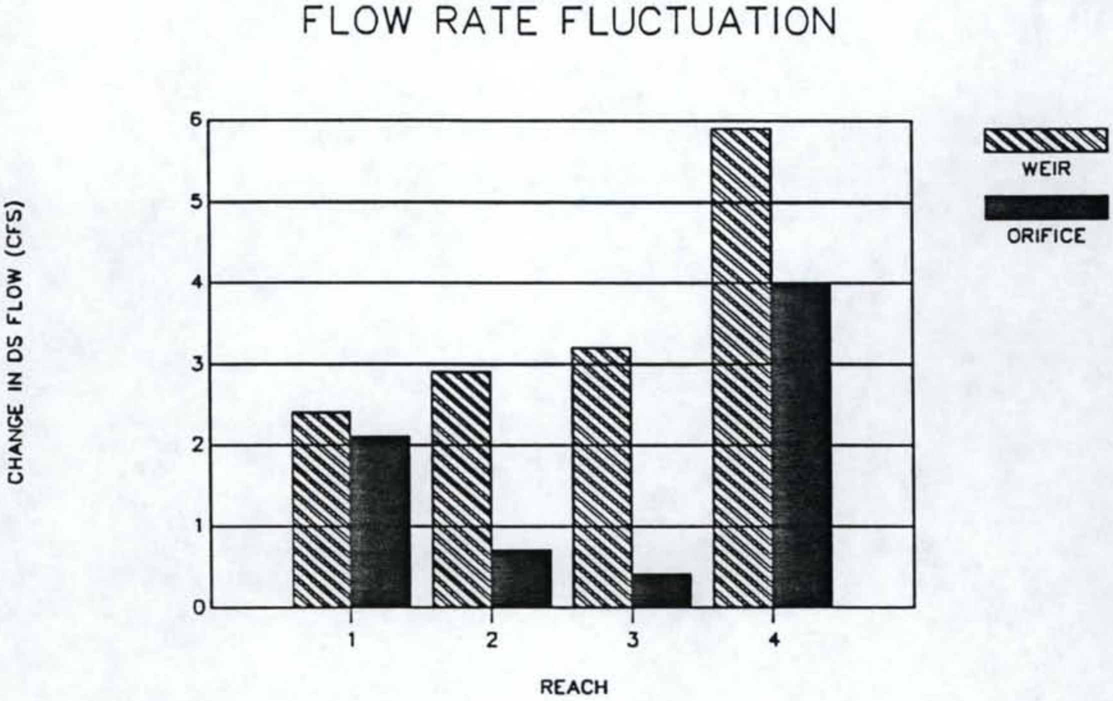


Figure 13(a) and (b). V Lateral flow rate and water surface fluctuation with increasing base width, 2 hour non-continuous turnout shutoff.



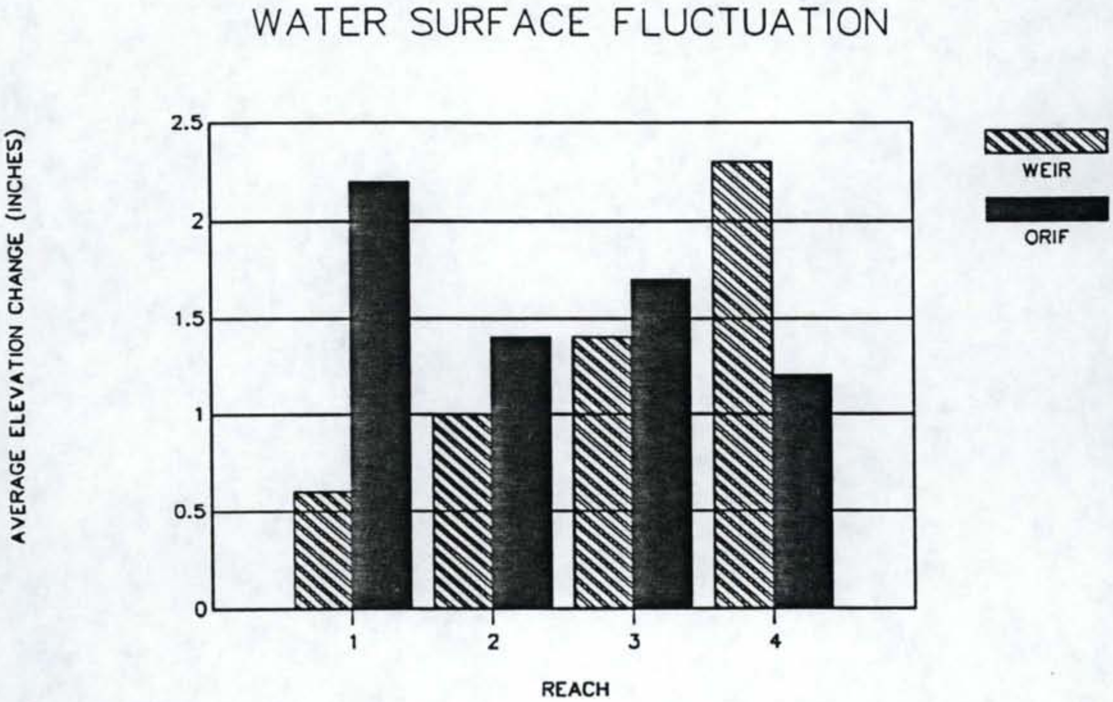
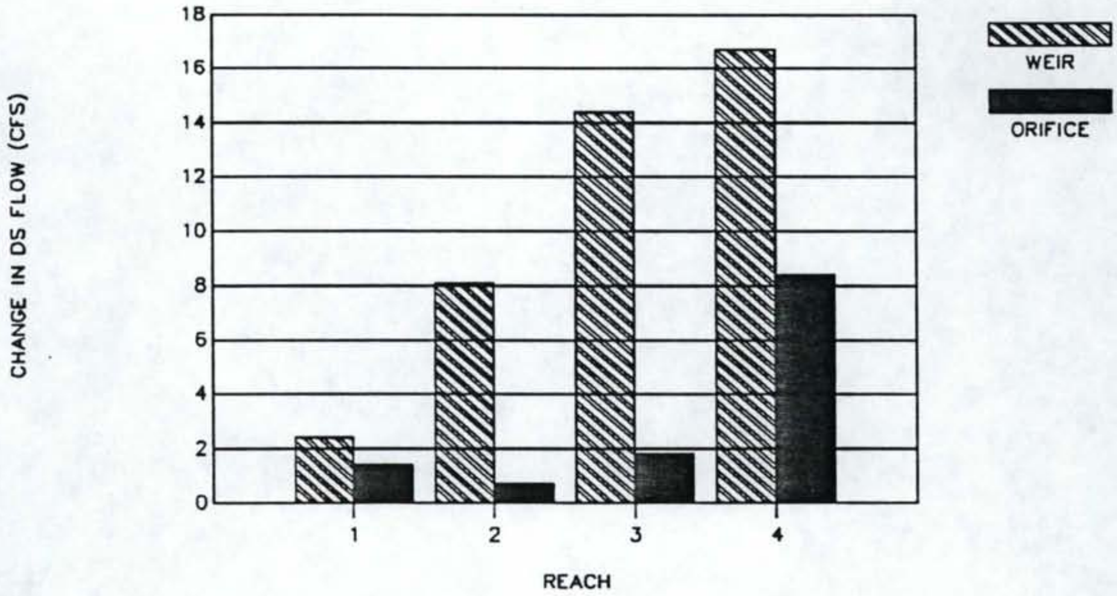


Figure 14(a) and (b). V Lateral flow rate and water surface fluctuation with orifice control structures, 2 hour non-continuous turnout shutoff.

FLOW RATE FLUCTUATION



WATER SURFACE FLUCTUATION

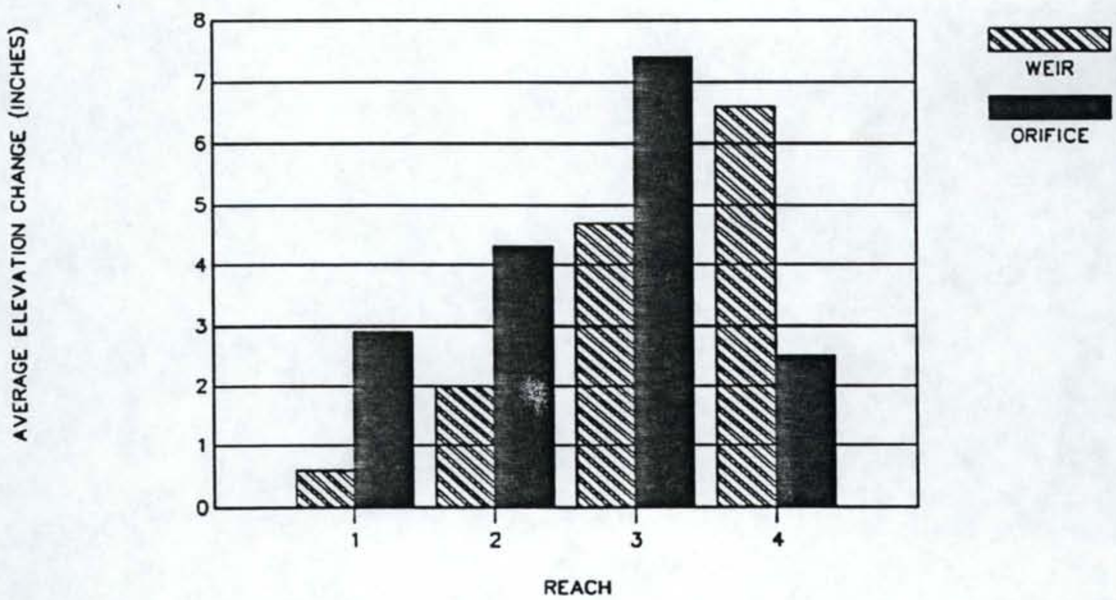
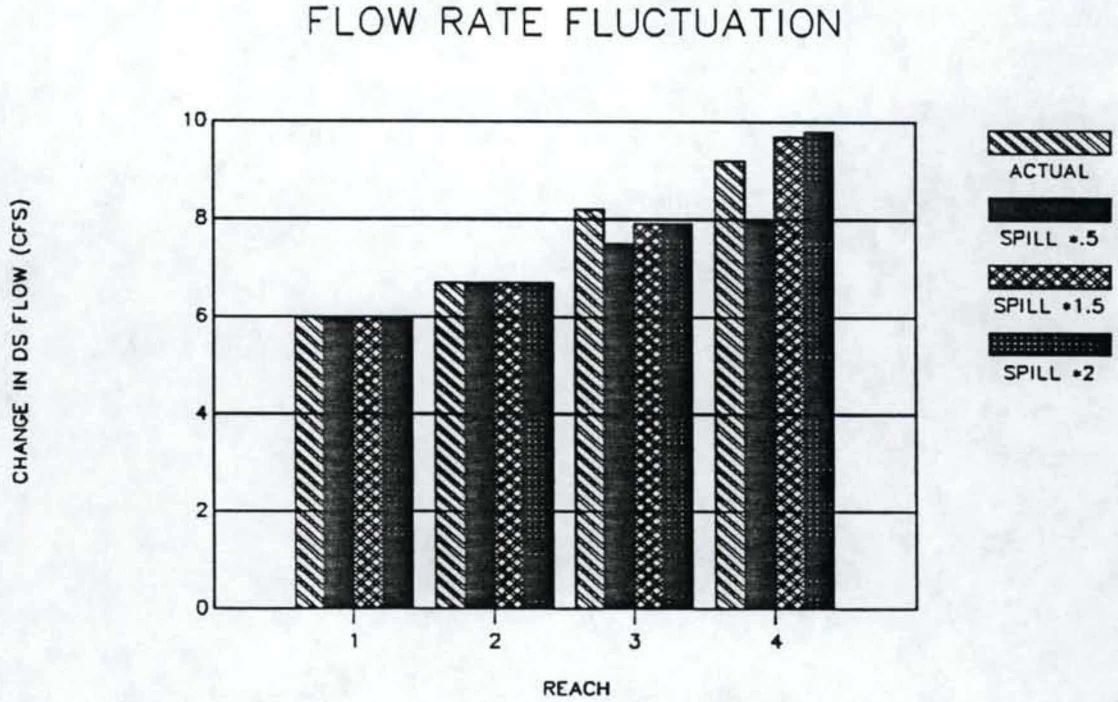


Figure 15(a) and (b). V Lateral flow rate and water surface fluctuation with orifice control structures, 2 hour 100% turnout shutoff.



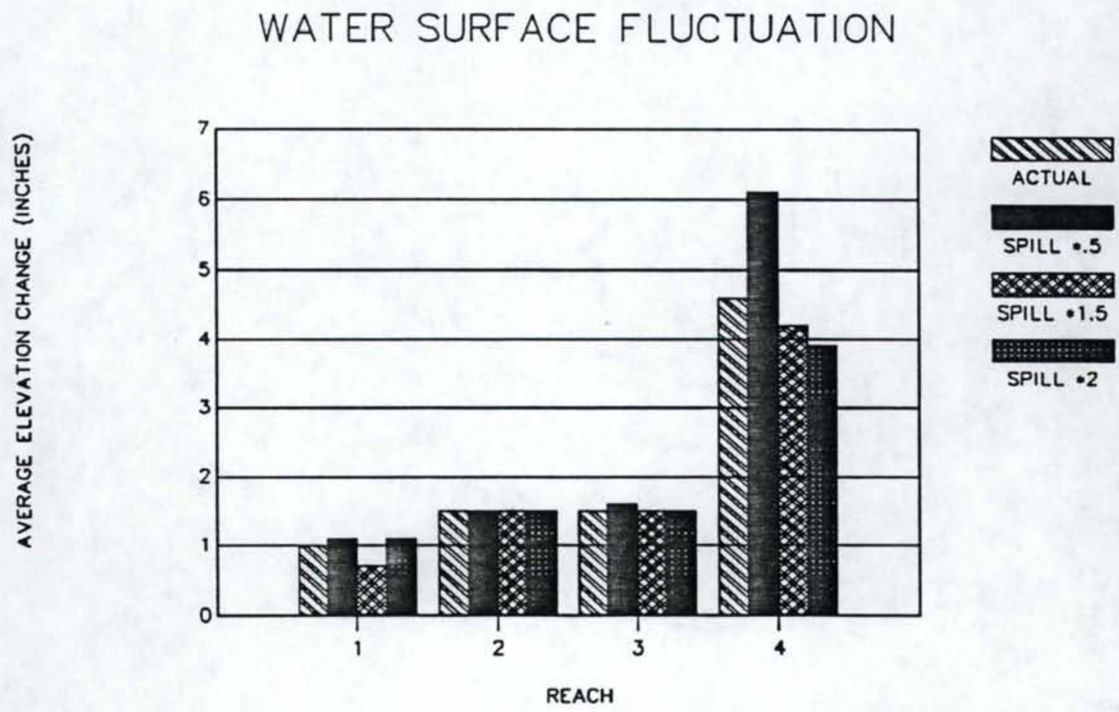


Figure 16(a) and (b). V Lateral flow rate and water surface fluctuation with varied spillway width and automatic control structures, 2 hour noncontinuous turnout shutoff.

(B) Main Line

The configuration data files for the Main Line appear in Appendix B. Figure 17 shows the Branch and Reach configuration used in the simulations. Data are presented in terms of these Branch and Reach numbers. Note that adjustable control structures appear only at the tail end of Branch 1 Reach 4, Branch 1 Reach 5, Branch 2 Reach 4, and Branch 2 Reach 9. Automatic gate control was used to control the four control structures during the simulations, although in reality only the first three are automatically controlled. The water level at the fourth is controlled by an automatic sluice gate on the spillway immediately upstream. Thus the results for downstream flow variation at Branch 2 Reach 9 are actually results for variation in the flow rate going through the spillway and into the Big Fill Reservoir. This is the only spillway used in daily operations on the Main Line.

The turnout and lateral flow rates used in the simulations are shown table 2. The turnout flow rates are those listed in company records for July 7, 1990. Lateral flow rates are determined using the lateral turnout flow rates and the calibrated seepage rates. The model was run until steady state conditions existed before turnout flows were varied. Inflow into the Main Line was 1300 cfs.

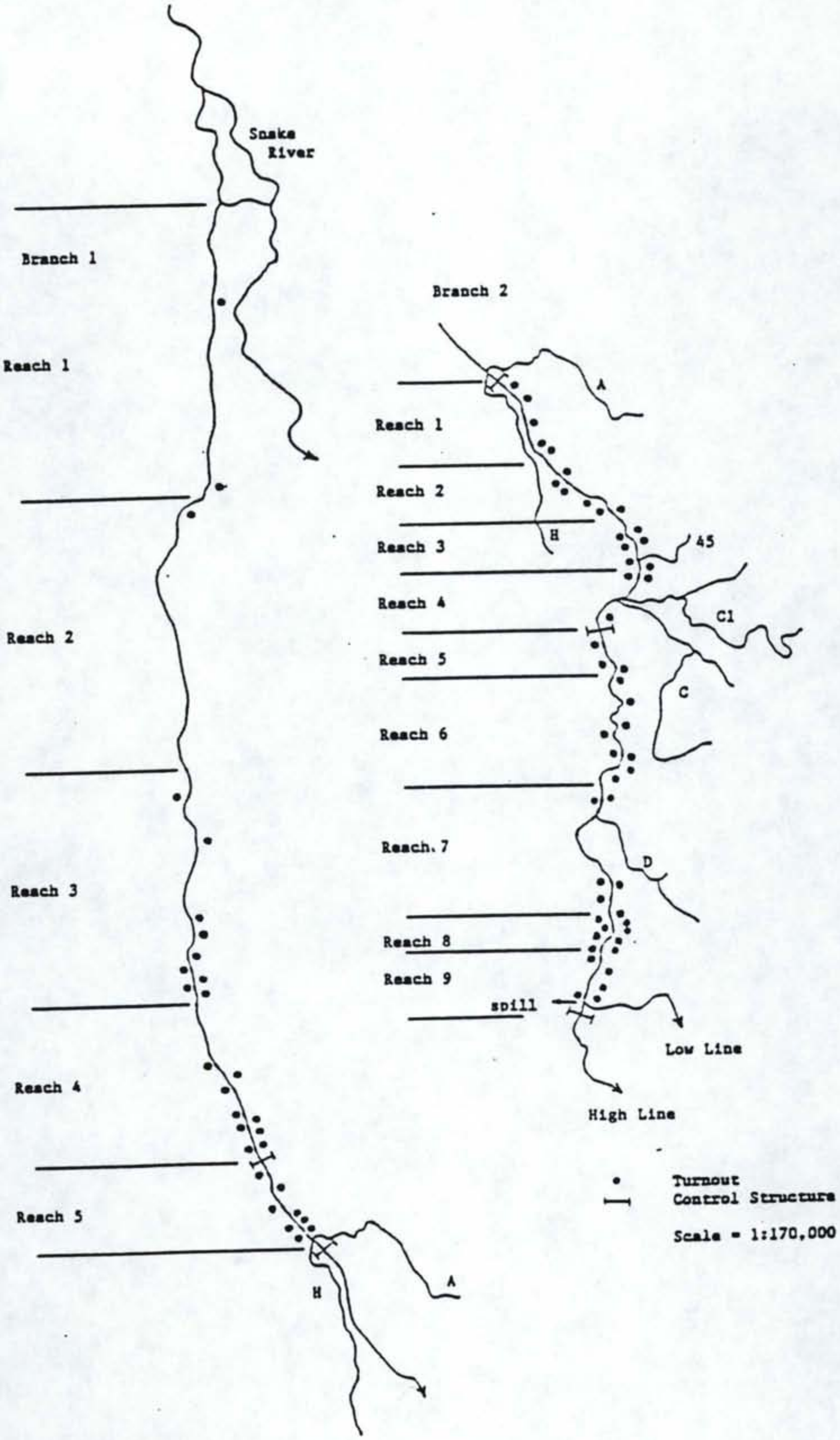


Figure 17. Reach configuration used for Main Line simulation.

Branch # Reach #	Turnout	Flow Rate (cfs)	Outlet
B1 R1	M 36-7	0.0	C
	M 36-8	0.0	C
	M 36-9	3.0	C*
B1 R2	M 37-1	0.0	C*
	M 37-2	2.0	C
	M 37-4	0.0	C
	M 37-3	0.0	C
	46 LATERAL	7.4	C
	M 37-7	0.0	C
	M 37-8	0.0	C
B1 R3	M 38-9	1.5	C
	E LATERAL	24.0	C
	M 39-1	3.6	C*
	F LATERAL	5.0	C
B1 R4	M 39-9	0.5	NC
	M 39-2	1.2	C*
	M 40-2	0.0	C
	M 40-4	0.1	C*
	M 40-7	0.0	NC
B1 R5	M 41-5	1.8	NC
	M 41-7	1.8	NC*
	M 41-14	0.0	C
	M 41-15	2.0	C
	M 41-16	0.0	C
	M 42-2	2.7	C*
	M 42-4	0.0	C
	M 42-9	2.0	C*
	M 43-1	2.4	NC
B1 R6	M 44-8A	0.3	NC*
	M 44-8	0.5	C
	M 44-9	4.5	C*
	M 44-10	0.0	C
	M 44-11	1.2	C*
	M 44-12	1.2	C
	M 45-1	1.0	C*
B1 R7	M 45-9	3.2	C
	G LATERAL	4.6	C
	M 46-1	1.6	C*
	I LATERAL	3.0	C
	J LATERAL	11.0	C
	M 46-6	0.0	C
	M 46-9	1.4	NC*
	M 47-3	0.0	C
	M 47-7	0.0	C
B2 R1	M 47-6	0.8	NC
	M 48-1	1.8	NC*
	M 48-2	1.2	NC
B2 R2	M 48-12	2.4	NC*
	M 49-8	5.2	NC*
	M 49-6	1.6	NC
	M 49-1	0.0	NC
B2 R3	M 49-9	1.7	NC*
	E SPILL	0.0	
	M 50-5	9.2	C
	M 50-7	0.0	C
	M 50-3	0.3	C*
	M 51-1	0.0	C
	M 51-2	0.0	C
	M 51-6	1.8	NC*
	K LATERAL	6.0	C
B2 R4	M 51-10	1.0	C
	M 52-3	0.2	C*
	M 52-8	2.4	NC
	M 52-10	4.0	NC*
	47 LATERAL	0.0	C
	M 52-12	0.8	NC*
B2 R5	M 53-9	1.5	NC
	M 53-10	0.8	NC*
	M 53-12	0.7	NC
	M 53-17a	0.0	C
B2 R6	M 53-17	0.0	C
	M 54-5	0.6	NC*
	M 54-10	0.7	NC*
	N LATERAL	8.4	C
	M 55-2a	0.0	C
	M 55-2	0.0	C
	M 55-4a	1.6	C
B2 R7	M 55-5	4.0	C*
	M 55-6	0.0	C
	M 55-8	2.4	C
	M 56-2	0.8	NC*
	M 56-7	0.0	C
	O LATERAL	5.0	C
B2 R8	SPILL	10.0	
	P LATERAL	16.0	C
	M 58-12	3.0	NC*
	M 58-15	0.0	C
	M 58-17	0.7	NC
	M 59-9	2.0	NC*
	M 59-2	1.5	NC
	M 59-10	2.0	C*
	M 59-10a	2.4	C*
B3 R1	M 60-6	1.0	C
	Q LATERAL	28.2	C
	M 60-8	0.0	C
	M 60-9	0.0	C
	M 61-3	1.2	C*
	M 61-5	3.4	C
B3 R2	M 61-13	1.6	C*
	SPILL	6.1	
B3 R3	M 61-22	2.0	C*
	M 62-3	0.0	C
	M 62-6	1.0	C
	M 62-10	4.5	C*
	M 62-15	1.0	NC
	SPILL	2.5	
	M 62-19	1.6	NC*
B3 R4	M 63-4	2.0	C*
	M 63-8	0.5	NC
	M 63-15	2.5	C*
B3 R5	M 64-4	1.6	C
	R LATERAL	3.0	C
	M 65-12	0.1	C*
B3 R6	M 65-15	3.0	C*
	S LATERAL	27.0	C
	66-6, 67-3	4.4	C
	SPILL	1.9	

Table 2. Turnout and Lateral flow rates used in Main Line Simulations. "*" indicates turnouts shut off for 60% non-continuous simulations.

Figures 18(a) and (b) show the effects of 60% and 100% of the turnouts being shut off for 2 hours. Lateral turnout settings remained constant. Water surface fluctuations in Branch 1 were almost nonexistent, due to the small number of turnouts and the automatic control structures at Reach 4 and Reach 5 sending the excess flow downstream. Water level fluctuation was higher in Branch 2, but was still below one inch for 60% non-continuous turnouts, and below two inches for 100% non-continuous.

Figures 19(a) and (b) show the effects of shutting 100% of the turnouts off for 2, 3, and 6 hours. The change in flow rate increases slightly from 2 to 3 hours, but there is no change from 3 to 6 hours, indicating that maximum flow rate variation occurs within 3 hours of 100% turnout shutoff. Water surface was still increasing at 6 hours, as the volume of water stored in the canal changed. Water surface fluctuations were still relatively small, under 2 inches in most cases, although at 6 hours shutoff time fluctuation approached 3 inches in some downstream reaches.

The effect of variation in the canal inflow rate compared with the effect of shutting off all turnouts for 2 hours is shown in figures 20(a) and (b). Actual inflow rate into the canal for July 7, 1990 from U.S. Bureau of Reclamation records was used. The inflow hydrograph is shown below. Turnout settings were constant. The figures show that the effects of variation

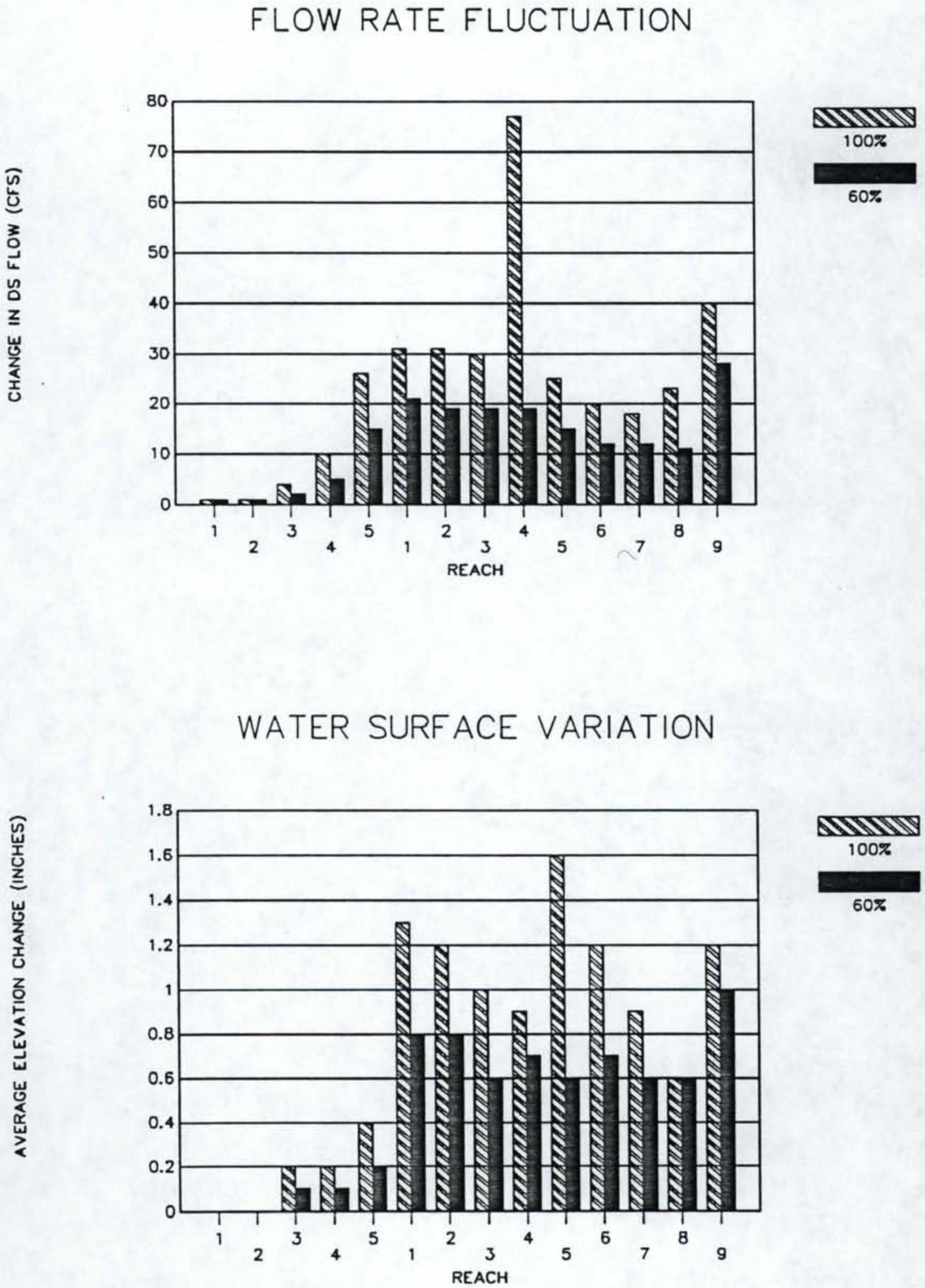
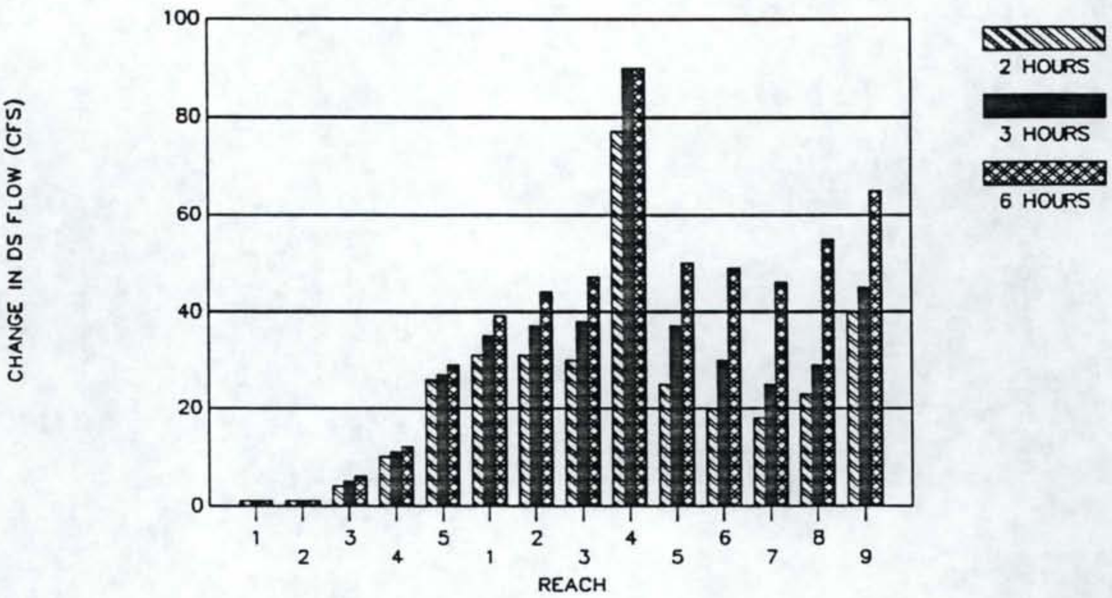


Figure 18(a) and (b). Main Line flow rate and water surface fluctuation from 2 hour turnout shutoff.

FLOW RATE FLUCTUATION



WATER SURFACE FLUCTUATION

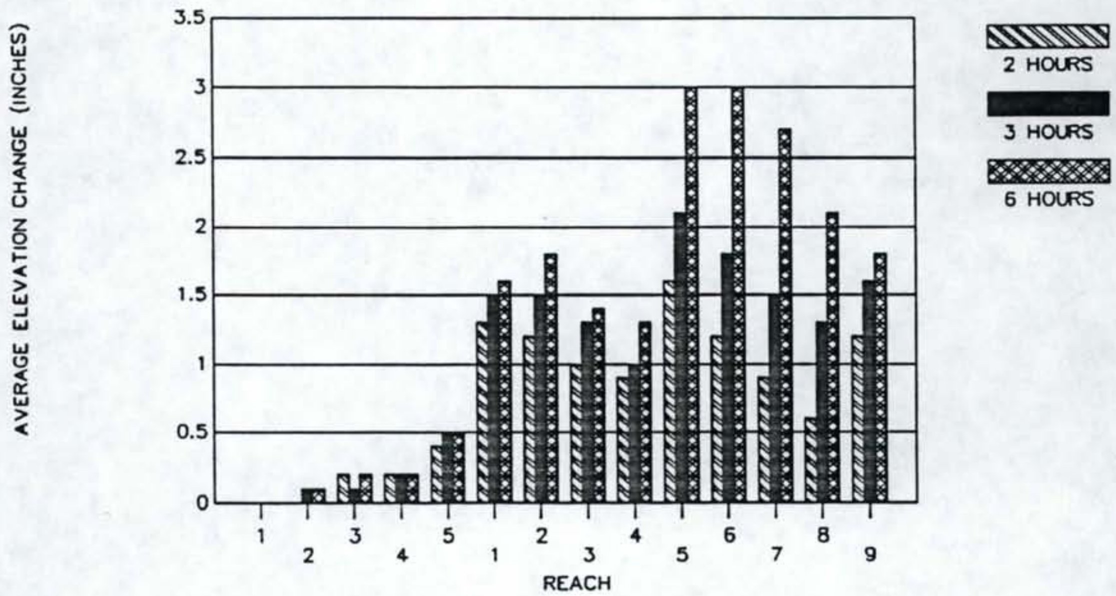


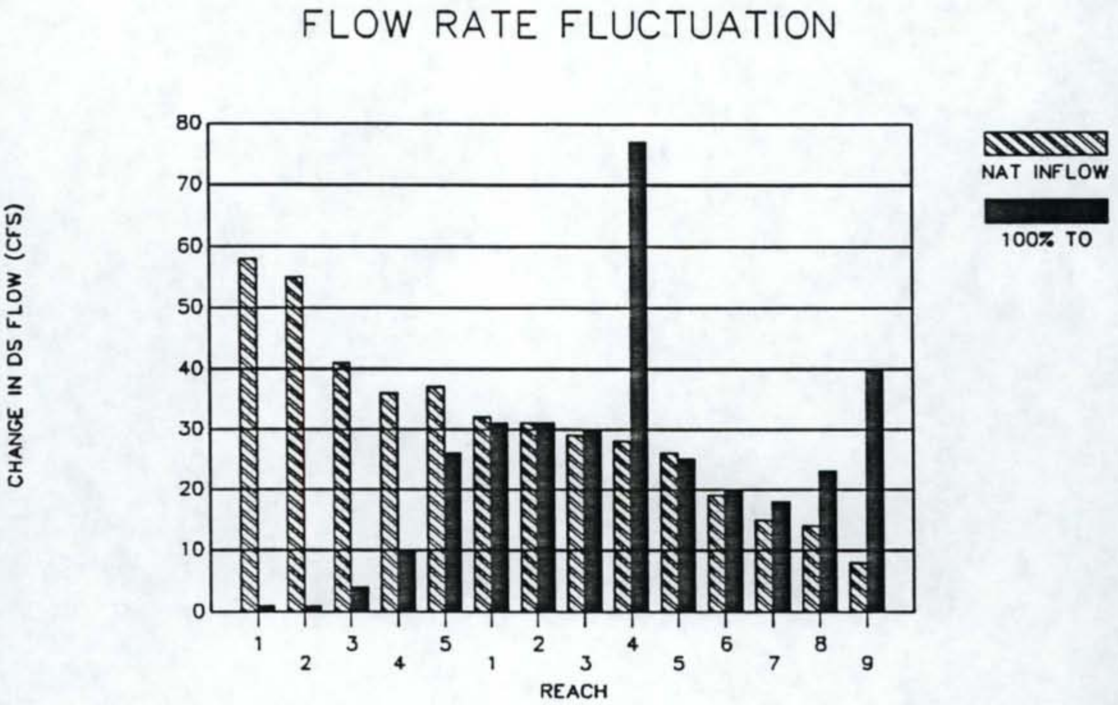
Figure 19(a) and (b). Main Line flow rate and water surface fluctuation with increasing shutoff time for 100% turnout shutoff.

in canal inflow decreases as the flow progresses down the canal. Non-continuous turnout flow has more effect on the lower reaches.

Figures 21(a) and (b) show the effects of natural varying inflow rates and shutting 60% of the turnouts off for 2,6, and 10 hours. Note the large increases in water surface and flow variation from 6 to 10 hours in Branch 2. This is an indication that maximum fluctuation has not yet been reached.

The effects on flow rate and water surface variation when the control structures are operated manually are shown in figures 22(a) and (b). The manually operated structures were not adjusted during the simulation, as 60% of the turnouts were shut off for 2 hours. While there are some slight variations, water surface fluctuation is below 1 inch for all reaches for both methods of control.

Figures 23(a) and (b) show the predicted effect of modifying the control structures so that they are entirely sluice gates, with no weirs. Total structure width remained the same. 100% of the turnouts were shut off for 2 hours, and the structures were controlled automatically. Slight variations in the water surface exist, but there are no significant differences.



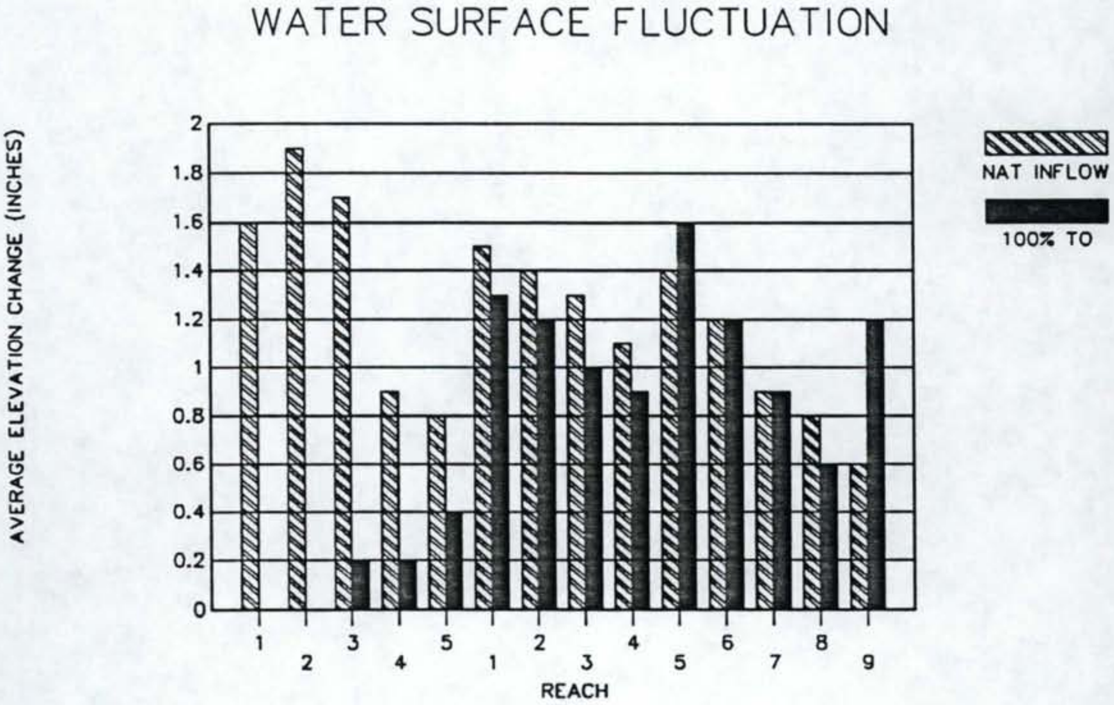
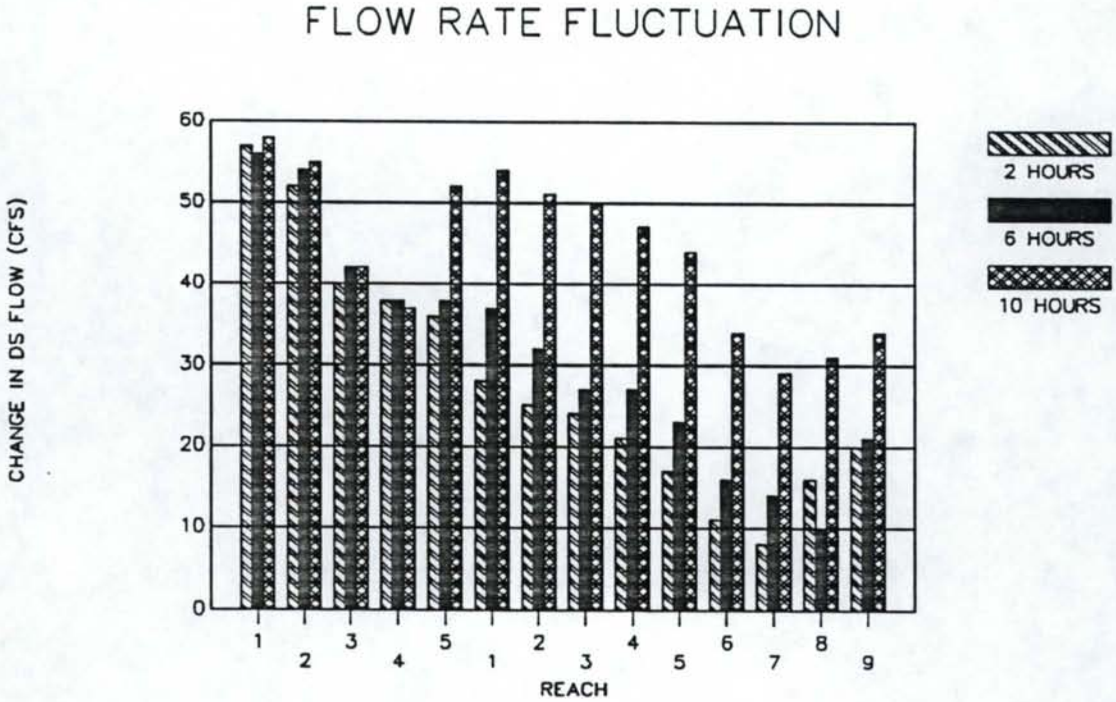


Figure 20(a) and (b). Main Line flow rate and water surface fluctuation from canal inflow and from 2 hour, 100% turnout shutoff.



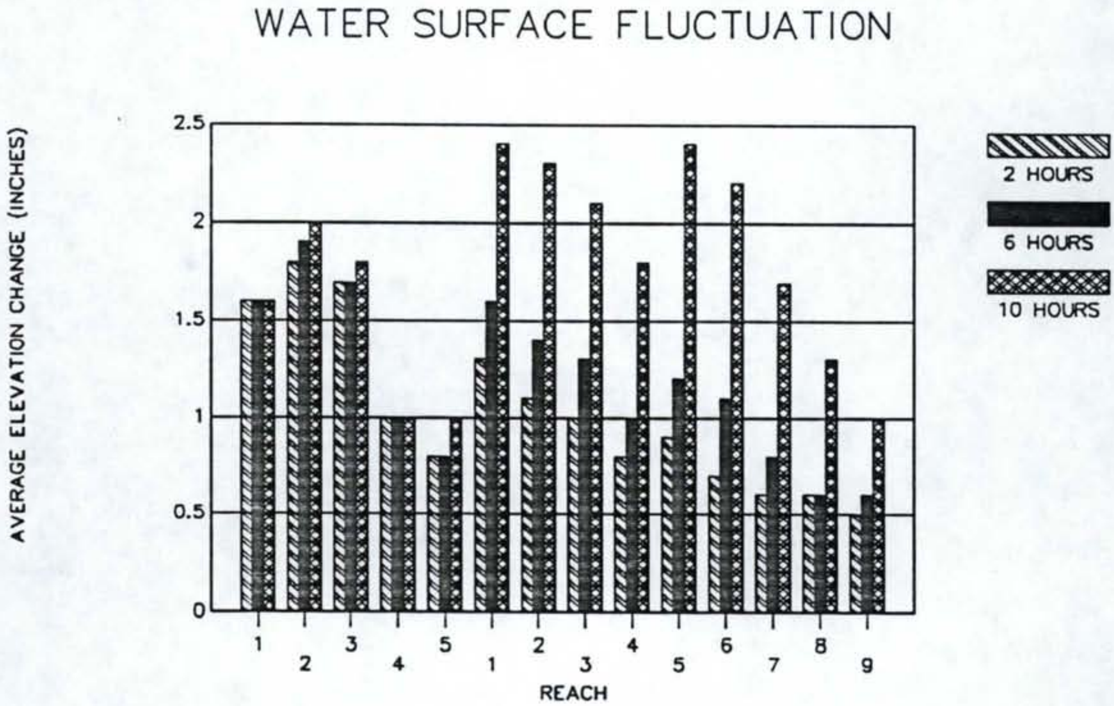


Figure 21(a) and (b). Main Line flow rate and water surface fluctuation with increasing shutoff time for non-continuous turnouts.

