University of Washington
Department of Civil and Environmental Engineering

THE USE OF SIMULATION MODELING TO EVALUATE THE EFFECT OF REGIONALIZATION ON WATER SYSTEM PERFORMANCE

Lawrence M. Karpack

Water Resources Series
Technical Report No.134
March 1992

Seattle, Washington
98195

Department of Civil Engineering
University of Washington
Seattle, Washington 98195

THE USE OF SIMULATION MODELING TO EVALUATE THE EFFECT OF REGIONALIZATION ON WATER SYSTEM PERFORMANCE

Lawrence M. Karpack

Water Resources Series
Technical Report No. 134

March 1992

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

Acknowledgments

The author would like to express sincere thanks to a number of individuals and groups who made the accomplishment of this thesis possible. Thanks go to the author's Committee members, Drs. Richard N. Palmer, Richard R. Horner, and Brian W. Mar. Special thanks go to Committee Chairman Dr. Richard Palmer who provided information, encouragement, and direction that helped make this effort successful. Financial support that enabled the author to complete this thesis was provided by the Center for Urban Water Resources Management directed by Richard Horner. Thanks go to the Tacoma Water Division, particularly John Kirner and Craig Gibson for providing information that enabled the author to understand and model the Tacoma water system. Likewise, thanks go to the Seattle Water Department, particularly David Parkinson and Brian Spindor for supplying information about the operation of the Seattle water system. Thanks go to the author's colleagues, especially those with whom he shared an office, for helping to make the past two years an enjoyable experience. Finally, the author would like to express a very special word of thanks to his wife Karen, who provides support and love (and commas) and stands beside him in all that he does.

Chapter I
Introduction

This thesis investigates physical performance measures and their practical application to a regional water resource system for western Washington. Specific criteria for characterizing the physical yield and reliability of a system are developed. The impact that a regionally intertied and jointly operated water supply network would have on system performance as measured by these criteria is analyzed.

Simulation models are developed to evaluate the benefits of this intertie. These models allow comparison of the performance of a combined system with that of the existing individual systems with respect to firm water yield and probability of failure at alternative demand levels. This study documents the trade-offs among specific performance measures of different methods of operation and discusses the implications for various competing interests.

In the introductory chapter the problem is identified, and the scope and approach of the study are defined. Chapter 2 provides a review of literature describing performance criteria, modeling, operating policies, and instream flow requirements. The physical characteristics of the systems that were studied are detailed in Chapter 3. A historical perspective of the concept of an intertie, as well as anticipated benefits of such a connection, are discussed in Chapter 4. The next chapter describes the graphical simulation environment, STELLA II™, which was used to model the systems and highlights important features of the formulation for this study. Results of the research are presented in Chapter 6, while Chapter 7 documents the conclusions of this study and provides directions for future research.

Problem definition

Development and population growth in western Washington have occurred at an unprecedented rate over the last decade. This growth has placed stress on the region's infrastructure, creating a situation where current facilities are not always capable of meeting the population's needs. This phenomenon is most visibly evidenced by the difficulties in commuting on overcrowded freeways. Other impacts can be seen in the region's air quality problems and land management practices. Perhaps less obvious, is the effect that rapid growth has had on the area's water supply systems. Increasing competition for current water diversions (water removed from the source for consumptive purposes), coupled with heightened awareness of the value of instream flow (water retained in the river for biological, assimilative, recreational, and other purposes), have stressed existing systems to the point where reliability of the water supply is no longer assured.

The allocation of limited water resources must address a wide variety of competing uses. In the river basins of western Washington, instream uses include fish and wildlife habitat protection, navigation, water quality considerations, recreation, hydroelectric power generation, and preservation of scenic beauty. Diversionary demands include municipal and industrial (M&I) supply and agricultural withdrawals. The benefits of each of these uses must also be weighed against flood control capabilities and federally mandated storage volumes when determining release policies for the region's reservoirs. The complex interaction among economic and environmental concerns underscores the need for a comprehensive management plan to maintain the delicate balance among diversions, storage and instream flow.

Municipal water supplies in western Washington are vulnerable to failure due to the effects of three hydrologic scenarios. These are (1) a light winter snowpack combined with below average spring rains, (2) a warm, dry, late spring and early summer or (3) a dry fall [CH2M Hill, 1974]. In the first scenario, inadequate spring runoff does not permit refilling reservoirs from winter flood pool level to the higher summer operating pool. This can lead to shortfalls in any month, particularly high demand months such as July and August. The second scenario is similar to the first. Spring runoff is adequate, but it occurs early in the spring while the reservoir flood pool is maintained. If this early runoff is followed by a warm, dry period, with low rainfall and high demands, the stored water can be depleted quickly, and the system is particularly vulnerable to failure in June, July and August. The third scenario is probably the most common cause for utilities' concern. If fall rains begin late or are significantly below normal, the lack of sufficient storage can leave the system without an adequate supply. This scenario can lead to failures in November or December. It should be noted that as regional demands escalate, these phenomena will intensify. The usual low summer inflows, combined with high residential peak demands, will cause rapid drawdown in August, September, and October.

In the past, municipal water system reliability questions were often addressed by developing new sources. Since the early 1970's, increased understanding of the extensive environmental impacts of many source options has forced municipalities to seek other solutions. Single purpose water supply reservoirs are no longer a viable answer for meeting escalating municipal demands. Creative operational strategies designed to make the best use of existing sources, combined with improved conservation measures, offer promising alternative means for meeting these demands.

One such strategy is the cooperative operation of existing water supplies among several utilities within western Washington. One coalition of utilities that could benefit from an intertie would include the region's two largest municipal and industrial water suppliers; Seattle and Tacoma. The geographic proximity of service areas and supply source diversity of these utilities provide the potential for maximum benefits at reasonable costs.

Consideration for improved use of the existing water supply is not without precedent in the area. The municipalities of western Washington have long sought non-structural methods for managing water resources. In the early 1970's the Municipality of Metropolitan Seattle (Metro) and the Seattle Water Department (SWD) coordinated an extensive environmental management study of the Cedar and Green River basins. This study, called the Water Resource Management Study (WRMS), was the first attempt at developing a comprehensive program for the management of surface water diversions and water quality in these basins. It is important to note that the first item listed in the plan evaluation guidelines of the WRMS was that "non structural solutions to a problem are preferable to structural solutions" [CH2M Hill, 1974]. This approach has been exemplified in recent years by the prevalence of conservation campaigns coordinated by local utilities. A second important principal stressed by the authors of the WRMS study was that area-wide management is preferred over fragmented management. The possibility of increasing the region's reliable water supply through joint management, without new source development, is clearly in accord with these long range planning goals.