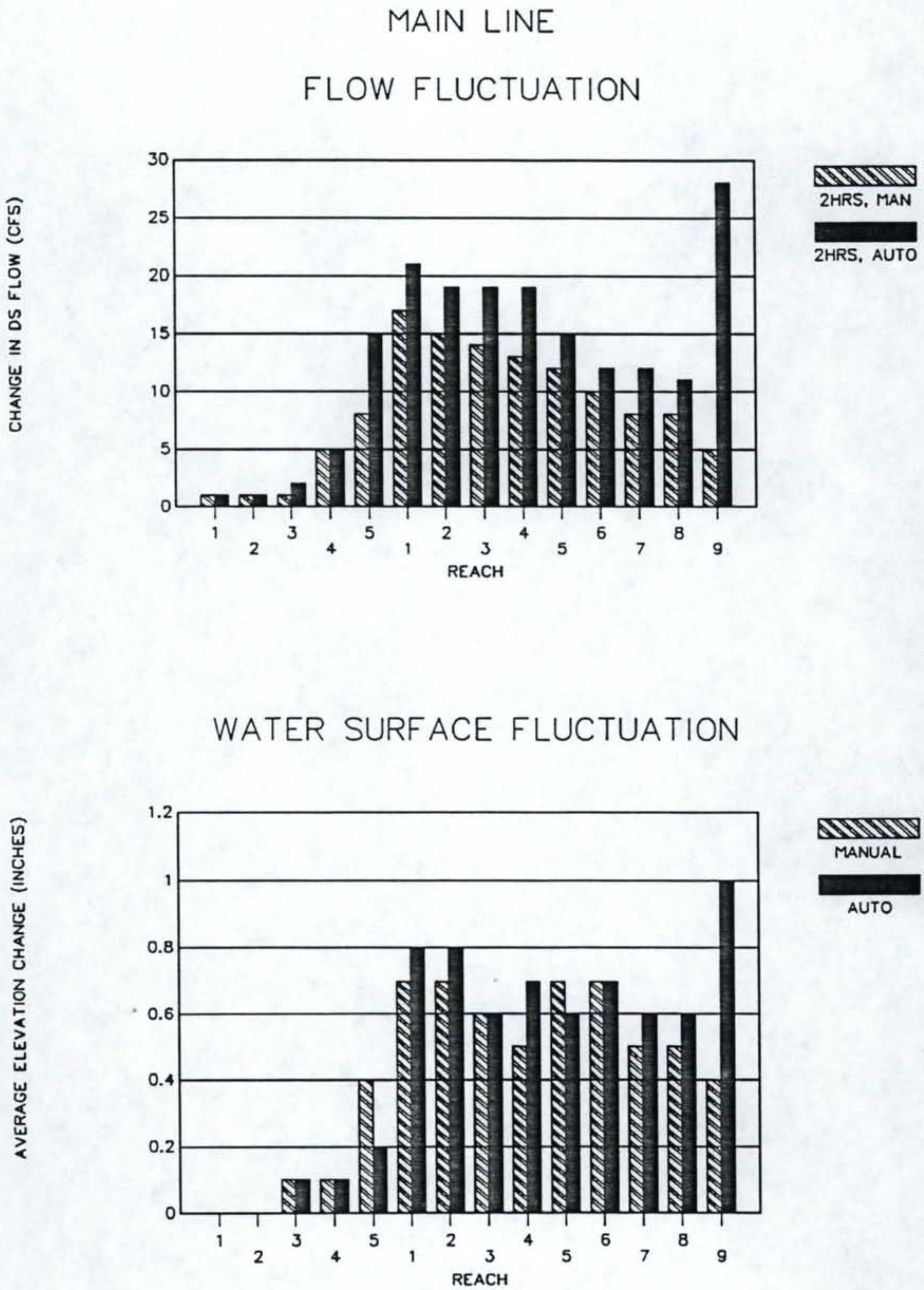
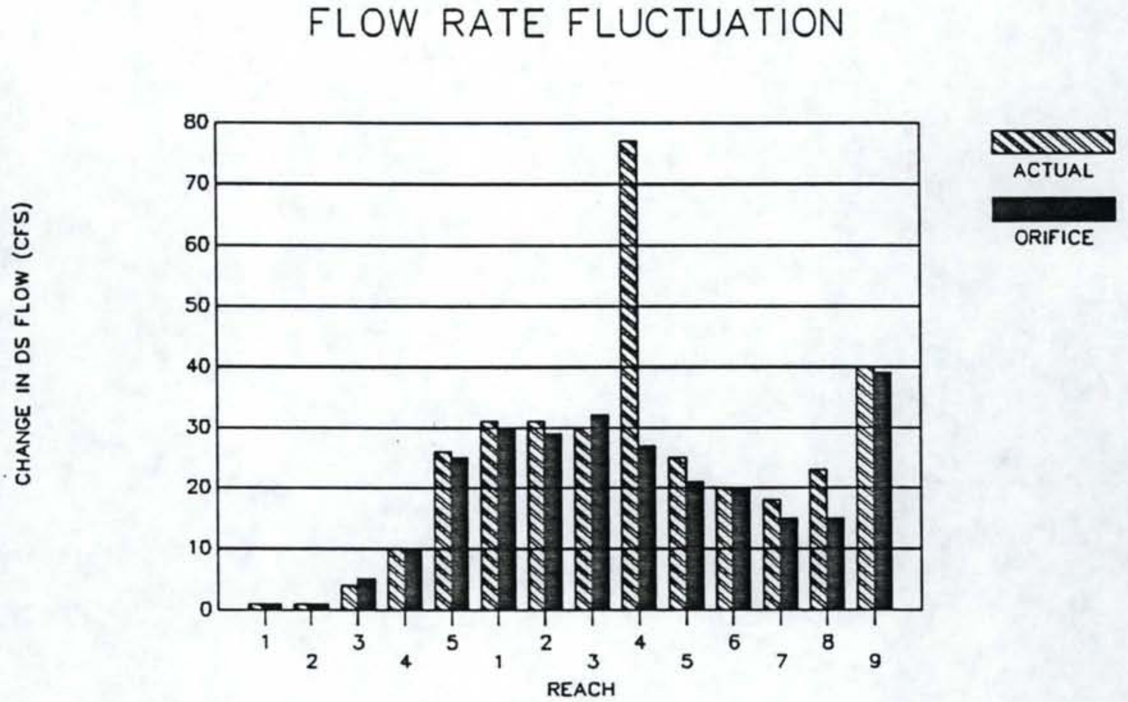


Figure 22(a) and (b). Main Line flow rate and water surface fluctuation with manual and automatic control structures, 2 hour, 60% turnout shutoff.



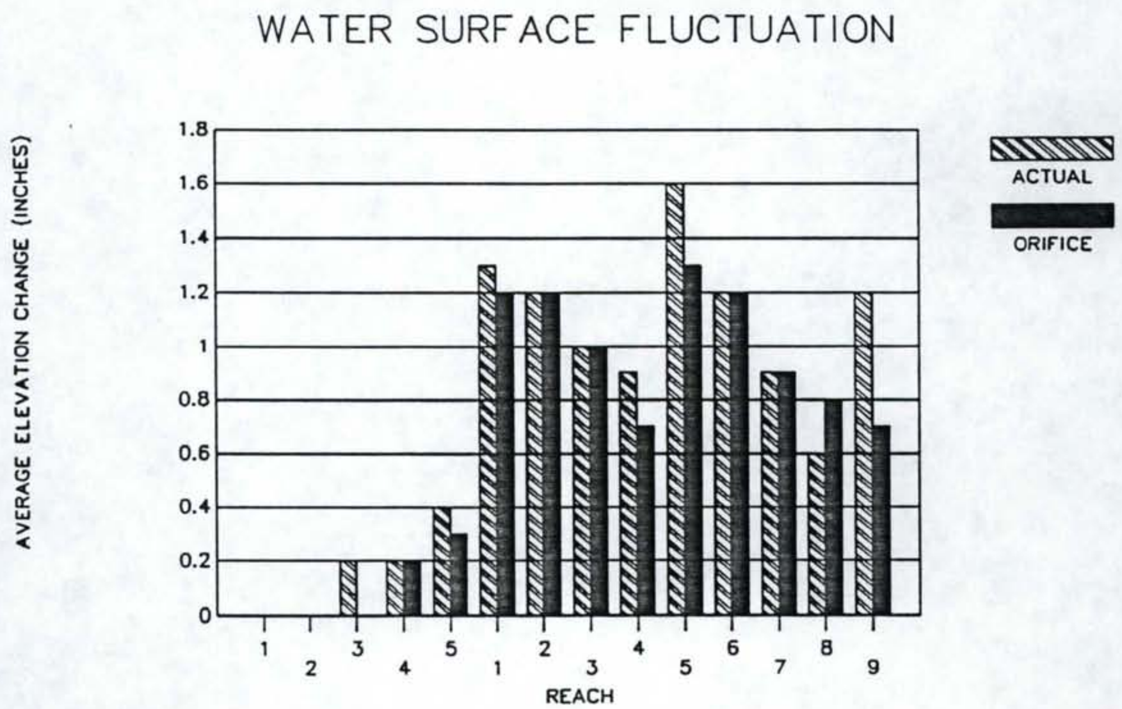
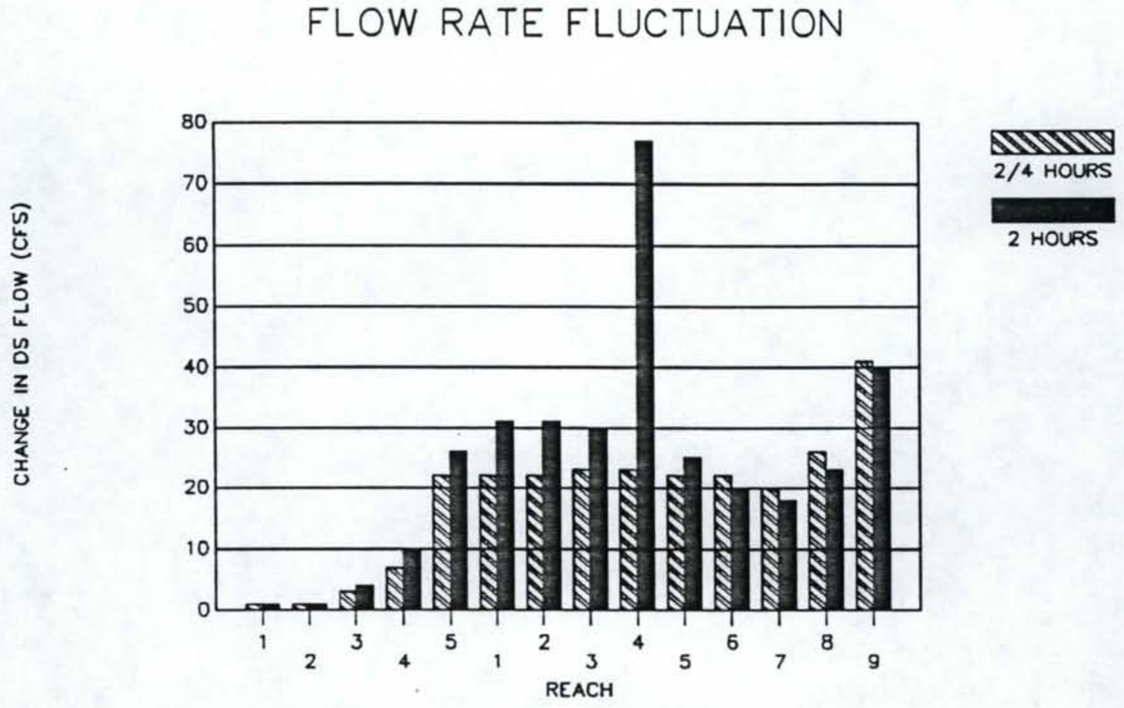


Figure 23(a) and (b). Main Line flow rate and water surface fluctuation with orifice control structures, 2 hour, 100% turnout shutoff.

Figures 24(a) and (b) show the results of staggering the period of non-continuous flow. All turnouts were shut off for 2 hours in both simulations. In the first, however, turnout shut off times were staggered. 20% were turned off from 1-3 hours, 60% from 2-4 hours, and 20% from 3-5 hours. While most reaches show decreased variation in water surface and flow rate, several reaches in Branch 2 show the opposite effect. This is due to the increased effect of upstream turnouts as the increased flow rates have travelled downstream far enough to effect these lower reaches.



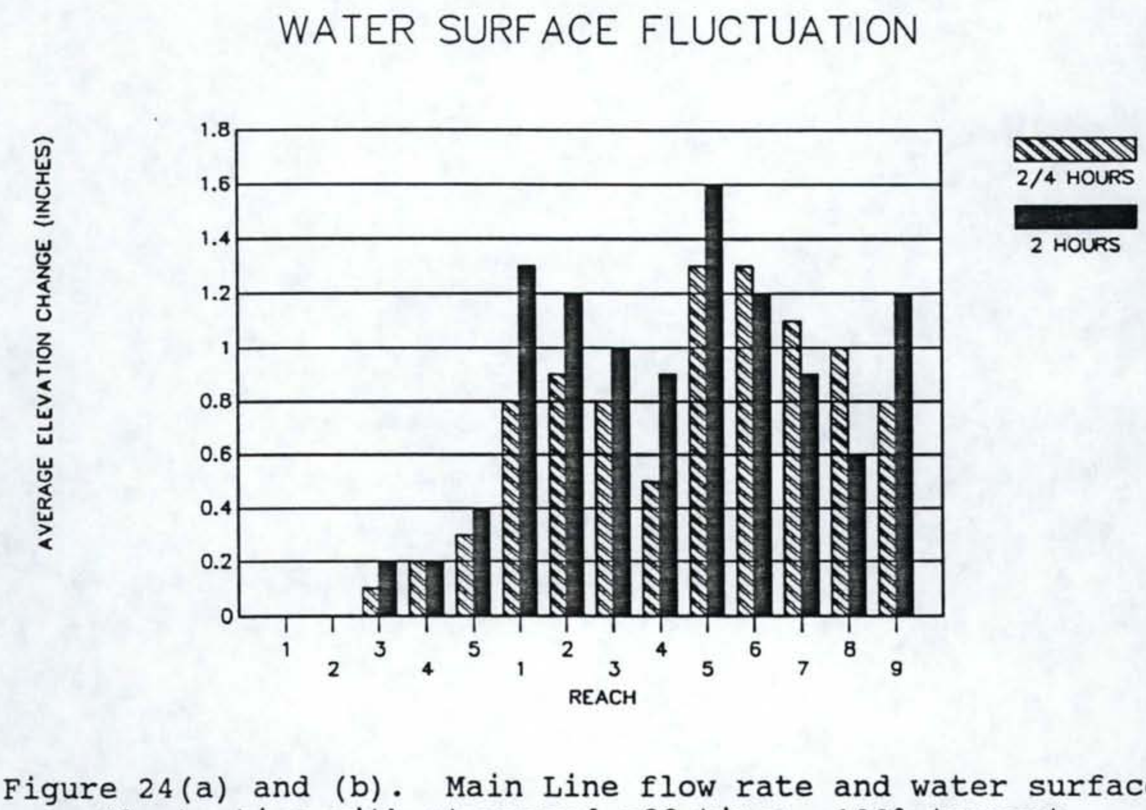


Figure 24(a) and (b). Main Line flow rate and water surface fluctuation with staggered off times, 100% turnout shutoff.

(C) High Line

The configuration files for the high line appear in Appendix B. Figure 25 illustrates the branch and reach configuration used. There are adjustable control structures at the downstream end of all but 7 reaches. These reaches are marked on the figures. The control structures are automatically adjusted to control the water surface immediately upstream of the control structure at the following locations: Branch 1, Reach 3 and 7; Branch 2, Reach 1 and 4; and Branch 3, Reach 1 and 2. The simulations were run in the manual control mode because not all control structures were automatically adjusted. Those that are automatically adjusted were adjusted manually during the simulation to attempt to keep the water level fluctuation small.

There are 5 spillways on the High Line, located in: Branch 2, Reach 3 and 8; and Branch 3, Reach 2, 3 and 6. The spillway in Branch 2 Reach 3 is not used in everyday operations. For the simulation it was not used at all.

Inflow into the High Line is not measured. It is controlled by manual adjustments at the control structure at the junction between the High Line and the Main Line. Flow is kept relatively constant by the automatic gate on the spill to the Big Fill Reservoir, directly upstream of this junction. Flow rate used in the simulations was 360 cfs, based on 281 cfs for

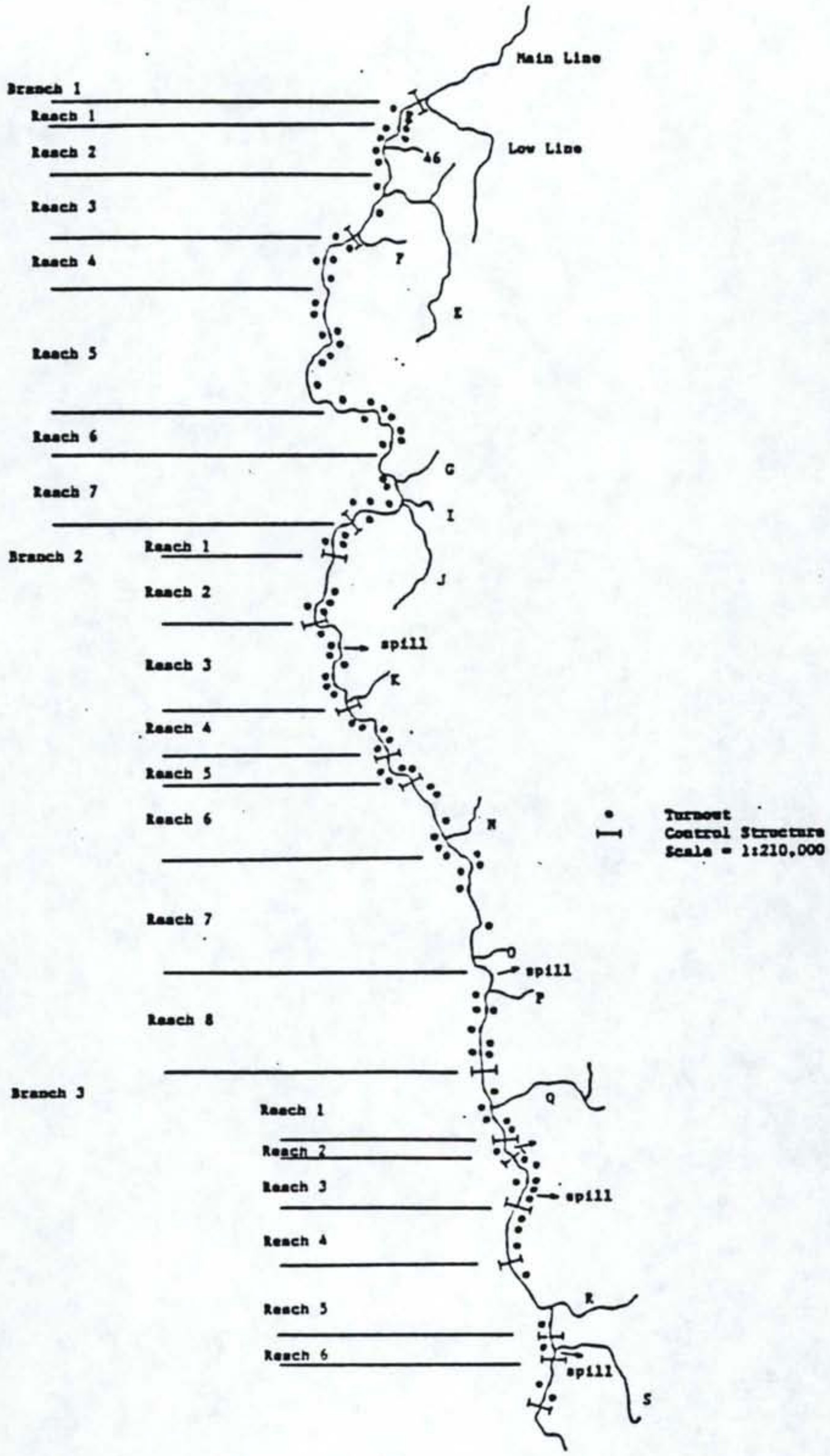


Figure 25. Reach configuration used in High Line simulations.

individual and lateral turnouts, 20 cfs for spills, and 59 cfs for seepage.

Table 3 shows the turnout flows from July 7, 1990, used in the simulations. Approximately 34% of the individual turnouts on the High Line are currently non-continuous turnouts. For the July 7 data, these turnouts accounted for 17% of the total turnout flow (including flow into laterals). Turnouts shut off during 60% non-continuous simulations are marked with a "*" in the "Outlet" column in the table.

Figures 26(a) and (b) show the effects of shutting off turnouts for 2 hours. 100%, 60%, and current non-continuous turnouts were shut off. In several reaches, fluctuation caused by current non-continuous users all shutting off for 2 hours was greater than that caused by 60% of the turnouts shutting off. Fluctuations in flow rate peak in Branch 2, while water surface fluctuation increased through Branch 1 and then remained relatively stable. Note the decreased fluctuation in reaches which have active spillways.

The effects of shutting off 100% of the turnouts for varying periods of time is shown in figures 27(a) and (b). Fluctuation with flow rate and water surface increases steadily from 2 to 4 to 8 hour shutoff times, indicating maximum fluctuation would be larger than that shown. Again,

Branch # Reach #	Turnout Number	Flow Rate (cfs)	Outlet
B1 R1	M 1-6	1.4	C*
	M 7-4	0.0	C
B1 R2	M 8-2	0.5	NC*
B1 R3	M 14-2	0.0	C
	M 14-4	0.0	C
	M 16-2	0.0	C
	M 16-4	1.4	C
	M 16-8	0.0	C
	M 16-10	0.0	C
	M 17-1	2.0	C*
	M 17-3	1.1	C
	M 17-4	1.0	C*
B1 R4	M 18-5	1.7	NC*
	M 18-9	0.0	C
	M 18-10	0.0	C
	M 19-3	1.4	C
	M 19-9	0.0	C
	M 19-6	0.2	C*
	M 19-10	1.7	C
	M 20-3	1.2	NC*
	M 20-5	0.3	C*
B1 R5	M 20-6	2.4	NC
	M 20-4	1.7	C*
	M 21-6	2.5	NC
	M 21-7	3.2	C*
	M 21-8	0.7	NC*
	M 22-8	1.6	C
	M 22-9	2.0	C*
	M 22-10	1.5	C*
	A LATERAL	18.0	C
	H LATERAL	14.0	C

Branch # Reach #	Turnout Number	Flow Rate (cfs)	Outlet
B2 R1	M 22-12	2.0	C*
	M 23-1	2.4	C*
	M 23-5	1.7	NC
	M 23-8	6.0	C*
	M 23-9	0.0	C
B2 R2	M 24-5	3.0	C
	M 24-7	0.0	C
	M 24-8	0.0	C
	M 24-10	1.5	C*
	M 24-14	0.3	C*
	M 25-1	0.7	C
B2 R3	M 25-7	0.0	C
	M 25-8	0.0	C
	M 25-9	0.0	C
	M 26-3	1.0	C*
	45 LATERAL	7.0	C
	M 26-8	2.0	C
	M 26-8A	0.7	NC*
B2 R4	M 26-9	0.0	C
	M 26-10	2.0	C*
	C1 LATERAL	23.2	C
	C LATERAL	26.0	C
	M 27-5	0.0	C
	SPILL	0.0	
B2 R5	M 28-6	4.0	C
	M 28-7	0.0	C
	M 29-2	1.2	NC*

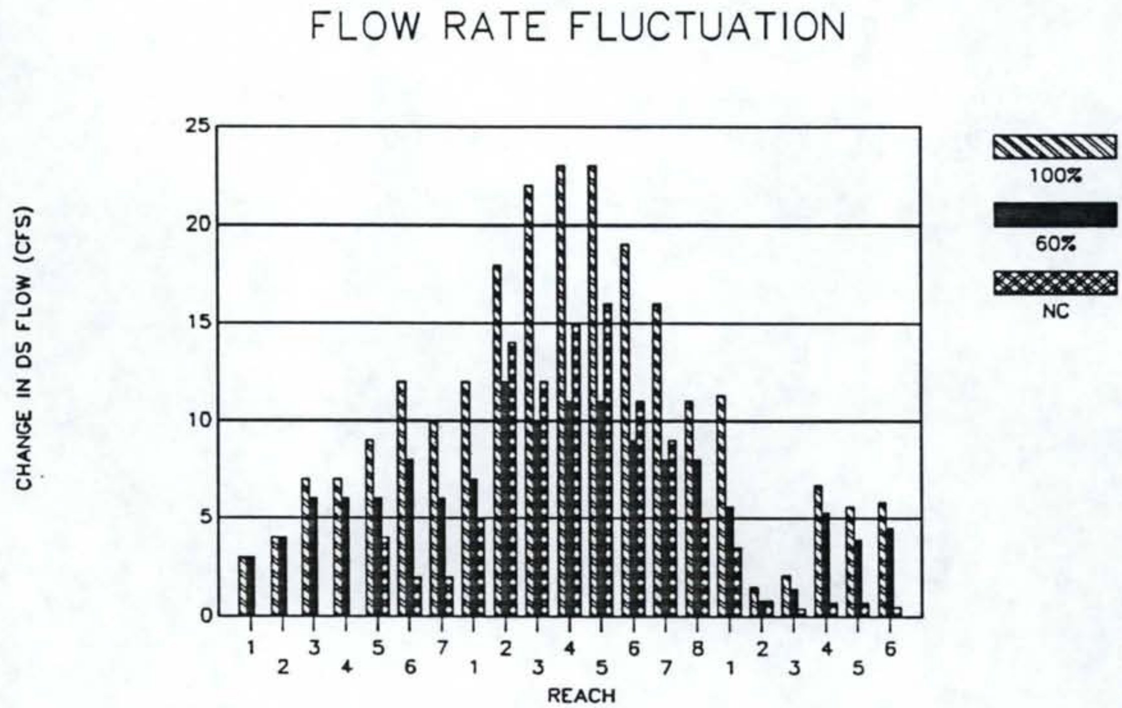
Branch # Reach #	Turnout Number	Flow Rate (cfs)	Outlet
B2 R6	M 29-4	1.5	C
	M 29-6	1.5	NC*
	M 30-8	0.0	C
	M 30-9	0.0	C
	M 30-10	0.6	C*
	M 31-1	1.0	NC
	M 31-2	2.3	C*
	M 31-3	2.3	C*
B2 R7	M 32-3	1.0	C
	M 32-5	1.0	NC*
	D LATERAL	18.0	C
	M 33-9	0.0	C
	M 33-10	0.0	C
	M 34-3	2.5	NC
	M 34-4	0.0	C
B2 R8	M 34-6	0.0	C
	M 34-5	1.5	NC*
	M 34-7	0.0	C
	M 34-8	1.0	C*
	M 34-8a	2.5	C
	M 34-10	0.0	NC
	M 34-9	1.4	C*
	M 35-2a	4.8	C
	M 35-2	2.5	C*
B2 R9	M 35-4	2.0	NC*
	M 35-6	0.0	NC
	M 35-9	0.0	C
	M 36-1	0.0	C
	SPILL	58.0	
	SPILL	13.0	
	LOW LINE	217.0	C

Table 3. Turnout and lateral flow rates used in High Line simulations. "*" indicates turnouts shut off for 60% non-continuous simulations.

fluctuation is substantially reduced in reaches with active spillways.

Figure 28(a) and (b) illustrate the effect of 100% of the turnouts shutting off for 2 hours simultaneously, and for 2 hours staggered over a four hour period. For the staggered off-times, 20% were shut off from 1-3 hours, 60% from 2-4 hours, and 20% from 3-5 hours. Fluctuation decreases only slightly for flow rate and water surface.

Figures 29(a) and (b) illustrate the effect of variations in spillway width. The spillways are overflow weirs with several bays. Using only half the spillway width, as is often done during actual canal operations, results in only slight changes in flow rate variation when 100% of the turnouts are shut off for 2 hours. Water surface data indicate increases up to nearly one inch in water surface when only half the spill is used. There is only a slight difference in Branch 2 Reach 8 as the spillway there is over 50 feet long, and 25 feet is still a relatively large spillway.



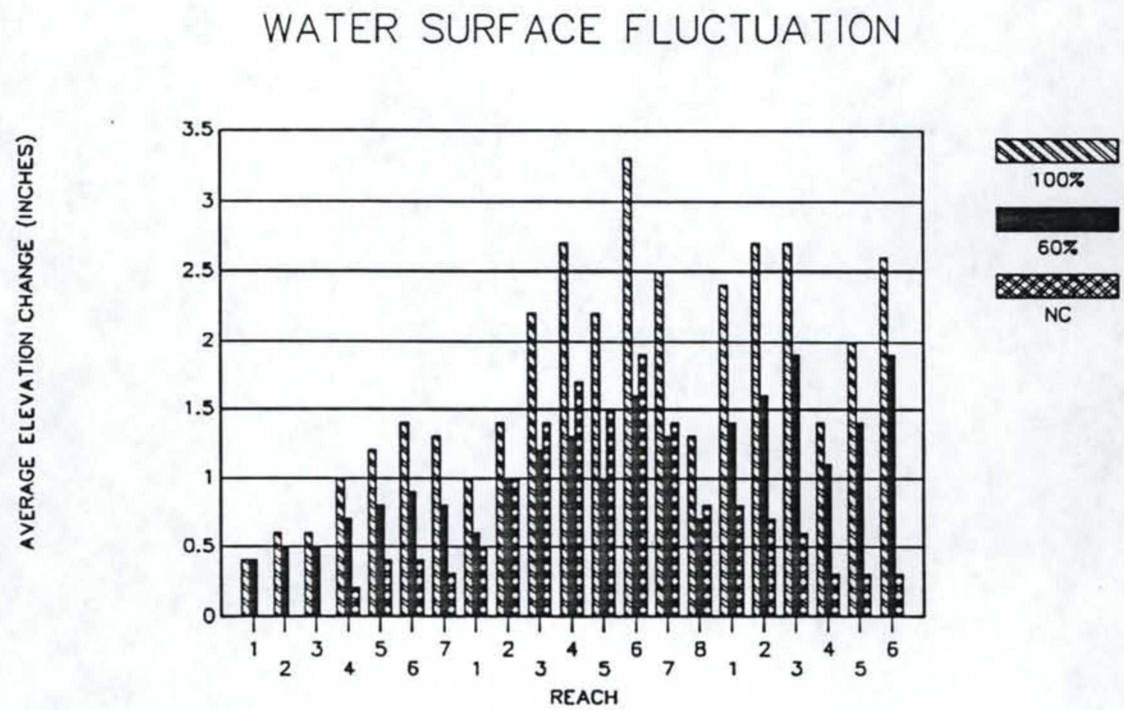
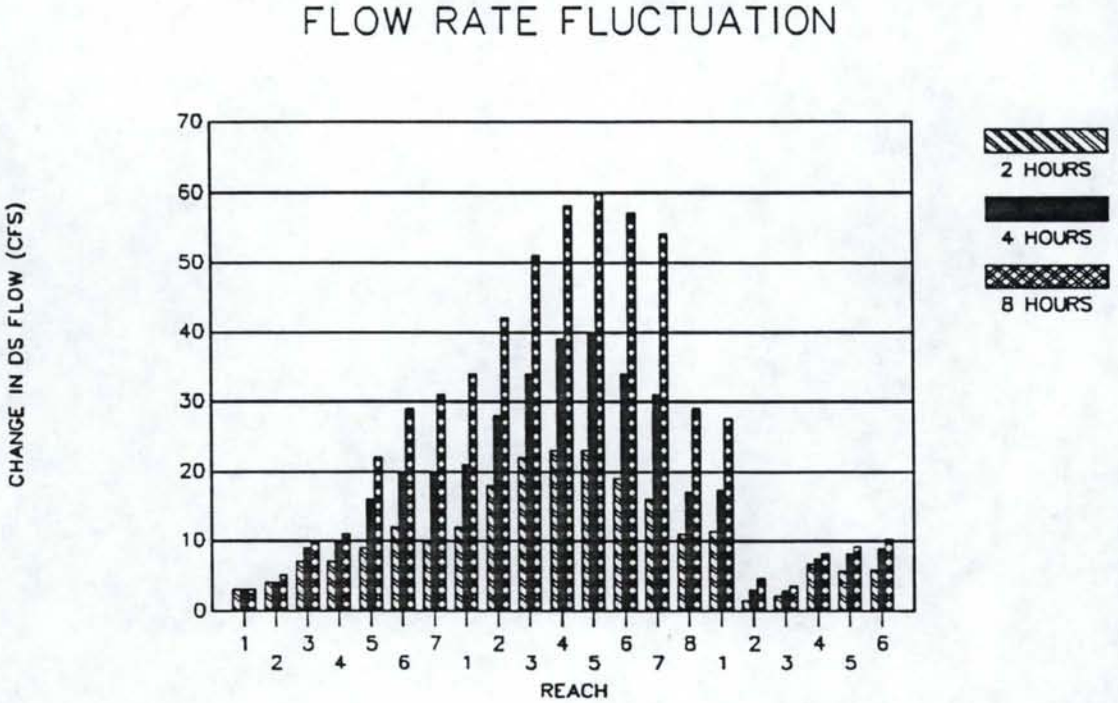


Figure 26(a) and (b). High Line flow rate and water surface fluctuation from 2 hour turnout shutoff.



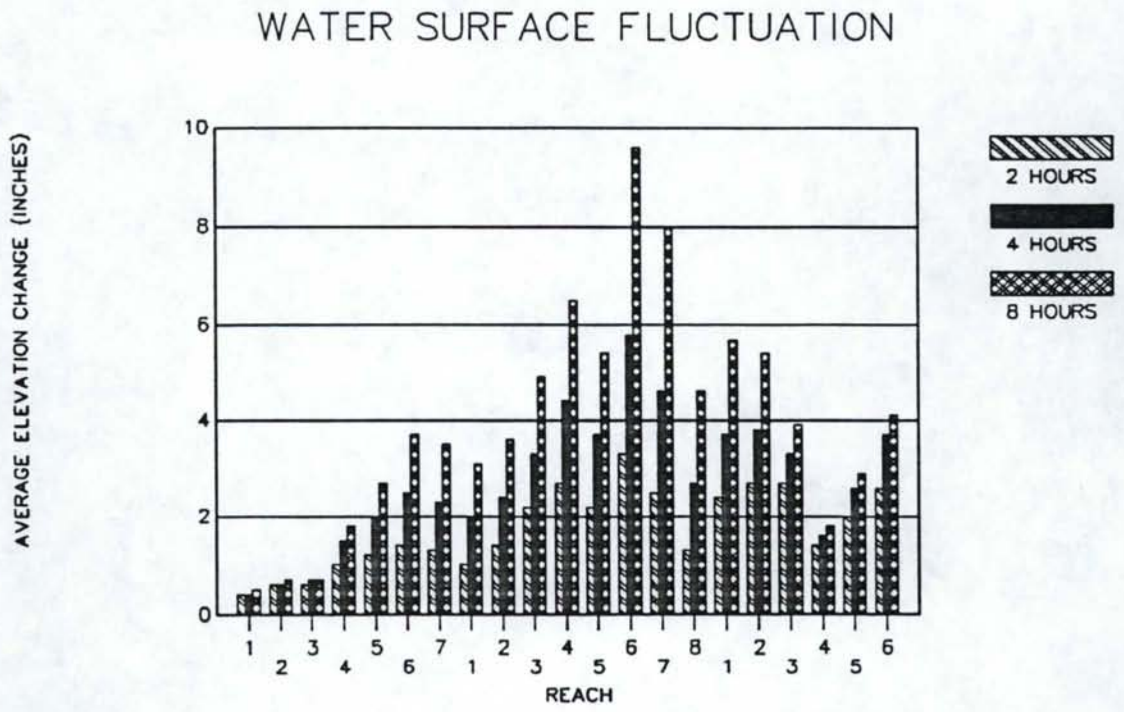
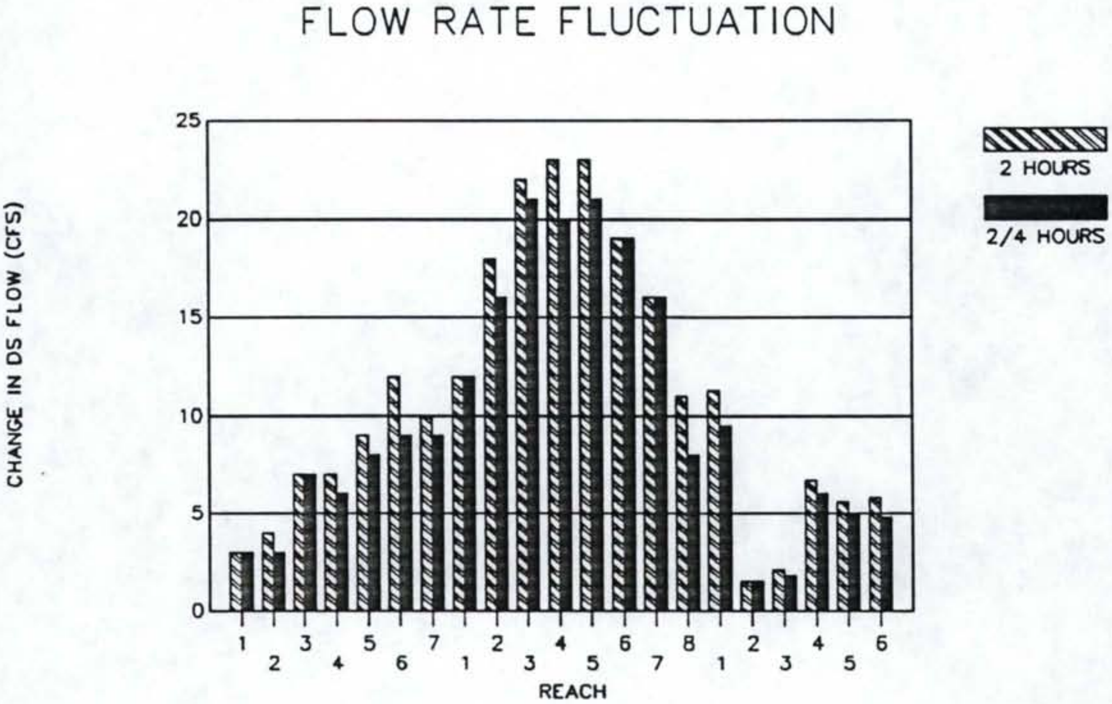


Figure 27(a) and (b). High Line flow rate and water surface fluctuation with increasing shutoff time for 100% turnout shutoff.



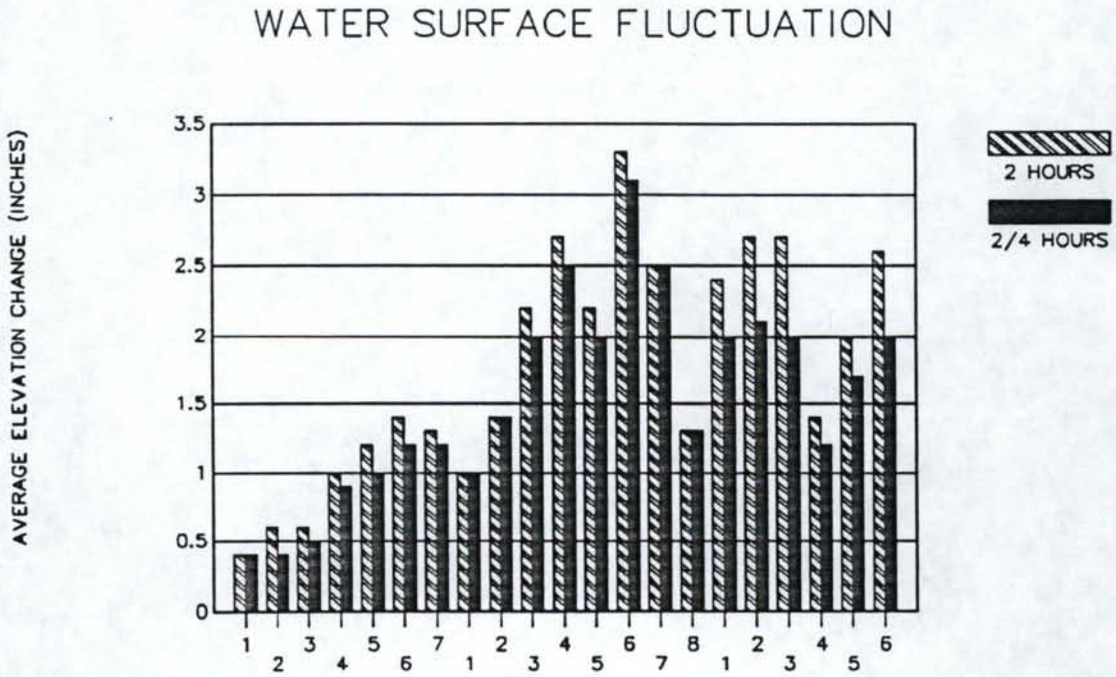
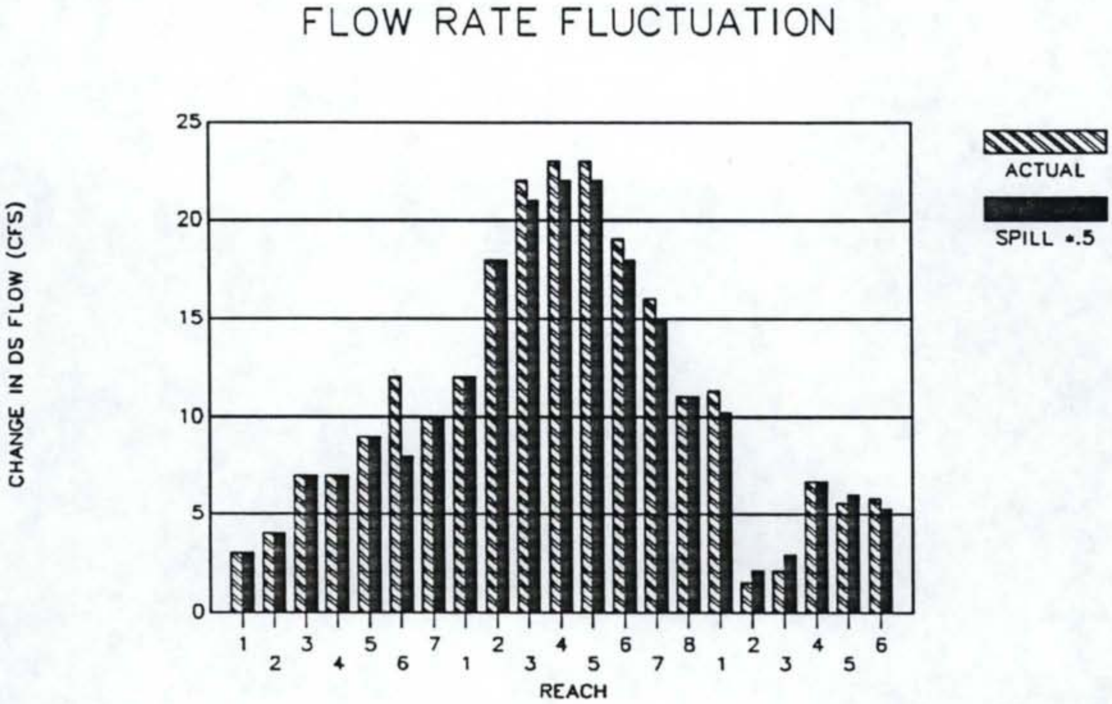


Figure 28(a) and (b). High Line flow rate and water surface fluctuation with staggered off times, 100% turnout shutoff.