Cooperative use of existing supplies could provide multiple benefits. First, if the current configurations can be operated to supply a greater volume of water, costly system upgrades can be delayed indefinitely, yielding substantial savings in capital expenditures. Due to the controversial nature of many source development options, better use of the existing systems would also offer political advantages over many other proposals. Also, if the combined system can be operated more efficiently, instream flow needs will benefit from the additional water that will remain in the river until demands rise further. This is especially significant as the environmental impacts of low-flow are not yet well understood, and any delay in instream flow reduction will allow better quantification of its effects.

Cooperative use of the existing infrastructure can also offer political advantages when compared to costly and controversial new source development. Joint use, however, requires a substantial shift in current operational policy. Rather than two autonomous utilities attempting to optimize the performance of their individual systems, the joint system requires a commitment to a common purpose in order to be successful. In a cooperative use system, the two municipalities will share not only the benefits of improved performance, but also the risk of failure. The political receptiveness of these utilities to such a shift is an issue that is beyond the scope of this study, but one that requires careful consideration in the future.

Before an intertied system can be evaluated effectively, extensive analysis of the problem is needed. This analysis must consider a broad realm of questions, such as:

- What is the maximum increase in safe yield that can be expected from a cooperatively used system?
- Using a realistic operating policy, how large a benefit can be achieved?

- Is a connection physically feasible?
- Is a connection economically feasible?
- What impacts will the intertie be expected to have on instream flow?
- Will the joint system offer performance benefits for a wide range of criteria or merely for safe yield.
- Can operating policies be designed to create a system that is oriented towards a safe fail rather than fail safe?

This thesis develops formal decision aids to facilitate consideration of a regional water resource system for Seattle and Tacoma. Preliminary to the investigation of any proposal is the need to gain increased knowledge about the character of the systems. Information that is necessary for any study includes the identification of significant system constraints, expansion possibilities, and likely improvements, as well as knowledge about the interest groups that are affected by operational policies and the nature of deficits currently being encountered.

Scope

This study analyzes the advantages offered by cooperative use of the existing sources of supply for the municipalities of Seattle and Tacoma. These benefits are measured in terms of the physical performance of the combined system, as compared to the individual benefits currently received. System specific performance criteria are developed, and the effect of a proposed intertie is evaluated using these as a yardstick. Using system-wide loss functions as an alternative means of gauging system performance was considered, but due to their inherent difficulties and time and monetary constraints, this method was not selected. This issue is discussed further in Chapters 4 and 6.

The Seattle and Tacoma systems are modeled using a simulation environment, and their performances are evaluated over the historic data record. Synthetic streamflow traces were not used in this analysis because the historic records were sufficient to adequately include several significant drought events [Jettmar and Young, 1975]. Consideration was also given to the fact that the ability of synthetic streamflow models to accurately replicate the effects of droughts is a matter of continuing controversy [Mandelbroot and Wallis, 1968; Hirsch, 1979]. Use of the historic streamflow record has been shown to provide sufficient results in the optimization of operating policy [Jettmar and Young, 1975]. Allocations between reservoirs within the Seattle system or supplies between systems were made based on simple heuristic operational policies.

Monthly inflow data were used for the simulation for several reasons. First, these are the most reliable data available for the Cedar and Tolt Rivers. Records from October 1929 through September 1989 were obtained from the SWD. Also, the Corps of Engineers was able to supply similar monthly flow data for the Green River from 1913 through 1991. For examining the effects of droughts, monthly modeling seemed to be an appropriate scale, as implementation of conservation programs or curtailments would require substantial lead time. The use of a monthly decision horizon led to some difficulties in the development of operational strategies, as will be discussed later, but these were not felt to be significant enough to alter the scope. The individual systems were modeled for the entire length of their data records, while the intertied system was modeled for the sixty years of historic record overlap. Due to the nature of the Tacoma system and the lack of extreme events outside the overlap period, this did not affect the results.

Because this study is concerned with quantifying the potential benefits of interties to the existing systems, only the current source configurations and imminent improvements were considered. However, the results of the study, particularly when system constraints can be identified as the cause of failures, may provide insight into which future source development would be most advantageous. Seattle Water Department and Tacoma Public Utility predictions of expected yearly demands through the year 2035 were used for the simulation [Tacoma, 1987; Seattle, 1985]. Although this study did not attempt to quantify failures in economic terms, an implicit assumption was made that the economic loss function is convex (larger magnitude failures cause disproportionately larger losses). This suggests that the volume of supply shortfall can be of equal or greater importance compared to the number of failures which occur [Loucks, 1975].

The extent of this study, and its goals, must be put into the proper perspective. An attempt was made to understand the intricacies of the two systems under study and to model them accurately. However, there are a number of factors, as explained in the body of this thesis, that make modeling a difficult and sometimes unreliable task. Gauging inaccuracies in obtaining data, the effect of previous record extension techniques, uncertainty regarding both major aquifers in the study area along with many other issues limit the ability of this or any other study to provide definitive conclusions about the effect of a proposed system modification. Considering these uncertainties the goals of this study were defined as follows:

- to provide a preliminary review of the impact of system regionalization on water resource reliability
- gain an improved understanding of the role of physical performance measures in water resource management
- to document the ability of a graphical-interface driven model to enhance the decision making process

Models are developed to investigate the theory of physical performance measures and their practical application to a regional water resource system for western Washington. A multi-faceted definition of reliability is introduced, and user specific performance measures are developed. Several experiments are conducted to demonstrate the applicability and flexibility of the approach used for this study. The experiments are intended to illustrate the applicability of the current modeling approach to real problems. In some cases, more detailed information would be required to allow definitive conclusions. These issues are left for future research.

Approach

STELLA II™ (Systems Thinking Experimental Learning Laboratory with Animation II) was used to construct computer simulation models for this analysis. Simulation modeling allows the user to examine the implications of a particular operational policy over a broad range of scenarios. Using the models that were developed in this study, the effect of changes in operating policies can also be qualitatively and quantitatively assessed from a number of perspectives. Significant effort in the development of these models was related to failure analysis. The frequency, mode, length, and magnitude of system failures are all important measures of performance. Graphical and tabular output data are produced to allow the evaluation of alternative solution options.

Individual water supply modules were constructed for each component of each of the systems. The major modules included the Cedar River, Tolt River, Lake Washington, Cedar moraine aquifer, Green River, Tacoma local wells, and South Tacoma well-fields. Each of these modules incorporated relevant constraints and included release guidelines based on historically defined priorities. Each supply module was then tested to determine its firm yield if operated independently. Next, components were joined, and the performances of larger sub-systems were evaluated. Finally, the entire intertied system was constructed and tested.