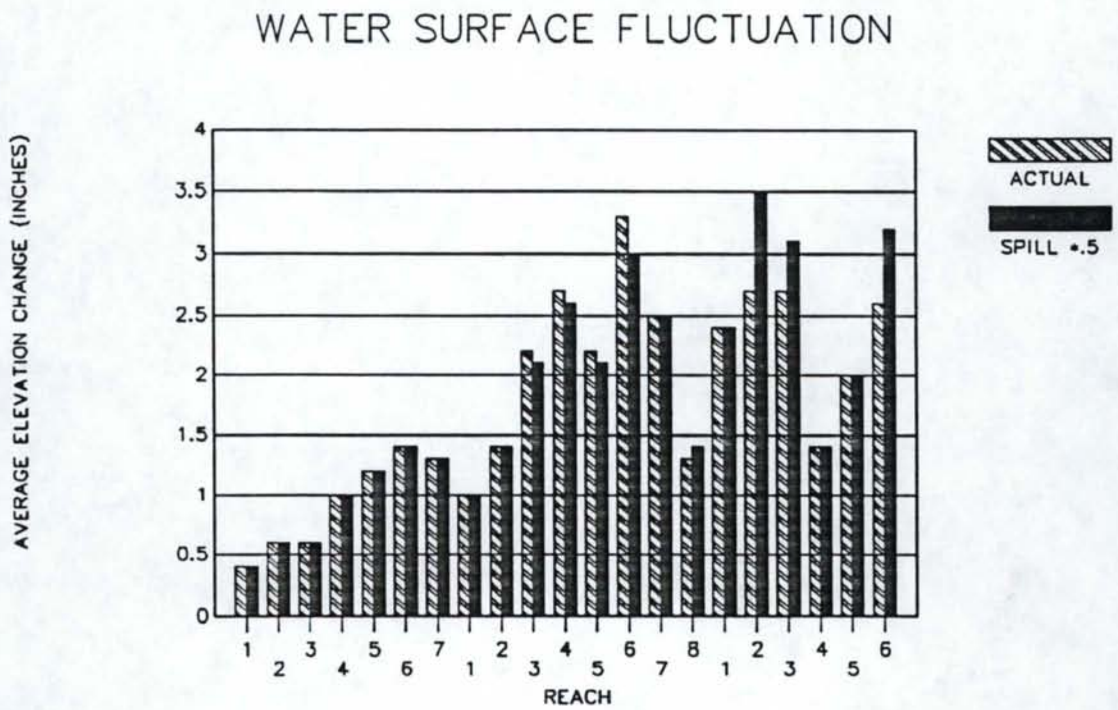


Figure 29(a) and (b). High Line flow rate and water surface fluctuation with smaller spillways, 2 hour 100% turnout shutoff.

(D) Low Line

The configuration files for the low line appear in Appendix B. Figure 30 illustrates the branch and reach configuration used in the simulations. There are automatic control structures in Branch 1, Reach 1 and 7. There is also a spillway with an automatic gate in Branch 2, Reach 2. There are manually adjusted spillways in Branch 1 Reach 6 and 7 and at the downstream end of the Low Line.

Flow rate into the Low Line is not measured. There is a staff gage at the head which measures depth for day to day comparisons. Flow rate into the Low Line comes directly from the Main Line over a manually adjusted weir, and from the Big Fill Reservoir. Flow over the weir is held relatively constant by the automatic gate on the spill into the reservoir. The flow from the reservoir to the Low Line is also controlled automatically. Flow rate used in the simulations was 225 cfs, based on 183 cfs for individual and lateral turnouts, 12 cfs for spills, and 30 cfs for seepage.

Table 4 shows the turnout flows from July 7, 1990, used in the simulations. Approximately 49% of the individual turnouts on the Low Line are currently non-continuous turnouts. For the July 7 data, these turnouts accounted for 33% of the total turnout flow (including flow into laterals).

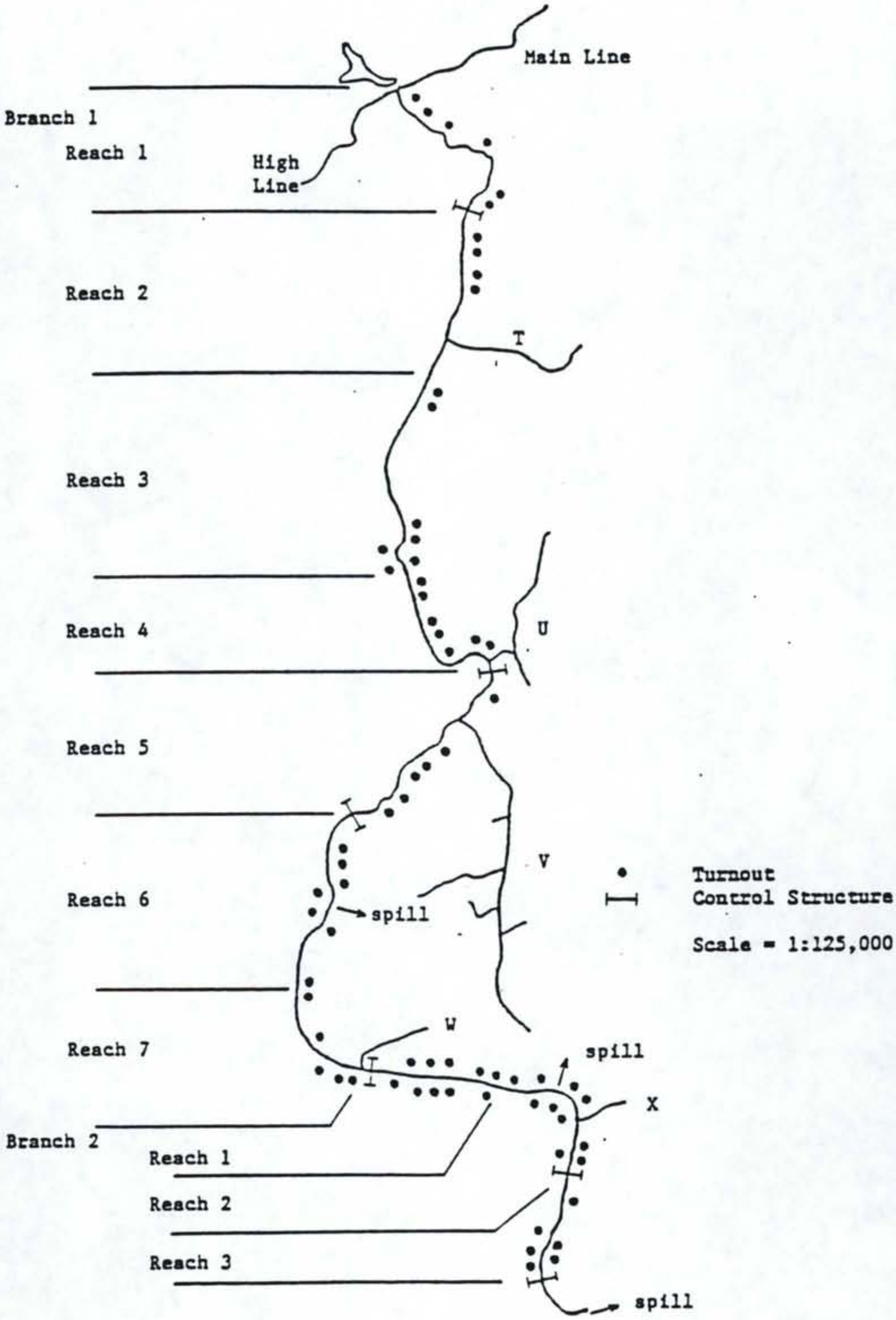


Figure 30. Reach configuration used in Low Line simulations.

Branch # Reach #	Turnout	Flow Rate cfs	Outlet
B1 R1	L 0-1	1.5	NC+
	L 0-4	0.0	C
	L 0-6	0.0	C
	L 0-8	1.0	NC*+
	L 1-8	3.9	C
	L 1-10	2.5	C+
B1 R2	L 2-6	3.5	C
	L 2-8	5.2	C*+
	L 3-1	0.0	C
	L 3-1A	2.0	NC+
	T LATERAL	5.0	C
B1 R3	L 4-2	1.3	NC
	LT 0-9	0.6	NC*+
	L 6-4	2.0	C
	L 6-5	1.6	NC+
	L 6-8	1.6	C+
	L 6-9	1.6	C*
	L 6-10	1.2	C+
B1 R4	L 7-2	1.4	NC
	L 7-3	2.0	C+
	L 7-6	0.9	NC*+
	L 7-7	0.0	C
	L 8-2	0.0	C
	L 8-6	1.0	NC
	L 8-4	1.6	NC+
	U LATERAL	20.7	C
B1 R5	L 9-5	1.8	NC
	V LATERAL	40.0	C
	L 9-9	0.0	NC
	L 10-5	1.5	NC*+
	L 10-6	1.0	NC
	L 10-9	3.4	C+
	L 11-2	1.0	C
B1 R6	L 11-9	1.6	NC*+
	L 12-3A	1.6	NC+
	L 12-5	0.0	C
	L 12-6	3.2	C
	SPILL	1.7	
	L 12-8	0.0	C
	L 12-9	0.0	NC
	L 12-10	4.0	NC+
	L 13-6	1.6	NC*
B1 R7	L 13-3	2.8	NC+
	L 14-6	1.3	C+
	L 14-8	1.8	C
	E SPILL	0.0	
	L 15-1	0.6	C*+
	L 15-2	2.8	NC
	W LATERAL	10.0	C
B2 R1	L 15-4	0.8	NC+
	L 15-8	0.6	NC*+
	L 16-1	2.0	NC
	L 16-1A	0.8	C+
	L 16-2	1.0	C
	L 16-2B,2A	4.5	NC*+
	L 16-7A	1.3	C+
	L 16-8,9	3.7	NC
	L 16-13	0.0	C
B2 R2	L 17-2	0.0	C
	L 17-3	0.0	C
	SPILL	1.7	
	L 17-10	3.9	NC+
	L 17-7	4.0	NC*
	L 17-9	0.8	NC+
	X LATERAL	0.0	C
	L 18-3	4.9	NC+
	L 18-7,5	5.6	C
B2 R3	L 18-12	0.8	C*+
	L 19-6	2.0	NC
	L 19-4	3.0	C+
	L 19-9	0.9	NC+
	L 19-10A	1.6	C*
	L 19-11	1.4	C+

Table 4. Turnout and lateral flow rates used in Low Line simulations. "*" indicates turnouts shut off for 25% non-continuous simulations. "+" indicates turnouts shut off for 60% simulations.

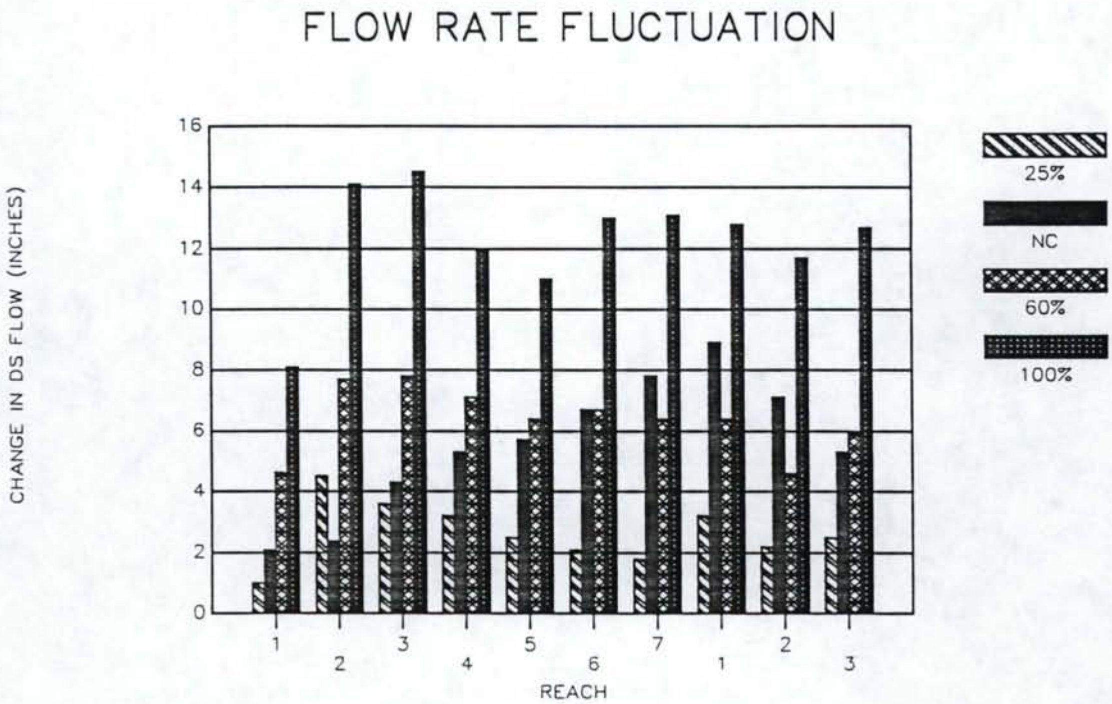
There are several locations along the Low Line where flow is added by spillways from High Line laterals, drainage pipes, and pumped groundwater. The simulation model CANAL will not accept inflow to a canal except at the head, so these inflows could not be simulated. The estimated magnitude of inflow is between 0 and 10 cfs at all times.

Figures 31(a) and (b) show the consequences of shutting off turnout flows for 2 hours. 25%, 60%, 100%, and the non-continuous turnouts were shut off. Water surface fluctuations as large as 6 inches are projected for Branch 3 when 100% of the turnouts are off for 2 hours.

Figures 32(a) and (b) illustrate the predicted fluctuations resulting from 100% turnout shutoff for periods of 2, 4, and 8 hours. Fluctuation increases as the turnouts are shut off for longer time periods throughout the Low Line, therefore, shut off periods longer than 8 hours would cause even larger fluctuations than simulated. Water surface fluctuates over 12 inches in the downstream reaches when 100% of the turnouts shut off for 8 hours.

Figures 33(a) and (b) show the effects of adding 3 additional control structures to the Low Line. The control structures were added at Branch 1 Reach 2 and 3, and Branch 2 Reach 1. The new simulated structures are manually adjusted rectangular weirs with a width of approximately two thirds of the canal bottom. This places an adjustable control structure at the tail end of each reach. 100% of the turnouts were shut off for 2 hours during the simulation. There is only a slight difference at the upper 2 control structures, but a difference of 4 cfs in flow variation and 1 inch in water surface variation at the new control structure in Branch 2 Reach 1. However, the decreased water surface fluctuation in that reach is offset by the increased fluctuation in the reach below.

Consequences of only shutting of the non-continuous turnouts for 2 hours is illustrated in figures 34(a) and (b). The 3 additional control structures appear to have little effect on fluctuation. The downstream check also illustrates the same properties as before, decreasing fluctuation in the upstream reach while increasing fluctuation below the check.



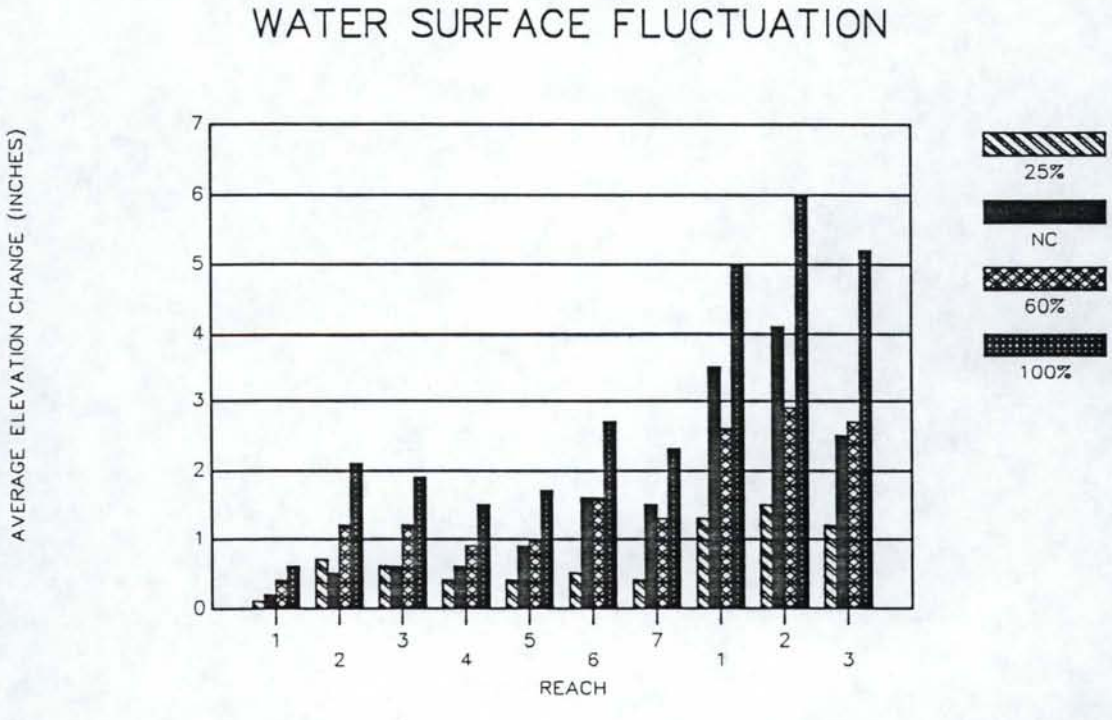
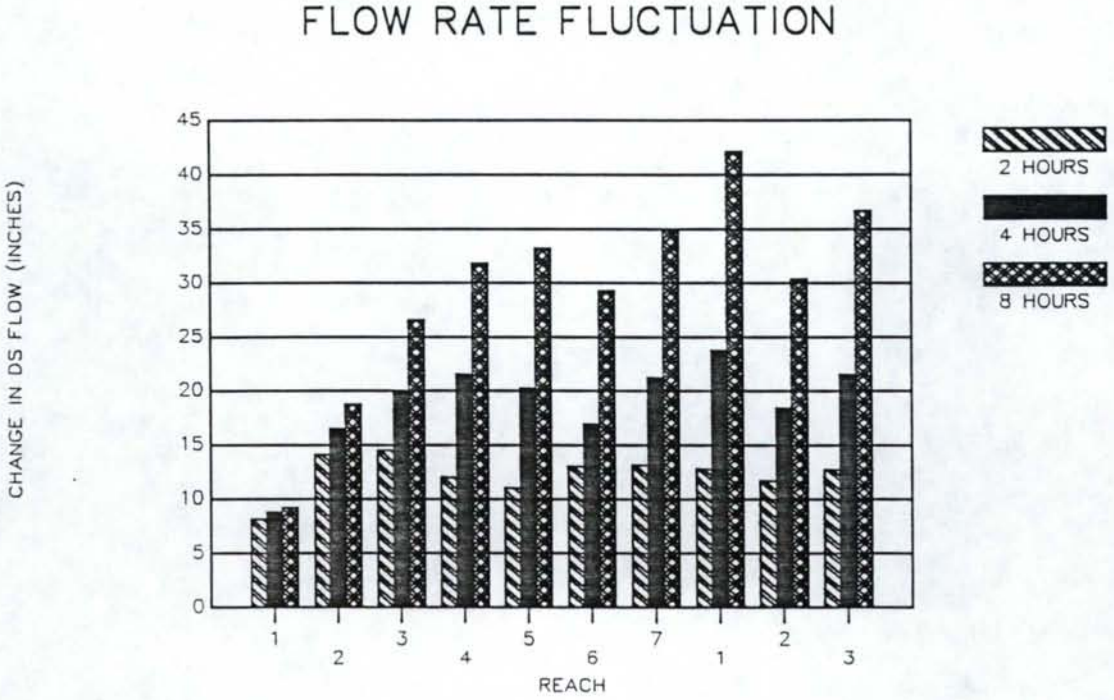


Figure 31(a) and (b). Low Line flow rate and water surface fluctuation from 2 hour turnout shutoff.



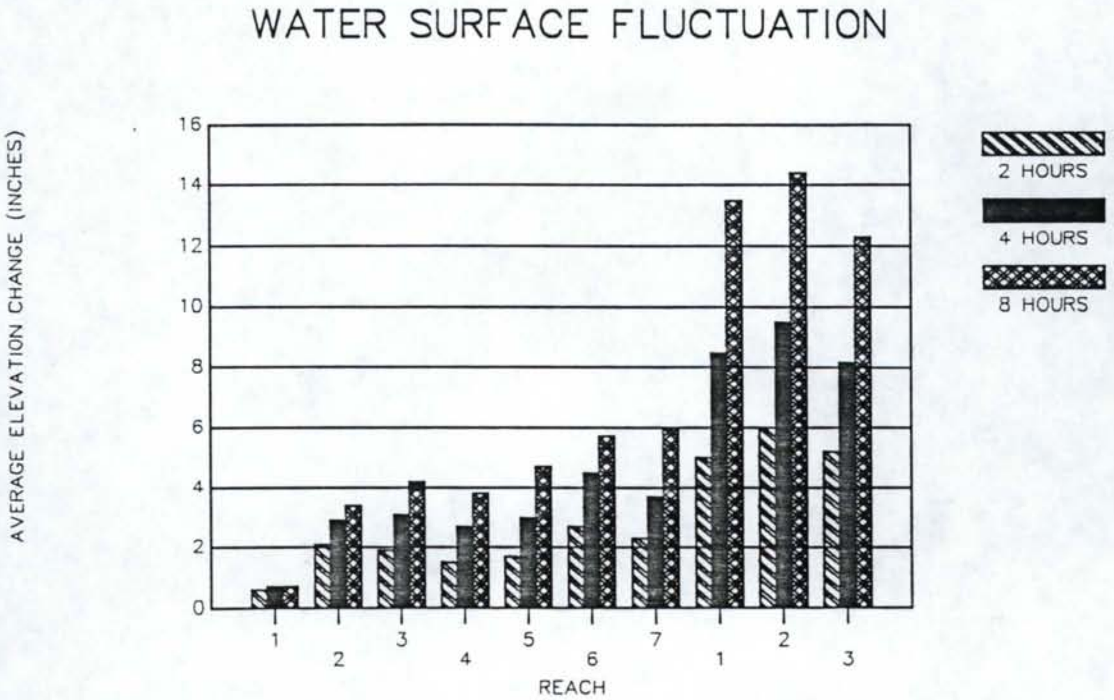
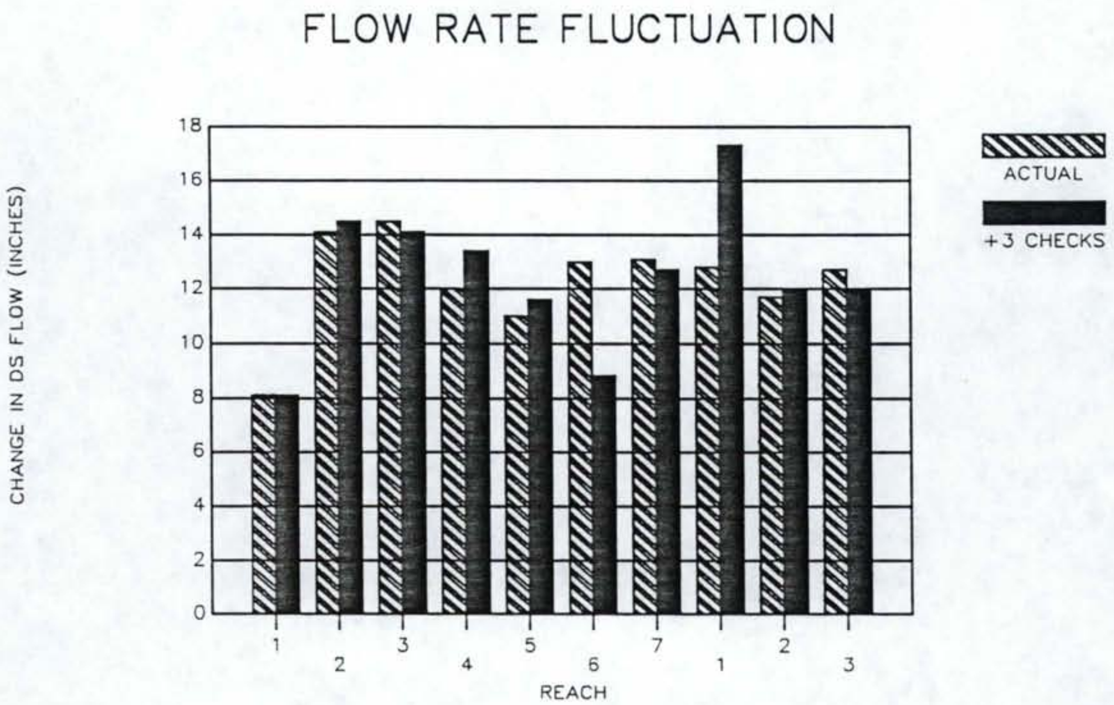


Figure 32(a) and (b). Low Line flow rate and water surface fluctuation with increasing shutoff time for 100% turnout shutoff.



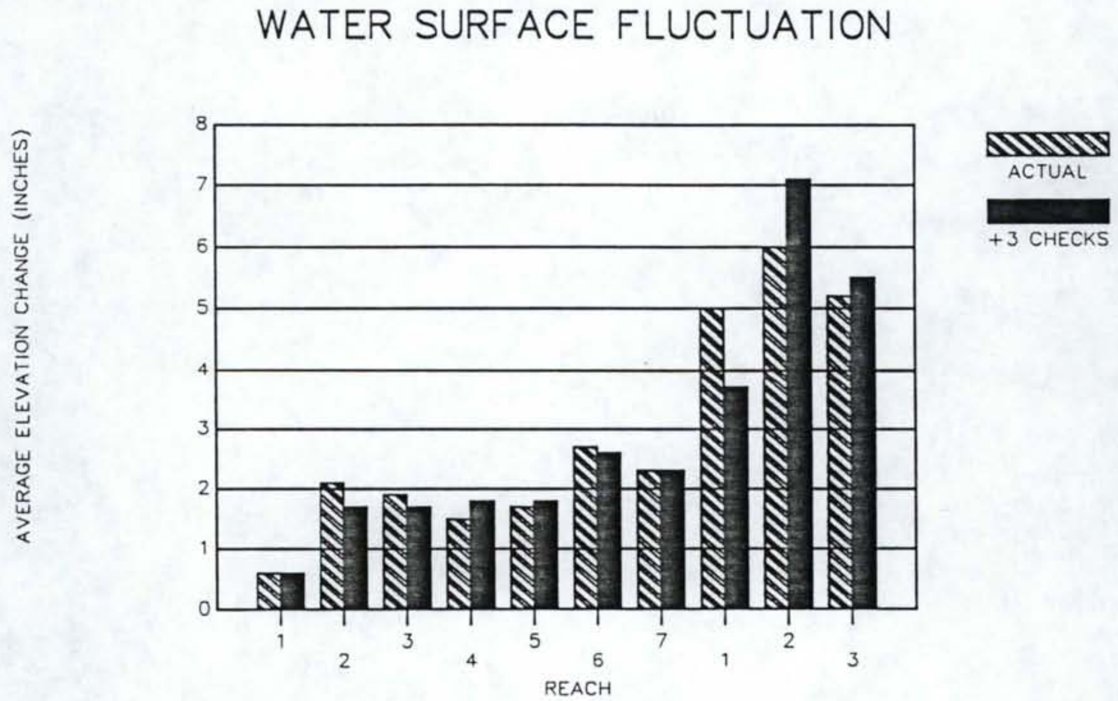
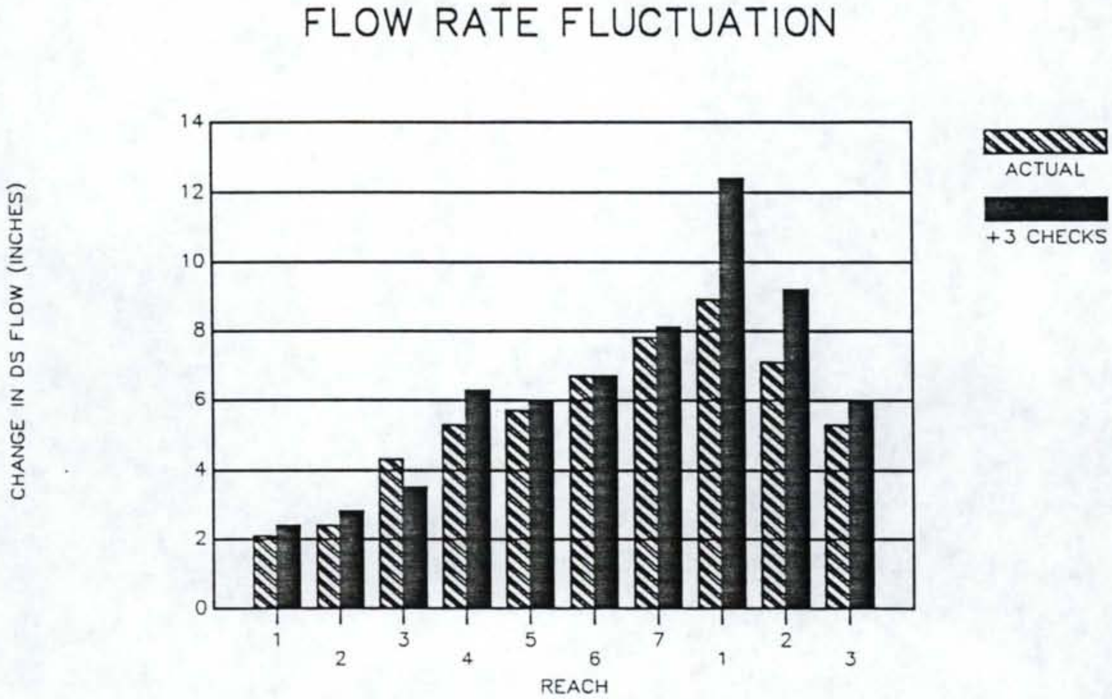


Figure 33(a) and (b). Low Line flow rate and water surface fluctuation with 3 additional control structures and 100% turnouts shut off for 2 hours. "*" indicates reach with new control structure.



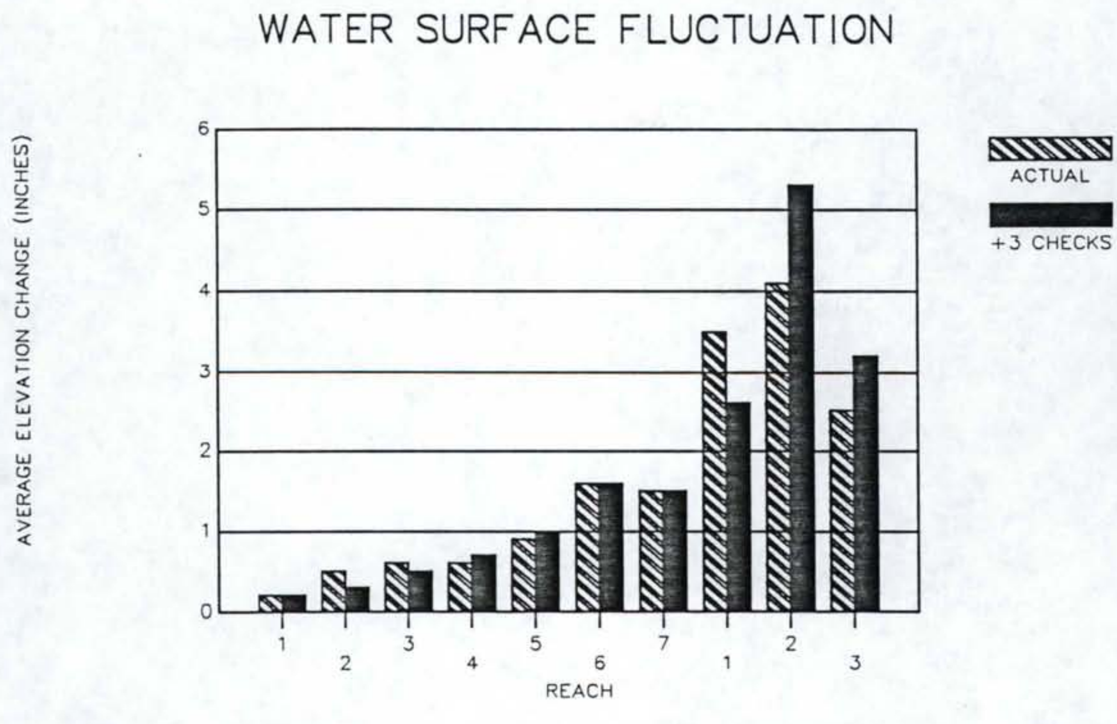


Figure 34(a) and (b). Low Line flow rate and water surface fluctuation with 3 additional control structures and all non-continuous turnouts shut off for 2 hours. "*" indicates reach with new control structure.

(E) C LATERAL

The configuration data files for C Lateral appear in Appendix B. Figure 35 shows the Reach configuration used in the simulations. Turnout flow rates from July 7, 1990, were used in the simulation and are shown in table 5. The turnouts labeled "NC" under "Outlet" are currently non-continuous turnouts, while those labeled "C" are continuous flow turnouts. All the control structures on C Lateral are manually adjusted weirs, and the simulations were ran using manual control mode. There is a rectangular weir spillway at the downstream end of the lateral. Flow variation leaving Reach 8 is flow over this spillway.

Water is diverted into C Lateral from the Main Line through a circular orifice 4 feet in diameter. Flow rate is controlled by an adjustable gate. No measurement of flow rate is possible, but it is estimated based on the experience of the ditchrider. Inflow used in the simulations was 27.3 cfs, based on 23.7 cfs for turnouts, 1.2 cfs for the spill, and 2.4 cfs for seepage.

Figures 36(a) and (b) show the effects on flow rate and water surface variation of shutting off various quantities of turnouts. The turnouts were shut off for 2 hours. Levels of non-continuous flow evaluated were 100%, 60%, 25%, and the turnouts which currently operate non-continuously.

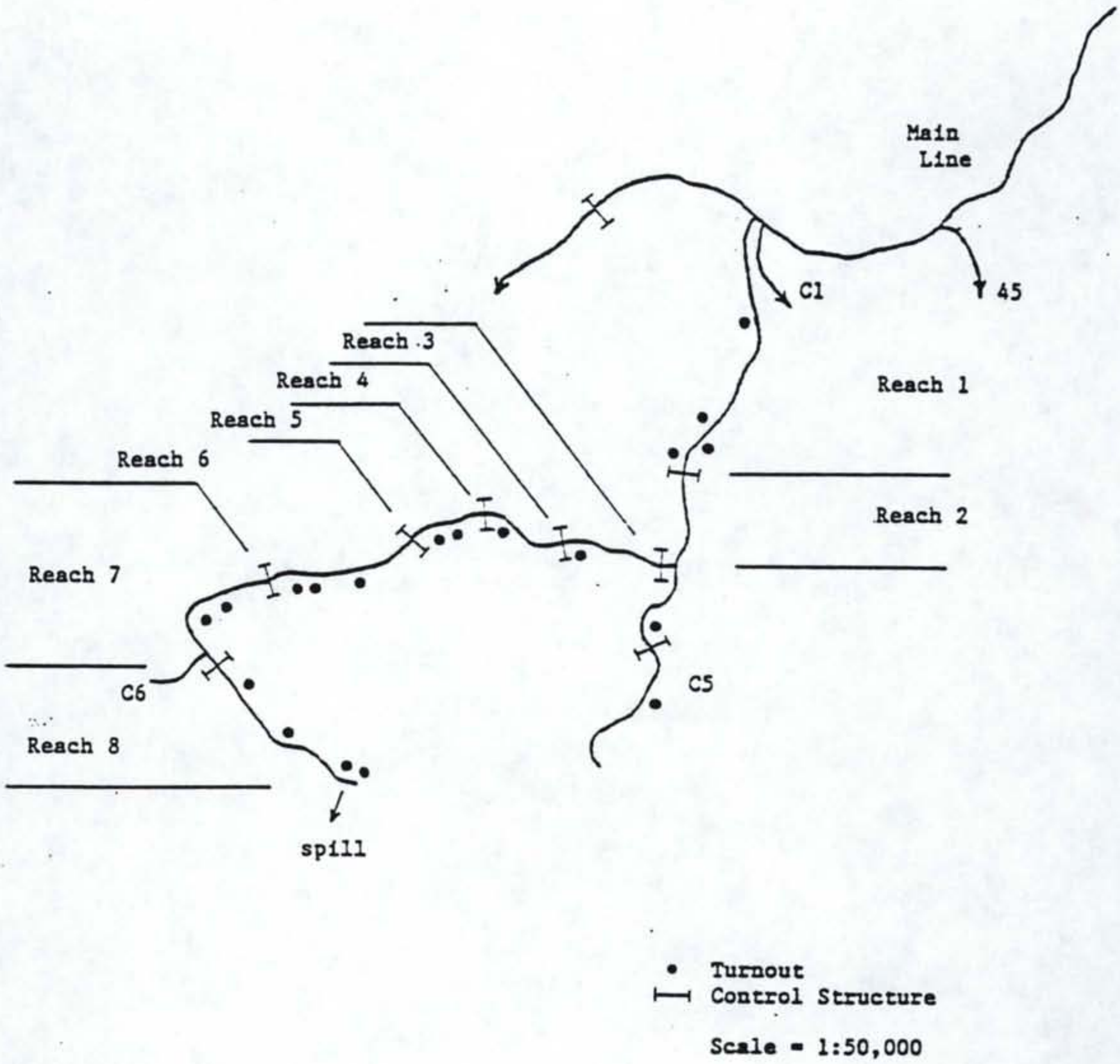


Figure 35. Reach configuration used in C Lateral simulations.

	Turnout	Flow Rate (cfs)	Outlet
Reach 1	C 0-5	1.5	C
	C 0-6	0.0	C
	C 1-5	0.0	C
	C 1-7	0.0	C
Reach 2	C5 LATERAL	0.5	C
Reach 3	C 1-14	3.6	NC*
Reach 4	C 2-1	1.2	NC
Reach 5	C 2-5	0.9	NC
	C 2-4	3.5	NC*
Reach 6	C 2-7	0.0	NC
	C 2-8	0.0	NC
	C 3-4	0.0	NC
Reach 7	C 3-5	0.6	C
	C 3-8	0.0	C
	C6 LATERAL	2.2	C
Reach 8	C 4-1	2.3	NC*
	C 4-3	1.5	C
	C 4-8	4.5	C
	SPILL	1.9	
	C 4-10	0.4	C

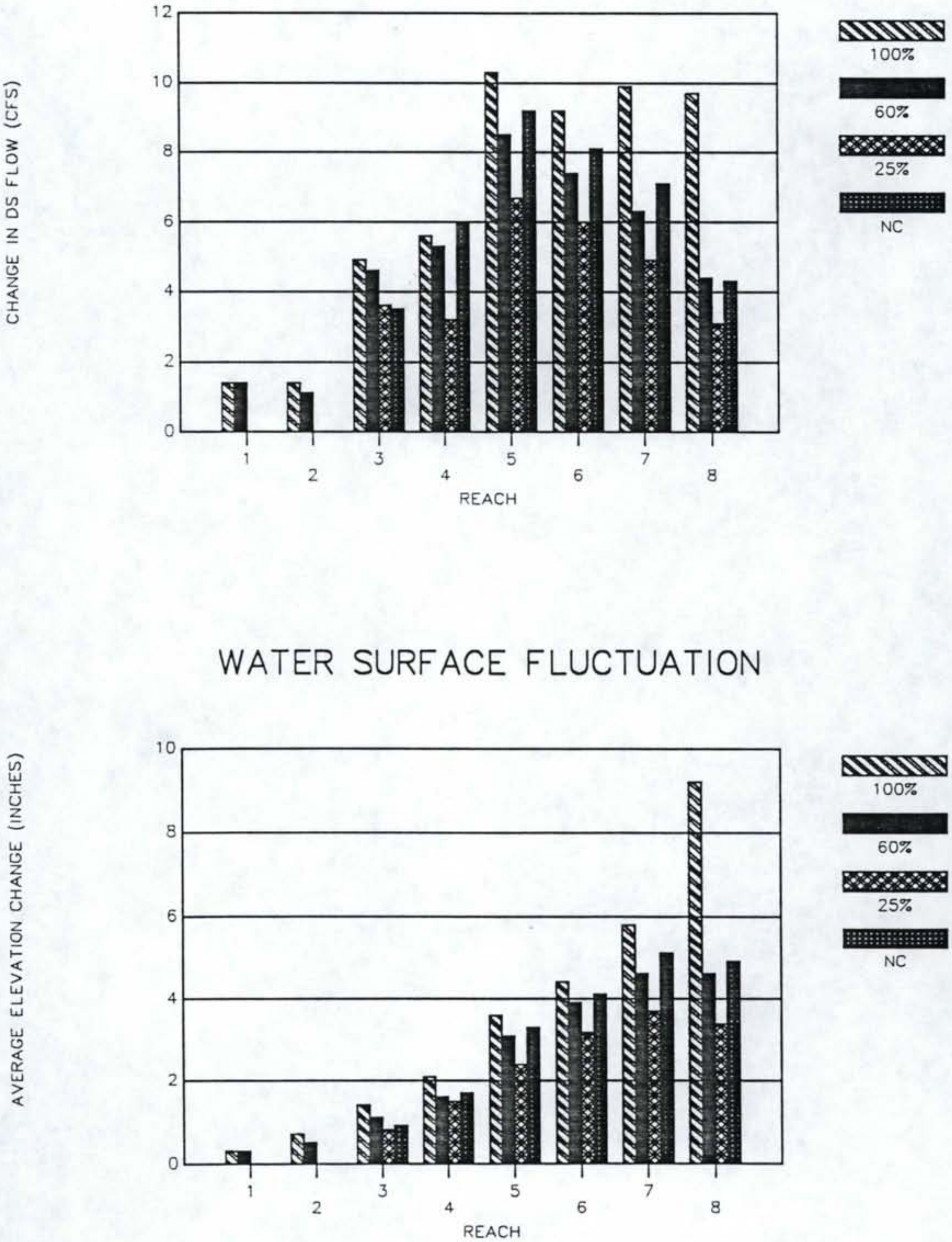
Table 5. Turnout flow rates used in C Lateral simulations. "*" indicates turnouts shut off for 25% non-continuous simulations.

Approximately 42% of the turnouts on C Lateral operate non-continuously. On July 7, 1990, these turnouts accounted for nearly half the total turnout flow. Flow rate and water surface fluctuation for the actual non-continuous turnouts and 60% of all the turnouts being shut off are nearly equal over most of C Lateral.

Figures 37(a) and (b) show the effects of shutting off 25% of the turnouts for 2, 4, and 8 hour periods. Note that except for Reach 8, maximum fluctuation is nearly reached within 2 hours. Water surface variation in this reach is over 5 inches for shutoff times greater than 2 hours.

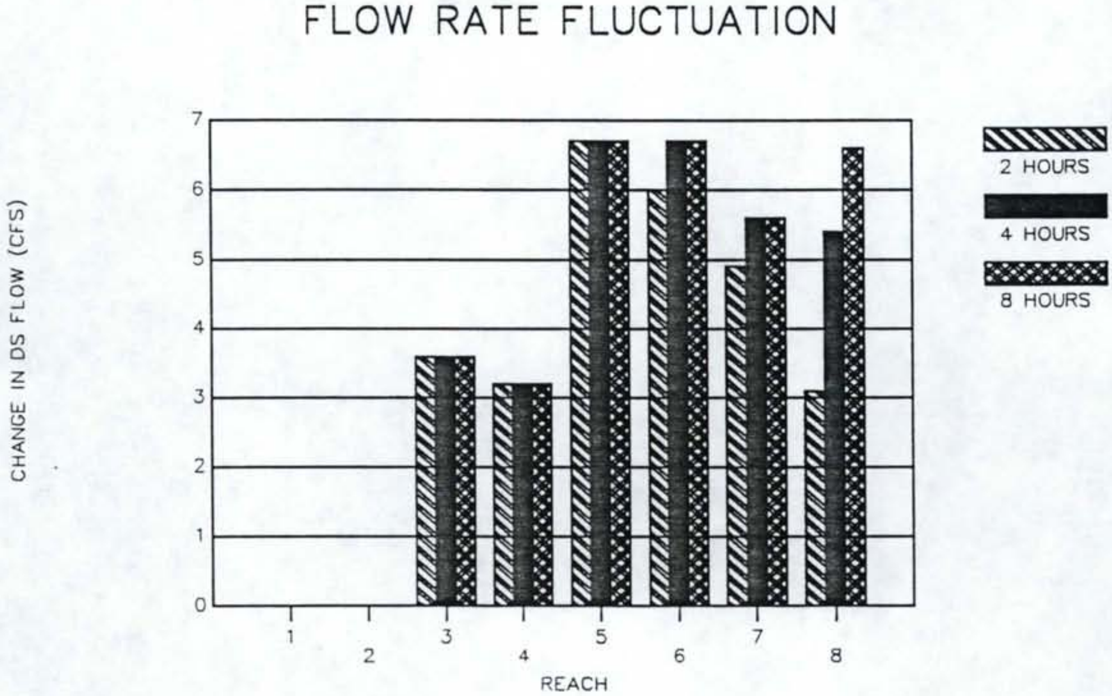
Figures 38(a) and (b) show the effects of some basic canal alterations on the fluctuations. 100% of the turnouts were shut off for 2 hours in these simulations. Widening the base of the lateral 7 feet does decrease the variations in the lower reaches, but these variations are still substantial, over 6 inches in Reach 8. Widening the spillway at the end of Reach 8 has no effect on the upstream reaches, but decreases water surface fluctuation in Reach 8 from 9 to 7 inches.

FLOW RATE FLUCTUATION



WATER SURFACE FLUCTUATION

Figure 36(a) and (b). C Lateral flow rate and water surface fluctuation from 2 hour turnout shutoff.



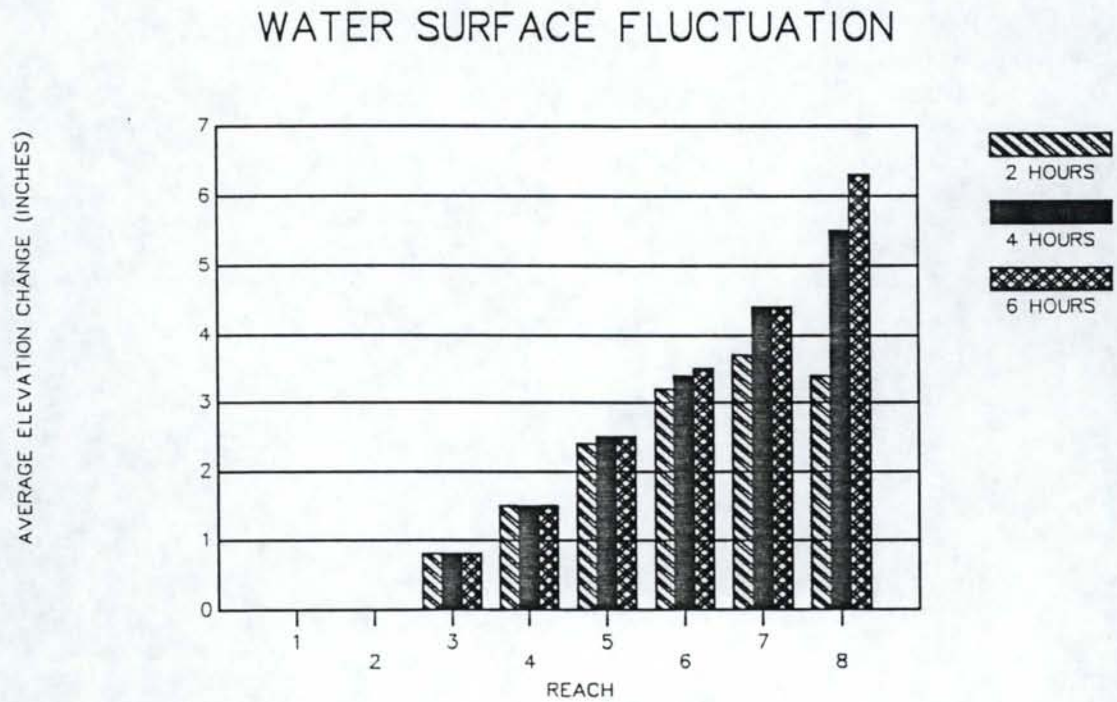
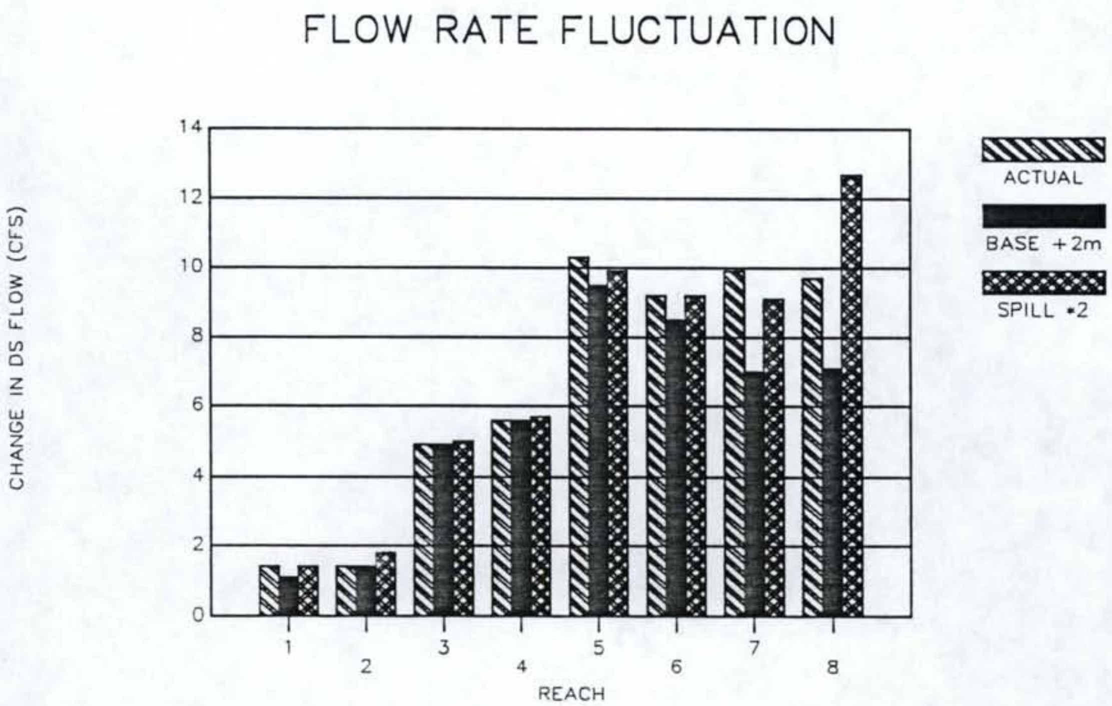


Figure 37(a) and (b). C Lateral flow rate and water surface fluctuation with increasing shutoff time for 25% turnout shutoff



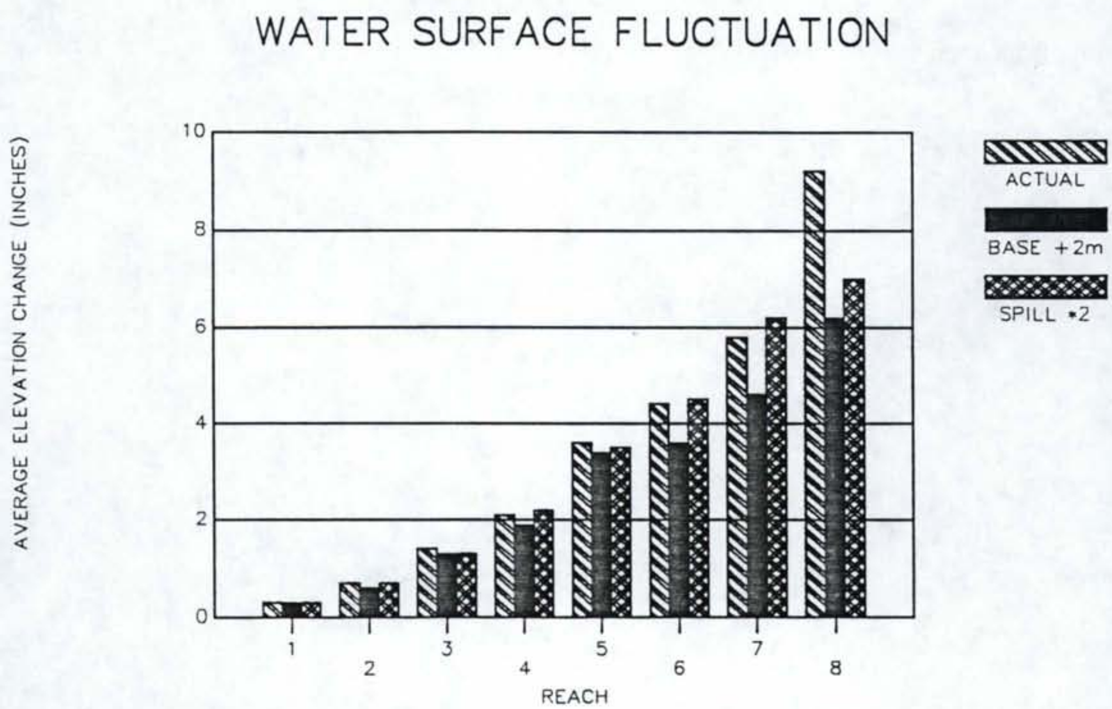


Figure 38(a) and (b). C Lateral flow rate and water surface fluctuation with increased base width and with larger spillway. 2 hour, 100% turnout shutoff.

(F) <u>C1 Lateral</u>

The configuration files for C1 Lateral appear in Appendix B. Figure 39 illustrates the reach configuration used in the simulations. The control structures on C1 Lateral are all manually adjusted weirs, and the simulations were run using manual control mode. There is a control structure at the downstream end of every lateral except the last one, which has a rectangular weir spillway near the end. Flow rate variation in Reach 7 is over this spill. Turnout flow rates from July 7, 1990, used in the simulation, are shown in table 6. Flow into C1 Lateral from the Main Line is through a 4 foot diameter circular orifice. There is no measuring device to determine the flow rate, so it is estimated based on ditchrider experience. Flow rate used for the simulations was 25.2 cfs, based on 20.3 cfs for turnouts, 2.5 cfs for the spill, and 2.4 cfs for seepage.

Figures 40(a) and (b) illustrate the predicted fluctuation in C1 Lateral caused by shutting off turnouts for 2 hours. 100%, 60%, 25%, and the current non-continuous turnouts were shut off simultaneously for 2 hours. Approximately 40% of the turnouts on C1 Lateral operate non-continuously. On July 7, 1990, these turnouts accounted for 39% of the total turnout flow. Fluctuation increases as the percentage of turnouts operated non-continuously increases. Fluctuation also increases in the downstream reaches.

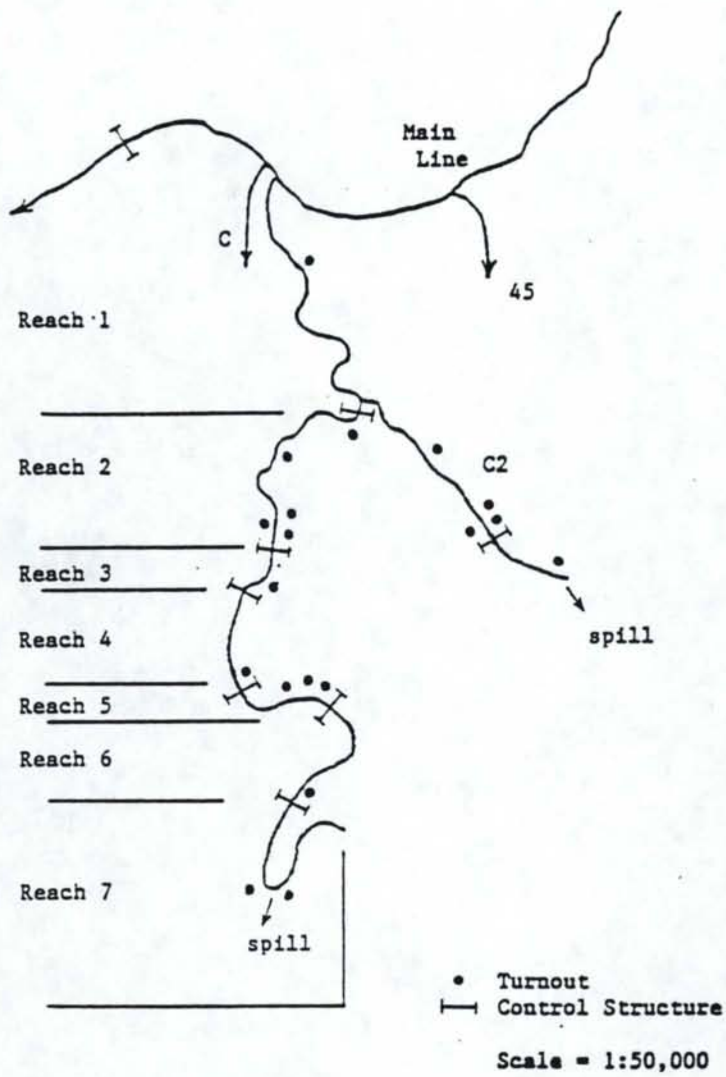


Figure 39. Reach configurations used for C1 Lateral simulations.

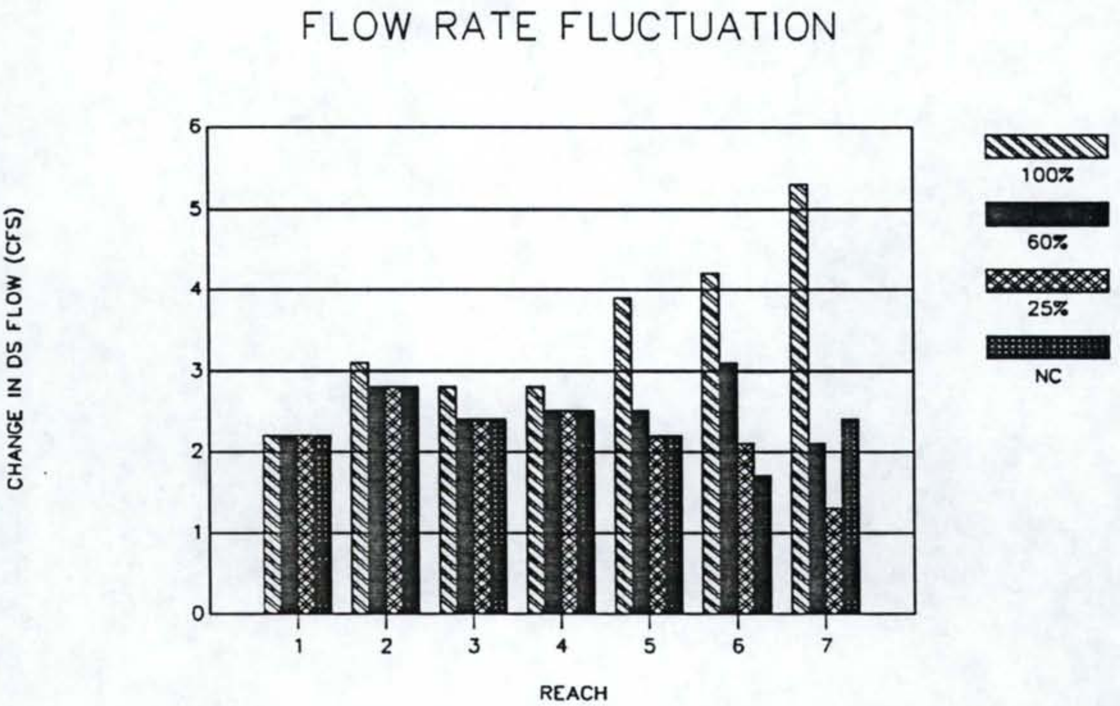
Figures 41(a) and (b) illustrate the effects of shutting 25% of the turnouts off for periods of 2, 4, and 8 hours. Fluctuation in flow rate and water surface increases as the turnouts are shut off for longer periods of time. Fluctuation in water surface elevation is greater in the lower canal reaches.

The effect on the fluctuations of increasing the width of the canal base and of doubling the spill width is shown in figures 42(a) and (b). 100% of the turnouts were shut off for 2 hours in the simulations. The base width was increased 7 feet in

	Turnout	Flow Rate (cfs)	Outlet
Reach 1	C1 0-3	2.6	NC*+
	C2 LATERAL	7.2	C
Reach 2	C1 1-5	0.9	NC*+
	C1 1-7	0.4	C
	C1 2-1	0.0	NC
	C1 2-2	1.1	NC*+
	C1 2-3	0.0	C
Reach 3	C1 2-6	0.0	C
Reach 4	C1 2-9	0.0	C
Reach 5	C1 3-1	0.0	C
	C1 3-2	0.0	NC
	C1 3-3	2.4	C+
Reach 6	C1 3-8	1.4	C
Reach 7	C3 LATERAL	1.0	C+
	SPILL	2.5	C
	C1 4-1	3.3	NC

Table 6. Turnout flow rates used in C1 Lateral simulations. "*" indicates turnouts shut off for 25% non-continuous simulations, and "+" indicates turnouts shut off for 60%.

all reaches (original base width listed in configuration files - appendix). Flow rate fluctuation decreased an average 17%, but water level fluctuation decreased only an average 0.7 inches. Increasing spill width had no effect except in the reach containing the spill, where the water surface fluctuation was decreased by approximately 2 inches. Figures 43(a) and (b) show the effects of changing weir control structures to orifice control structures (sluice gates). 25% of the turnouts were shut off for 2 hours during the simulations. The control structure at Reach 2 was replaced by 2 rectangular orifices with the same dimensions for one simulation. A simulation was also ran replacing the control structures at Reach 2 and Reach 3 with rectangular orifices, again with the same dimensions as the actual weir structures. The orifice control structures do not pass the increased flow rate fluctuation downstream as efficiently as the weir control structures. The result is increased water surface fluctuation in the reaches with an orifice control structure at the downstream end, but decreased fluctuation in reaches further downstream.



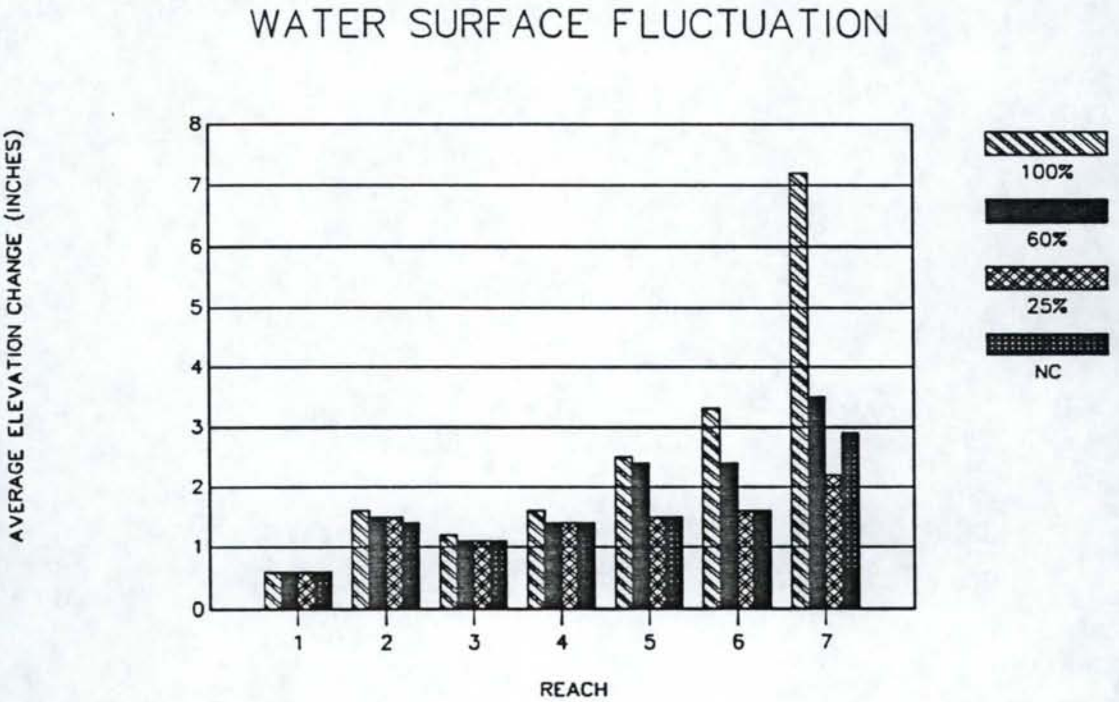
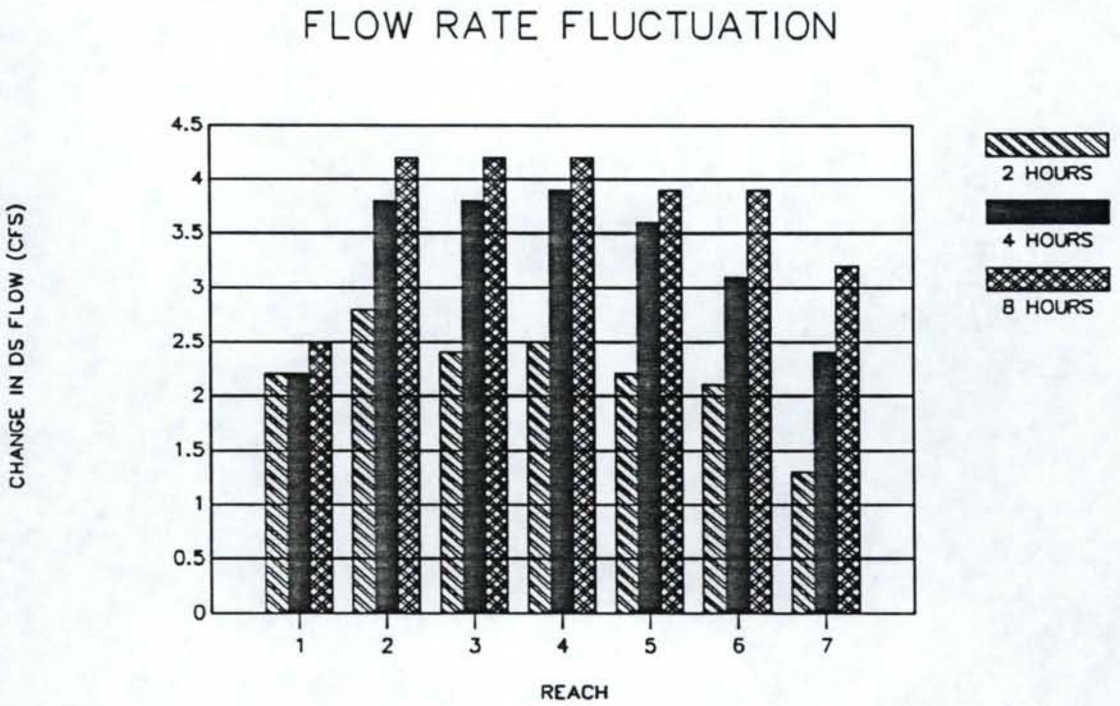


Figure 40(a) and (b). C1 Lateral flow rate and water surface fluctuation from 2 hour turnout shutoff.



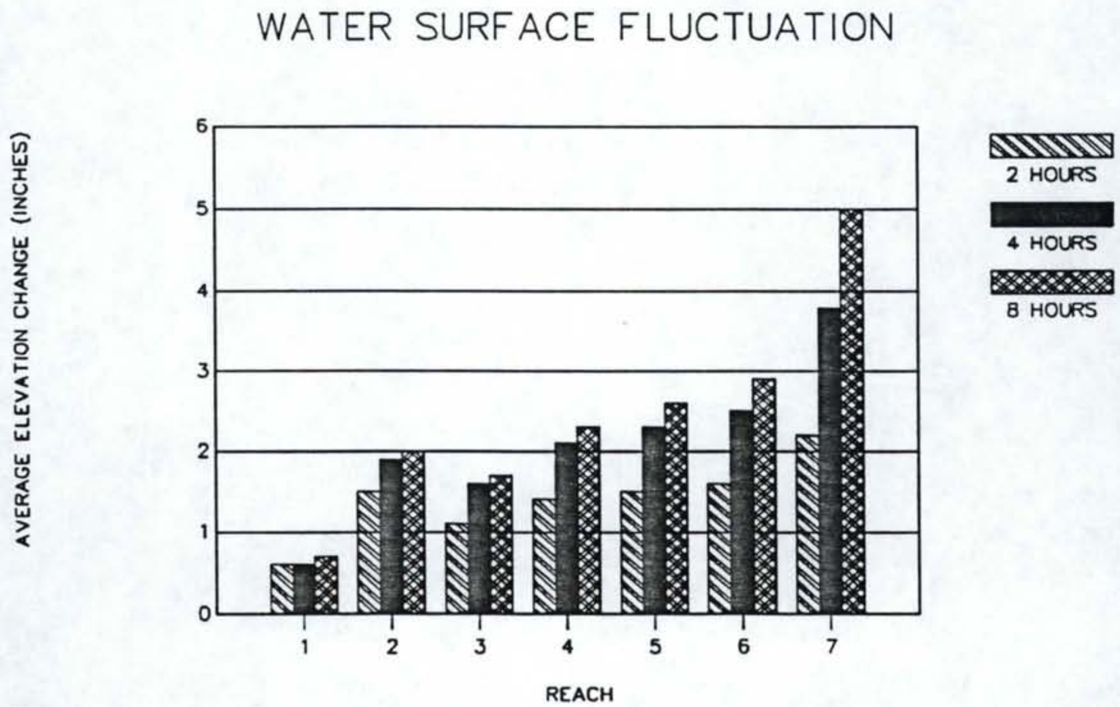
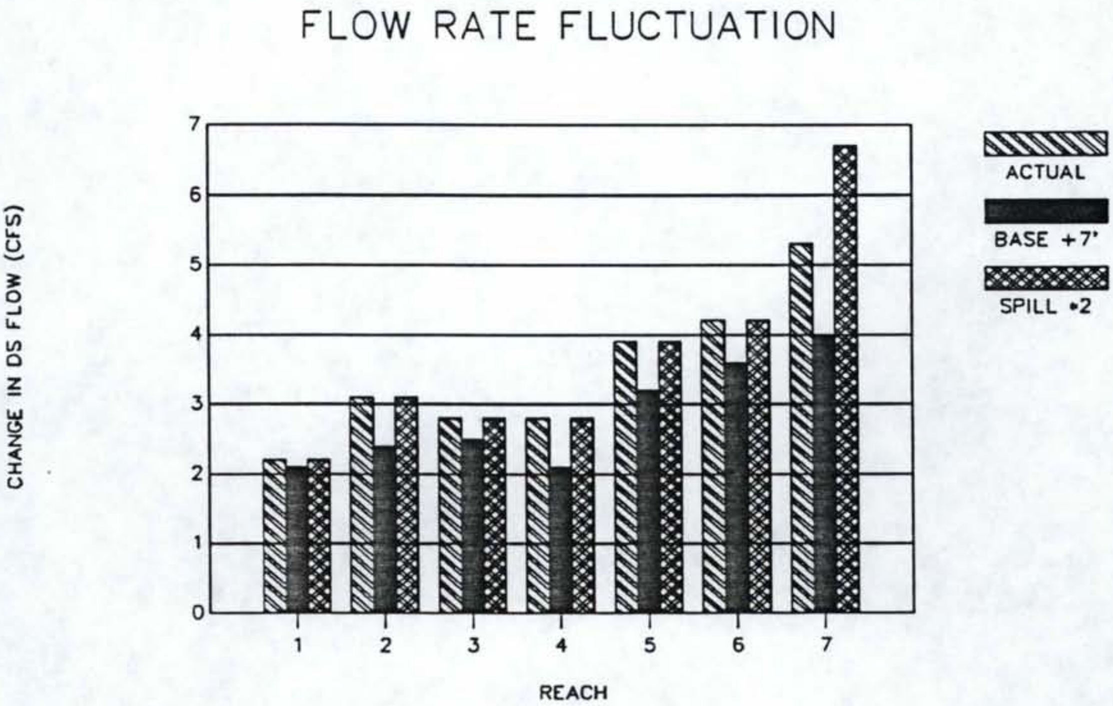


Figure 41(a) and (b). C1 Lateral flow rate and water surface fluctuation with increasing shutoff time for 25% turnout shutoff.



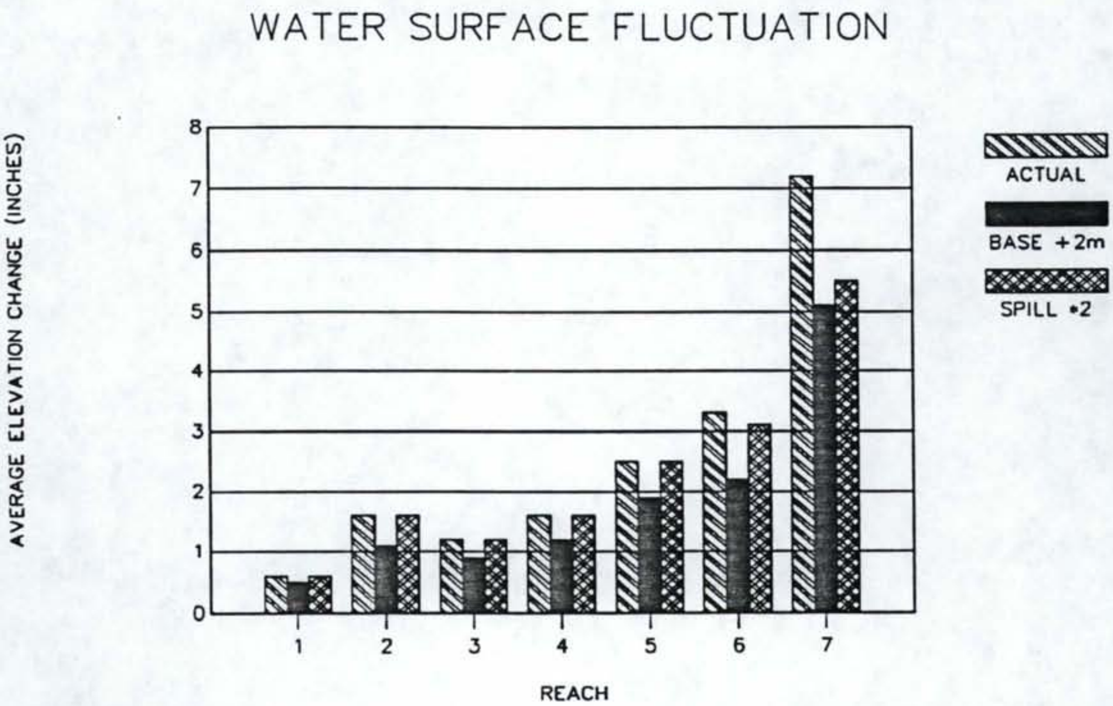
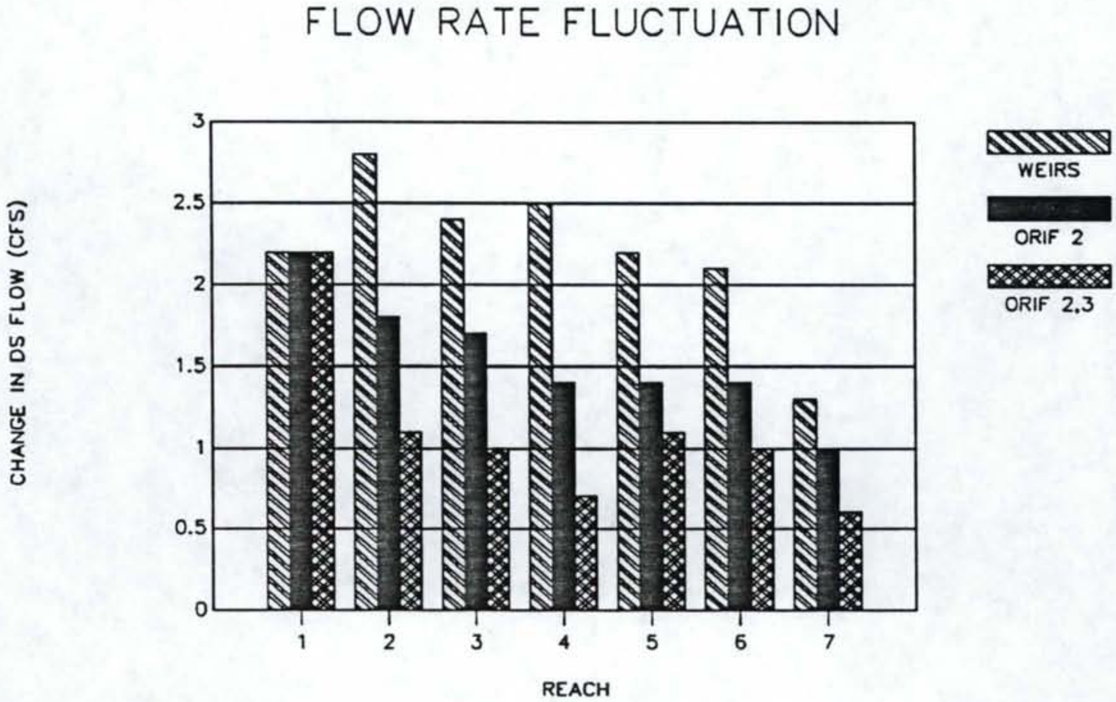


Figure 42(a) and (b). C1 Lateral flow rate and water surface fluctuation with increased base width and with larger spillway. 2 hour, 100% turnout shutoff.



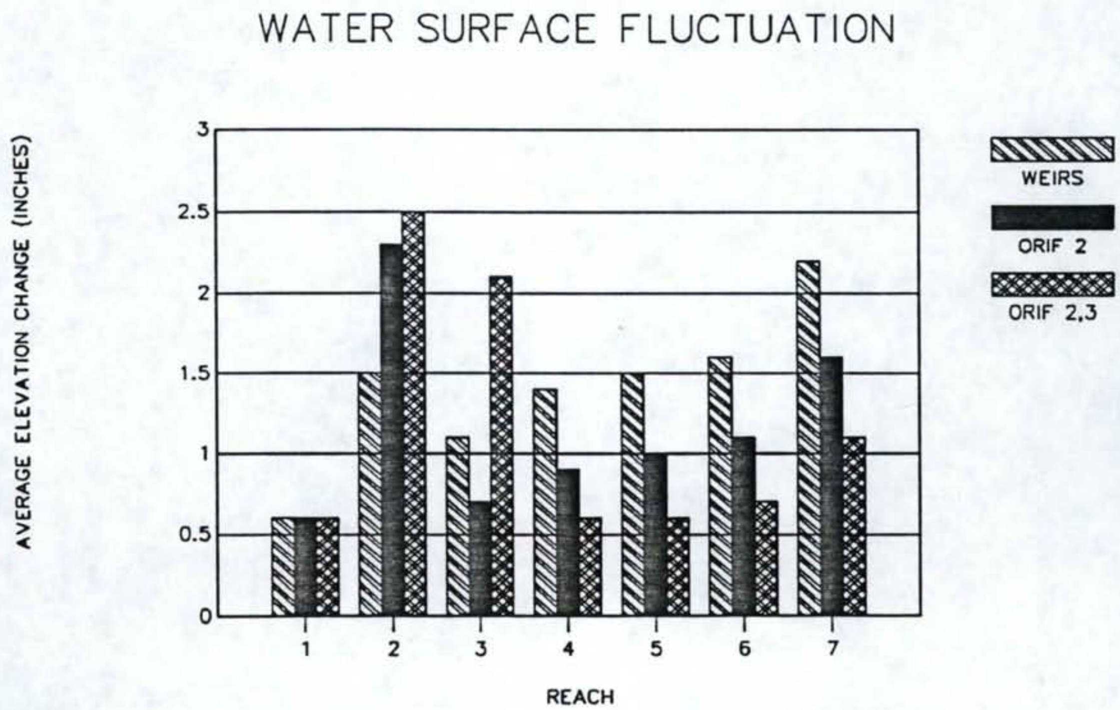


Figure 43(a) and (b). C1 Lateral flow rate and water surface fluctuation with orifice control structures at Reach 2 and at Reach 2 and 3. 2 hour, 25% turnout shutoff.

(G) E Lateral

The configuration data files for E Lateral appear in Appendix B. Figure 44 shows the branch and reach configuration used in the simulations. There are manually adjustable rectangular weirs at the downstream end of the first 3 reaches in Branch 1 (E Lateral). There is an adjustable rectangular weir spillway in the last reach to remove excess flow. Flow variation in this reach is over the spill. There is no spillway in Branch 2 (E1 Lateral), and excess flow must be stored in the canal. The turnout flows used in the simulation are shown in table 7. These values are not actual July 7, 1990 values but have been modified to reflect full lateral operation. Flow is diverted into E Lateral from the High Line through a circular orifice 4 feet in diameter. Actual flow rate is not measured, but estimates are made based on ditchrider experience. E Lateral inflow used in the simulations was 32.2 cfs, based on 26.8 cfs for turnouts, 1.0 cfs for the spill, and 4.4 cfs for seepage.

Predicted values for flow rate and water surface fluctuation resulting from various levels of turnouts shutting off is shown in figures 45(a) and (b). 100%, 25%, and the current non-continuous turnouts were shut off for 2 hours during the simulations. Water surface fluctuates in the downstream reach of E1 Lateral as much as 30 inches when 100% of the turnouts are shut off for 2 hours, since water entering the reach must be stored within the reach. The canal overflowed the banks in the simulation model when 100% of the turnouts were shut off.

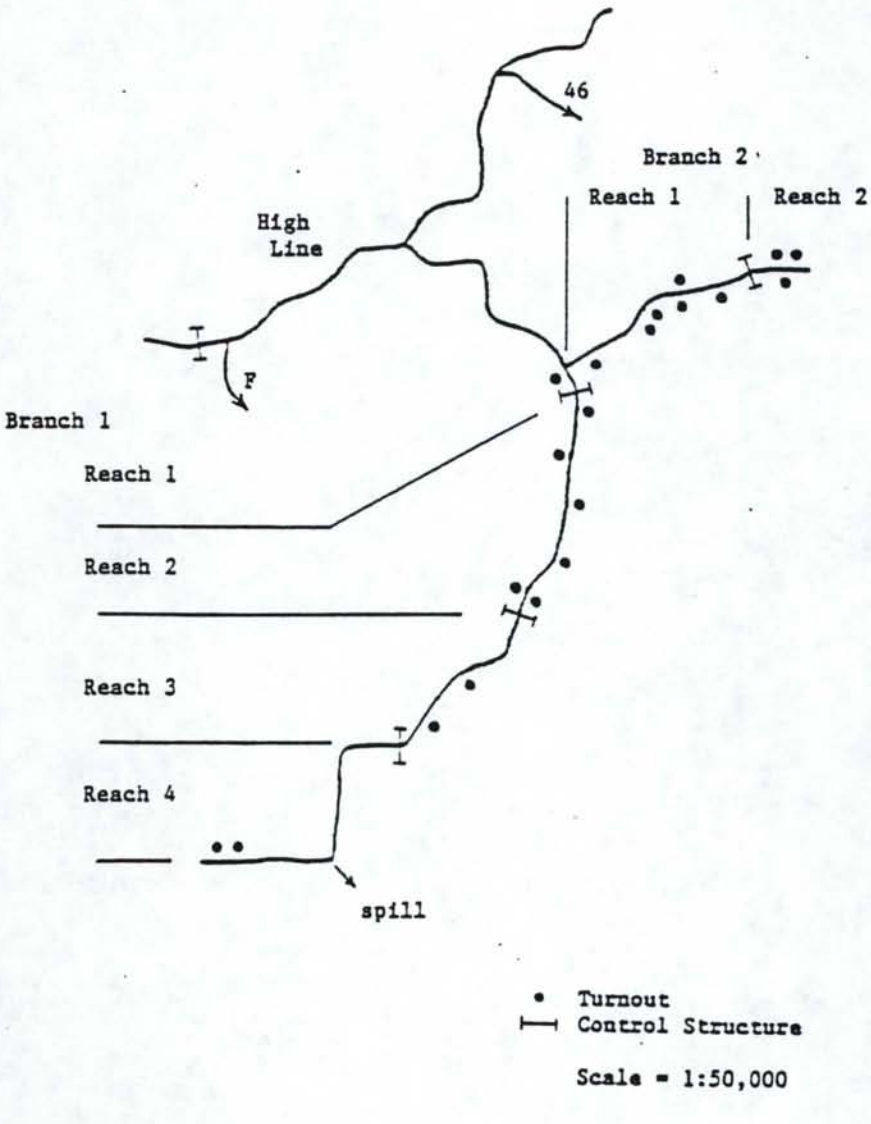


Figure 44. Reach configurations used for E Lateral simulations.

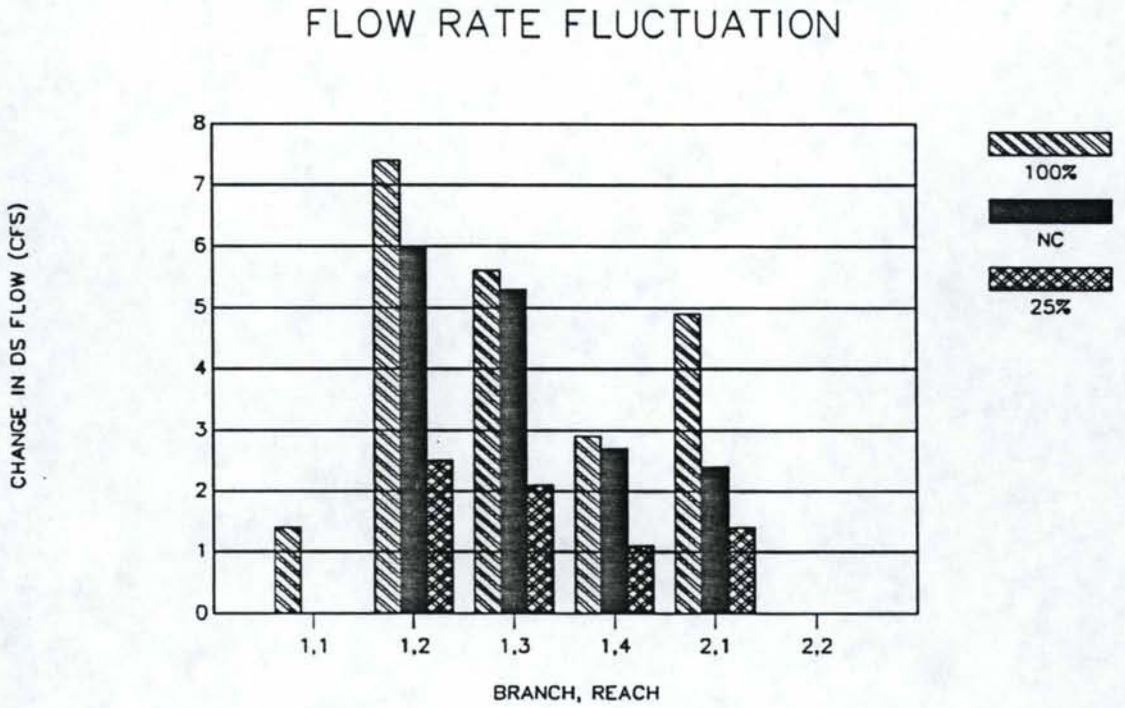
The effects of turning off 25% of the turnout flows for different time periods is shown in figures 46(a) and (b). Fluctuation increases as the length of shutoff time increases. Fluctuation is also more severe in the downstream reaches of both branches.