There were a number of operational issues considered when the benefits of the fully intertied system were maximized. Allocations of releases between reservoirs were estimated using the space rule which limits the amount of spill from the system [Bower, 1962]. Consideration is also given to system constraints such as pipeline capacities, requiring more complex rules. The prioritization of water allocations between systems is also influenced by the nature of the sources. If a supply is only available for instantaneous use, without the potential for storage, a realistic operating policy utilizes this water before accessing reservoir storage. Likewise, if local inflow can be used to meet instream demands, reservoir releases are not made. Also, hedging in the utilization of specific sources is necessary to optimize the system's performance during extreme events. Consideration was given to each of these fundamental guidelines when the models' operating policies were developed.

Allocation priorities among the various sources were determined dependent on the distribution of stored water in the system, as well as predictions of future inflow. The time of year was also considered in developing an indicator of the relative condition of the system with respect to its ability to meet future demands. A reasonable number of system states were identified and based on these states. Alternative allocation procedures were followed. These procedures were designed to restore the system to as satisfactory a state as possible. The complete list of states developed and the target releases for each are listed in Chapter 4.

Appropriate operating policies that improved the performance of the combined system with respect to yield were developed in the course of this research. It must be noted that this approach does not consider the political ramifications of the implementation of such a policy. By sharing the benefits of a proposed system intertie, the two municipalities would also be sharing the possibility of economic loss should failure occur. Relinquishing operational autonomy is an issue that utilities are not likely to welcome; therefore, demonstrable benefits of cooperative control are a requirement to gain political acceptability.

The distribution of benefits from an alternative management plan is also a subject that is beyond the scope of this thesis. As mentioned earlier, these basins support a number of diverse competing uses, both diversionary and instream, and any additional water that is made available would need to be allocated prudently. In the past, the focus of source development and operational policy has been primarily geared towards improving availability of municipal supplies. More recently, however, there has been an increased recognition of the value of adequate instream flow. Equitable distribution of the gains from coordinated management would require extensive analysis and would likely require adjudication. The objective of this study is to quantify potential benefits and develop a tool through which negotiation may be enhanced.

Chapter II
Literature Review

This chapter begins with an examination of the relevant literature relating to evaluation of the performance of water resource systems. One of the most commonly used performance measures, safe yield, is defined and its shortcomings in practical applications are discussed. Methods that have been used in the past to establish the yield of the two systems in this study, are also cited. These affirm the difficulty of relying on safe yield as the primary indicator of reliability. The section concludes with a multi-faceted definition of system reliability which considers the number, length, and magnitude of failures at alternative demand levels.

The second section provides a discussion of the advantages and disadvantages of various classes of computer models used to examine system performance over a wide range of scenarios. Specific criteria for choosing a simulation model are highlighted, and difficulties in their practical application are addressed. To effectively utilize a simulation model, operating heuristics must be developed. The third section of this chapter details a number of heuristics that have been used previously in modeling studies to improve operational efficiency. Finally, because instream flow releases are a critical constraint on each of these systems, a discussion of the significance of maintaining adequate instream flow is included.

Performance criteria

When evaluating the performance of a water resource system there are a number of criteria that can be used. One measure that is typically of concern to water users is termed "safe" or "firm" yield. This yield is normally defined as the maximum quantity of water that can be guaranteed for delivery from any source or combination of sources [Loucks, 1976]. Although safe yield is sometimes used to imply the maximum withdrawal at a specific reliability level, in this study, the terms safe and firm yield will be used interchangeably. The firm yield of a system allows planners to judge when a system has substantial cushion to meet demands under drought conditions, or when the system is seriously stressed.

Despite this definition of firm yield, the stochastic nature of streamflow implies that virtually no level of supply is 100% safe. Historic data records are often used to determine the safe yield of a system. If the length of record is adequate, this method is normally considered to be acceptable [Loucks, 1976]. If the historic record is brief, record extension or synthetic generation techniques are commonly used to augment the data [Hirsch, 1982]. In either case, the system firm yield is computed as the amount of water that could have been withdrawn without failure over the entire study period. In this case, failure is defined as the inability to meet all demands placed on the water supply system. Firm yield provides no indication of the impacts of failures should demand exceed this amount.

One difficulty in applying the concept of safe yield to any real water supply is identifying the most severe droughts for alternative system configurations and operating policies. Some drought events are a result of lingering, sub-average flows over several years, while others are caused by a few months of negligible flow and/or high demands. The choice of which droughts are the most severe is also influenced by the antecedent conditions, especially in regard to reservoir and aquifer storage levels. Deciding what constitutes a "design drought," and determining when a particular drought begins and ends, also presents a problem [Draper, 1981]. For these reasons, selecting a few representative events and developing an operating policy which maximizes yield for these specific conditions can lead to serious errors.

As noted previously, the definition of safe yield is sometimes altered to reflect the specification of the American Water Works Association (AWWA) that water supplies be 98% reliable [AWWA, 1959]. The manner in which this definition is applied to actual water resource systems has been subject to numerous interpretations. One understanding asserts that the amount of water that can be supplied in 49 years out of 50 is the system safe yield. As a practical application of this, the Seattle Water Department (SWD) calculates safe yield as the amount of water that can be delivered in every year exclusive of the historic drought of record [CH2M Hill, 1974]. This results in a one dimensional description of system performance that may not be capable of fully characterizing the impact of failure. It also relies on the subjective determination of a drought of record that may be unique for each system configuration and operating plan.

There are several other methods of determining system yield that have been applied to the systems discussed here. In the WRMS study, individual monthly flow were ranked in regard to recurrence interval, and the composite probability of drought events were determined by considering the probabilities of individual monthly flow [CH2M Hill, 1974]. The firm yield was then determined as the amount of water that could be supplied in a drought event of 1 in 50 year recurrence (98% reliability). Questionably, this study disregards its own finding that the yield during a 1 in 10 year summer drought (90% reliability) is lower than the amount they label the firm yield. This study also overlooks much of the complexity involved in determining the recurrence interval of a sequence of flows in a correlated series.

In a recent study for the Tacoma Water Division, the 98% reliability criterion was approximated by assuming that the largest single pump did not contribute to the system's firm yield [Economic and Engineering Services, 1991]. This had the effect of lowering the system's firm yield by 8 million gallons per day (MGD). In a system such as Tacoma's, where water supply comes from a nearly guaranteed run of river diversion, in addition to pumped groundwater, mechanical reliability of pumps might be the single most important feature limiting system performance. A complex risk assessment of the type used in the evaluation of nuclear power plants may provide insight into the system's susceptibility to failure. However, in a hydrologically dependent system such as Seattle's, or the proposed intertied system, this approximation is not as justifiable. In these cases consideration must be given to the uncertain availability of water.

Alternative methods of characterizing the reliability of a water supply system are required. Because future inflow scenarios may be more extreme than previous ones, firm yield based on selective use of the historic record is not always an adequate definition of system performance. Also, due to the economic impacts of deficits of varying magnitudes and length of failure episodes, it is not sufficient to simply know how often a system fails to meet demands, but also by how much, and for how long. Hashimoto et al. [1982a] defined three alternative measures of system performance, reliability, resiliency and vulnerability.