		Turnout	Flow Rate (cfs)	Outlet
Branch 1				
	Reach 1	E1 LATERAL	12.4	C
		E 1-3	1.6	C
	Reach 2	E 1-6	0.4	C
		E 1-7	0.0	NC
		E 1-10	2.8	NC*
		E 1-11	2.0	NC
		E 1-16	0.0	C
		E 1-17	2.0	NC
	Reach 3	E 2-5	0.0	NC*
		E 2-6	1.6	NC
	Reach 4	SPILL	1.0	
		E 3-10	1.8	NC*
		E 3-15	2.2	NC
Branch 2				
	Reach 1	E1 0-1	0.0	C
		E1 0-2	2.0	NC*
		E1 0-6	1.3	NC
		E1 0-8	1.5	C
		E1 0-9	1.5	C
		E1 1-1	0.0	C
	Reach 2	E1 1-4	0.0	C
		E1 1-7	4.0	C
		E1 1-8	1.4	C

Table 7. Turnout flow rates used for E Lateral Simulations. "*" indicates turnouts shut off for 25% non-continuous simulations.

The effects of modifying and adding spillways to the lateral is shown in figures 47(a) and (b). The spillway in Branch 1 Reach 4 was doubled in size, from 4.4 feet to 8.8 feet. A 10 foot rectangular weir spillway was added at the downstream end of Branch 2 Reach 2. The changes in the spill capabilities have no effect except in the reaches where the spillways are located. In those reaches, outgoing flow rate is increased and the resulting water surface fluctuation is decreased.

The effects of increasing the base width of the laterals is shown in figures 48(a) and (b). Base width was increased by 7 feet in all reaches of the laterals, while 25% of the turnouts were shut off for 2 hours. Flow rate and water surface fluctuations show slight decreases with the additional canal capacity.



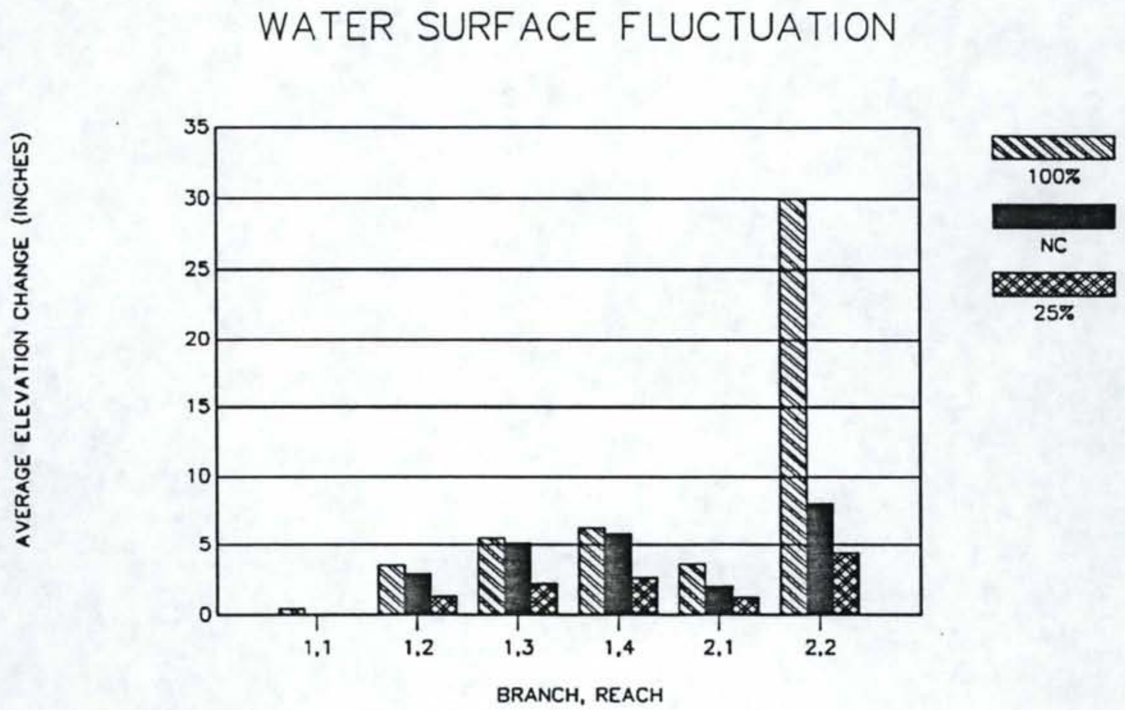
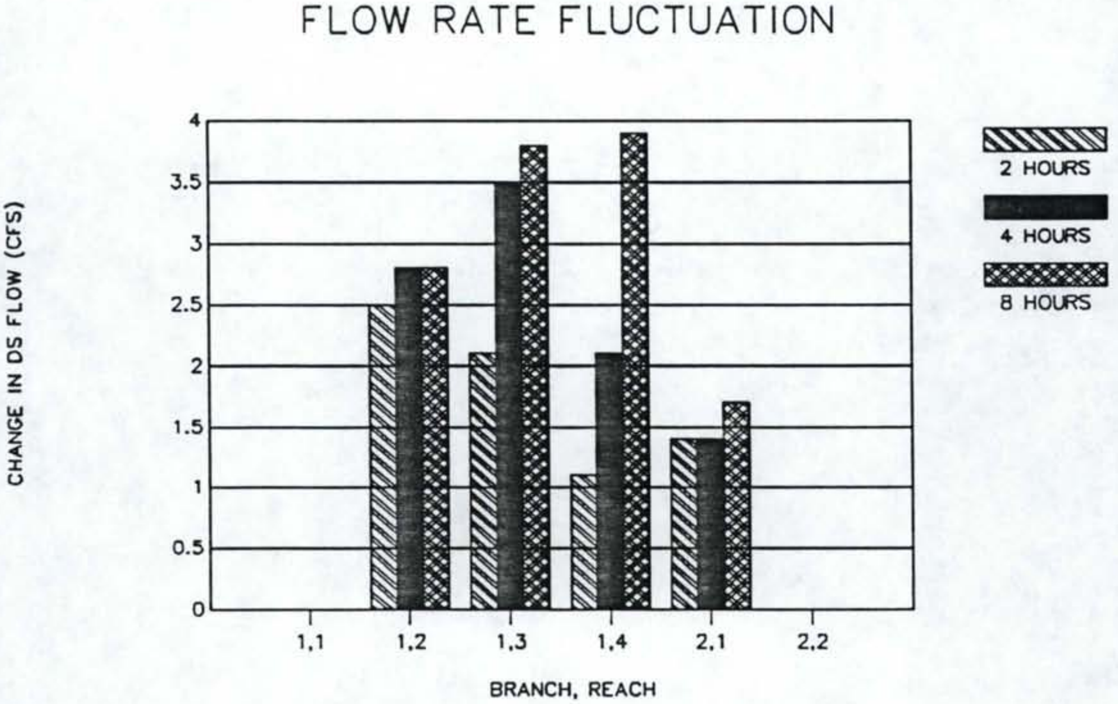


Figure 45(a) and (b). E Lateral flow rate and water surface fluctuation from 2 hour turnout shutoff.



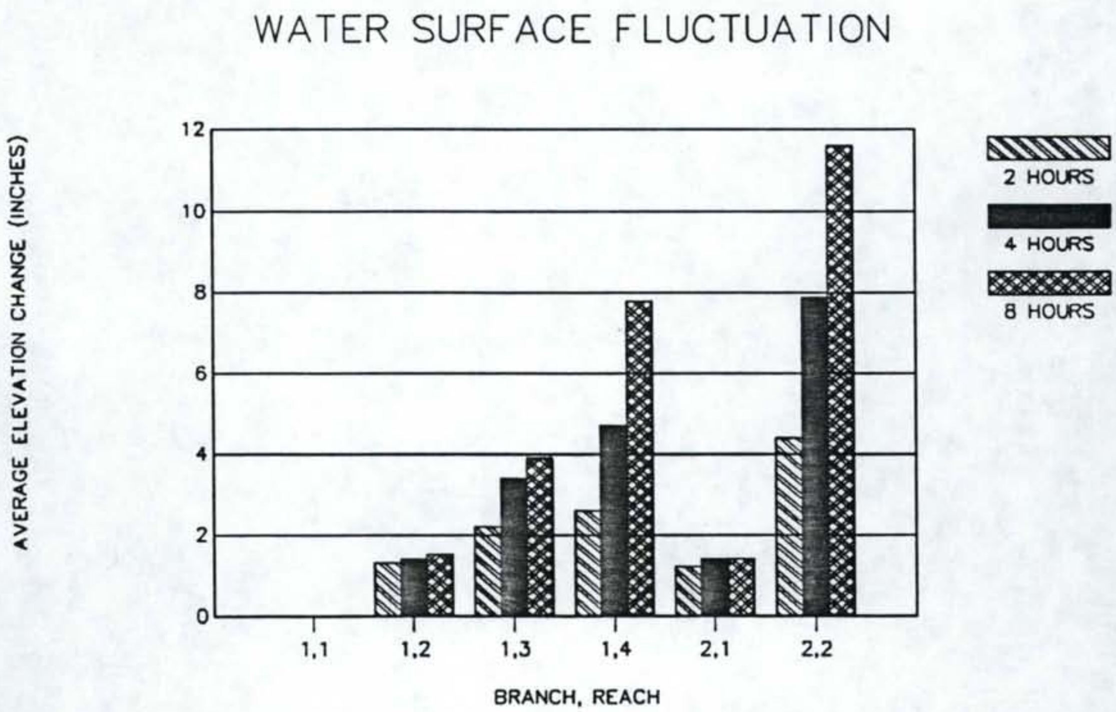
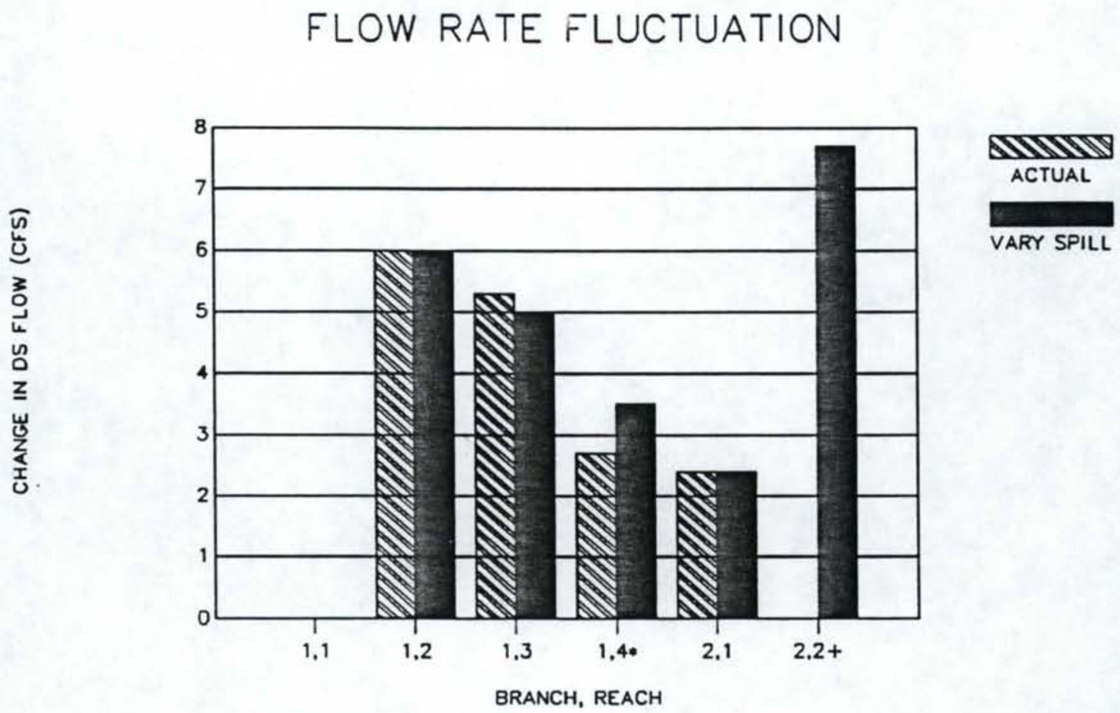


Figure 46(a) and (b). E Lateral flow rate and water surface fluctuation with increasing shutoff time for 25% turnout shutoff.



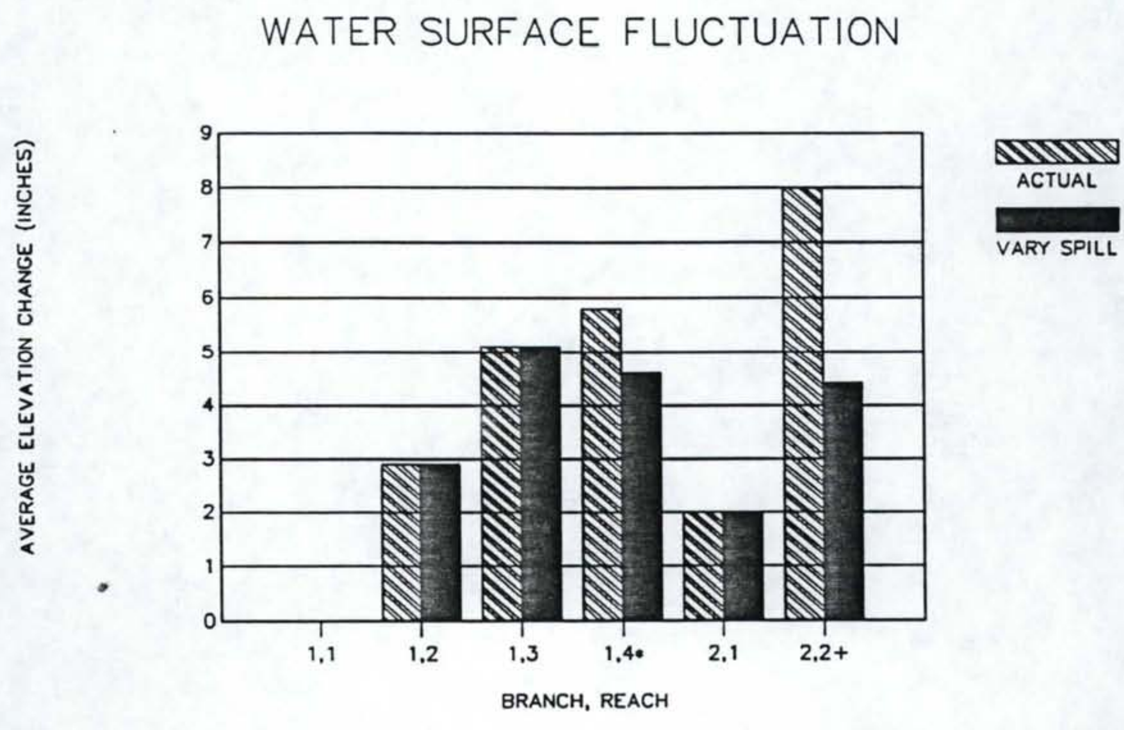
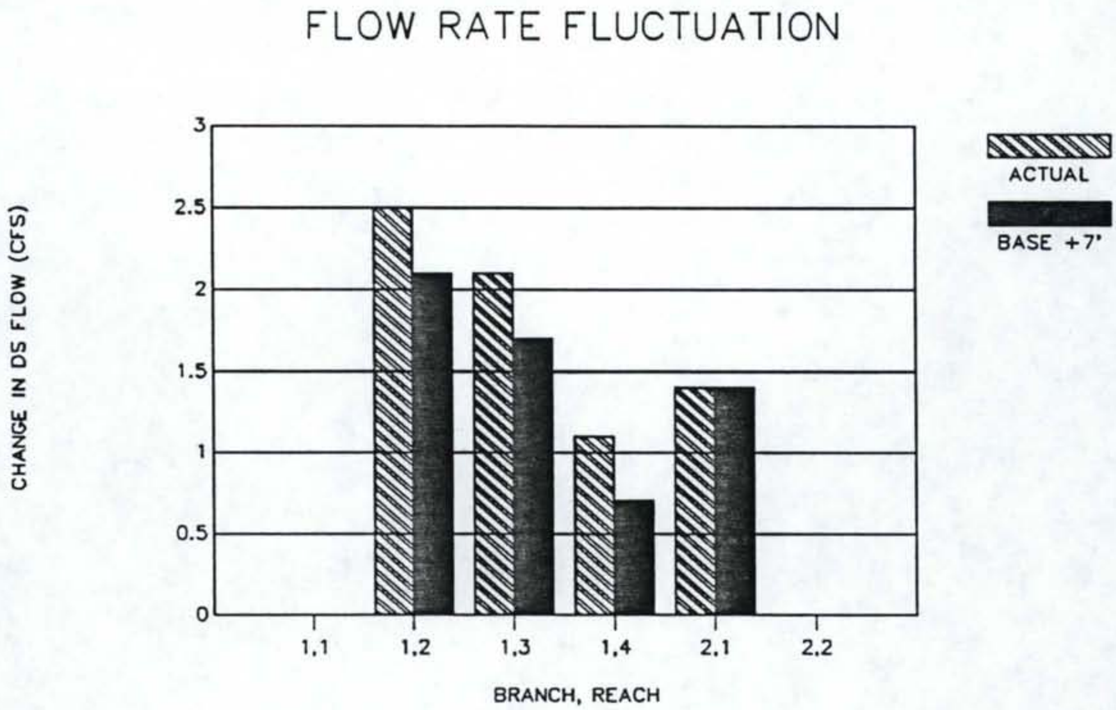


Figure 47(a) and (b). E Lateral flow rate and water surface fluctuation with larger spill in E Lateral (*) and added spill in E1 Lateral (+).



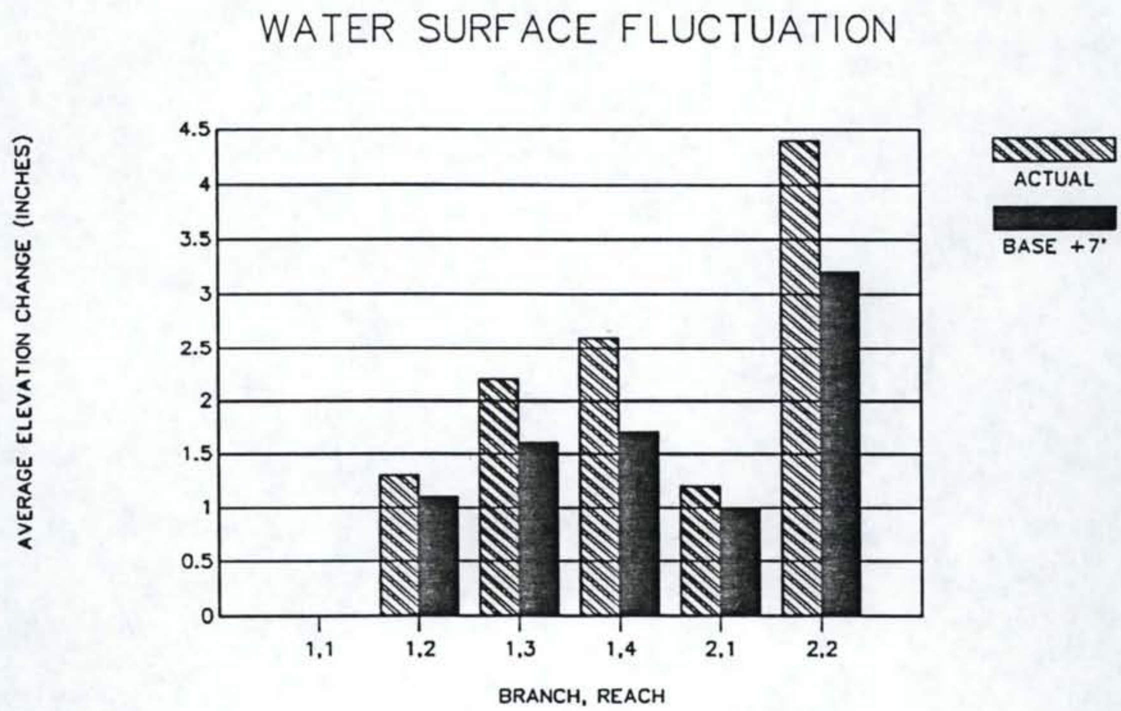


Figure 48(a) and (b). E Lateral flow rate and water surface fluctuation with increased base width. 2 hour, 25% turnout shutoff.

(H) G Lateral

The configuration files for G Lateral are located in Appendix B. Figure 49 shows the branch and reach configuration used in the simulations. The control structures on G Lateral are both manually adjusted rectangular weirs. There is a spillway at the downstream end of the third reach which removes excess flow. Flow variation in Reach 3 is over this spillway.

Inflow into G Lateral is controlled and measured through a box submerged orifice, like the standard turnouts on the system. Inflow used in the simulations was 7.0 cfs, 5.2 cfs for the

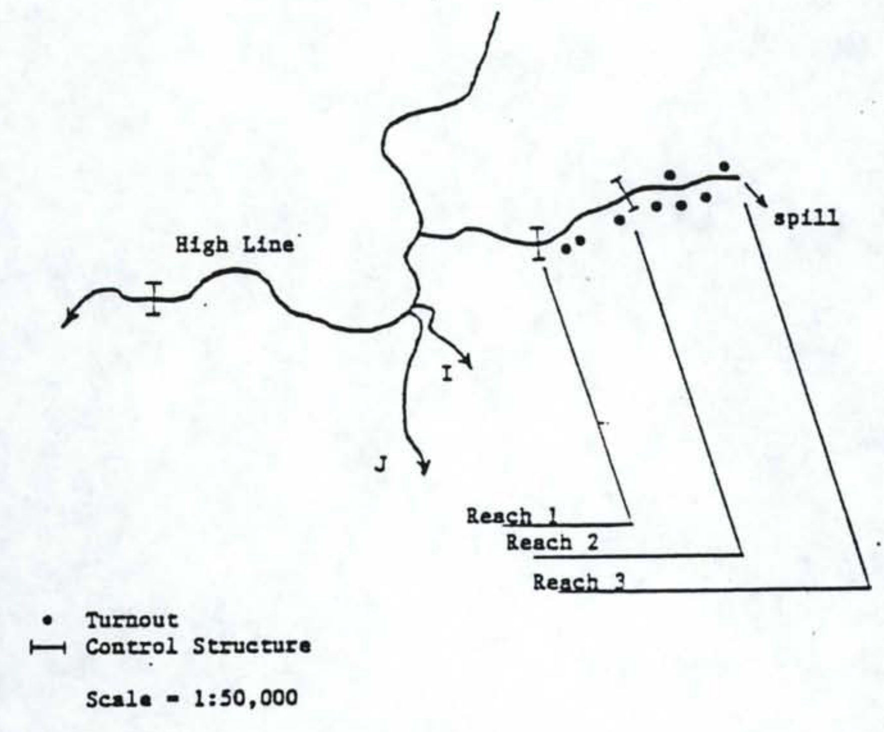


Figure 49. Reach configuration used in G Lateral simulations.

turnouts, 1.0 cfs for the spill, and 0.8 cfs for seepage. Outflow through the turnouts is listed in table 8. This is slightly modified from July 7, 1990 values in order to increase the number of active turnouts.

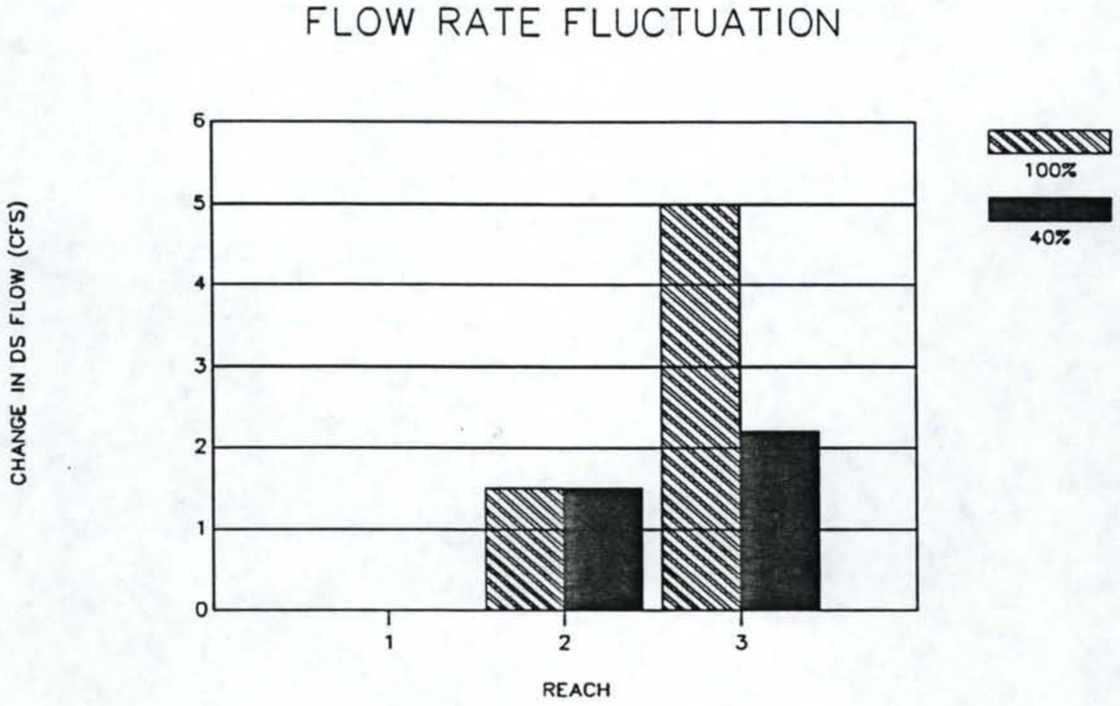
Shutting off the turnouts for 2 hours results in the fluctuations shown in figures 50(a) and (b). 100% and 40% of the turnouts were shut off. 40% of the turnouts is only the 2 turnouts noted in table 8. The total non-continuous flow which is rejected into the canal is only 2.6 cfs, but this is 37% of the total flow diverted into the lateral.

	Turnout	Flow Rate (cfs)	Outlet
Reach 1	NO TURNOUTS		
Reach 2	G 0-7	1.6	C*
	G 0-10	0.0	C
	G 0-10A	0.0	C
Reach 3	G 0-12	1.2	C
	G 1-7	1.0	C*
	G 1-8A	0.7	C
	G 1-9	0.0	C
	G 1-10	0.7	C
	SPILL	1.0	

Table 8. Turnout flow rates used in G Lateral simulations. "*" indicates turnouts shut off for 40% non-continuous simulations.

Figures 51(a) and (b) show the effects of shutting 40% of the turnouts off for increasing time periods. Flow fluctuation has stabilized by 2 hours. Water surface in the downstream reach continues to increase as the length of time the turnouts are shut off increases.

The effect of varying the width of the spillway in Reach 3 is shown in Figures 52(a) and (b). 40% of the turnouts were shut off for 2 hours. The actual spillway is a rectangular weir 6.8 feet wide. The width was doubled (13.6 feet) and cut in half (3.4 feet) during the simulations. The size of the spillway has no effect on the upstream reaches, but affects the amount of water spilled and the water surface fluctuation in Reach 3.



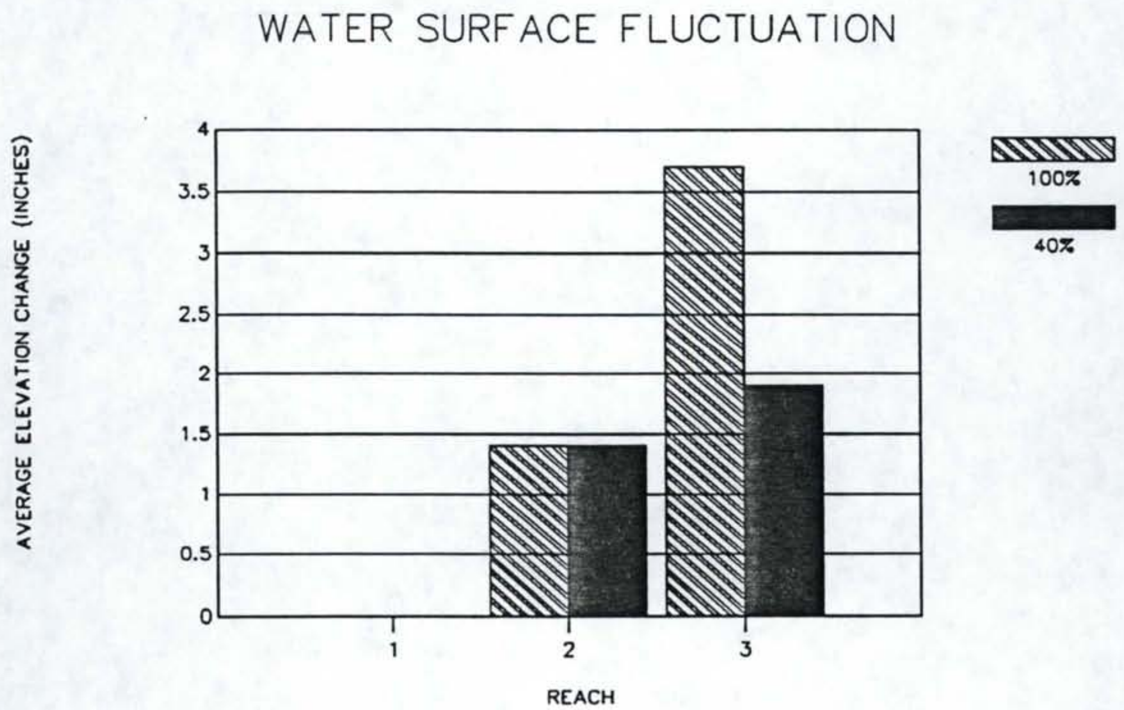
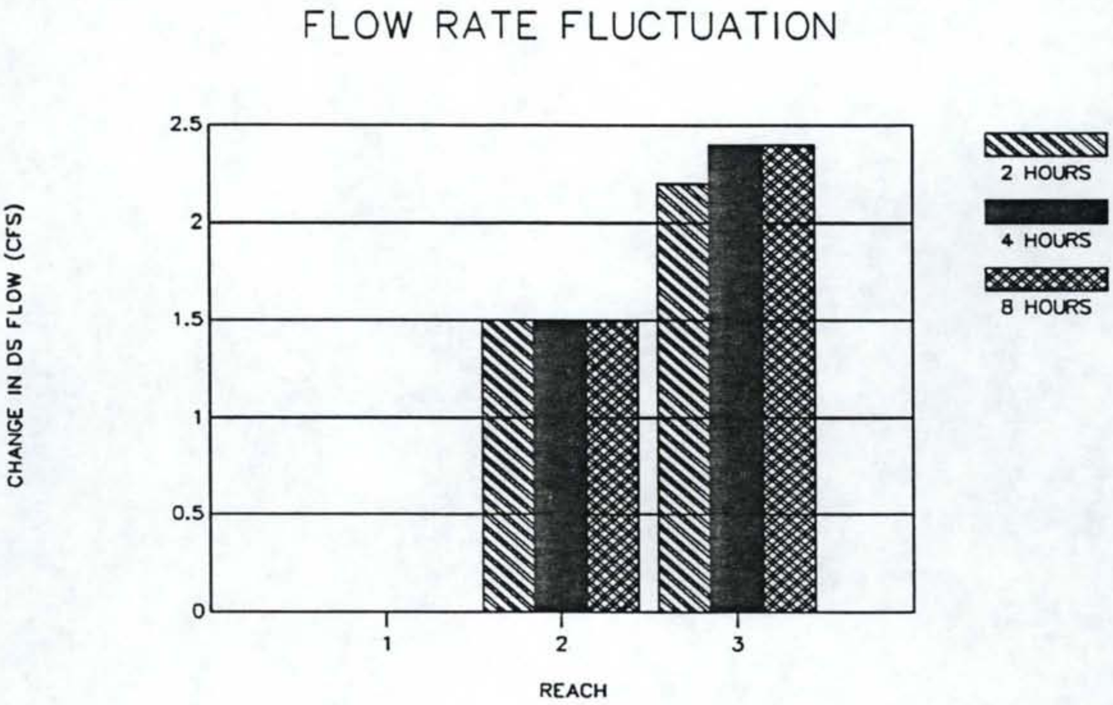


Figure 50(a) and (b). G Lateral flow rate and water surface fluctuation from 2 hour turnout shutoff.



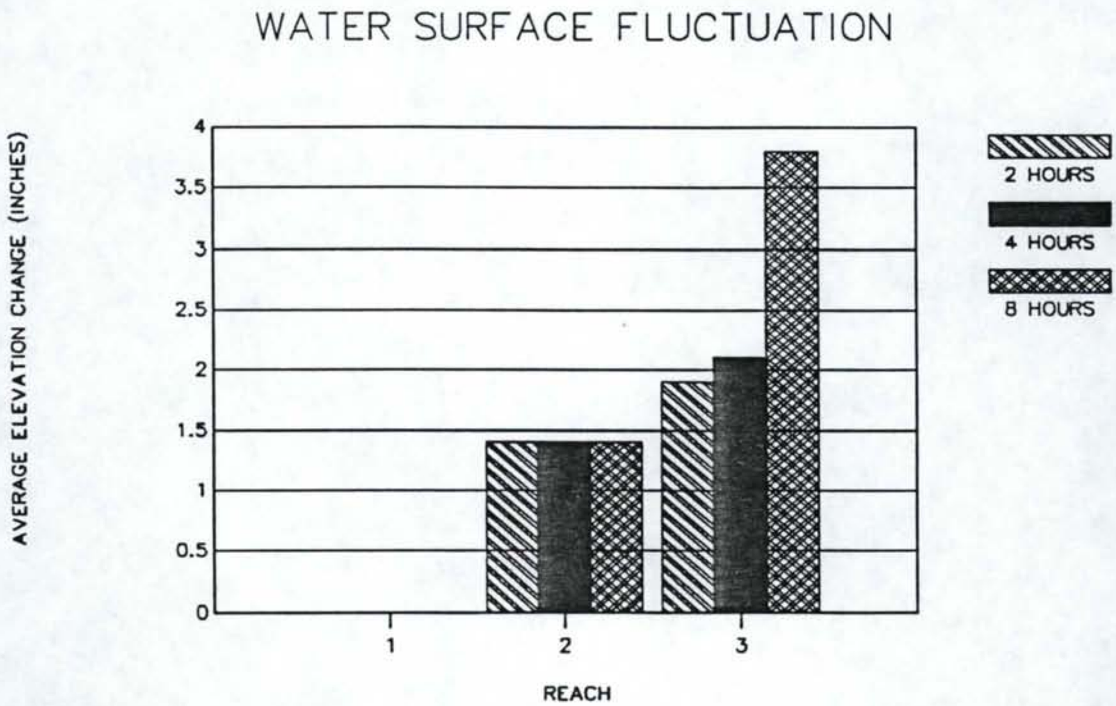
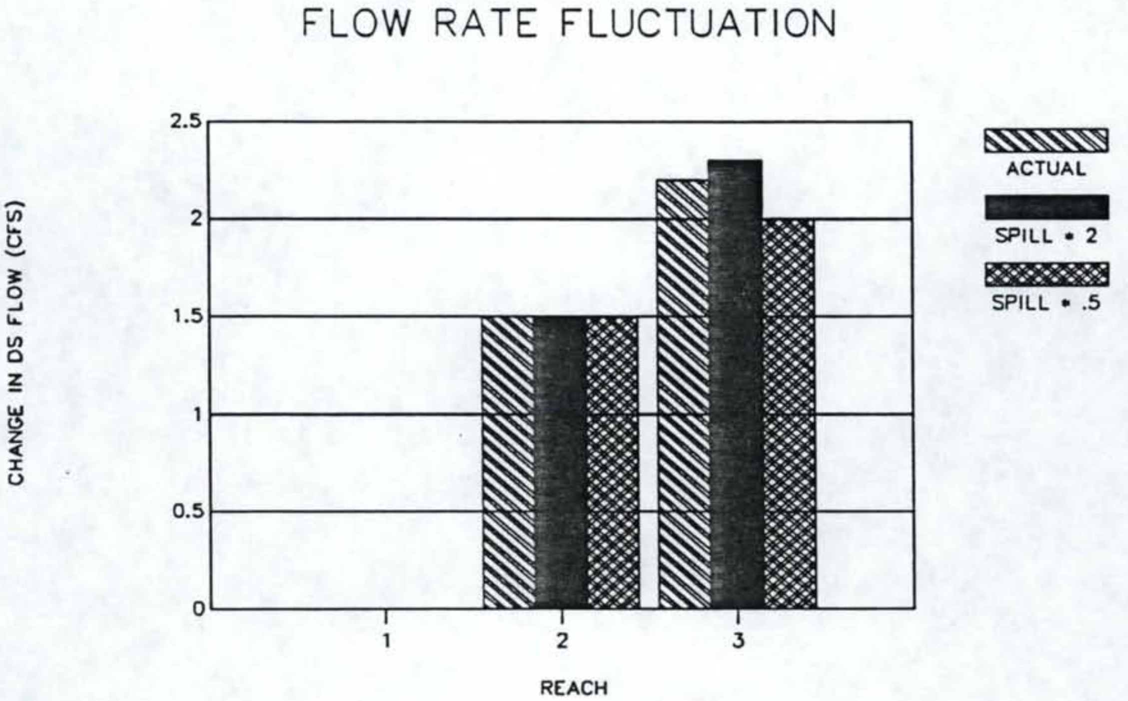


Figure 51(a) and (b). G Lateral flow rate and water surface fluctuation with increasing shutoff time for 40% turnout shutoff.



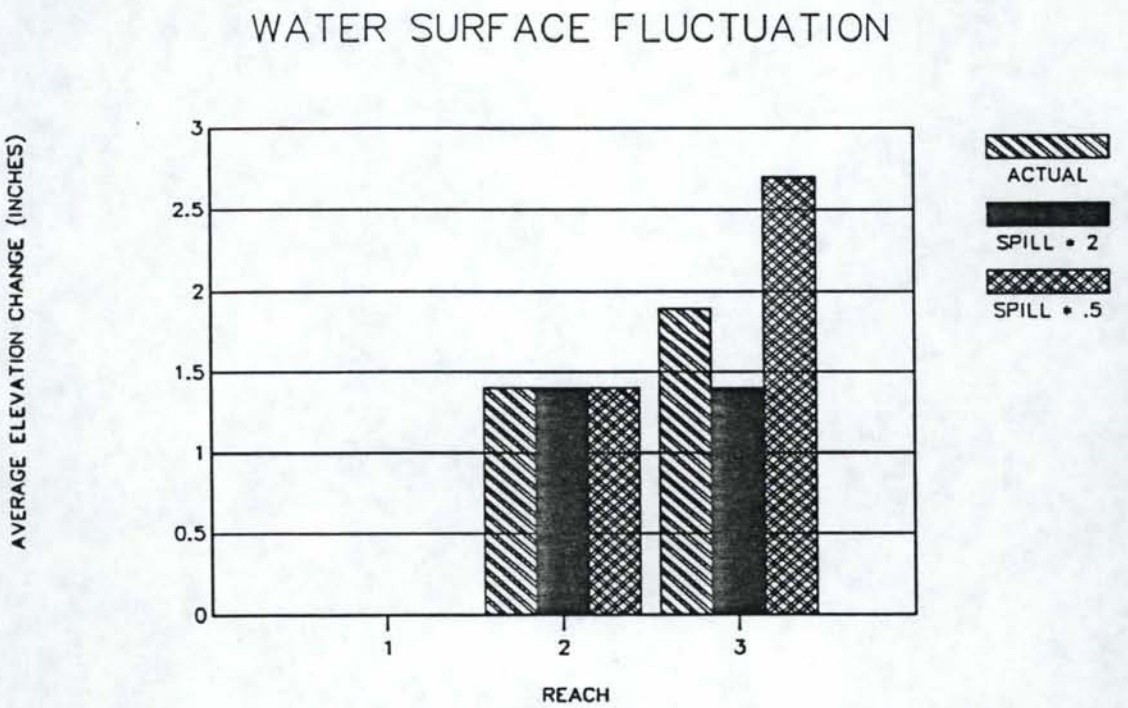


Figure 52(a) and (b). G Lateral flow rate and water surface fluctuation with varied spillway width. 2 hour, 40% turnout shutoff.

(I) Q Lateral

The configuration files for Q Lateral appear in Appendix A. Figure 53 shows the branch and reach configuration used in the simulations. The control structures are all manually adjusted rectangular weirs, and all simulations were run using manual control mode. There is a rectangular weir spillway at the upstream end of Branch 1 Reach 5, and another at the downstream end of Branch 2 Reach 1. Variations in flow rate in these reaches is variation in flow rate over the spillway.

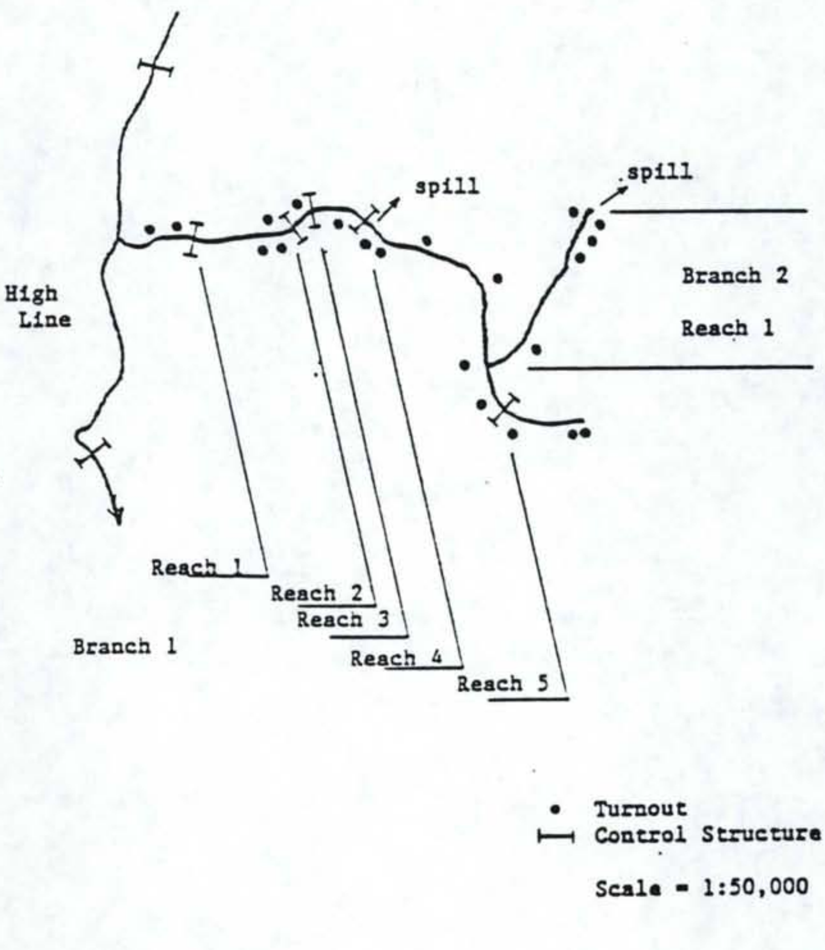


Figure 53. Reach configuration used in Q Lateral simulations.

Inflow into Q Lateral is through a 4 foot diameter circular orifice. There is no measuring device. Flow rate is estimated based on ditchrider experience. Inflow rate used for the simulations was 28 cfs, based on 23.6 cfs for turnouts, 1.5 cfs for the spills, and 2.9 cfs for seepage. Turnout flow rates from July 7, 2990, used in the simulations are shown in table 9.

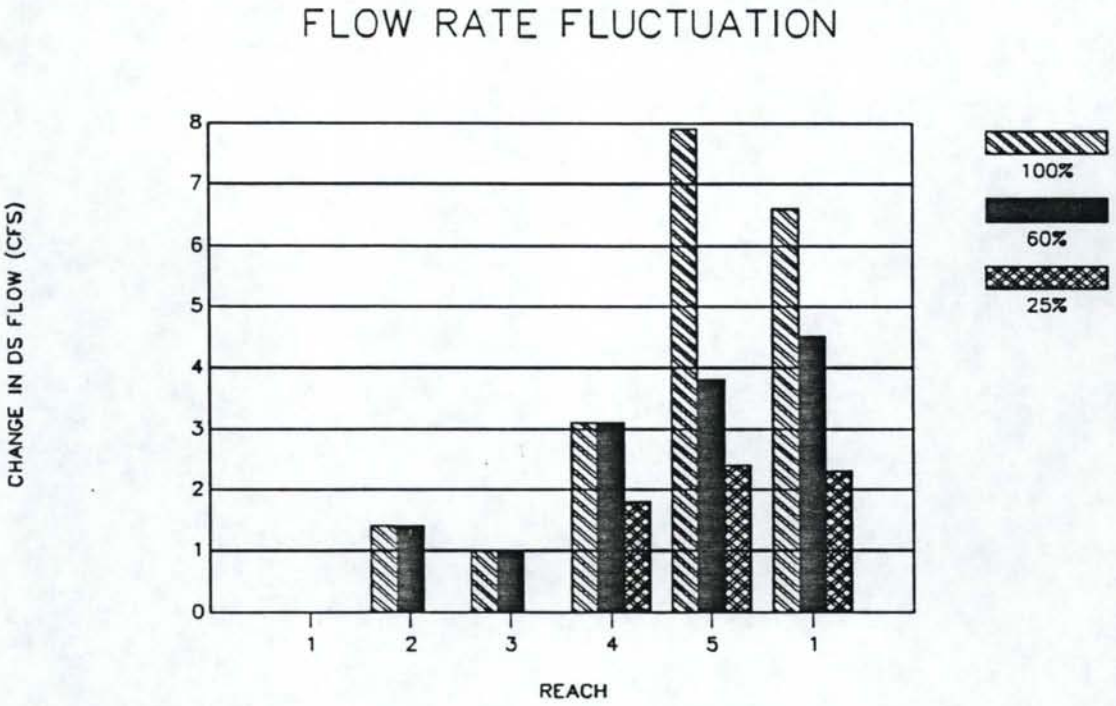
		Turnout	Flow Rate (cfs)	Outlet
Branch 1				
	Reach 1	Q 0-2	0.0	NC
		Q 0-3	0.0	C
	Reach 2	Q 0-5 +	1.0	C
		Q 0-7	0.0	C
		Q 0-9 +	0.2	C
	Reach 3	Q 0-10	0.0	C
	Reach 4	Q 1-1*+	2.0	C
	Reach 5	SPILL	1.3	
		Q 1-3	0.0	C
		Q 1-5	0.1	C
		Q 1-6	2.5	C
		Q 1-7 +	1.5	C
		Q 1-8*+	1.6	C
		Q1 LATERAL	8.0	C
		Q 2-3	1.8	C
		Q2-6,12,10	6.0	C
Branch 2				
	Reach 1	Q1 0-4	0.0	C
		Q1 0-10 +	2.4	C
		Q1 0-9*+	3.3	C
		Q1 0-12	0.0	C
		Q1 0-13	1.2	C
		SPILL	0.2	

Table 9. Turnout flow rates used in Q Lateral simulations. "*" indicates turnouts shut off for 25% non-continuous simulations, and "+" indicates turnouts shut off for 60%.

Shutting off the turnouts for 2 hours results in the fluctuations shown in figures 54(a) and (b). 100%, 60%, and 25% of the turnouts were shut off.

Figures 55(a) and (b) illustrate the effects of increasing the length of time for which turnouts are operated non-continuously. 25% of the turnouts were shut off for 2, 4, and 8 hour periods. Fluctuation of flow rate and water surface increases as the length of shutoff time increases.

Figures 56(a) and (b) illustrate the effects of flow fluctuation in Branch 1 on fluctuations in Branch 2. The two branches are separated by a circular orifice 2 feet in diameter. 100% of the turnouts in Branch 1 were shut off, while no turnouts were adjusted on Branch 2. This is compared with 100% of the turnouts being shut off on both branches. Only a small fluctuation is passed through the orifice into Branch 2.



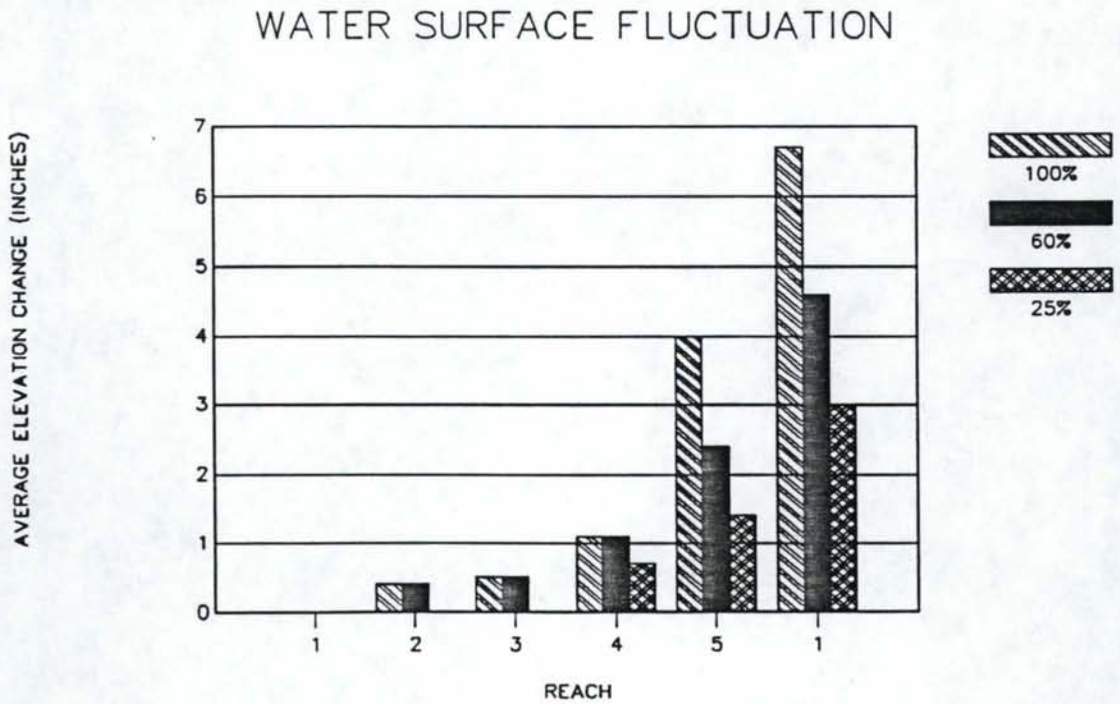
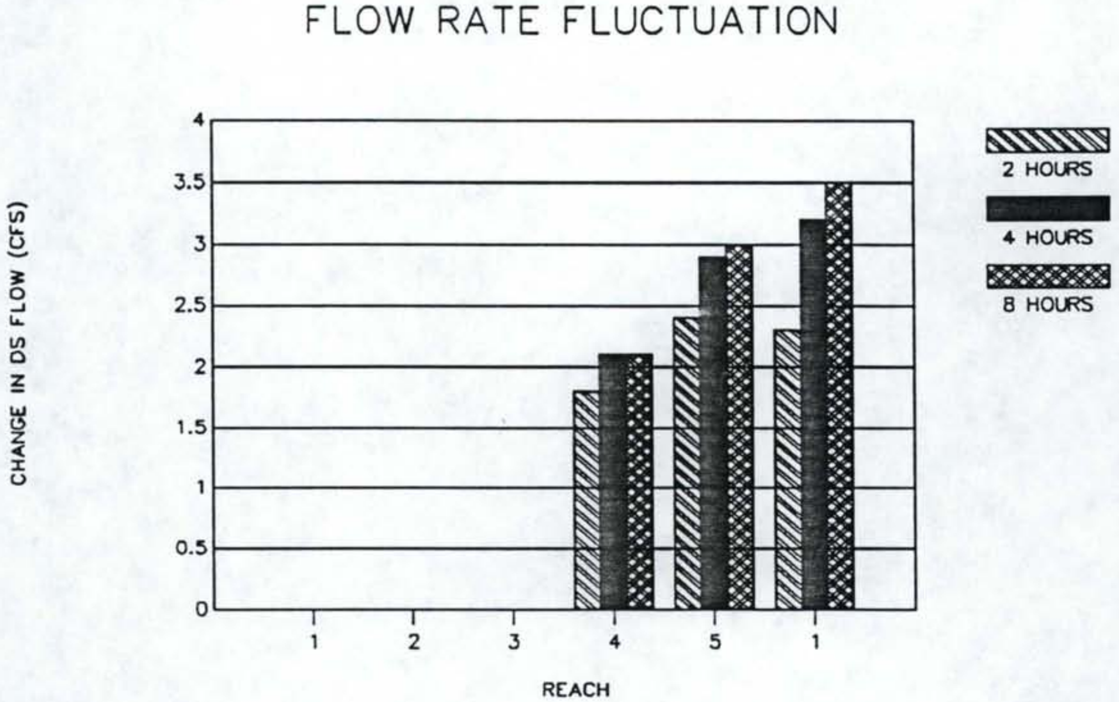


Figure 54(a) and (b). Q Lateral flow rate and water surface fluctuation from 2 hour turnout shutoff.



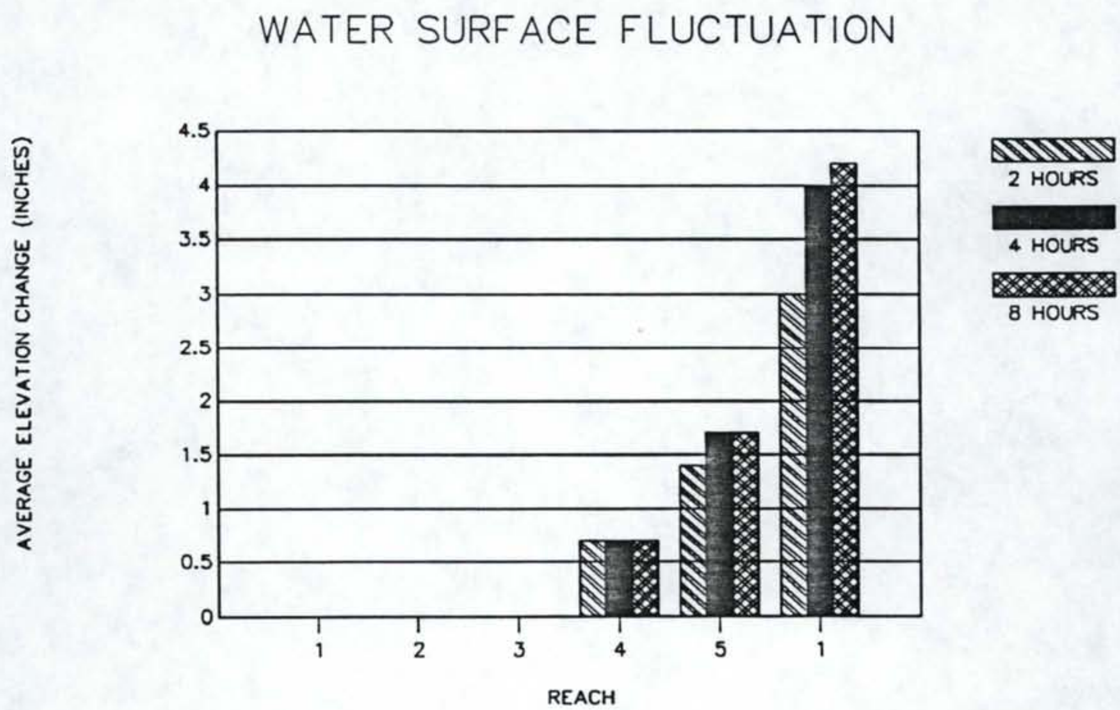
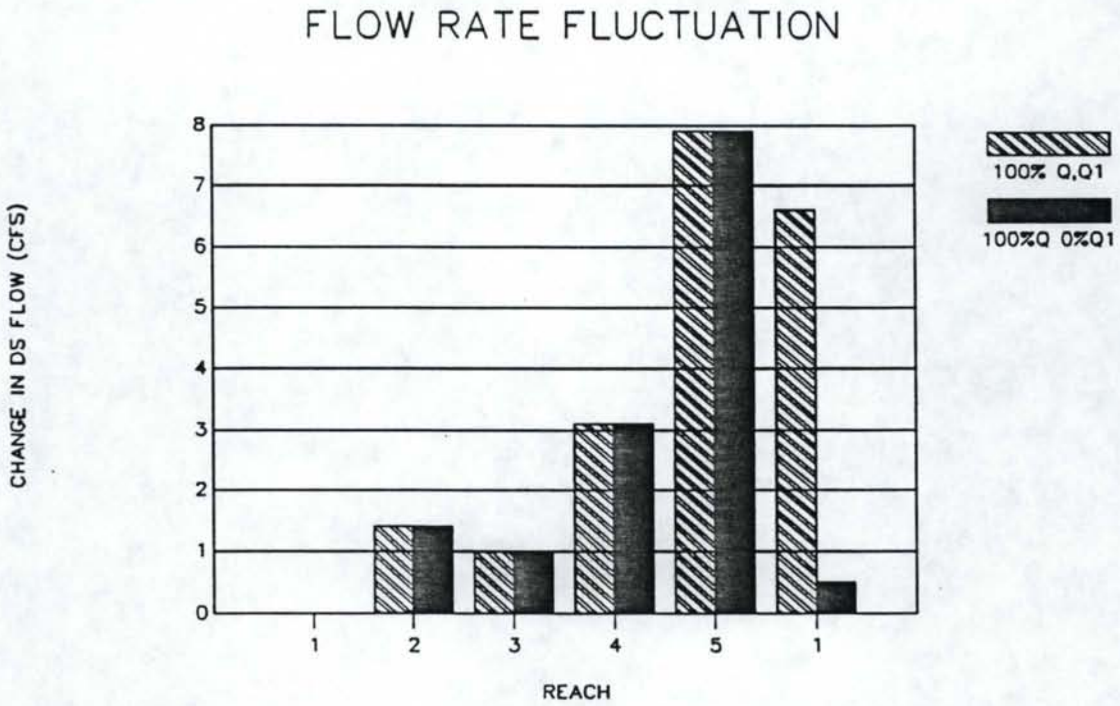


Figure 55(a) and (b). Q Lateral flow rate and water surface fluctuation with increasing shutoff time for 25% turnout shutoff.



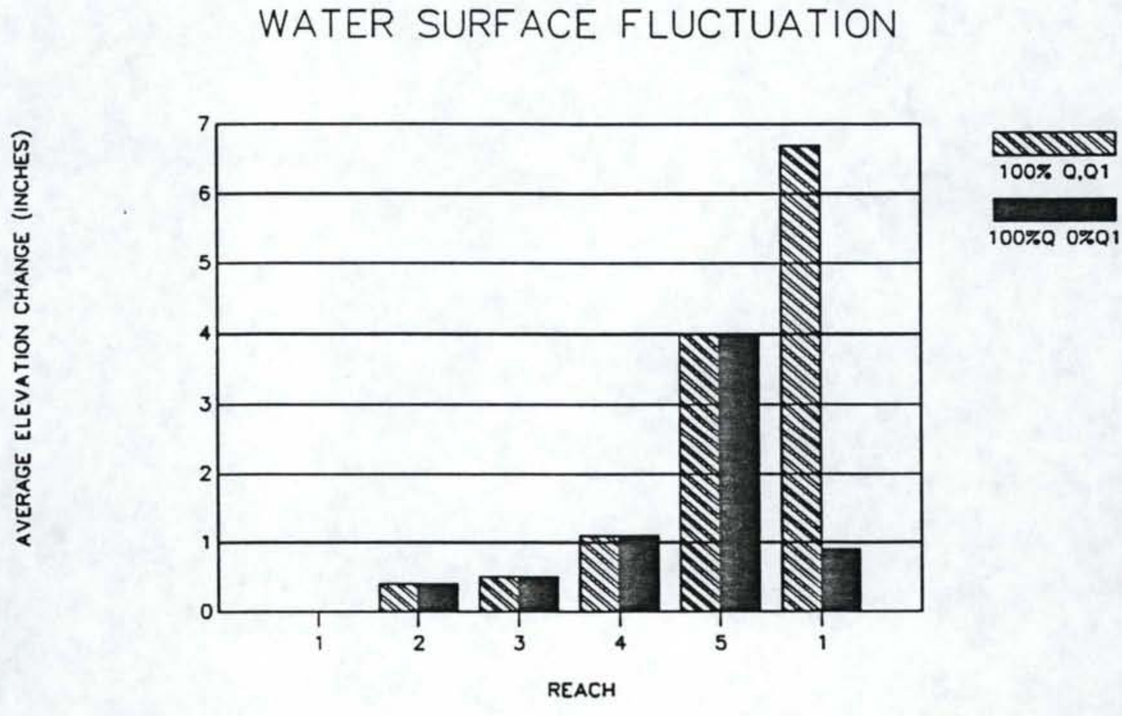


Figure 56(a) and (b). 53 Lateral flow rate and water surface fluctuation caused by 2 hour 100% turnout shutoff in Q Lateral and by 2 hour 100% turnout shutoff in Q and 53 Laterals.

CHAPTER 5

DISCUSSION

The results given showed the consequences of non-continuous flow in several different parts of the Aberdeen-Springfield canal system, and the results of several different modifications to the system. The general effects of the different non-continuous flow scenarios are discussed below.

(A) Vary Percentage of 2 Hour Non-continuous Flow

As expected, as the percentage of turnouts operating non-continuously increases, the fluctuation in the canals increases. The fluctuation tends to increase in the downstream reaches as the channel narrows and the volume of water which has been rejected into the canal upstream increases. Shutting off 100% of the turnouts in the laterals increased water surface on the order of 4 to 7 inches in the downstream reaches with spillways. The increase was over 30 inches in E1 Lateral, which has no spillway, and over 16 inches in V Lateral when the spillway was not used. In both cases this fluctuation caused the laterals to overflow.

The volume of flow rate increases is greater on the Main Line, High Line, and Low Line canals, but as they are larger canals with greater capacity, the resulting water surface fluctuation is not as large. Lower reaches of both the High Line and Low Line, however, where the channel capacity is diminished, exhibit fluctuations much like those in the laterals. Water surface fluctuations of about 3 inches on the High Line and 5 to 6 inches on the Low Line are predicted for 100% turnout shutoff.

A relatively small percentage of non-continuous turnouts causes significant fluctuation in the laterals. In C1 Lateral, shutting off only 3 turnouts (25%) results in water surface increases of 2 inches. In C Lateral, 3 non-continuous turnout flows (25%) yield water surface fluctuations of nearly 4 inches, and in G Lateral, shutting off 2 turnout flows (40%) results in a 2 inch water surface increase.

(B) <u>Vary Length of Shutoff Time</u>

In every canal and lateral simulated, increasing the length of time that turnouts were shut off resulted in increased water surface fluctuation. Water levels in upstream reaches sometimes leveled out, but downstream reaches continued to rise while turnouts rejected flow into the canal. The average fluctuation in the downstream reaches of C, C1, E, and Q laterals for 25% 8 hour non-continuous turnout operation was 80% of the 2 hour, 100% non-continuous fluctuation. Resulting water surface in E Lateral was actually higher with 25% of the turnouts shutting off for 8 hours than with 100% shutting off for 2 hours.

Even in the main canals, extended shutoff times create significant fluctuations in water level. Shutting 100% of the turnouts off for 6 hours yields 3 inch fluctuations in some reaches of the Main Line. For 100%, 8 hour turnout shutoff, fluctuations of over 8 inches occur on the High Line in the reaches above the spillways. Water surface would increase over 12 inches in the lower reaches of the Low Line under these same operating conditions.

(C) Stagger 2 Hour Shutoff Times

Staggering the shutoff times so that 20% of the turnouts shut off from 1 to 3 hours, 60% shut off from 2 to 4 hours, and 20% from 3 to 5 hours has little effect on the resulting water surface fluctuation. On the Main Line, using automatic control structures, water level fluctuation decreased 40% in some reaches, but actually rose in others, indicating that a different series of gate adjustments produced the varying fluctuations. Using manual control mode on the High Line, a slight decrease (0.5 inches) was predicted. On V Lateral the water surface fluctuation decreased only from 6.6 to 5.8 inches in the downstream reach when shutoff times were staggered.

(D) Vary Canal Base Width

Widening the base of the canals creates more in-stream storage for non-continuous turnout flow, which in turn results in smaller flow rate fluctuations and decreased water surface fluctuation. In V Lateral, increasing the base width 10 feet over the entire length of the canal resulted in less than 1 inch decreased water level fluctuation when all non-continuous turnouts on the lateral were shut off for 2 hours. Increasing canal base widths 7 feet in C Lateral decreased fluctuation in the downstream reach from 9 to 6 inches, but substantially less in the other reaches when 100% of the turnouts were shut off for 2 hours. Increasing the base width in C1 Lateral by 7 feet decreased water surface fluctuation from 7 to 5 inches in the downstream reach when 100% of the turnouts were shut off for 2 hours. Again, reductions in water surface fluctuation in the upper reaches was substantially less.

(E) Spillway Modification

Increasing the width of spillways in the laterals increases the capability of the spillways to remove the excess water from the canal while decreasing the maximum water surface fluctuation. Increasing the size of the spillway has no effect on reaches upstream of the reach in which the spillway is located. By doubling the width of the spillway, predicted water surface fluctuation decreases by 2 inches in C and C1 Laterals when 100% of the turnouts are shut off for 2 hours. Water surface fluctuations decrease about 1 inch on V and E Laterals when all non-continuous turnouts were shut off for 2 hours. Doubling the width of the spillway on G Lateral yields half an inch less water surface fluctuation when 40% of the turnouts are shut off for 2 hours.

Decreasing the width of the spillway results in increased water surface fluctuations in the reach containing the spillway, but has no effect on upstream reaches. In G Lateral, where doubling the size decreased water surface fluctuation by half an inch, decreasing the size of the spillway to half its actual width resulted in nearly 1 inch increased fluctuation. In V Lateral, decreasing the spillway to half its actual width resulted in nearly 1.5 inches greater water surface fluctuation when all non-continuous turnouts were shut off for 2 hours.

Decreasing the width of spillways on the High Line by 50% yielded increased water surface fluctuation in the reaches with spillways, but had no effect on other canal reaches. Water surface fluctuation increased about one half inch when 100% of the turnouts were shut off for 2 hours.

Using the spillway located in Branch 2 Reach 3 of the High Line (currently not used by canal company) resulted in slightly decreased fluctuation in the downstream reaches. The fluctuation decreased about one quarter inch when 100% of the turnouts were shut off for 2 hours.

Adding a spillway at the end of a lateral which does not possess one results in significant reductions in water surface fluctuation in the downstream reach. A 10 foot wide spillway at the downstream end of E1 Lateral reduces predicted water surface fluctuation by nearly 50%, from 8 inches to 4.2 inches, when all non-continuous turnouts are shut off for 2 hours. Shutting off 100% of the turnouts for 2 hours on V Lateral results in nearly 16 inches fluctuation in water surface when the spillway is removed from the model, and only 6 inches when the spillway is operating.

(F) <u>Orifice Control Structures</u>

Replacing weir or overflow type control structures with orifice or sluice gate type control structures has significant effects on fluctuation in the canals. Flow rate over a weir is a function of the head on the weir to the 1.5 power, while flow rate through an orifice is a function of the square root of the head, as shown in the equations below.

Rectangular Weir

$$Q = C_d * W * H^{1.5} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4)$$

Orifice

$$Q = C_d * A * H^{0.5} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

Where Q = Flow rate

C_d = Discharge coefficient

W = Width of weir sill

H = Head above weir sill or across orifice

A = Area of orifice opening

Thus an increase in water surface upstream of an orifice control structure results in a smaller increase in flow rate through the structure. The decreased flow rate out of the reach results in smaller water surface fluctuations downstream, but increases water surface fluctuation upstream of the structure.

Replacing all four rectangular weir control structures on V Lateral with rectangular orifices results in increased water surface fluctuation in the three upstream reaches, but decreased fluctuation in the downstream reach. When 100% of the turnouts were shut off for 2 hours, the fluctuation increased from 0.6 to 2.9 inches in Reach 1, 2.0 to 4.3 inches in Reach 2, 4.7 to 7.4 inches in Reach 3, and decreased from 6.6 to 2.5 inches in Reach 4.

Inserting orifice control structures into C1 Lateral at the downstream end of Reach 2 increased water surface fluctuation from 1.5 to 2.3 inches in that reach, but decreased fluctuation in the downstream reaches by about half an inch when 25% of the turnouts were shut off for 2 hours. Inserting orifices at Reach 2 and Reach 3 increased the water surface fluctuation from 1.5 to 2.5 inches in Reach 2, from 1.1 to 2.1 inches in Reach 3, and decreased fluctuation about 1 inch in the downstream reaches.

(G) <u>Manual and Automatic Control Structures</u>

The automatic control structures in the model do not accurately represent the automatic control structures in the system. While the automatic mode does try to keep the water level upstream of the control structures constant as the actual automatic controls do, the model links all the control structures together, which is not done in the actual system. The model attempts to control the fluctuation in the downstream reaches not only by adjusting the downstream gate, but also by adjusting the upstream structures. This results in some differences in flow rates and fluctuations between the model and what would actually occur on the system. The effect of automatic checks as they occur on the system would be to stabilize the water surface in the reach upstream of the control structure, while passing any fluctuations downstream. This would result in magnified fluctuations in the downstream reach, or in reaches without automatic control structures.

CHAPTER 6

FLOW MEASUREMENT

(A) CANAL SYSTEM

The exact flow rate at any location in the Aberdeen-Springfield Canal system is unknown except at the head of some laterals. The depth of flow is measured on a daily basis at several locations and used as a reference for day-to-day operations, but the measurements do not represent an accurate flow rate measurement. Even near the head of the canal, at the USGS gage, only an estimate of the actual flow rate is made. This value is subject to change due to conditions in the canal. The canal is current metered approximately once a month to determine the necessary 'shift' to get an accurate measurement at the gage.

Flow measurement at strategic locations on the canal system would provide several benefits. A better estimate of seepage losses could be made, and high loss areas pinpointed. The exact amount of water diverted into the canal system would be known. This would eliminate any guesswork regarding storage water. Manageability of the system would increase, as the correct amount of water could be delivered every day. Better management of the system could provide some water savings.

There are five locations on the Aberdeen-Springfield Canal where accurate water measurements would provide the most benefit.

1. Head of the Main Line Canal at USGS gage.
2. Head of the High Line Canal near Big Fill.
3. Head of the Low Line Canal near Big Fill.
4. High Line Canal near Nash Spill.
5. High Line Canal below the reservoir at mile 62 (north of Center Pleasant Valley Road).

Measurement at location (1) would provide an accurate record of flow diverted into the system. Only the necessary flow would be diverted into the system on a daily basis, eliminating any guesswork due to a 'shift' in the existing rating table. Measurement at location (2) and (3) would allow the correct amount of water to be diverted into the High Line and Low Line Canals. Excess flow could be diverted into the Big Fill Reservoir, and shortages could be recovered from the reservoir. In addition, a good idea of seepage losses in the Main Line could be made by subtracting High Line, Low Line, Lateral, turnout, and reservoir flows from the inflow measured at the head. Measurement at location (4) would increase the efficiency at the Nash Spill. Excess water would be spilled instead of sent down the canal, and excessive spill loss could be managed better. The seepage loss from the head of the High Line to this point could be determined. Measurements at location (5) would allow the correct flow rate to be sent into the downstream reaches of the High Line. Excess water could be stored in the reservoir, and shortages made up from the reservoir.

Measurement locations (1), (2), (3), and (5) would allow better system management without wasting any water. Water diverted back to the Snake River at location (1) is not charged to the Aberdeen-Springfield Canal Company. The water may be stored in reservoirs at locations (2), (3), and (5) and recovered for later use.

The recommended measuring structures are broad crested weirs, (Figure 57. Clemmens and others (1987) list the following advantages associated with this type of structure.

1. A rating table can be calculated with an error of less than 2 percent in the listed discharge.

2. The complete range of discharge can be measured accurately.

3. The head loss over the weir or flume is minimal.

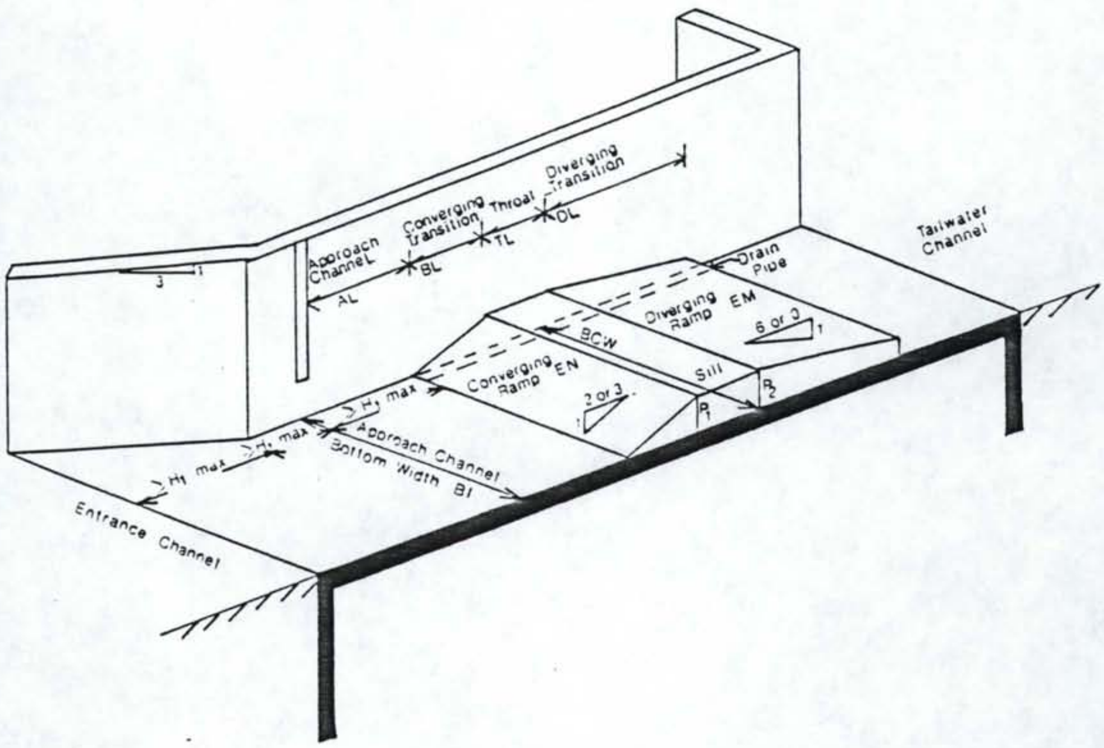
4. Because of the gradually converging transition, there is little problem with floating debris.

5. The structure can be designed to pass sediment transported in channels with suberitical flow.

6. An accurate rating table can be produced even if the flume is not constructed to design dimensions.

7. Broad crested weirs are usually the most economical for accurately measuring open channel flows.

The dimensions for structures at locations (1), (2), (3), and (5) are provided in figure 57. Not enough data was collected to design the structure at location (4).



Location	1	2	3	5
Length to Gage (HL) feet	10	5	5	5
Converging Ramp Length (BL) feet	6	18	10.5	9
Throat Length (TL) feet	7	6	5	3.5
Diverging Ramp Length (DL) feet	0	0	0	0
Sill Height (P_1 & P_2) feet	2	6	3.5	3
Converging Ramp Slope (EN)	3:1	3:1	3:1	3:1
Throat Width feet	51	26	24	10
Measurable Flow Rage (cfs)	150–400	35–600	20–400	5–110
Upstream Wall Length	10	5	5	5
Downstream Wall Length	15	10	10	10
Total Structure Length	48	44	35.5	32

Figure 57. Ramped Broad Crested Weir and Dimensions for Aberdeen-Springfield Canal Company structures.

The diverging section on the broad crested weir is for reducing head loss across the structure. The weirs have been designed so that this section is not necessary. The weirs will have a vertical diverging ramp.

The elevation of the top of the weir sill has been determined relative to local benchmarks. At location (1) the benchmark is the top of the concrete sidewall at the downstream side of the USGS Gaging Station. The weir sill should be 5.1 feet below the benchmark. The concrete structure could be used as part of the broad crested weir.

At location (2) the benchmark is on the control structure at the head of the High Line, the metal plate supporting the gear which moves the south sluice gate (left side looking downstream). The weir sill should be 3.0 feet below the benchmark. The weir should be located immediately below the gage house.

At location (3) the benchmark is the top of the concrete box at turnout L-0-1. The weir sill should be 2.0 feet below the benchmark. The weir should be located just below the convergence of the 2 channels. Turnout L-0-1 may have to be moved to accommodate the structure.

At location (5) the benchmark is the top of the concrete pad supporting the pump on the east side of the canal. The pump is located approximately 400 feet downstream of the culvert which delivers water from the reservoir into the canal. The weir sill should be 7.0 feet below this benchmark. The weir should be constructed near this benchmark.

The rating table for the four weirs are given in figures 58-61.

Depth of flow over weir sill (feet)	Flow Rate (cfs)
1.0	157
1.2	209
1.4	266
1.6	328
1.8	395
2.0	467
2.2	543
2.4	624
2.6	708
2.8	798
3.0	891
3.2	988
3.4	1089
3.6	1194
3.8	1303
4.0	1415

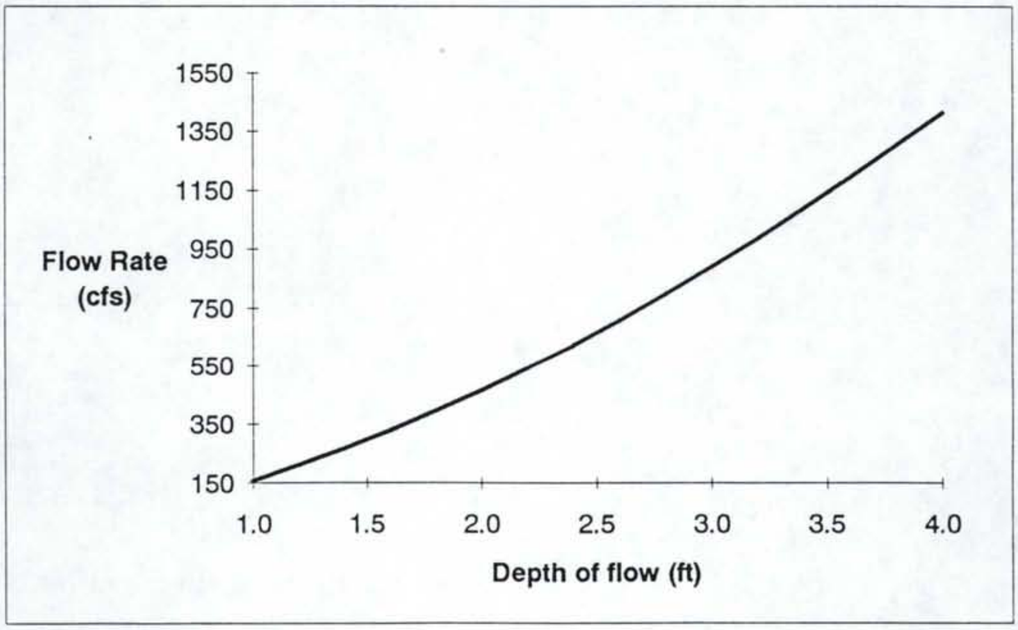


Figure 58. Rating table and curve for Broad Crested Weir located near head of Main Line Canal (location 1).

Depth of flow over weir sill (feet)	Flow Rate (cfs)
0.6	35
0.8	55
1.0	78
1.2	103
1.4	131
1.6	161
1.8	193
2.0	226
2.2	262
2.4	300
2.6	339
2.8	380
3.0	423
3.2	468
3.4	514
3.6	561
3.8	611

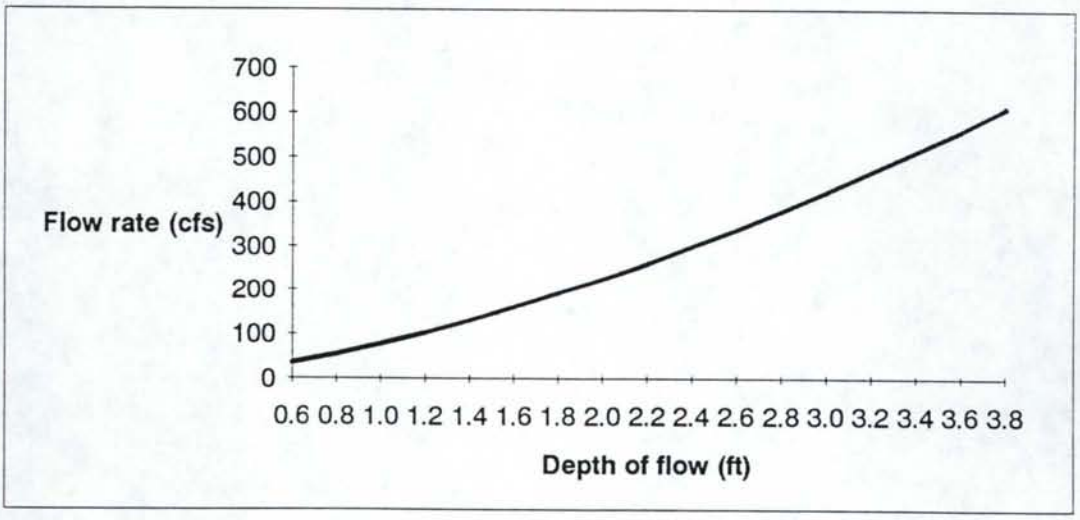


Figure 59. Rating table and curve for Broad Crested Weir located near head of High Line Canal (location 2).

Depth of flow over weir sill (feet)	Flow Rate (cfs)
0.4	18
0.6	33
0.8	52
1.0	73
1.2	97
1.4	123
1.6	151
1.8	181
2.0	213
2.2	248
2.4	284
2.6	321
2.8	361
3.0	402

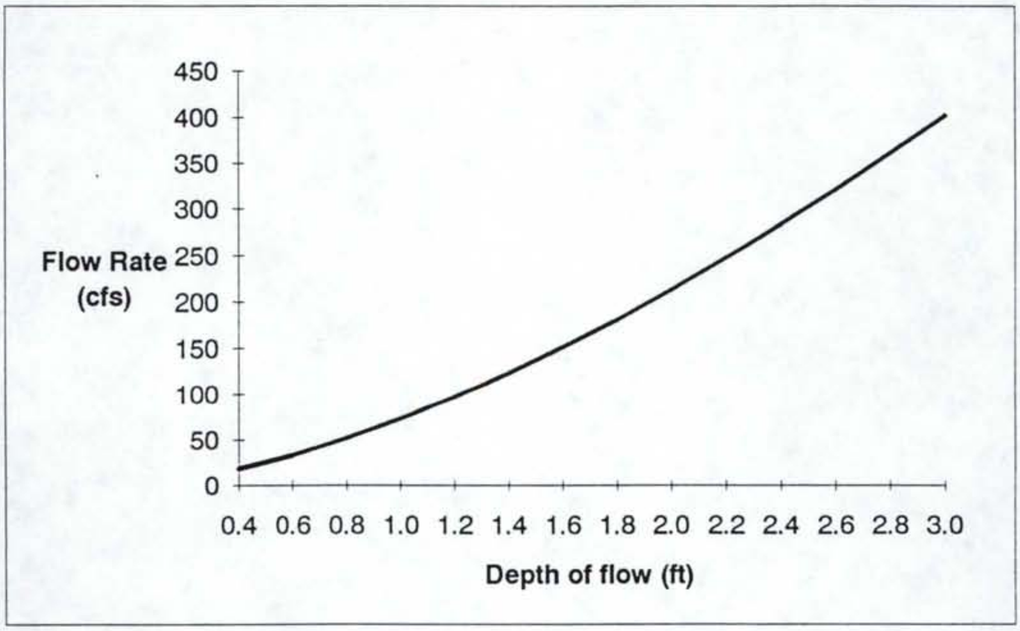


Figure 60. Rating table and curve for Broad Crested Weir located near head of Low Line Canal (location 3).

Depth of flow over weir sill (feet)	Flow Rate (cfs)
0.3	5
0.4	8
0.5	11
0.6	14
0.7	18
0.8	22
0.9	26
1.0	31
1.1	36
1.2	41
1.3	46
1.4	52
1.5	57
1.6	63
1.7	70
1.8	76
1.9	83
2.0	90
2.1	97
2.2	104
2.3	112

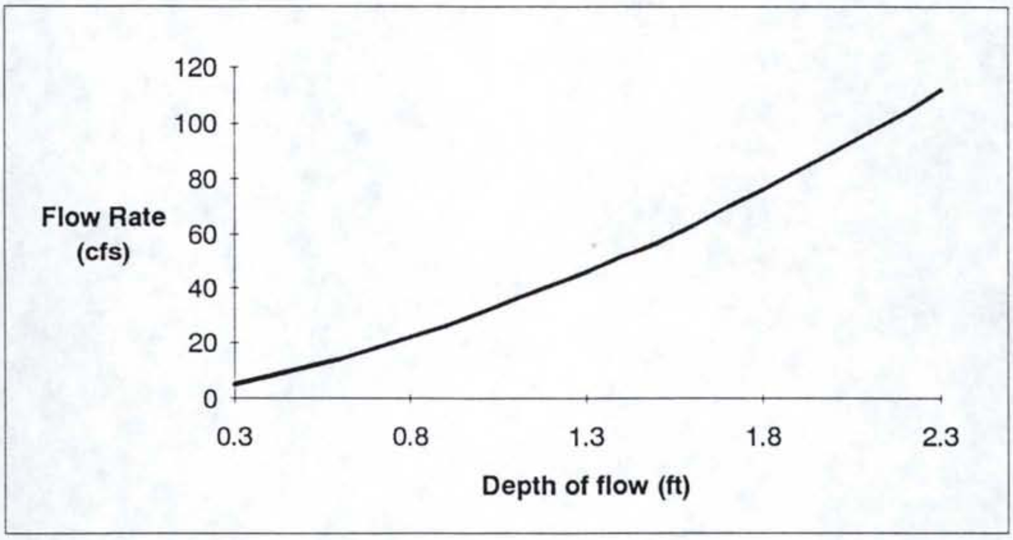


Figure 61. Rating Table and curve for Broad Crested Weir located near Reservoir at mile 62, above center (location 5).

Rough cost estimates for the structures, based on research by France and Brockway (1987) are as follows.

location # 1	$12,000
location # 2	$6,500
location # 3	$6,000
location # 5	$2,500

Annual maintenance cost on the structures is minimal.

(B) SNAKE RIVER DIVERSION STRUCTURE

The diversion structure for the Aberdeen Springfield Canal at the Snake River consists of seven (7) radial gates operated by manual gate lifts. Two of the gates are not currently used and the remainder are in a poor state of repair. The Company contemplates replacement of the structure in the near future. The current structure is located on the outside of a major bend of the river with water level control achieved by a low overflow section downstream of the diversion structure. Sediment and gravel removal have been a major maintenance problem to maintain adequate canal flows when discharge in the river is low.

There are three alternative sites for the new structure. Site one, approximately 300 feet upstream of the current structure, is the preferred site relative to the hydraulics of the river. Sediment and gravel accumulation would be reduced since the curvature of the river is not as severe as the current site. It would be necessary to raise the overflow structure downstream to maintain river water levels slightly higher than the current structure requires. This site would likely require purchase of right-of-way; however, this need has not been explored. If the structure were built at this site, an additional 300+ feet of canal would need to be constructed to convey flow to the existing canal.

Site two is immediately below the existing structure in the existing channel. This site has the advantage in that it will require no construction within the floodway of the river and could be built in the "dry" by sealing off the gates of the existing structure. Only minimal pumping of ground-water would be necessary for construction of footings and cutoff walls. The existing structure could either be removed entirely or the gate assemblies and concrete bays removed after construction of the new structure. This site has the disadvantage of being subject to the same hydraulic environment as the current structure regarding channel maintenance and gravel removal.