Reliability is the probability that a system will be in a satisfactory state in any time period. The definition of satisfactory can be defined several ways but is often chosen to indicate if the system is able to meet the target yield in any time period [Hashimoto, 1982a]. Mathematically:

$$\text{Reliability} = \alpha = \text{Prob}\,[\, X_t \in S]$$

Where: X_t = state of system at time t

S = the set of successes

Alternative definitions include whether instream flow requirements were met or whether or not demand reductions were necessary. An alternate definition of reliability is the probability that the system will be able to meet demands in every time period within the planning horizon, usually taken to be 50 years [CH2M Hill, 1974].

Resiliency is a measure of the ability of a system to recover from a failure condition to a success state, once a failure has occurred. Mathematically:

$$\text{Resiliency} = \gamma = \frac{\rho}{1-\alpha} = \frac{\text{Prob}\,[\, X_t \in S \text{ and } X_{t+1} \in F]}{\text{Prob}\,[\, X_t \in F]}$$

Where: X_t=state of system at time t

S=the set of successes

F=the set of failures

Hashimoto [1982a] notes that if the occurrence of a failure and a subsequent success are probabilistically independent, resiliency reduces to the same definition as reliability. This is very seldom a proper assumption when success is dependent on highly correlated occurrences including monthly streamflow. A highly resilient system would rebound from each failure to meet demands in the subsequent time period. This type of operation, however, might lead to future failures and could complicate the institution of water use restrictions [Loucks and Sigvaldson, 1982]. Loucks and Sigvaldson also note that even if a system fails infrequently (i.e., reliability is high), there can be serious implications if it tends to remain in a failure state for several time periods .

Hashimoto defines vulnerability as a measure of the expected magnitude of water shortfall should a failure occur. His definition states that vulnerability is the sum of the products of the severity of failures times the probability that the failure is the most severe outcome in a failure sojourn. Mathematically:

$$\text{Vulnerability} = v = \sum_{j \in F} s_j e_j$$

Where: s_j=severity of failure state
e_j=prob x_j corresponding to s_j is most severe outcome in a sojourn into F

Alternative definitions of vulnerability could be the maximum shortfall over the planning horizon, the average shortfall, or the number of shortfalls greater than some magnitude that is considered appropriate. A highly reliable system often fails to consider the non-linear economic losses associated with failing to supply target volumes. Because large deficits tend to cause disproportionately larger costs, the vulnerability of a system is as crucial to consider as its reliability.

Modeling

A wide range of approaches have been utilized in the analysis and modeling of water resource systems [Yeh, 1985]. Yeh concluded that the choice of method depends on the characteristics of the system being modeled, the availability of data, and the objectives and constraints specified. Similarly, Loucks noted that the problem is not identifying the best class of model, but characterizing the allowable trade-offs in information each model is suitable for providing and selecting an appropriate one based on temporal, monetary and computational constraints [Loucks, 1976].

Models fall into two general classifications; simulation and optimization. Optimization models are most useful for defining the appropriate level of releases to be made for a particular inflow scenario to maximize the value of some objective function. Two broad categories of optimization models that have been applied extensively in water resources management are linear programming and dynamic programming [Yeh, 1985]. Deterministic, probabilistic, static, and dynamic variants of each category of model have been used to treat numerous specific concerns.

Linear programs are best applied when all relations among the variables are linear, both in the constraints and the objective function. This type of model has been successfully applied to the Seattle water system to aid in the selection of new source development [Palmer, 1984]. Palmer notes that unlike simulation models, linear programming can determine the maximum yield of a system for a wide variety of configurations without trial and error solutions. There are, however, several reasons why linear programs may be unable or inappropriate for determining the exact optimal solution.

The first of these reasons is mathematical. The algorithms used by linear programs often require simplification of system components and objectives that may limit its effectiveness. In the Seattle system, for example, the rate of seepage and return from the aquifer is a highly non-linear process, as are lake evaporation and economic benefits and losses. Piecewise linearization or successive approximation techniques may allow solution of these problems, but additional uncertainty is introduced. A second criticism is that the specification of an objective function that correctly values end of period storage, and thereby releases, is difficult if not impossible [Johnson, 1991]. Loucks [1976] notes that agreement among decision makers is rarely achieved in setting objectives, let alone determining when requirements are satisfied.

Dynamic programming offers advantages over linear programming with respect to the inclusion of non-linear relationships. The same difficulty exists with the formulation of an explicit objective function that correctly addresses all interests. Additionally, both dynamic and linear programming are subject to the "curse of dimensionality", wherein computational constraints limit the models' ability to accurately capture the relevant system characteristics. The choice of critical period hydrology can often overcome this effect in linear programs, but the corresponding loss of generality, as discussed earlier, may create problems. In dynamic programs discretization of continuous variables, such as reservoir storage, can lead to substantially increased computational complexity.

Optimization algorithms can provide insight into the maximum yield that can be obtained from a specific system configuration and set of inflows. This information is useful when examining the performance of a system as operated under alternative management plans. However, it is important to note that although optimization models can determine the best releases for a particular inflow scenario, they do not lead to the development of heuristic operating guidelines that can be translated into actual operating policies.

Finally, although sensitivity analysis can often be helpful in extrapolating the results of an optimization study, the lack of flexibility in the model formulation can be a serious hindrance in facilitating trade-offs among competing interests. These limitations restrict the use of optimization techniques to preliminary analysis of complex water supply systems [Loucks, 1976]. Further analysis with a simulation model is normally required to fully explore the impacts of a specific operating policy.

Simulation modeling is a technique to approximate the behavior of a system based on a set of mathematical relationships between the system components. By using appropriate decision rules (heuristics) and inputs, simulation models allow the user to investigate the performance of a system over a wide range of inflow scenarios and a variety of operating policies. Because it is less computationally demanding than optimization modeling, the simplifications required in a simulation model are typically less pronounced. This allows for more realistic characterization of system responses and a better understanding of the impacts of any decision.

Simulation models do not attempt to define an optimal release policy. Therefore, their value is greatest when a limited number of operating options can be identified for investigation. The identification of alternative policies can be accomplished with a preliminary optimization [Erikson, 1982]. Other simulations, such as SIMLYD-II [Evenson and Mosely, 1970] employ an optimization sub-model to implicitly specify the operating policy. Johnson et al [1991] used a single period optimization in conjunction with heuristic guidelines to determine the appropriate releases from reservoirs in California's Central Valley Project. Consideration of the physical constraints and hydrologic inputs to a system, in addition to simple heuristics, such as the space rule or multiple zoning, can also be used to design an operating plan.