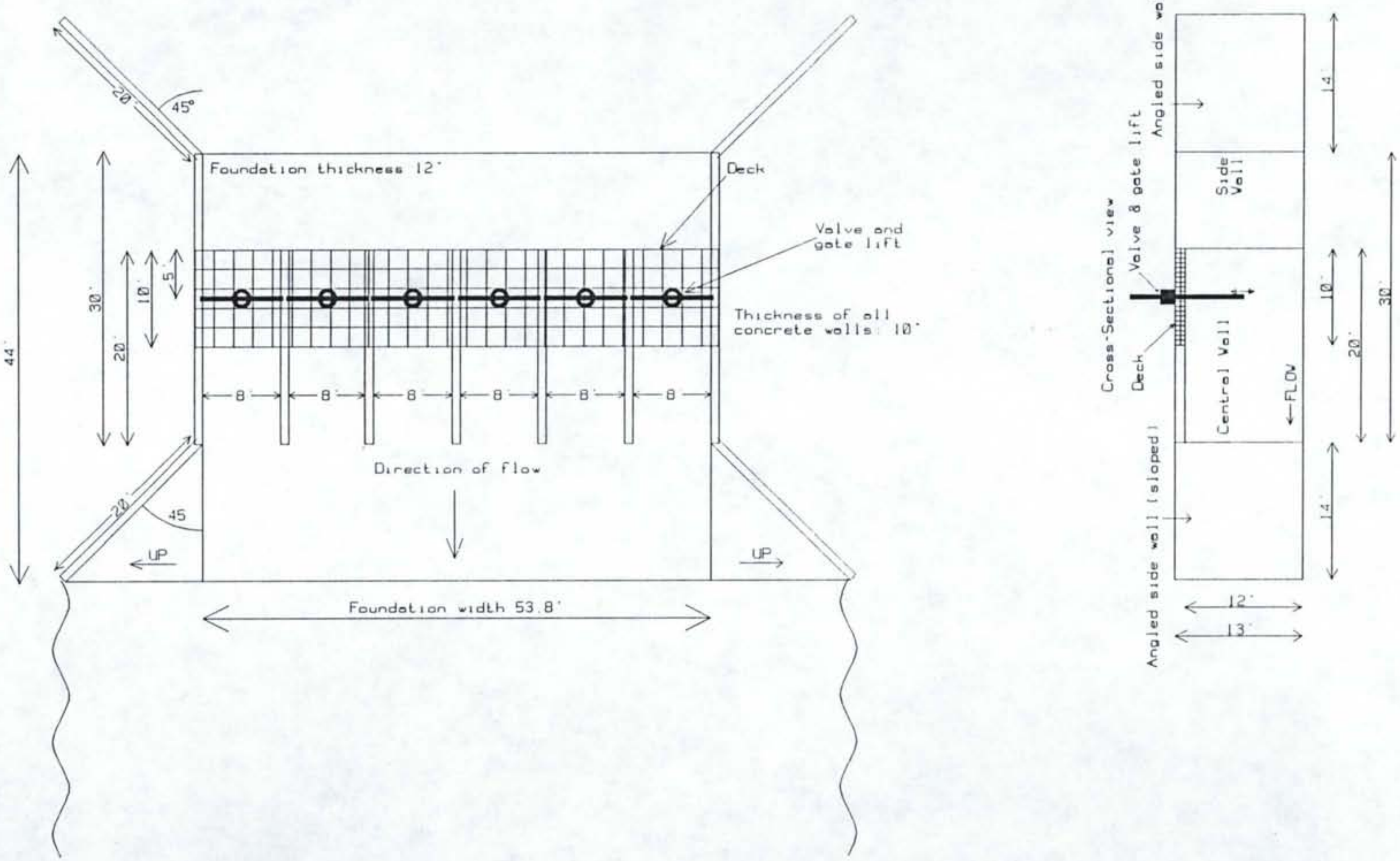
The advantages of construction cost savings for the existing channel would appear to outweigh the potential savings in maintenance for the upstream site.

A hydraulic design for the potential new structure was performed to provide the Company with a gate size and layout for assistance in planning and cost estimating. The design discharge suggested by the Board of Directors and Manager is 1500 cfs. A survey of water levels in the Snake River and below the existing structure was performed to determine design water levels for the new structure. Based on the survey, the minimum available head, or difference in upstream and downstream water levels, is approximately 3 feet.

Several configurations of gates and gate sizes are possible to achieve the desired discharge with the available head. Vertical slide gates are recommended rather than radial gates because of their simplicity, cost, and amenability to hydraulic gate lift mechanisms. It is recommended that hydraulic gate lifts, operated by portable power units or manual gate wheels be utilized because of the lack of electricity at the site. Possible gate configurations include: four (4) twelve (12) foot wide gates, five (5) ten (10) foot wide gates, or six (6) eight (8) foot wide gates.

Utilization of six, eight foot wide gates offers reasonable flexibility in flow control and is likely the most cost effective. Figure 62 shows a schematic layout for a six gate structure using eight foot wide slide gates with hydraulic lifts. <u>This is not a design drawing and a full design layout and structural analysis will be required prior to bid and construction</u>. A full cost-benefit analysis of the various gate configurations has not been performed. This could be done at final design. A preliminary estimate of costs based

FIGURE 62. SCHEMATIC DIAGRAM FOR
ABERDEEN-SPRINGFIELD SNAKE RIVER DIVERSION



on concrete in-place at $150 per cubic yard and estimated gate costs of $4,000 per gate is $51,750 for the structure.

Figure 63 shows the rating table and curve for a 6 gate structure. Utilizing six, eight foot gates, the desired discharge of 1500 cfs can be passed with a 3.0 ft differential head and a gate opening of 3.8 feet.

ABERDEEN-SPRINGFIELD CANAL CO.

DISCHARGE RATING FOR SNAKE RIVER DIVERSION GATES
ASSUME SUBMERGED GATES

NO. OF GATES	6
GATE WIDTH	8 FT
GATE COEFF.	0.61

DIFF	GATE OPENING FEET — DISCHARGE CFS																			
	1.0	1.2	1.4	1.6	1.8	2.0	2.2	2.4	2.6	2.8	3.0	3.2	3.4	3.6	3.8	4.0	4.2	4.4	4.6	4.8
1.0	235	282	329	376	423	470	517	564	611	658	705	752	799	846	893	940	987	1034	1081	1128
1.2	257	309	360	412	463	515	566	618	669	721	772	824	875	927	978	1030	1081	1133	1184	1236
1.4	278	334	389	445	500	556	612	667	723	778	834	890	945	1001	1056	1112	1168	1223	1279	1335
1.6	297	357	416	476	535	594	654	713	773	832	892	951	1011	1070	1129	1189	1248	1308	1367	1427
1.8	315	378	441	504	567	630	694	757	820	883	946	1009	1072	1135	1198	1261	1324	1387	1450	1513
2.0	332	399	465	532	598	665	731	798	864	930	997	1063	1130	1196	1263	1329	1396	1462	1529	1595
2.2	349	418	488	558	627	697	767	836	906	976	1046	1115	1185	1255	1324	1394	1464	1533	1603	1673
2.4	364	437	510	582	655	728	801	874	946	1019	1092	1165	1238	1310	1383	1456	1529	1602	1674	1747
2.6	379	455	530	606	682	758	834	909	985	1061	1137	1212	1288	1364	1440	1516	1591	1667	1743	1819
2.8	393	472	550	629	708	786	865	944	1022	1101	1180	1258	1337	1415	1494	1573	1651	1730	1809	1887
3.0	407	488	570	651	733	814	895	977	1058	1140	1221	1302	1384	1465	1547	1628	1709	1791	1872	1954
3.2	420	504	588	673	757	841	925	1009	1093	1177	1261	1345	1429	1513	1597	1681	1765	1849	1934	2018
3.4	433	520	607	693	780	867	953	1040	1126	1213	1300	1386	1473	1560	1646	1733	1820	1906	1993	2080
3.6	446	535	624	713	802	892	981	1070	1159	1248	1337	1427	1516	1605	1694	1783	1872	1962	2051	2140
3.8	458	550	641	733	824	916	1008	1099	1191	1283	1374	1466	1557	1649	1741	1832	1924	2015	2107	2199
4.0	470	564	658	752	846	940	1034	1128	1222	1316	1410	1504	1598	1692	1786	1880	1974	2068	2162	2256

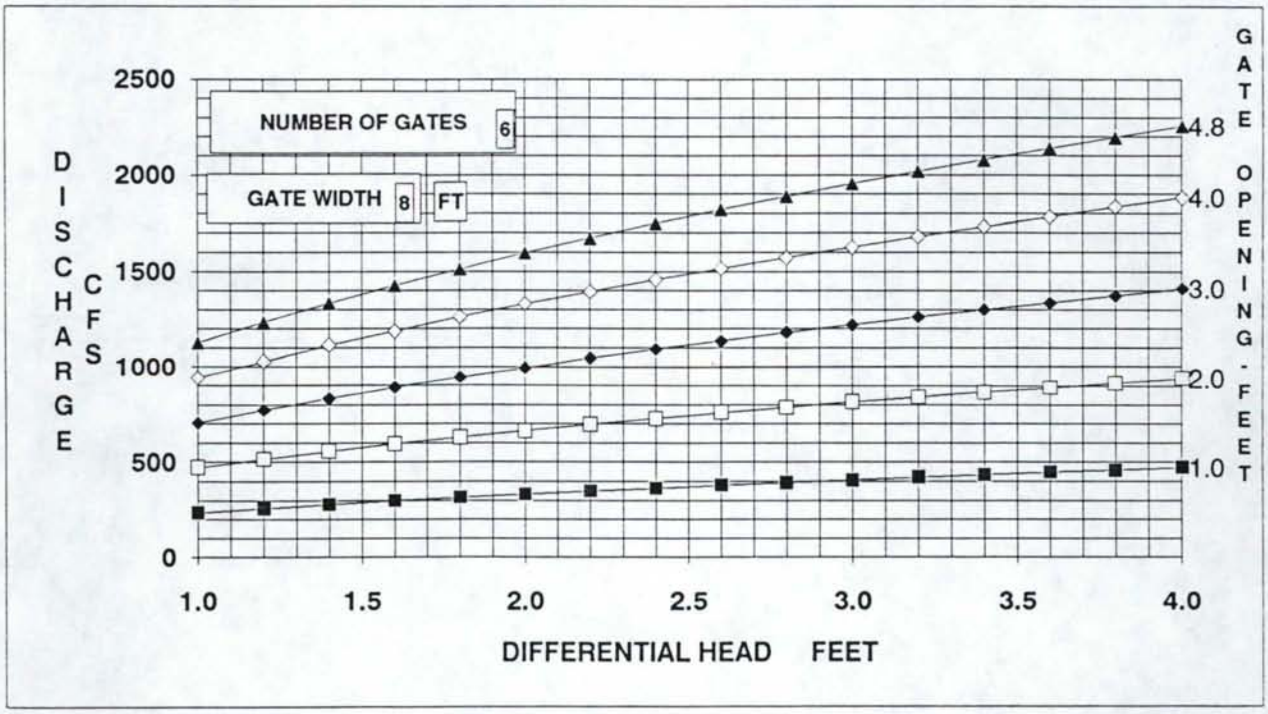


Figure 63. Discharge Rating for Snake River Diversion Gates.

CHAPTER 7

REVIEW OF BYLAWS

The By-laws of the Aberdeen-Springfield Canal Company consist of nine (9) articles dealing with operations and management of the Company. These articles were reviewed relative to possible modifications which would enhance water management within the system and potential changes to update the bylaws. This review should not be construed as a legal evaluation of the document,

Article IV deals with the delivery of water from the system and outlines the duty of water and delivery rates applicable to specific lands served either from the main canal or from laterals. Section 1 specifically empowers the corporation to "provide for the delivery of said water by a proper system of rotation,--". This implies that if the corporation deemed it necessary for equitable and safe distribution of flows that it could require a system of rotation within a lateral or sections of channel on a time basis between users or on proportionate share if necessary. It the corporation decided to allow non-continuous pumping within a lateral and that a reasonable rigid time schedule for off-times was necessary to preserve the integrity and safety of the lateral, then a rotation system could be required.

The point is that the corporation has the authority to require a rotation system or timing of periods when pumps are off if it is determined that rotation is necessary to provide the service and protect the system.

No articles or sections of articles in the current by-laws speaks to a requirement for equitable measurement of water to individual stockholders. Since Article IV Section 1 defines a share in terms of flow or discharge, it implies measurement of those deliveries by corporation personnel. In addition, since the maximum seasonal delivery is limited to two and one-half acre feet per acre, daily measurement is implied in order to compute the accumulated volume delivery to each stockholder.

An addition to Article IV which would require' equitable measurement of flow and volume delivered to all types of users' utilizing recognized measuring devices approved by the corporation' could be considered. This would require different types of measuring devices depending on the type of delivery that the stockholder chooses. Standard gravity flow deliveries could be made through orifice gates or weirs, whereas pipe deliveries could require in-line flow meters with instantaneous and accumulated volume readout.

It may be desirable to include in the by-laws an article or section on general conservation policies to expand on the requirement for beneficial use as outlined in Article IV. A conservation policy could include the intent of the corporation to minimize spills from the system whenever possible, utilize on-project storage where possible to reduce diversion requirements and spill, provide for water measurement at strategic locations within the delivery system to facilitate water management and allocation, minimize canal and lateral seepage rates where possible, and encourage on-farm water conservation. These 'policies' do not constrain the corporation to any specific structural or management procedures, but would put the corporation on record as supporting water conservation and environmental concerns.

CHAPTER 8

CONCLUSIONS & RECOMMENDATIONS

The hydraulic simulation model CANAL provided an invaluable tool for evaluating the many possible scenarios involving non-continuous flow on the Aberdeen-Springfield canal system. The results of model calibration indicate that the model was capable of accurately simulating the system and predicting the resulting fluctuation in water surface and flow rate. Evaluation of CANAL by the program author and others (Merkley, Walker and Gichuki, 1990, and Merkley and Rogers, 1991) also demonstrate its ability.

Using CANAL to simulate the Aberdeen-Springfield canal system has shown the potential effects of non-continuous turnout operation upon the canal system. The model shows that the fluctuation in flow rate caused by non-continuous turnouts results in water surface fluctuation. The magnitude of the fluctuation is dependent upon the physical dimensions of the canal, the degree of non-continuous turnout operation, and the length of time the turnouts are operated non-continuously. The model showed that staggered shutoff times of 2 hours during a 4 hour period resulted in fluctuations nearly as great as when all the turnouts were shut off simultaneously.

The model showed that increasing the width of canal laterals, and thus increasing the storage, decreased canal fluctuations somewhat. Increasing the canal width enough to significantly reduce or eliminate the fluctuation would involve significant construction costs, however, and is not a viable solution for reducing or eliminating the fluctuation.

The model showed that increasing the width of current overflow spillways only slightly decreased the resulting fluctuation in the canal. Decreasing the width of the structure, as is sometimes done during canal operations, results in slightly increased water surface fluctuation. Variation of spillway width had no effect on reaches upstream of the spillway, and only small effects on downstream reaches.

Having a spillway at the end of a lateral was shown to have a significant effect upon the resulting fluctuation in the lateral. Non-continuous turnout operation in laterals without a spillway in the downstream reach resulted in overflow of the canal banks. Operation of the existing spill in Branch 2 Reach 3 of the High Line was shown to have only minimum effects on the resulting fluctuations in the High Line because the fluctuations at the point of the spill were small. Canal company staff reported there were no new sites for spillways available, so the addition of new spillways was not investigated further.

Orifice control structures, as opposed to weir control structures, were shown to significantly reduce the fluctuation in flow rate passing through the structure. Resulting water surface fluctuation was greater upstream of the orifice control structure, but decreased downstream of the structure. The orifice controls spread the fluctuation out across the entire lateral instead of channelling it towards the downstream reach.

Automatic controls on the canal could not be simulated exactly, but the simulation indicated that the automatic structures pass any fluctuations downstream in an attempt to stabilize conditions upstream of the check. This results in increased fluctuations in the downstream reach or in reaches without an automatic control structure.

There are several options available to the canal company for reducing or eliminating the fluctuation caused by non-continuous turnout flows. Each of the options has some cost, either political or financial, to the company.

Requiring all turnouts to take continuous flow would eliminate the problem entirely. This option would require all irrigators to provide their own storage or spillway if they wanted to have the option of using the water non-continuously. Soil Conservation Service provides cost sharing and help with the design of on-farm storage ponds. Water savings increase substantially when turnouts delivering water to pumps are converted from non-continuous to continuous with a pond for storage. SCS staff and several irrigators estimate 30-35% savings. The increased efficiency on the farm would provide extra water for the canal company to use during dry years. The company could also lease the extra water for additional income.

The company could also require only those non-continuous turnouts causing or contributing to unacceptable fluctuation to convert to continuous flow. This would likely include most turnouts drawing water from laterals, the High Line, and the Low Line. Turnouts on the Main Line could be exempted from non-continuous flow because the resulting fluctuation in the Main Line is minimal. Also water rejected into the Main Line is diverted into the Big Fill Reservoir where it can be used in the Low Line. Flow rates into the High Line and Low Line canals is held constant by the automatic gate on the spillway into the reservoir.

It is likely some other turnouts could also be exempted from continuous flow, including; (1) turnouts at the downstream end of a lateral with a spillway, providing there are no other turnouts nearby, (2) turnouts near diversions into storage ponds, and (3) turnouts drawing only a small volume of water with regard to the flow rate in the canal going past the turnout. These exemptions would have to be decided on a case by case basis.

To reduce the fluctuation caused by non-continuous turnouts, the company could try to regulate the periods of non-continuous flow. The model illustrated that the length of time turnout flow was shut off was directly related to the resulting fluctuation. A limit on the length of time that flow could be rejected into the canal could decrease fluctuations. The model also showed that staggering the period of non-continuous flow so that not all turnouts were shut off at the same time caused some reduction in fluctuation. Enforcing any regulations on the period of non-continuous flow would be difficult.

The fluctuation in the canals is directly related to the number of turnouts operating non-continuously (actually the volume of flow being diverted non-continuously). Regulating the number of non-continuous turnouts in any reach of the canal system would restrict the maximum fluctuation occurring. Simulations showed, however, that a small number of non-continuous turnouts on the laterals can cause significant fluctuations in the laterals. Shutting off all current non-continuous turnouts would create fluctuations equal to 60% of the turnouts being shut off on some parts of the system.

The effects of physical alterations to the canal system on fluctuation caused by non-continuous flow are limited. Increasing the width of canals decreases fluctuation, but would require high construction costs and a long time. Limited benefits could be gained through an aggressive cleaning program. Increasing spillway widths also has only limited effects on fluctuation. Operation of existing spillways using the entire width of the overflow weir would reduce fluctuations somewhat. Inserting spillways into laterals currently without one would greatly reduce fluctuations in the lateral, and removal of spillways would greatly increase fluctuation.

Orifice control structures shift the fluctuation to the upstream reach, while weir control structures tend to pass the fluctuations downstream. Some alteration of existing control structures could be done to control fluctuation. In most cases turnouts are located on the upstream side of control structures, where an orifice structure would cause more fluctuation. It is believed that switching from weir to orifice control structures would only reduce one set of problems while aggravating another.

It is recommended that flow measurement at various locations throughout the system be improved. Four broad-crested weirs have been designed for installation at critical locations along the system. No flow would be wasted at any of these locations, but management of flow rates could be greatly increased. These weirs would also provide data regarding seepage in the various canals.

A broad-crested weir at the head of the Main Line, near the current USGS gage, will allow the company to determine the flow rate into the canal at any time, without waiting for USGS reports regarding the appropriate "shift" in the rating curve. Flow rate into the canal can be adjusted at the control structure directly above the proposed weir location. The company is charged for all water passing the USGS gage. An accurate measuring device would allow them to divert only what is needed.

Broad-crested weirs at the head of the High Line and Low Line canals would eliminate uncertainties in the volume of water entering these canals. Flow rates could be controlled at the control structures at the head of the canals. Any excess water could be stored in the Big Fill Reservoir, and water deficiencies could be made up by increasing the diversion from the reservoir into the Low Line.

A broad-crested weir below the check at the downstream end of Branch 3 Reach 3 in the High Line would increase water management into the tail end of the High Line and S Lateral. Flow rate over the weir could be adjusted at the control structure, with excess flow being diverted into the storage reservoir and flow deficiencies corrected by increasing flow rate from the reservoir back into the High Line.

It is also recommended that measurement of flow into the laterals be improved. By knowing the flow rate into any section of canal, the company ensures that enough water is going into the section to meet the demand. Providing turnouts are properly set, this reduces the chances of pump damage caused by insufficient flow.

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APPENDIX A

HYDRAULIC SIMULATION MODEL CANAL

LIST OF FIGURES

The hydraulic simulation model CANAL may be operated on an IBM-AT compatible computer with a fixed (hard) disk drive, a math co-processor, and at least 640K RAM memory. An enhanced graphics adapter (EGA) and a high resolution color monitor are necessary for colored display. The program will also run with a CGA or Hercules graphics card but the display will be monochrome. The program will also run without a math co-processor, but will run much slower due to the large number of mathematical calculations necessary.

The hydraulic simulation model consists of two executable files, CDAT and CANAL. CDAT is an editor where the physical dimensions of the canal system, the control structures, and the turnouts are entered. CANAL runs the hydraulic simulation and displays the results. More information about the function and operation of these two files is contained in the following sections. This information is intended only to illustrate some of the requirements for running the hydraulic simulation model, and some of the information that the model provides. For actual operation of the model, the reader is referred to "Users Manual for the Pascal Version of the USU Main System Hydraulic Model" (Merkley, 1987), from which this information has been drawn.

(A) CDAT

CDAT is the editor for the program CANAL. Data is entered into CDAT under four headings: (1) system dimensions; (2) configuration data; (3) control structure data; and (4) turnout structure data. The data files created using CDAT are collectively called configuration files in this report, and are located in Appendix A. The specific data which must be entered under each of the headings is listed below. All input into the program is in metric units (meters for length and cubic meters per second for flow rate). Other units are shown where used.

(1) System Dimensions
- Number of branches
- Number of reaches/branch
- Number of turnouts/reach

A reach is a section of canal separated from the adjoining sections of canal, generally by control structures, although a reach may also be defined with no control structure at the end. There is a maximum number of four branches, nine reaches/branch, and nine turnouts/reach.

(2) Configuration Data
- Base width
- Side slope
- Reach length
- Maximum canal depth
- Hydraulic roughness (Manning n)
- Longitudinal slope (m/100m)
- Operational supply level (%)
- Seepage rate (mm/day)

The cross section of the canal must be trapezoidal. If the actual canal cross section is not trapezoidal, the user must determine the trapezoidal dimensions which best represent the actual shape.

(3) Control Structure Data
- Control structure type
    - Circular culvert
    - Rectangular culvert
    - Sharp crested rectangular weir, fixed sill
    - Sharp crested rectangular weir, adjustable sill
    - Rectangular sluice gate
    - Circular sluice gate
- Number of control structures
- Cd1 -- free flow coefficient
- Cd2 -- submerged flow coefficient
- Cd3 -- weir overflow coefficient
- Width or diameter (sluice gates)
- Weir sill width
- Sill or culvert width
- Upstream elevation change
- Downstream elevation change

(4) Turnout Structure Data
- Turnout type
    - Circular orifice
    - Rectangular orifice
    - Sharp crested rectangular weir, adjustable sill
    - Sharp crested rectangular weir, fixed sill
- Discharge coefficient
- Width
- Height
- Distance from head of reach
- Position from canal bottom
- Downstream depth
- Downstream depth

(B) CANAL

CANAL runs the hydraulic simulation and displays the results. The simulation of a canal system begins with the filling of the canal. The program CANAL reads the file created with the editor CDAT. CANAL runs several checks on the data to make sure that the canal system which has been built is logically possible. The program runs six checks on turnout structures:

1) all turnouts must be within the reach specified.
2) the turnouts must be sequenced according to their location
3) weir sill height must be greater than one-half the maximum canal depth in each reach
4) turnout spacing must be greater than twice the combined widths of adjacent turnouts
5) turnout position must be below the top of the canal
6) downstream depth must be less than depth in the canal so backflow will not occur

The program also checks branch linkage data and compares it with system dimension data to ensure a feasible setup has been entered. If there are any errors the program displays an error message indicating the type and location of the error. The error must be corrected using CDAT before the canal may be filled. If there are no errors in the program the user can proceed with the simulation by inputting the inflow hydrograph to be used. The input is in 5 minute intervals, with a maximum time for each simulation of 12 hours and 30 minutes.

The operation of the control structures must also be specified before the simulation begins. There are three operational modes- manual, pre-set, and automatic. Manual mode allows the program operator to adjust check structure settings at any time during the simulation. If the pre-set mode is chosen the structure settings over the course of the simulation must be input at the beginning. In automatic mode control structure settings are calculated by the computer so that the downstream water level in each reach with an adjustable control structure matches the operational (full) supply level specified in CDAT. The computer uses an algorithm called "gate stroking", developed by Falvey and Luning (1979), to determine the correct settings.

(1) <u>Simulation Interruptions</u>

After a simulation has begun, it may be paused to change a number of the operating parameters. The user simply types one of the following letters, makes the adjustments necessary, and returns to the simulation at the same time step where the program was halted.

T - Turnout demands. User inputs the demand and the computer calculates the required setting based on water levels in the canal and the turnout parameters specified with CDAT. There is also a modified version of CANAL which allows the user to input the turnout setting, and the flow through the turnout is determined by the same methods.

C - Control structure setting. User may adjust any adjustable check in the reach currently being displayed.

P - Pause. Pauses the simulation at the current time step. No adjustments may be made.

H - Help. Loads the help menu onto the screen. The help menu is taken directly from the users manual.

I - Inflow. Allows the inflow hydrograph to be adjusted.

S - Scheduling mode. User can change operating mode to manual or automatic control for full supply levels, can freeze the current flow levels as the new full supply levels, or return to the simulation with no changes.

K - Keep. Stops the simulation at the current location and saves the data for viewing in the summary tables if desired. If this key is chosen it is not possible to continue the present simulation.

V - View. Displays the canal system configuration data file created with CDAT.

F - Full (operational) supply level. User may adjust specified full supply levels for the individual reaches in the branch currently being displayed.

(2) <u>Output during simulation</u>

CANAL provides information about the simulation while it is progressing. There are six different "alpha" screens which display numerical information in columns, with each column representing a reach. There is also a "graphic" screen, which provides a visual representation of the simulation. The program user may toggle from the graphic to the alpha screens using the "=" key.

(3) <u>Alpha Screens</u>

The screens are displayed by pressing the function keys F1 through F6. Information about one branch at a time is displayed. If there is more than one branch in the system being simulated, the user may move from branch to branch by typing the number of the branch (1, 2, 3, or 4). A brief description of the information contained in each screen is listed below.

1) System flow status. Screen lists the current flow conditions for each reach in the branch being displayed, including; upstream flow depth, downstream flow depth, target depth (specified by OSL), upstream flow rate, downstream flow rate, downstream flow over weir structures, turnout flow, seepage flow, control structure setting, and the "schedule" for the control structure. The screen also lists the system inflow and outflow.

2) System flow status. The screen is the same as (1) except the system inflow and outflow are replaced with numerical solution details, including flow regime and status, control structure type, computational node ranges, and various messages about the progress of the solution in each reach.

3) Turnout flow demands. Shows the demand for each turnout in the current branch and the sum of turnout demand in each reach. Also displays the total turnout flow in the current branch, total seepage flow in the current branch, and the system inflow and outflow. This screen has no meaning in the modified version of the program which allows the operator to input turnout structure settings instead of demand.

4) Actual turnout flow. Same display as (3) except the turnout flow demands are replaced by the actual flows. Branch turnout and seepage flow, as well as the system inflow and outflow are still displayed.

5) Turnout settings. Same display as (3) and (4) except the turnout settings replace the turnout demands (3) and actual flows (4).

6) Turnout Status. Displays the status of the turnouts in the current branch with one of the following codes.

OPEN - Turnout is completely open
---- - Turnout is a wasteway weir
WAIT - Turnout setting is changing
SHUT - Turnout is completely closed
BACK - Turnout backflow is impending
FREE - Turnout operates under free flow
SUBM - Turnout operates under submerged flow
SURF - Turnout is opened to upstream flow level

(4) Graphics Screen

The graphics screen displays three subplots. Flow profiles for the current branch are displayed across the top of the screen. Each reach is shown separated by a check structure, and the length of each reach is shown proportional to the branch length. An inflow bargraph is shown in the lower left corner of the screen. Two bars for each branch in the system are displayed. The left bar represents inflow into the branch, and the right bar represents average flow rate in the branch. The third subplot, displayed in the lower right corner of the screen, is an outflow bargraph. There are two bars shown for each reach in the current branch. The left bar represents the total turnout flow in the reach, and the right bar represents outflow at the downstream end of the reach. This bargraph may be replaced by outflow hydrograph curves that show the downstream flow for all reaches in the branch over the course of the simulation.

(1) SYSTEM FLOW STATUS   Configuration Data File: TEST
Branch 1 Step 123

	Reach Number								
	1	2	3	4	5	6	7	8	9
Ldepth	1.978	1.960	1.922	1.745	1.200	0.134			
Rdepth	2.125	2.083	2.040	2.040	1.459	0.000			
Tdepth	2.125	2.083	2.040	2.040	1.900	1.750			
Lflow	1.000	0.984	0.966	0.962	0.302	0.095			
Rflow	0.984	0.966	0.962	1.032	0.095	0.000			
Wflow	0.000	0.035	0.011	0.005	0.000	0.000			
Tflow	0.000	0.000	0.000	0.000	0.000	0.000			
Iflow	0.016	0.018	0.004	0.017	0.005	0.000			
Setting	0.448	0.446	0.133	0.220	0.057	0.030			
Schedule	On	On	On	On	Wait	Fill			

All units are in meters and seconds.   System Inflow = 1.000
System Outflow = 0.229

Figure 1. First alpha screen display.

(2) SYSTEM FLOW STATUS   Configuration Data File: TEST
Branch 1 Step 123

	Reach Number								
	1	2	3	4	5	6	7	8	9
Ldepth	1.978	1.960	1.922	1.745	1.200	0.134			
Rdepth	2.125	2.083	2.040	2.040	1.459	0.000			
Tdepth	2.125	2.083	2.040	2.040	1.900	1.750			
Lflow	1.000	0.984	0.966	0.962	0.302	0.095			
Rflow	0.984	0.966	0.962	1.032	0.095	0.000			
Wflow	0.000	0.035	0.011	0.005	0.000	0.000			
Tflow	0.000	0.000	0.000	0.000	0.000	0.000			
Iflow	0.016	0.018	0.004	0.017	0.005	0.000			
Setting	0.448	0.446	0.133	0.220	0.057	0.030			
Schedule	On	On	On	On	Wait	Fill			

NUMERICAL SOLUTION DETAILS

Reach : 1
Branch: 1
Status: Submerged Flow
Regime: Post Advance
Contrl: Rect Sluice
Nodes : 0 to 11

Figure 2. Second alpha screen display.

(3) TURNOUT FLOW DEMANDS Configuration Data File: TEST

Branch 1 Step 123

Reach Number

	1	2	3	4	5	6	7	8	9
1	-----	0.000	0.000	0.100	0.100	-----			
2	0.000	0.000	0.000	0.100	0.100	0.000			
3	0.000	0.000	0.000	0.200	0.250	0.100			
4	1.200	0.000	0.000	-----	0.300	0.000			
5	0.000	0.100	0.000		0.000	0.125			
6	0.500	0.100	-----		0.000	0.000			
7	0.000	-----			0.150	0.125			
8	-----	0.100				-----			
9	0.000	0.000							
SUM	1.700	0.300	0.000	0.400	0.900	0.350			

Total Turnout Flow 3.649 System Inflow = 7.000

Total Seepage Flow 0.085 System Outflow = 5.082

Units are cubic meters per second.

Figure 3. Third alpha screen display.

(4) ACTUAL TURNOUT FLOWS Configuration Data File: TEST

Branch 1 Step 123

Reach Number

	1	2	3	4	5	6	7	8	9
1	0.000	0.000	0.000	0.100	0.100	0.000			
2	0.000	0.000	0.000	0.100	0.100	0.000			
3	0.000	0.000	0.000	0.200	0.250	0.100			
4	1.199	0.000	0.000	0.000	0.300	0.000			
5	0.000	0.100	0.000		0.000	0.125			
6	0.500	0.100	0.000		0.000	0.000			
7	0.000	0.000			0.150	0.125			
8	0.000	0.100				0.000			
9	0.000	0.000							
SUM	1.699	0.300	0.000	0.400	0.900	0.350			

Total Turnout Flow 3.649 System Inflow = 7.000

Total Seepage Flow 0.085 System Outflow = 5.082

Units are cubic meters per second.

Figure 4. Fourth alpha screen display.

(5) TURNOUT SETTINGS Configuration Data File: TEST

Branch 1 Step 123

Reach Number

	1	2	3	4	5	6	7	8	9
1	-----	0.000	0.000	0.045	0.046	-----			
2	0.000	0.000	0.000	0.038	0.041	0.000			
3	0.000	0.000	0.000	0.088	0.100	0.035			
4	0.167	0.000	0.000	-----	0.114	0.000			
5	0.000	0.034	0.000		0.000	0.056			
6	0.101	0.033	-----		0.000	0.000			
7	0.000	-----			0.057	0.058			
8	-----	0.030				-----			
9	0.000	0.000							

Total Turnout Flow 3.649 System Inflow = 7.000

Total Seepage Flow 0.085 System Outflow = 5.082

Units are in meters.

Figure 5. Fifth alpha screen display.

(6) TURNOUT STATUS Configuration Data File: TEST

Branch 1 Step 123

Reach Number

	1	2	3	4	5	6	7	8	9
1	----	Shut	Shut	Free	Free	----			
2	Shut	Shut	Shut	Free	Subm	Shut			
3	Shut	Shut	Shut	Subm	Surf	Free			
4	Wait	Shut	Shut	----	Free	Shut			
5	Shut	Free	Shut		Shut	Free			
6	Subm	Free	----		Shut	Shut			
7	Shut	----			Free	Free			
8	----	Free				----			
9	Shut	Shut							
SUM	1.700	0.300	0.000	0.400	0.900	0.350			

Total Turnout Flow 3.649 System Inflow = 7.000

Total Seepage Flow 0.085 System Outflow = 5.082

Figure 6. Sixth alpha screen display.

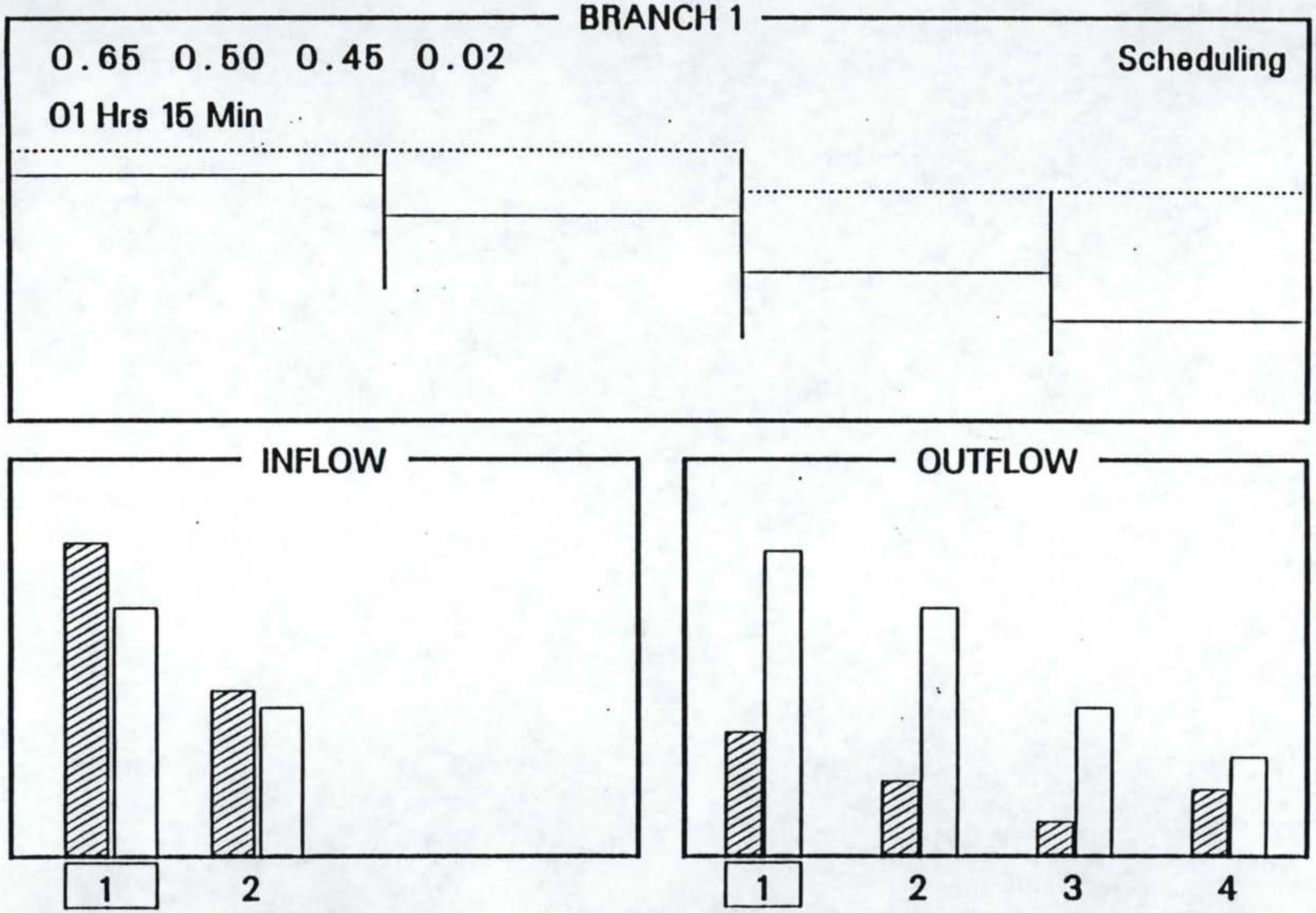

Figure 63. Graphics screen display with outflow bars.

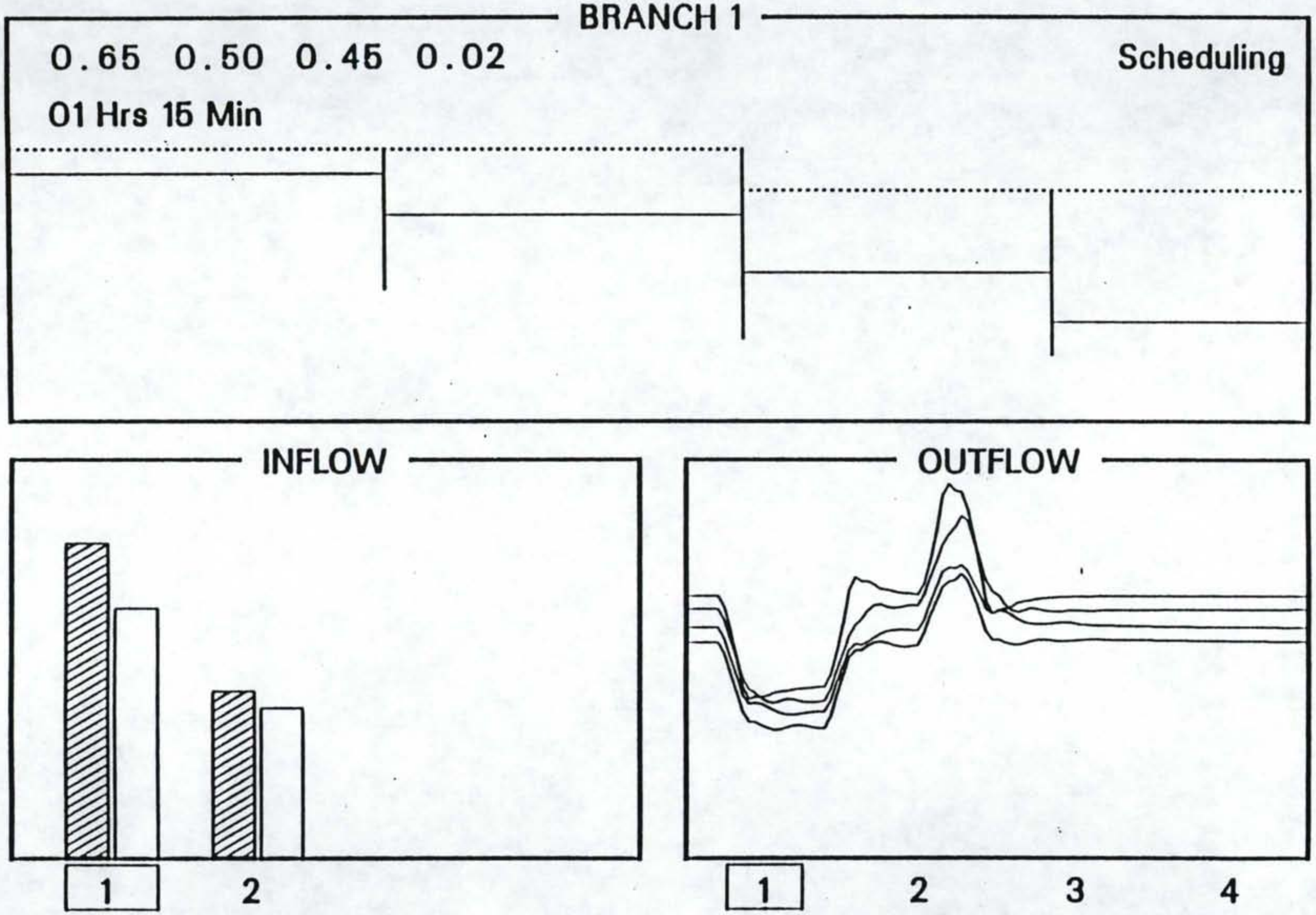

Figure 64. Graphics screen display with outflow curves.

APPENDIX B

CANAL CONFIGURATION FILES

LIST OF TABLES

2 BRANCHES		# TO	END POINT	LOCATION m	LENGTH m
BRANCH 1					
	Reach 1	2	10' DROP	8617	8617
	Reach 2	1	HIWAY 26	18206	9589
	Reach 3	9	ARMY WAY	25027	6821
	Reach 4	9	CHECK	29670	4643
	Reach 5	9	CHECK	32745	3075
BRANCH 2					
	Reach 1	5	400 SOUTH	36243	3498
	Reach 2	6	HILL RD	38682	2439
	Reach 3	7	1600 WEST	40123	1441
	Reach 4	5	CHECK	42572	2449
	Reach 5	3	JUDGE RD	43977	1405
	Reach 6	8	SPRFLD TABER	47445	3468
	Reach 7	7	STERLING NORTH	52681	5236
	Reach 8	9	IMAGINARY	53604	923
	Reach 9	5	CHECK	55217	1613

Table 10. Main Line System Dimensions.

BRANCH		1				
REACH		1	2	3	4	5
BASE WIDTH	m	18.6	15.8	20.4	18	18.5
SIDE SLOPE	run\rise	1	1	1	1	1
LENGTH	m	8617	9589	6821	4643	3075
MAX DEPTH	m	2.1	2.3	2.6	3	3.3
n		0.03	0.03	0.04	0.04	0.04
LONG SLOPE	m/100m	0.07	0.07	0.05	0.03	0.02
OSL	%	87	86	94	89	80
SEEPAGE	mm/day	350	500	2000	500	350

BRANCH		2								
REACH		1	2	3	4	5	6	7	8	9
BASE WIDTH	m	20.7	18.9	20.1	20.7	18.9	18.9	18	18.6	17.2
SIDE SLOPE	run\rise	1.25	1	1	1	1	1	1	1	1.25
LENGTH	m	3498	2439	1441	2449	1405	3468	5236	923	1613
MAX DEPTH	m	2.5	2.5	2.5	2.6	2.5	2.5	2.5	2.4	2.5
n		0.035	0.035	0.035	0.04	0.05	0.05	0.045	0.045	0.045
LONG SLOPE	m/100m	0.025	0.025	0.025	0.025	0.03	0.03	0.02	0.02	0.02
OSL	%	86	85	86	84	89	88	82	82	74
SEEPAGE	mm/day	350	350	500	500	500	2000	2000	1000	1000

Table 11. Main Line Configuration Data.

BRANCH	1				
REACH	1	2	3	4	5
Type	2	2	2	5	5
# Controls	1	1	1	1	1
Cd1-free	0.61	0.61	0.61	0.61	0.61
Cd2-submerged	1	1	1	0.61	0.61
Cd3-weir				1.83	1.83
Structure Wid	30	30	30	2.21	2
Weir Sill Wid				14.18	13
Weir Sill Height	3	3	3	1.6	1.6
US delta Z	0	0	0	0	0
DS delta Z	0	0	0	0.3	0.3
Setting Correction				0	0

BRANCH	2								
REACH	1	2	3	4	5	6	7	8	9
Type	2	2	2	5	2	2	2	2	5
# Controls	1	1	1	1	1	1	1	1	4
Cd1-free	1	1	1	0.61	1	1	1	1	0.61
Cd2-submerged	1	1	1	0.61	1	1	1	1	0.61
Cd3-weir				1.83					1.83
Structure Wid	25	25	25	1.75	20	20	20	20	1.22
Weir Sill Wid				12.25					2.28
Weir Sill Height	3	3	3	1	3	3	3	3	1.1
US delta Z	0	0	0	0	0	0	0	0	0
DS delta Z	0	0	0	0.3	0	0	0	0	0.3
Setting Correction				0					0

Table 12. Main Line Control Structures.