A number of simulation models have been constructed primarily in the FORTRAN language. Generic water resources models include the HEC series (HEC 2-HEC 5), developed and supported by the COE. Specific site models have been developed for a number of basins including SIMLYD II, used in the Texas Water Development Study; Acres, which models Canada's Trent River basin; and CVPower, which was developed for the Central Valley Project. These models have been designed to facilitate understanding the strengths and inadequacies of present or proposed operating plans. Unfortunately, these formulations still tend to separate the underlying assumptions and equations with which the model was developed, from the results that it generates. Complex codes and bulky output act to segregate those involved with simulation into model builders and model users.

A second option for simulation modeling is to use a conventional spreadsheet program. Most spreadsheets offer substantial advantages over FORTRAN programming for manipulation of data, necessary to produce graphs and tables. Also, spreadsheets are often easier to update than a coded simulation model. The effects of minor changes can be determined through simple plots generated within a typical spreadsheet package. However, the underlying assumptions of the model builder are often lost within the equations which operate "behind the scenes" to produce output. Understanding these governing assumptions is difficult, and modifying them may be next to impossible for anyone other than the model's author. For this reason, spreadsheets do not offer the flexibility that is often required to deal with competing interests in a complex multi-dimensional problem such as water resource allocations. Once again, the level of effort required to understand the model limits its applicability for effective problem solving.

Recent developments in the field of graphics driven simulation environments offer an opportunity to improve this situation. STELLA II™ is one of the leading tools in this market. It has been used extensively in the simulation of biological treatment processes [Abbott, 1992], pharmacokinetic testing [Ramsay, 1991], healthcare management [DeMello, 1990], landscape dynamics [Boumans, 1990], and a wide range of physiological and ecological systems. Despite STELLA's inherent applicability for modeling complex interactions, there is no literature documenting its previous use in the simulation of water resource systems. This study will attempt to show how graphical simulation models, in general, and STELLA, in particular, can be used effectively in the evaluation and characterization of approaches to meeting western Washington's future water supply demands.

Operating Policy

The development of an adequate operating policy, both for simulation and for real time management, is an issue of great importance. Hirsch [1977] emphasizes that when evaluating alternative policies, any benefits (yield) offered by following a specific policy must be compared to the maximum possible benefits attainable within the physical and hydrologic constraints. He also suggested a means by which the maximum firm yield of a system could be calculated without the need for an optimization model and its corresponding approximations. His technique simplifies the system being studied into a single reservoir, into which all inflows enter, and from which required releases are made. This allows a simplified operating policy that eliminates failures resulting from non-optimal system management. Therefore, the maximum hydrologic potential of a system can be evaluated.

System robustness, or the ability of a system to minimize expected loss under a wide variety of future events, is a critical issue when planning expansion alternatives [Hasimoto, 1982b]. This principle may also be applied to operating policies. Rigid or drought specific operating rules can cause a situation wherein substantially poorer performance occurs in time periods other then the one for which the rules were developed. Johnson et al. [1991] demonstrate how flexible operating heuristics are preferable to rigid policies in management of a complex multi-reservoir system. These heuristics not only make simulation modeling of complex systems feasible, but they also offer guidance in the development of real time management policies.

Several operating heuristics are valuable when allocating water between sources or systems. These include the space rule, or some means to allocate releases among reservoirs in parallel and hedging, or some means to allocate releases across time. Bower et al. [1962] provide a discussion of the theories behind these rules and a qualitative assessment of their expected benefits. Johnson et al. offer proof that the space rule optimizes releases with respect to system spillage, which can be thought of as release without benefit. Bayazit [1990] analyzed the effects of a policy of hedging on the performance criteria defined by Hashimoto. Loucks provides a more complete discussion of the effects of various operating policies and system constraints on performance.

Rule curves are often used to define desired storage and release rates in the actual operation of reservoirs and, likewise, provide useful targets for simulation model policies [Loucks and Sigvaldson, 1982]. These rule curves can be adjusted seasonally to allocate storage space to competing uses such as flood control, conservation, or enhancement of instream flow. Loucks and Sigvaldson [1982] also list a number of potentially useful components in the development of operating policies including the following:

- Target storage levels or volumes - end of month goals that allow the system manager to moderate releases based on historic seasonal trends.
- Multiple zoning - the use of several rule curves to identify the state in which a reservoir is at a particular time and alter releases accordingly. Some example zones include flood control zone, conservation zone, buffer zone, and inactive zone.
- Flow ranging - prescriptive use of the relationship between reservoir level and channel flow in the determination of releases.
- Conditional rule curves - the incorporation of expected future inflows, in addition to season and reservoir level, into the development of operational rules.

Instream Flow Requirements

In western Washington, instream flow requirements are set in conjunction with river basin analysis by the Department of Ecology (DOE). This analysis is intended to balance instream needs with diversionary uses of water. However, the analysis process is a long and costly undertaking and detailed examinations have been accomplished on only a few rivers. Even after completing analysis, the DOE must work with local utilities or through the courts to establish instream release requirements. For many rivers, there is no legal mechanism by which senior water rights may be appropriated to provide sufficient instream flow. This predicament results in the current situation where instream flow is commonly viewed as a secondary concern. Ongoing research has shown that this viewpoint is both ecologically and economically unwise.

Maintenance of an adequate level of instream flow is critical for many reasons including numerous ecological concerns. Biological populations of rivers and streams require sufficient water to survive, and many species have a threshold below which their existence is threatened. Also, the waste assimilation capabilities of rivers are directly related to the flow volume, and poor water quality further stresses the rivers inhabitants. Finally, lakes and estuaries at the mouths of these rivers support diverse activities that rely on substantial inflow. Lake Washington, for example, is dependent on the Cedar River for over 70% of its inflow, which is essential to the maintenance of water quality, ecological diversity, and the prevention of saltwater intrusion.

The supply source rivers of Seattle and Tacoma's systems are home to numerous species of resident and anadromous fish. These fish populations, particularly the anadromous runs, "are of significant importance to the social and economic well being of the Pacific Northwest and the nation" [PL 96-501]. The Cedar River sockeye salmon run is the largest sockeye run in the conterminous United States. The Green River supports one of the state's largest fisheries of steelhead trout and is also the spawning grounds for chinook, coho and chum salmon. The Tolt system provides extensive habitat for resident trout and steelhead.

Determination of the volume of flow required for habitat preservation is a complex and difficult process. Increased residential development near streams and rivers has altered hydrologic characteristics and introduced higher pollutant loadings to these water courses. Increases in peak runoff, coupled with decreased base flow, are a consequence of past development practices. Restoration and preservation of the remaining habitat require careful planning. Effective timing of releases from reservoirs for instream needs must be developed on a basin-wide scale, considering the impacts of alternative flow levels on the physical characteristics of the streams. This study considered the requirements of low-flow augmentation but did not analyze the effects of increased peaks. Peak and flood flow analysis would necessitate the choice of a shorter modeling time step to provide suitable results.

Instream flow must also be sufficient to assimilate current and expected future pollutant loads coincident with further development. In addition to adequate volumes of flow for waste assimilation, many species of fish are dependent on high flow velocities to facilitate spawning. Numerous methods have been advocated to quantify the areal extent of spawning ground provided at alternative flow rates [Snyder, 1983]. These methods attempt to incorporate physical characteristics and flow patterns to derive optimal flow rates. The basins within the area of this study have been examined previously in some detail [Collings, 1972; Stober, 1974; Stober, 1983]. The results of these studies and the recommendations of the DOE and the Corps of Engineers were used in the development of the models in this study.

At its terminus, the flow in the Cedar River enters Lake Washington. The lake requires adequate inflow to maintain a high water level necessary for both waterborne and bridge transportation. The lake also receives a substantial quantity of local inflow primarily from the storm drainage systems of the surrounding communities. This water is typically higher in pollutant loading than inflow from river sources and can stress the delicate balance of the lake's natural ecosystem. The high quality of Cedar River water is essential for preserving this balance.

Cedar River inflow is also critical to the maintenance of water quality at the Hiram Chittenden Locks, where Lake Washington water is released into Puget Sound. The locks provide the only water-borne transportation passage between the lake system and the Sound. This link is vital to the economic well being of the region. Releases through the locks use a significant quantity of water; more than 210 MGD during the summer. Additionally, in the lockage process some Puget Sound water enters the lake and must be flushed from the system through a salt water drain. Flushing requires the release of fresh water in addition to draining the salt water. If lockage releases are reduced, salt water may intrude up the channel below the locks further than is desirable. If this happens, additional releases are made to prevent intrusion into the lake and degradation of water quality.

The lock complex also has a fish ladder which is operated year round for passage of several anadromous species. This ladder releases water at a constant rate of 45 cubic feet per second (cfs). Species which rely on this ladder include the Cedar River sockeye salmon and steelhead trout. These runs are essential to recreational and commercial fisheries that yield regional economic benefits. The Cedar River is the major source of supply to this outlet. Therefore, instream flow in the river must consider lake needs, in addition to river demands.

Clearly, determination of the proper level of release for instream flow needs is an involved process. This process must consider several other impacts in addition to those mentioned above. The rate of sediment transport and deposition is a function of flow characteristics and altering the natural flow pattern can have serious consequences on usable spawning habitat. Also, nearly all biological processes, including assimilative capacity and fishery production, are affected by temperature. River temperature is affected by the timing, volume and manner in which releases are made.

Considering the scope and intent of this study, instream releases from reservoirs were based on current DOE recommendations. This choice streamlined model development and allowed validation of the results against other system models. Further study into the impact of alternative instream requirements could easily be accomplished with the tools developed for this analysis. The manner in which these models may be useful in negotiating and scenario testing is demonstrated in a parametric comparison of instream flows versus M&I diversions (Chapter 6).

Chapter III
System description

In this chapter, the physical characteristics of the two municipalities' water supply systems are described. The Seattle system, the larger and more complex of the two, is presented first. Pertinent attributes, including reservoir storage and transmission constraints, are detailed, and their effect on system performance is explained. Hydrologic conditions that affect reliability and unique features of each component of the system are also discussed. The most significant competing uses for the water and the current operating policy are detailed. This is followed by a similar description of the Tacoma system, which highlights the most significant constraints on water supply reliability in that system. The final section of this chapter discusses the most promising route for constructing an intertie and the expected cost of such a connection.

Seattle with a population of nearly 500,000 people is the largest city in the State of Washington, while Tacoma with over 150,000 residents is the third largest. Located in western Washington, these cities lie thirty miles apart in the north-south corridor formed between the Cascade mountains and the Puget Sound. Being the largest municipalities in the region, Seattle and Tacoma maintain substantial infrastructures, including expansive water supply networks. Together, their water distribution systems supply over 1.4 million consumers, nearly one-third of the state's residents. The service areas of the two utilities are contiguous and include most of the Puget Sound region from just south of Everett to southeast Pierce County (see Figure 3.1).

Most of the water used to meet the demands of this service area is diverted from rivers which originate in the Cascades. The majority of Seattle's water comes from the Cedar and Tolt Rivers, while Tacoma's primary source is the Green River. The Cedar and Green Rivers flow northeasterly across the southern portion of King County approximately 10 miles apart. The Tolt flows southwesterly from near the border with Snohomish County, just to its north, to its confluence with the Snoqualmie River. The primary diversion points for the Seattle and Tacoma M&I supplies are within 8 miles of each other, near the towns of Landsburg and Palmer, respectively.