BRANCH	1		1	1								
REACH	1		2	3								
TURNOUT	1	2	1	1	2	3	4	5	6	7	8	9
	M1-6	M7-4	M8-2	M14-2	M14-4	M16-2	M16-4	M16-8	M16-10	M17-1	M17-3	M17-4
Type	1	1	1	1	1	1	1	1	1	1	1	1
Disch Coeff	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61
Width												
Height	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45
Distance	2691	8186	598	1658	2181	4438	4824	5525	6018	6295	6389	6523
Position	1	1.2	0	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3
DS Depth	1	1.2	0	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3
DS Slope	1	1	0	1	1	1	1	1	1	1	1	1

BRANCH	1									1								
REACH	4									5								
TURNOUT	1	2	3	4	5	6	7	8	9	1	2	3	4	5	6	7	8	9
	M18-5	M18-9	M18-10	M19-3	M19-9	M19-6	M19-10	M20-3	M20-5	M20-6	M20-4	M21-6	M21-7	M21-8	M22-8	22-9,10	ALAT	HLAT
Type	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
Disch Coeff	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61
Width																		
Height	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	1.22	1.22
Distance	1466	2301	2318	2864	3374	3394	3711	4488	4639	170	740	1401	2029	2348	2702	2854	2978	3020
Position	0	1.9	1.9	1.9	1.9	1.9	1.9	0	1.9	0	2.2	0	2.2	0	2.2	2.2	0	0
DS Depth	0	1.9	1.9	1.9	1.9	1.9	1.9	0	1.9	0	2.2	0	2.2	0	2.2	2.2	0	0
DS Slope	0	1	1	1	1	1	1	0	1	0	1	0	1	0	1	1	0	0

BRANCH	2					2						2						
REACH	1					2						3						
TURNOUT	1	2	3	4	5	1	2	3	4	5	6	1	2	3	4	5	6	7
	M22-12	M23-1	M23-5	M23-8	M23-9	M24-5	M24-7	M24-8	M24-10	M24-14	M25-1	M25-7	M25-8	M25-9	M26-3	45LAT	M26-8	M26-8A
Type	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
Disch Coeff	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61
Width																		
Height	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.61	0.45	0.45
Distance	775	1144	2019	2891	2964	308	647	667	1242	1863	2194	194	393	434	696	1349	1374	1427
Position	1.4	1.4	0	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	0.37	1.4	0
DS Depth	1.4	1.4	0	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	0.37	1.4	0
DS Slope	1	1	0	1	1	1	1	1	1	1	1	1	1	1	1	1	1	0

Table 13. Main Line Turnout Data.

BRANCH	2						2			2							
REACH	4						5			6							
TURNOUT	1	2	3	4	5	6	1	2	3	1	2	3	4	5	6	7	8
	M26-9	M26-10	C1LAT	CLAT	M27-5	SPILL	M28-6	M28-7	M29-2	M29-4	M29-6	M30-8	M30-9	M30-10	M31-1	M31-2	M31-3
Type	1	1	1	1	1	3	1	1	1	1	1	1	1	1	1	1	1
Disch Coeff	0.61	0.61	0.61	0.61	0.61	1.83	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61
Width						7.47											
Height	0.45	0.45	1.22	1.22	0.45		0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45
Distance	214	391	1245	1258	1857	2442	377	733	1160	52	988	1973	2195	2339	2974	3042	3221
Position	1.4	1.4	0.09	-0.3	1.4	1.3	1.4	1.4	0	1.4	0	1.4	1.4	1.4	0	1.4	1.4
DS Depth	1.4	1.4	0.09	-0.3	1.4		1.4	1.4	0	1.4	0	1.4	1.4	1.4	0	1.4	1.4
DS Slope	1	1	1	2	1		1	1	0	1	0	1	1	1	0	1	1

BRANCH	2							2								
REACH	7							8								
TURNOUT	1	2	3	4	5	6	7	1	2	3	4	5	6	7	8	9
	M32-3	M32-5	DLAT	M33-9	M33-10	M34-3	M34-4	M34-6	M34-5	M34-7	M34-8	M34-8A	M34-10	M35-3	M35-2A	M35-2
Type	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
Disch Coeff	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61
Width																
Height	0.45	0.45	0.91	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45
Distance	30	39	1610	4029	4097	4701	5235	12	116	243	275	323	565	721	731	738
Position	1.4	0	0.73	1.4	1.4	0	1.4	1.4	0	1.4	1.4	1.4	0	1.4	1.4	1.4
DS Depth	1.4	0	0.73	1.4	1.4	0	1.4	1.4	0	1.4	1.4	1.4	0	1.4	1.4	1.4
DS Slope	1	0	1	1	1	0	1	1	0	1	1	1	0	1	1	1

BRANCH	2							
REACH	9							
TURNOUT	1	2	3	4	5	6	7	8
	M35-4	M35-6	M35-9	M36-1	M36-2	SPILL	SPILL	LOW
Type	1	1	1	1	1	2	3	3
Disch Coeff	0.61	0.61	0.61	0.61	0.61	0.61	1.83	1.83
Width						1.5	8	6
Height	0.45	0.45	0.45	0.45	0.45	2		
Distance	188	394	1035	1435	1470	1540	1560	1590
Position	0	0	1.4	1.4	1.4	0.2	1.3	1.9
DS Depth	0	0	1.4	1.4	1.4	0		
DS Slope	0	0	1	1	1	0		

Table 13 (continued). Main Line Turnout Data.

3 BRANCHES

		# TO	END POINT	LOCATION m	LENGTH m
BRANCH 1					
	Reach 1	3	ANDERSON RD	1099	1099
	Reach 2	7	DIRT RD	3019	1920
	Reach 3	4	CHECK	5381	2362
	Reach 4	5	GRANDVIEW RD	7489	2108
	Reach 5	9	KENDALL RD	12196	4707
	Reach 6	7	CRATER SUB RD	14970	2774
	Reach 7	9	CHECK	18726	3756
BRANCH 2					
	Reach 1	3	CHECK	20093	1367
	Reach 2	4	CHECK	22290	2197
	Reach 3	9	CHECK	25600	3310
	Reach 4	6	CHECK	27869	2269
	Reach 5	4	CHECK	28891	1022
	Reach 6	7	FAIRVIEW RD	31827	2936
	Reach 7	6	IMAGINARY	36391	4564
	Reach 8	8	CHECK	39146	2755
BRANCH 3					
	Reach 1	6	CHECK	41225	2079
	Reach 2	2	CHECK	41776	551
	Reach 3	7	CHECK	43733	1957
	Reach 4	3	CHECK	45399	1666
	Reach 5	3	CHECK	48413	3014
	Reach 6	4	CHECK	48969	556

Table 14. High Line System Dimensions.

BRANCH		1						
REACH		1	2	3	4	5	6	7
BASE WIDTH	m	8.2	8.8	10	11.3	11.3	10.4	9.1
SIDE SLOPE	run\rise	1.25	1.25	1.25	1.25	1.25	1.25	1.25
LENGTH	m	1099	1920	2362	2108	4707	2774	3756
MAX DEPTH	m	3	2.8	2.1	2.1	2.1	2.1	2.1
n		0.04	0.04	0.04	0.04	0.04	0.04	0.04
LONG SLOPE	m/100m	0.02	0.02	0.02	0.02	0.02	0.02	0.02
OSL	%	85	85	85	85	85	85	85
SEEPAGE	mm/day	300	300	300	300	300	300	300

BRANCH		2							
REACH		1	2	3	4	5	6	7	8
BASE WIDTH	m	9.3	7	7.6	7	7	7	7	7
SIDE SLOPE	run\rise	1.25	1.25	1.25	1.25	1.25	1.25	1	1
LENGTH	m	1367	2197	3310	2269	1022	2936	3500	3819
MAX DEPTH	m	2.1	2.1	1.8	1.8	1.8	1.8	1.8	1.8
n		0.04	0.04	0.04	0.035	0.035	0.035	0.035	0.035
LONG SLOPE	m/100m	0.02	0.02	0.02	0.02	0.02	0.015	0.015	0.015
OSL	%	85	85	85	85	85	85	85	85
SEEPAGE	mm/day	300	300	300	300	300	300	300	300

BRANCH		3					
REACH		1	2	3	4	5	6
BASE WIDTH	m	7	5.8	5.8	4	4	3.7
SIDE SLOPE	run\rise	1	1	1	1	0.8	1
LENGTH	m	2079	551	1957	1666	3014	556
MAX DEPTH	m	1.7	1.6	1.5	1.3	1	1
n		0.035	0.035	0.035	0.035	0.035	0.035
LONG SLOPE	m/100m	0.015	0.015	0.015	0.015	0.015	0.015
OSL	%	85	85	85	85	85	85
SEEPAGE	mm/day	350	350	350	350	350	350

Table 15. High Line Configuration Data.

BRANCH	1						
REACH	1	2	3	4	5	6	7
Type	2	2	5	2	2	2	5
# Controls	1	1	1	1	1	1	1
Cd1-free	1	1	0.61	1	1	1	0.61
Cd2-submerged	1	1	0.61	1	1	1	0.61
Cd3-weir	1	1	1.83	1	1	1	1.83
Structure Wid	9	9	1.52	10	10	10	1.52
Weir Sill Wid			10				5.6
Weir Sill Height	3	3	1.1	3	3	3	1.1
US delta Z	0	0	0	0	0	0	0
DS delta Z	0	0	0.5	0	0	0	0.5
Setting Correction			0			0	

BRANCH	2							
REACH	1	2	3	4	5	6	7	8
Type	5	4	4	5	4	2	2	4
# Controls	1	6	6	1	5	1	1	2
Cd1-free	0.61			0.61		1	1	
Cd2-submerged	0.61			0.61		1	1	
Cd3-weir	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83
Structure Wid	1.55			1.42		8	10	
Weir Sill Wid	7.75	1.55	1.4	5.48	1.4			3.1
Weir Sill Height	1.1			1		2	2	
US delta Z	0	0	0	0	0	0	0	0
DS delta Z	0.3	0.3	0.3	0.3	0.3	0	0	0.3
Setting Correction	0			0				

BRANCH	3					
REACH	1	2	3	4	5	6
Type	5	5	5	4	4	3
# Controls	1	3	2	2	2	1
Cd1-free	0.61	0.61	0.61			
Cd2-submerged	0.61	0.61	0.61			
Cd3-weir	1.83	1.83	1.83	1.83	1.83	1.83
Structure Wid	1.52	1.19	1.22			
Weir Sill Wid	3.04	1	1	1.78	1.42	1.09
Weir Sill Height	1	2	2			1
US delta Z	0	0	0	0	0	0
DS delta Z	0.15	0.15	0.15	0.15	0.15	0.15
Setting Correction	0	0	0			

Table 16. High Line Control Structures.

BRANCH	1			1							1			
REACH	1			2							3			
TURNOUT	1	2	3	1	2	3	4	5	6	7	1	2	3	4
	M36-7	M36-8	M36-9	M37-1	M37-2	M37-4	M37-3	46LAT	M37-7	M37-8	M37-9	ELAT	M39-1	FLAT
Type	1	1	1	1	1	1	1	1	1	1	1	1	1	1
Disch Coeff	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61
Width														
Height	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.76	0.45	0.45	0.45	1.22	0.45	1.22
Distance	871	1030	1035	380	385	700	840	1156	1165	1381	612	765	1247	2218
Position	1.2	1.2	1.2	1.2	1.2	1.2	1.2	0.34	1.2	1.2	1.2	0.12	1.2	-0.61
DS Depth	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.16	1.2	1.2	1.2	0.12	1.2	0.37
DS Slope	1	1	1	1	1	1	1	0.5	1	1	1	0.5	1	1

BRANCH	1					1								
REACH	4					5								
TURNOUT	1	2	3	4	5	1	2	3	4	5	6	7	8	9
	M39-9	39-12	M40-2	M40-4	M40-7	M41-5	M41-7	41-14	41-15	41-16	M42-2	M42-4	M42-9	M43-1
Type	1	1	1	1	1	1	1	1	1	1	1	1	1	1
Disch Coeff	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61
Width														
Height	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45
Distance	283	393	1068	1276	1745	554	693	1586	1712	1835	2319	2483	3537	4393
Position	0	1.2	1.2	1.2	0	0	0	1.2	1.2	1.2	1.2	1.2	1.2	0
DS Depth	0	1.2	1.2	1.2	0	0	0	1.2	1.2	1.2	1.2	1.2	1.2	0
DS Slope	0	1	1	1	0	0	0	1	1	1	1	1	1	0

BRANCH	1							1							
REACH	6							7							
TURNOUT	1	2	3	4	5	6	7	1	2	3	4	5	6	7	8
	44-8A	M44-8	M44-9	44-10	44-11	44-12	M45-1	45-1A	GLAT	M46-1	ILAT	JLAT	M46-6	M46-9	M47-3
Type	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
Disch Coeff	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61
Width															
Height	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.53	0.45	0.45	1.07	0.45	0.45	0.45
Distance	803	966	1155	1762	2036	2324	2596	1170	1255	1286	1825	1839	2308	3082	3545
Position	0	1.2	1.2	1.2	1.2	1.2	1.2	1.2	0.61	1.2	0.76	-0.67	1.2	0	1.2
DS Depth	0	1.2	1.2	1.2	1.2	1.2	1.2	1.2	0.61	1.2	0.76	0.61	1.2	0	1.2
DS Slope	0	1	1	1	1	1	1	1	0.5	1	0.5	0.25	1	0	1

Table 17. High Line Turnout Data.

BRANCH	2			2				2							
REACH	1			2				3							
TURNOUT	1	2	3	1	2	3	4	1	2	3	4	5	6	7	8
	M47-6	M48-1	M48-2	48-12	M49-8	M49-6	M49-1	M49-9	ESPIL	M50-5	M50-7	M50-3	M51-1	M51-2	M51-6
Type	1	1	1	1	1	1	1	1	3	1	1	1	1	1	1
Disch Coeff	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	1.83	0.61	0.61	0.61	0.61	0.61	0.61
Width									4.57						
Height	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45		0.45	0.45	0.45	0.45	0.45	0.45
Distance	991	1017	1022	998	1282	1937	1942	371	1444	1458	1555	1727	2200	2433	2575
Position	0	0	0	0	0	0	0	-0.6	1.7	0.5	1	1	1	1	-0.6
DS Depth	0	0	0	0	0	0	0	0		0.5	1	1	1	1	0
DS Slope	0	0	0	0	0	0	0	0		1	1	1	1	1	0

BRANCH	2						2			
REACH	4						5			
TURNOUT	1	2	3	4	5	6	1	2	3	4
	51-10	M52-3	M52-8	52-10	47LAT	52-12	M53-9	53-10	53-12	5317A
Type	1	1	1	1	1	1	1	1	1	1
Disch Coeff	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61
Width										
Height	0.45	0.45	0.45	0.45	0.61	0.45	0.45	0.45	0.45	0.45
Distance	235	745	1364	1817	2254	2259	468	527	777	1010
Position	1	1	-0.6	-0.6	0	-0.6	-0.6	-0.6	-0.6	1
DS Depth	1	1	0	0	0	0	0	0	0	1
DS Slope	1	1	0	0	0	0	0	0	0	1

BRANCH	2							2					
REACH	6							7					
TURNOUT	1	2	3	4	5	6	7	1	2	3	4	5	6
	53-17	M54-5	54-10	NLAT	55-2A	M55-2	55-4A	M55-5	M55-6	M55-8	M56-2	M56-7	OLAT
Type	1	1	1	1	1	1	1	1	1	1	1	1	1
Disch Coeff	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61
Width													
Height	0.45	0.45	0.45	0.61	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45
Distance	514	880	1812	2089	2099	2342	2375	258	360	729	1249	2268	3399
Position	1	-0.6	-0.6	0.15	1	1	1	1	1	1	-0.6	1	0.9
DS Depth	1	0	0	0.76	1	1	1	1	1	1	0	1	0.9
DS Slope	1	0	0	0.5	1	1	1	1	1	1	0	1	1

Table 17 (continued). High Line Turnout Data.

BRANCH	2								
REACH	8								
TURNOUT	1	2	3	4	5	6	7	8	9
	SPILL	PLAT	58-12	58-15	58-17	M59-9	M59-2	59-10	5910A
Type	3	1	1	1	1	1	1	1	1
Disch Coeff	1.83	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61
Width	10								
Height		0.73	0.45	0.45	0.45	0.45	0.45	0.45	0.45
Distance	1020	1284	1729	2063	2188	2862	3127	3238	3248
Position	0.9	-0.46	-0.6	1	-0.6	-0.6	-0.6	1	1
DS Depth		-0.49	0	1	0	0	0	1	1
DS Slope		0.5	0	1	0	0	0	1	1

BRANCH	3						3		3						
REACH	1						2		3						
TURNOUT	1	2	3	4	5	6	1	2	1	2	3	4	5	6	7
	M60-6	QLAT	M60-8	M60-9	M61-3	M61-5	61-13	SPILL	61-22	M62-3	M62-6	62-10	62-15	SPILL	62-19
Type	1	1	1	1	1	1	1	3	1	1	1	1	1	3	1
Disch Coeff	0.61	0.61	0.61	0.61	0.61	0.61	0.61	1.83	0.61	0.61	0.61	0.61	0.61	1.83	0.61
Width								5.6						3	
Height	0.45	1.22	0.45	0.45	0.45	0.45	0.45		0.45	0.45	0.45	0.45	0.45		0.45
Distance	718	1087	1147	1516	1827	2075	413	549	483	951	1410	1636	1729	1945	1956
Position	1	-0.91	1	1	1	1	1	0.8	0.9	0.9	0.9	0.9	-0.9	0.8	-0.9
DS Depth	1	0.34	1	1	1	1	1		0.9	0.9	0.9	0.9	0		0
DS Slope	1	0.5	1	1	1	1	1		1	1	1	1	0		0

BRANCH	3			3			3			
REACH	4			5			6			
TURNOUT	1	2	3	1	2	3	1	2	3	4
	M63-4	M63-8	63-15	M64-4	RLAT	65-12	65-15	SLAT	HIGH	SPILL
Type	1	1	1	1	1	1	1	1	1	3
Disch Coeff	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	1.83
Width										3.28
Height	0.45	0.45	0.45	0.45	0.61	0.45	0.45	1.22	0.61	
Distance	744	924	1660	949	2471	2990	500	519	545	554
Position	0.7	-0.9	0.7	0.4	0.06	0.4	0.4	-1.65	0.1	0.5
DS Depth	0.7	0	0.7	0.4	-0.09	0.4	0.4	-0.34	0.1	
DS Slope	1	0	1	1	1	1	1	0.5	1	

Table 17 (continued). High Line Turnout Data.

2 BRANCHES

	# TO	END POINT	LOCATION m	LENGTH m
BRANCH 1				
Reach 1	6	CHECK	3612	
Reach 2	5	1100 SOUTH	7094	3485
Reach 3	7	1300 SOUTH	11421	4329
Reach 4	8	CHECK	14384	2964
Reach 5	7	CHECK	18681	4300
Reach 6	9	1700 SOUTH	22756	4075
Reach 7	6	CHECK	25087	2331
BRANCH 2				
Reach 1	9	1900 SOUTH	27999	2912
Reach 2	9	CHECK	30311	2312
Reach 3	6	CHECK	32421	2110

Table 18. Low Line System Dimensions.

BRANCH		1						
REACH		1	2	3	4	5	6	7
BASE WIDTH	m	8.4	7.6	8.5	7.6	6.3	6.9	7.3
SIDE SLOPE	run\rise	1	1	1	1.25	1	1.25	1
LENGTH	m	3612	3485	4329	2964	4300	4075	2331
MAX DEPTH	m	2.3	2	2	2	1.8	1.6	1.3
n		0.035	0.035	0.035	0.035	0.035	0.035	0.035
LONG SLOPE	m/100m	0.015	0.015	0.015	0.015	0.015	0.015	0.015
OSL	%	85	85	85	85	85	85	85
SEEPAGE	mm/day	300	300	300	300	300	300	300

BRANCH		2		
REACH		1	2	3
BASE WIDTH	m	7.5	6.6	5.9
SIDE SLOPE	run\rise	1	1	1
LENGTH	m	2912	2312	2110
MAX DEPTH	m	1.3	1.4	1.4
n		0.035	0.035	0.035
LONG SLOPE	m/100m	0.015	0.015	0.015
OSL	%	85	85	85
SEEPAGE	mm/day	300	300	300

Table 19. Low Line Configuration Data.

CONTROL STRUCTURES LOW LINE

BRANCH	1							2		
REACH	1	2	3	4	5	6	7	1	2	3
Type	4	2	2	4	4	2	4	2	5	4
# Controls	4	1	1	4	4	1	4	1	1	2
Cd1-free		1	1			1		1	0.61	
Cd2-submerged		1	1			1		1	0.61	
Cd3-weir	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83
Structure Wid		9	9			7		7	1.47	
Weir Sill Wid	1.53			1.36	1.68		1.22		1.47	1.4
Weir Sill Height		2	2			2		2	1	
US delta Z	0	0	0	0	0	0	0	0	0	0
DS delta Z	0.15	0	0	0.15	0.15	0	0.15	0	0.15	0.15
Setting Correction									0	

Table 20. Low Line Control Structures.

BRANCH	1						1				
REACH	1						2				
TURNOUT	1	2	3	4	5	6	1	2	3	4	5
	L0-1	L0-4	L0-6	L0-8	L1-8	L1-10	L2-6	L2-8	L3-1	L3-1A	TLAT
Type	1	1	1	1	1	1	1	1	1	1	1
Disch Coeff	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61
Width											
Height	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.61
Distance	170	697	1293	2180	3206	3606	630	660	1433	1444	2740
Position	0	1.1	1.1	0	1.1	1.1	0.8	0.8	0.8	0	0.24
DS Depth	0	1.1	1.1	0	1.1	1.1	0.8	0.8	0.8	0	0
DS Slope	0	1	1	0	1	1	1	1	1	0	1

BRANCH	1							1							
REACH	3							4							
TURNOUT	1	2	3	4	5	6	7	1	2	3	4	5	6	7	8
	L4-2	Lt0-9	L6-4	L6-5	L6-8	L6-9	L6-10	L7-2	L7-3	L7-6	L7-7	L8-2	L8-6	L8-4	U LAT
Type	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
Disch Coeff	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61
Width															
Height	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.76
Distance	99	495	3308	3313	3727	4050	4203	12	16	1158	1165	1964	2558	2820	2830
Position	0	0	0.8	0	0.8	0.8	0.8	0	0.8	0	0.8	0.8	0	0	0.09
DS Depth	0	0	0.8	0	0.8	0.8	0.8	0	0.8	0	0.8	0.8	0	0	0.12
DS Slope	0	0	1	0	1	1	1	0	1	0	1	1	0	0	1

Table 21. Low Line Turnout Data.

BRANCH	1						
REACH	5						
TURNOUT	1	2	3	4	5	6	7
	L9-5	VLAT	L9-9	L10-5	L10-6	L10-9	L11-2

Type	1	1	1	1	1	1	1
Disch Coeff	0.61	0.61	0.61	0.61	0.61	0.61	0.61
Width							
Height	0.45	1.22	0.45	0.45	0.45	0.45	0.45
Distance	339	1337	2026	2371	2638	3397	3734
Position	0	-0.46	0	0	0	0.8	0.8
DS Depth	0	0.3	0	0	0	0.8	0.8
DS Slope	0	0.5	0	0	0	1	1

BRANCH	1									1					
REACH	6									7					
TURNOUT	1	2	3	4	5	1	2	3	4	1	2	3	4	5	6
	L11-9	L12-3A	L12-5	L12-6	SPILL	L12-8	L12-9	L12-10	L13-6	L13-3	L14-6	L14-8	L15-1	L15-2	WLAT

Type	1	1	1	1	3	1	1	1	1	1	1	1	1	1	1
Disch Coeff	0.61	0.61	0.61	0.61	1.83	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61
Width					1.52										
Height	0.45	0.45	0.45	0.45		0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.76
Distance	1022	1199	1687	1798	2007	2327	2466	2648	4063	15	1199	1700	2115	2119	2202
Position	0	0	0.6	0.6	0.9	0.6	0	0	0	0	0.45	0.45	0.45	0	-0.24
DS Depth	0	0	0.6	0.6		0.6	0	0	0	0	0.45	0.45	0.45	0	-0.12
DS Slope	0	0	1	1		1	0	0	0	0	1	1	1	0	1

BRANCH	2								
REACH	1								
TURNOUT	1	2	3	4	5	6	7	8	9
	L15-4	L15-8	L16-1	L16-1A	L16-2	L16-2B	L16-7A	L16-8	L16-13
------------						L16-2A		L16-9	
Type	1	1	1	1	1	1	1	1	1
Disch Coeff	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61
Width									
Height	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45
Distance	660	1001	1099	1312	1502	1535	2333	2357	2843
Position	0	0	0	0.45	0.45	0	0.45	0	0.45
DS Depth	0	0	0	0.45	0.45	0	0.45	0	0.45
DS Slope	0	0	0	1	1	0	1	0	1

BRANCH	2									2					
REACH	2									3					
TURNOUT	1	2	3	4	5	6	7	8	9	1	2	3	4	5	6
	L17-2	L17-3	SPILL	L17-10	L17-7	L17-9	XLAT	L18-3	L18-7	L18-12	L19-6	L19-4	L19-9	L19-10A	L19-11
------------									L18-5						
Type	1	1	3	1	1	1	1	1	1	1	1	1	1	1	1
Disch Coeff	0.61	0.61	1.83	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61
Width			1.52												
Height	0.45	0.45		0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45
Distance	224	683	753	925	930	1355	1360	2253	2306	554	1524	1965	1970	2082	2102
Position	0.45	0.45	0.7	0	0	0	0.45	0	0	0.45	0	0.45	0	0.45	0.45
DS Depth	0.45	0.45		0	0	0	0.45	0	0	0.45	0	0.45	0	0.45	0.45
DS Slope	1	1		0	0	0	1	0	0	1	0	1	0	1	1

Table 21 (continued). Low Line Turnout Data.

1 BRANCH

		# TO	END POINT	LOCATION m	LENGTH m
BRANCH 1					
	Reach 1	4	CHECK	2009	2009
	Reach 2	1	CHECK	2793	784
	Reach 3	1	CHECK	3529	736
	Reach 4	1	CHECK	4282	753
	Reach 5	2	CHECK	4689	407
	Reach 6	3	CHECK	5836	1147
	Reach 7	3	CHECK	6696	860
	Reach 8	5	EOC	8447	1751

Table 22. C Lateral System Dimensions.

BRANCH		1							
REACH		1	2	3	4	5	6	7	8
BASE WIDTH	m	3.5	3.5	4	4	4.3	2.9	2	2
SIDE SLOPE	run\rise	1	1	1	1	1	1	1	1.25
LENGTH	m	2009	784	736	753	407	1147	860	1756
MAX DEPTH	m	1.5	1.3	0.9	0.9	0.9	0.9	1.1	1.2
n		0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04
LONG SLOPE	m/100m	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.005
OSL	%	55	75	75	75	75	75	75	80
SEEPAGE	mm/day	275	275	275	275	275	275	275	275

Table 23. C Lateral Configuration Data.

BRANCH	1							
REACH	1	2	3	4	5	6	7	8
Type	4	4	4	4	4	4	4	3
# Controls	2	1	2	2	1	2	1	1
Cd1-free								
Cd2-submerged								
Cd3-weir	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83
Structure Wid								
Weir Sill Wid	1.5	1.96	1.4	1.4	2.41	1.52	1.22	1
Weir Sill Height								1.2
US delta Z	0	0	0	0	0	0	0	0
DS delta Z	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0
Setting Correction								

Table 24. C Lateral Control Structures.

BRANCH	1				1	1	1	1		1		
REACH	1				2	3	4	5		6		
TURNOUT	1	2	3	4	1	1	1	1	2	1	2	3
	C0-5	C0-6	C1-5	C1-7	C5LAT	C1-14	C2-1	C2-5	C2-4	C2-7	C2-8	C3-4
Type	1	1	1	1	1	1	1	1	1	1	1	1
Disch Coeff	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61
Width												
Height	0.45	0.45	0.45	0.45	0.61	0.45	0.45	0.45	0.45	0.45	0.45	0.45
Distance	806	1542	1839	1931	644	730	576	153	347	500	794	1141
Position	0.45	0.45	0.45	0.45	0.15	-0.9	-0.9	-0.9	-0.9	-0.9	-0.9	-0.9
DS Depth	0.45	0.45	0.45	0.45	0.5	0	0	0	0	0	0	0
DS Slope	1	1	1	1	1	0	0	0	0	0	0	0

BRANCH	1			1				
REACH	7			8				
TURNOUT	1	2	3	1	2	3	4	5
	C3-5	C3-8	C6LAT	C4-1	C4-3	C4-8	SPILL	C4-10
Type	1	1	1	1	1	1	3	1
Disch Coeff	0.61	0.61	0.61	0.61	0.61	0.61	1.83	0.61
Width							1.22	
Height	0.45	0.45	0.45	0.45	0.45	0.45		0.45
Distance	301	476	855	337	808	1336	1623	1751
Position	0.45	0.45	0.45	-0.9	0.45	0.45	0.6	0.45
DS Depth	0.45	0.45	0.5	0	0.45	0.45		0.45
DS Slope	1	1	1	0	1	1		1

Table 25. C Lateral Turnout Data.

SYSTEM DIMENSIONS C1 LATERAL

1 BRANCH

		# TO	END POINT	LOCATION m	LENGTH m
BRANCH 1					
	Reach 1	2	CHECK	2247	2247
	Reach 2	5	CHECK	4247	2000
	Reach 3	1	CHECK	4731	484
	Reach 4	1	CHECK	5187	456
	Reach 5	3	CHECK	5724	537
	Reach 6	1	CHECK	6632	908
	Reach 7	3	CHECK	7745	1113

Table 26. C1 Lateral System Dimensions.

BRANCH REACH		1 1	2	3	4	5	6	7
BASE WIDTH	m	3.4	3.3	3.3	3.3	2.9	2.5	2.5
SIDE SLOPE	run\rise	1	1	1	1	1	1	1
LENGTH	m	2247	2000	484	456	537	908	1112
MAX DEPTH	m	1.4	1	1	1	1	1	1
n		0.04	0.04	0.04	0.04	0.04	0.04	0.04
LONG SLOPE	m/100m	0.03	0.03	0.03	0.03	0.03	0.03	0.015
OSL	%	70	80	80	80	80	80	85
SEEPAGE	mm/day	275	275	275	275	275	275	275

Table 27. C1 Lateral Configuration Data.

CONTROL STRUCTURES C1 LATERAL

BRANCH	1						
REACH	1	2	3	4	5	6	7
Type	4	4	4	4	4	4	3
# Controls	2	2	2	1	1	1	1
Cd1-free							
Cd2-submerged							
Cd3-weir	1.83	1.83	1.83	1.83	1.83	1.83	1.83
Structure Wid							
Weir Sill Wid	1.19	1.22	1.47	1.55	1.4	1.42	1
Weir Sill Height							
US delta Z	0	0	0	0	0	0	0
DS delta Z	0.15	0.15	4	0.15	0.15	0.15	0
Setting Correction							

Table 28. C1 Lateral Control Structures.

BRANCH	1		1					1	1	1			1	1		
REACH	1		2					3	4	5			6	7		
TURNOUT	1	2	1	2	3	4	5	1	1	1	2	3	1	1	2	3
	C1-0-3	C2LAT	C1-1-5	C1-1-7	C1-2-1	C1-2-2	C1-2-3	C1-2-6	C1-2-9	C1-3-1	C1-3-2	C1-3-3	C1-3-8	C3LAT	SPILL	C1-4-1
Type	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
Disch Coeff	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61
Width																
Height	0.45	0.61	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45
Distance	904	2237	211	486	1372	1379	1725	480	449	122	469	532	901	1090	1102	1107
Position	-0.9	0.15	-0.9	0.3	-0.9	-0.9	0.45	0.45	-0.9	0.45	-0.9	0.45	0.45	0.45	0.5	-0.9
DS Depth	0	0.5	0	0.3	0	0	0.45	0.45	0	0.45	0	0.45	0.45	0.45		0
DS Slope	0	1	0	1	0	0	1	1	0	1	0	1	1	1		0

Table 29. C1 Lateral Turnout Data.

2 BRANCHES		# TO	END POINT	LOCATION m	LENGTH m
BRANCH 1					
	Reach 1	2	CHECK	1865	1865
	Reach 2	6	CHECK	3281	1416
	Reach 3	2	CHECK	4599	1318
	Reach 4	3	EOC	6321	1722
BRANCH 2					
	Reach 1	6	CHECK	1816	1816
	Reach 2	3	EOC	2288	472

Table 30. E Lateral System Dimensions.

BRANCH		1				2	
REACH		1	2	3	4	1	2
BASE WIDTH	m	4.4	2.9	2.9	2.9	1.5	1.5
SIDE SLOPE	run\rise	1	1.25	1.25	1.25	1	1
LENGTH	m	1865	1416	1318	1722	1816	467
MAX DEPTH	m	1.3	1.2	1.2	1.2	1.2	1.2
n		0.04	0.04	0.04	0.04	0.04	0.04
LONG SLOPE	m/100m	0.01	0.01	0.01	0.01	0.01	0.01
OSL	%	85	85	85	85	85	85
SEEPAGE	mm/day	275	275	275	275	275	275

Table 31. E Lateral Configuration Data.

BRANCH	1				2	
REACH	1	2	3	4	1	2
Type	4	4	4	3	3	3
# Controls	2	1	1	1	1	1
Cd1-free						
Cd2-submerged						
Cd3-weir	1.83	1.83	1.83	1.83	1.83	1.83
Structure Wid						
Weir Sill Wid	1.07	1.68	0.76	1	1.52	1
Weir Sill Height				1.5	0.76	1.5
US delta Z	0	0	0	0	0	0
DS delta Z	0.15	0.15	0.15	0	0.15	0
Setting Correction						

Table 32. E Lateral Control Structures.

BRANCH	1		1						1		1		
REACH	1		2						3		4		
TURNOUT	1	2	1	2	3	4	5	6	1	2	1	2	3
	E1LAT	E1-3	E1-6	E1-7	E1-10	E1-11	E1-16	E1-17	E2-5	E2-6	SPILL	E3-10	E3-15
Type	1	1	1	1	1	1	1	1	1	1	3	1	1
Disch Coeff	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	1.83	0.61	0.61
Width											1.37		
Height	0.91	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45		0.45	0.45
Distance	1671	1766	127	345	685	1091	1392	1413	740	1193	1278	1712	1717
Position	0.3	0.45	0.45	-0.9	0.45	-0.9	0.45	-0.9	-0.9	-0.9	0.6	0.45	0.45
DS Depth	0.3	0.45	0.45	0	0.45	0	0.45	0	0	0		0.45	0.45
DS Slope	0.5	1	1	0	1	0	1	0	0	0		1	1

BRANCH	2						2		
REACH	1						2		
TURNOUT	1	2	3	4	5	6	1	2	3
	E10-1	E10-2	E10-6	E10-8	E10-9	E11-1	E11-4	E11-7	E11-8
Type	1	1	1	1	1	1	1	1	1
Disch Coeff	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61
Width									
Height	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45
Distance	41	325	522	978	983	1120	453	460	467
Position	0.45	-0.9	-0.9	0.45	0.45	0.45	0.45	0.45	0.45
DS Depth	0.45	0	0	0.45	0.45	0.45	0.45	0.45	0.45
DS Slope	1	0	0	1	1	1	1	1	1

Table 33. E Lateral Turnout Data.

1 BRANCH	# TO	END POINT	LOCATION m
BRANCH 1			
Reach 1	0	CHECK	853
Reach 2	3	CHECK	1617
Reach 3	6	EOC	2324

Table 34. G Lateral System Dimensions.

BRANCH		1		
REACH		1	2	3
BASE WIDTH	m	2.7	2.3	1.9
SIDE SLOPE	run\rise	1	1	1
LENGTH	m	853	764	702
MAX DEPTH	m	0.9	0.9	1.1
n		0.04	0.04	0.04
LONG SLOPE	m/100m	0.02	0.02	0.01
OSL	%	85	85	85
SEEPAGE	mm/day	275	275	275

Table 35. G Lateral Configuration Data.

BRANCH	1		
REACH	1	2	3
Type	4	4	3
# Controls	1	1	1
Cd1-free			
Cd2-submerged			
Cd3-weir	1.83	1.83	1.83
Structure Wid			
Weir Sill Wid	1.6	1.12	1
Weir Sill Height			1.5
US delta Z	0	0	0
DS delta Z	0.15	0.15	0
Setting Correction			

Table 36. G Lateral Control Structures.

BRANCH	1	1			1					
REACH	1	2			3					
TURNOUT		1	2	3	1	2	3	4	5	6
	NO TURNOUTS	G0-7	G0-10	G0-10A	G0-12	G1-7	G1-8A	G1-9	G1-10	SPILL
Type		1	1	1	1	1	1	1	1	3
Disch Coeff		0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	1.83
Width										2.06
Height		0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	
Distance		109	228	510	15	409	433	685	692	702
Position		0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.6
DS Depth		0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	
DS Slope		1	1	1	1	1	1	1	1	

Table 37. G Lateral Turnout Data.

2 BRANCHES				LOCATION	LENGTH
		# TO	END POINT	m	m
BRANCH 1					
	Reach 1	2	CHECK	609	609
	Reach 2	3	CHECK	1217	608
	Reach 3	1	CHECK	1408	191
	Reach 4	1	CHECK	1894	486
	Reach 5	9	CHECK	3616	1722
BRANCH 2					
	Reach 1	6	EOC	1049	1049

Table 38. Q Lateral System Dimensions.

BRANCH		1					2
REACH		1	2	3	4	5	1
BASE WIDTH	m	2.1	2.7	2.7	2.7	2.3	1.4
SIDE SLOPE	run\rise	1.25	1.25	1.25	1.25	1.25	1.25
LENGTH	m	609	608	191	486	1722	1049
MAX DEPTH	m	1.5	1.4	1.4	1.4	1.4	2.1
n		0.04	0.04	0.04	0.04	0.04	0.04
LONG SLOPE	m/100m	0.02	0.02	0.02	0.02	0.01	0.005
OSL	%	65	70	80	80	80	80
SEEPAGE	mm/day	300	300	300	300	300	300

Table 39. Q Lateral Configuration Data.

BRANCH	1					2
REACH	1	2	3	4	5	1
Type	4	4	4	4	3	3
# Controls	1	1	1	1	1	1
Cd1-free						
Cd2-submerged						
Cd3-weir	1.83	1.83	1.83	1.83	1.83	1.83
Structure Wid						
Weir Sill Wid	1.63	1.93	1.93	1.93	1.52	1
Weir Sill Height					1.5	2.5
US delta Z	0	0	0	0	0	0
DS delta Z	0.15	0.15	0.15	0.15	0.15	0.15
Setting Correction						

Table 40. Q Lateral Control Structures.

BRANCH	1		1			1	1	1								
REACH	1		2			3	4	5								
TURNOUT	1	2	1	2	3	1	1	1	2	3	4	5	6	7	8	9
	Q0-2	Q0-3	Q0-5	Q0-7	Q0-9	Q0-10	Q1-1	SPILL	Q1-3	Q1-5	Q1-6	Q1-7	Q1-8	Q1LAT	Q2-3	END Q
Type	1	1	1	1	1	1	1	4	1	1	1	1	1	1	1	1
Disch Coeff	0.61	0.61	0.61	0.61	0.61	0.61	0.61	1.83	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61
Width								3.45								
Height	0.45	0.45	0.45	0.45	0.45	0.45	0.45		0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.73
Distance	352	602	423	568	605	187	482	5	39	248	381	535	1008	1550	1707	1715
Position	0	0.45	0.45	0.45	0.45	0.45	0.45	1.1	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45
DS Depth	0	0.3	0.3	0.3	0.3	0.3	0.3		0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
DS Slope	0	1	1	1	1	1	1		1	1	1	1	1	1	1	0.5

BRANCH	2					
REACH	1					
TURNOUT	1	2	3	4	5	6
	Q10-4	Q10-10	Q10-9	Q10-12	Q10-13	SPILL
Type	1	1	1	1	1	3
Disch Coeff	0.61	0.61	0.61	0.61	0.61	1.83
Width						1.27
Height	0.45	0.45	0.45	0.45	0.45	
Distance	187	1006	1011	1029	1039	1044
Position	0.45	0.45	0.45	0.45	0.45	1.1
DS Depth	0.3	0.3	0.3	0.3	0.3	
DS Slope	1	1	1	1	1	

Table 41. Q Lateral Turnout Data.

1 BRANCH		# TO	END POINT	LOCATION m	LENGTH m
BRANCH 1					
	Reach 1	1	CHECK	576	576
	Reach 2	9	CHECK	3585	3009
	Reach 3	6	CHECK	5455	1870
	Reach 4	8	EOC	7118	1663

Table 42. V Lateral System Dimensions.

BRANCH REACH		1 1	 2	 3	 4
BASE WIDTH	m	3.6	3.7	3.2	2.8
SIDE SLOPE	run\rise	1.25	1.25	1.25	1.25
LENGTH	m	576	3009	1870	1672
MAX DEPTH	m	1.5	1.2	1.1	1
n		0.04	0.04	0.04	0.04
LONG SLOPE	m/100m	0.02	0.02	0.01	0.01
OSL	%	65	75	85	85
SEEPAGE	mm/day	275	275	275	275

Table 43. V Lateral Configuration Data.

BRANCH REACH	1 1	 2	 3	 4
Type	4	4	4	3
# Controls	3	2	2	1
Cd1-free				
Cd2-submerged				
Cd3-weir	1.83	1.83	1.83	1.83
Structure Wid				
Weir Sill Wid	1.41	1.42	1.83	1
Weir Sill Height				1
US delta Z	0	0	0	0
DS delta Z	0	0.07	0.15	0
Setting Correction				

Table 44. V Lateral Control Structures.

BRANCH	1	1									1					
REACH	1	2									3					
TURNOUT	1	1	2	3	4	5	6	7	8	9	1	2	3	4	5	6
	V0-2	V0-10	V1-4	V1-9	V1-10	V4LAT	V1-17	V1-18	V1-19	V3LAT	V2-7	V1LAT	V2LAT	V2-13	V3-5	V3-6
Type	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
Disch Coeff	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61
Width																
Height	0.45	0.45	0.45	0.45	0.45	0.61	0.45	0.45	0.45	0.61	0.45	0.61	0.45	0.45	0.45	0.45
Distance	330	809	959	1627	1661	1666	2550	2555	2560	2751	988	1013	1428	1822	1862	1867
Position	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.3	0.15	0.15	0.15
DS Depth	0	0.1	0	0.1	0.1	0.1	0	0.1	0.1	0.1	0	0.1	0.3	0	0.1	0.1
DS Slope	0	1	0	1	1	0.5	0	1	1	0.5	0	0.5	1	0	1	1

BRANCH	1							
REACH	4							
TURNOUT	1	2	3	4	5	6	7	8
	V3-8	V3-9	V3-12	V3-13	V3-15	SPILL	V4-6	V4-7
Type	1	1	1	1	1	3	1	1
Disch Coeff	0.61	0.61	0.61	0.61	0.61	1.83	0.61	0.61
Width						3.66		
Height	0.45	0.45	0.45	0.45	0.45		0.45	0.45
Distance	150	476	913	1124	1377	1653	1663	1668
Position	0.15	0.15	0.15	0.15	0.15	0.5	0.15	0.15
DS Depth	0	0	0.1	0.1	0.1		0	0
DS Slope	1	0	1	1	1		0	0

Table 45. V Lateral Turnout Data.