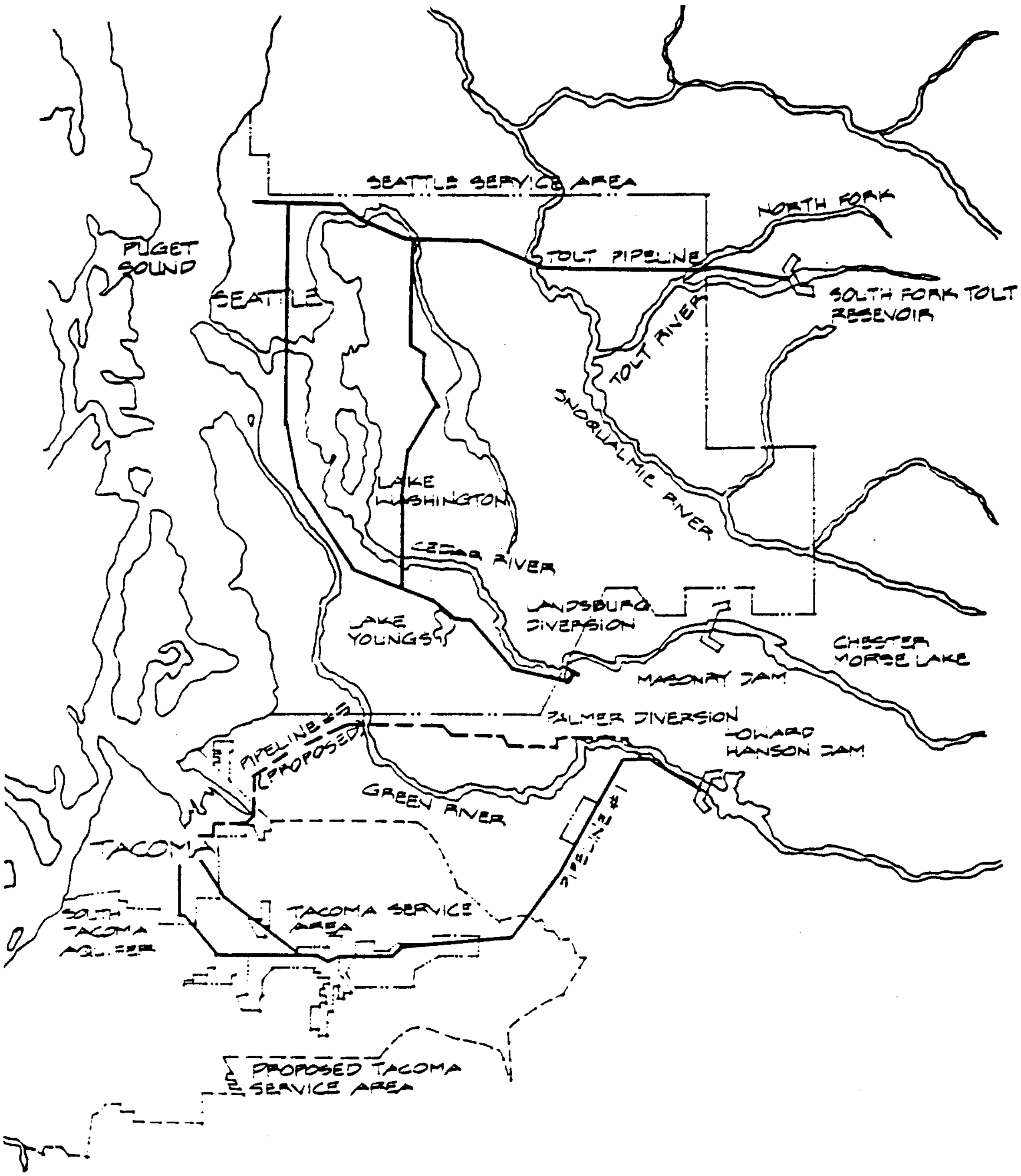

Figure 3.1 Seattle and Tacoma Water Service Areas

The demand for potable water in western Washington is increasing at an unprecedented rate. Estimates by the two utilities indicate that demand will grow by more than 25% in the next decade and by as much as 75% by the year 2035 [Tacoma Water System Plan, 1987; SWD Comp Plan, 1980]. If growth continues as projected, current water supply capacities will be inadequate before the year 2000. Further source development and improved operating strategies will be required to meet the escalating demand.

One potential solution for increasing the water supply capabilities of these two systems is an intertie between their distribution networks. Because their service areas and major sources are in proximity, such a connection has been shown to be economically and physically feasible [Tacoma Water System Plan, 1987]. The impact that such an intertie will have upon system performance requires further analysis.

Seattle

The Seattle Water Department (SWD) provides drinking water for nearly 1.2 million people living in the City of Seattle and the surrounding area. The existing supply source for this system is surface water from the Cedar and Tolt River basins. These rivers have their headwaters in the Cascade mountains east of Seattle and drain into Puget Sound (see Figure 3.1). The water from these supplies reaches the Seattle distribution network via gravity transmission lines. Because of the pristine nature of the catchments above the diversions, the water is high quality and no filtration is required. Treatment consists of chlorination, fluoridation, and pH control to minimize corrosion.

The Cedar River basin, located southeast of Seattle, encompasses an area of approximately 190 square miles. The highest point in the basin is Tinkham peak, at 5400 feet, while the floodplain near Renton is just above sea level. Annual precipitation ranges from 100-200 inches per year in the mountains to 30-50 inches per year at lower elevations. A large percentage of the precipitation in the mountains occurs as snowfall, which can exceed 500 inches per year, between the months of October and June. Streamflow are dependent on temperatures in the upper catchment but are typically greatest during the high precipitation period from December through February. A second period of high flow, fed primarily by snowmelt runoff, occurs in April and May. The lowest annual flow coincides with the minimum precipitation months from July through September.

The Cedar River is 50 miles long from its headwaters in the Cascade range, about 5 miles south of Snoqualmie Pass, to its outlet into Lake Washington below the City of Renton. The river was originally a tributary to the Black River but was diverted into Lake Washington in 1916 as part of construction of the Hiram Chittenden Locks. Flow from the Cedar provides over 70% of the lake's total inflow and is critical to the maintenance of adequate water levels and operation of the locks. The Cedar also supplies almost 70% of the Seattle area's annual municipal and industrial needs.

The Cedar River basin is a diverse multi-purpose system of streams, rivers, and lakes. In addition to municipal diversions and the needs of Lake Washington, flows in the Cedar River also generate hydroelectric power, maintain critical fisheries habitat, and assimilate waste from increasing streamside development. Additionally, the Cedar is the primary source of water to the Lake Washington/ Lake Union/ Ship Canal complex in the Puget Sound Lowlands. Hydropower is subordinate to the other uses of the system and is not a major consideration in water resources management. However, the complicated interactions among the remaining needs, coupled with the unique physical features of the catchment, add substantial complexity to the development of an appropriate operating plan.

Flow in the Cedar is regulated by two dams located in the upper catchment. The Chester Morse Crib Dam at river mile 37.2 forms Chester Morse Lake which provides an active storage capacity of 19,442 acre feet with dead storage totaling 36,064 acre feet (see Figure 3.2). The crest of the timber Crib Dam is at an elevation 1,546 feet. Located 7,000 feet downstream of the Crib Dam, at river mile 35.6, is Masonry Dam. This dam rises to 1,590 feet, creating Masonry Pool with a total storage capacity of 154,800 acre feet. Seventy-five percent of the catchment's land area, and an even greater portion of the total

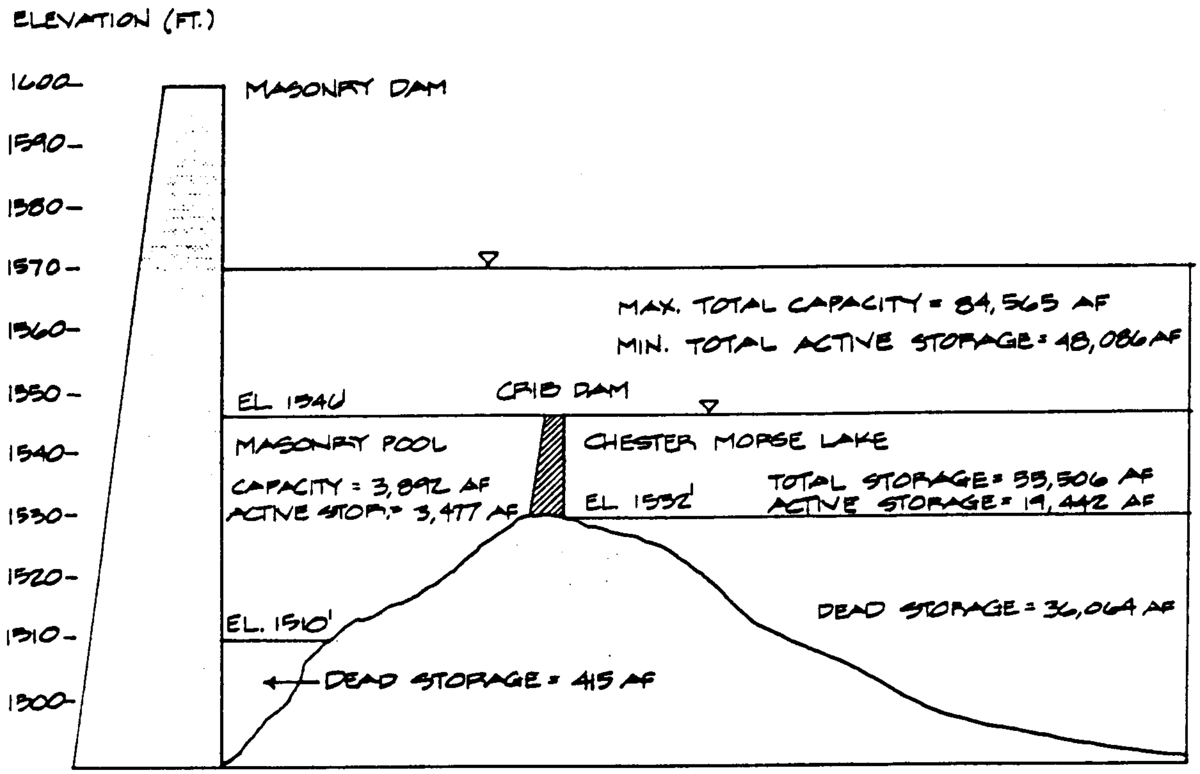

Figure 3.2 Chester Morse Dam

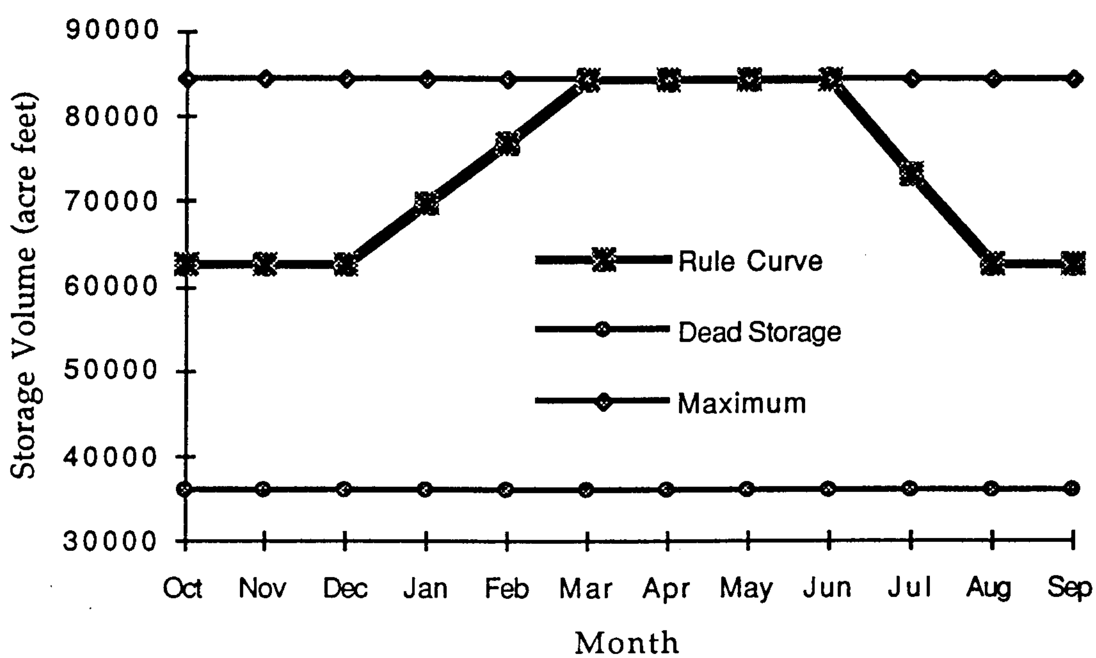

Figure 3.3 Cedar Reservoir Rule Curve

precipitation, contribute to inflow above the dams. Due to structural concerns, however, the maximum water level is maintained below 1,570 feet giving a total active storage of 48,086 acre feet.

When the water level in Masonry Pool rises above 1,546 feet the Crib Dam is inundated and a single reservoir results. The current operating plan allocates a portion of the total storage space to flood control (see Figure 3.3). The remaining space is utilized for storage to meet M&I demands and low-flow augmentation. The soil underlying Masonry Dam is a highly permeable glacial till. Water retained in Masonry Pool is subject to large seepage losses which recharge the glacial moraine aquifer. Approximately 80% of this water eventually returns to the Cedar system, while 20% is lost to the Snoqualmie basin (see discussion of moraine below). Therefore, whenever releases from storage are required, Masonry Pool is emptied first, leaving as much water as possible above the Crib Dam where seepage losses are minimal.

In addition to the seepage return, a substantial quantity of inflow occurs downstream of the Masonry Pool spillway. Water diverted to the power penstocks of Seattle City Light's Cedar Falls Powerhouse returns to the river at river mile 33.7. Additionally, several tributaries, including Taylor creek and Walsh Lake ditch, join the Cedar above the site of Seattle's diversion. In planning for low-flow augmentation, municipal water needs, or flood releases, the inflow below Masonry Dam is an important consideration.

Currently, the SWD has a water right claim for 300 MGD average annual withdrawal from the Cedar River [URS, 1981]. Legal questions remain as to the prioritization between Seattle's claim and instream flow requirements, and the SWD is working with the Department of Ecology (DOE) to settle this issue. Seattle's water supply diversion is located above the town of Landsburg at river mile 21.6. The diverted water is transported via a 220 MGD pipeline to Lake Youngs where it can be retained for settling or passed into the Seattle distribution network. Water from Lake Youngs can be directed either to Seattle or to communities on the eastside of Lake Washington.

Between the Landsburg diversion and the river's outlet near Renton, additional local inflow occurs. This inflow is used to meet the instream flow requirements at Renton and for maintaining Lake Washington's water level. If the inflow between Masonry Dam and Renton is insufficient to meet instream flow requirements, reservoir releases are required. The amount of this release is dependent on the level of storage in Masonry Pool and the average inflow in the upper catchment for the previous 90 days. Table 3.1 lists the reservoir storage levels and average monthly inflows that are used to switch release regimes. The releases required for each state are listed in Table 3.2. The maximum amount that can be mandated for release, however, is the natural inflow above Masonry Dam. Therefore, stored water does not have to be released to meet instream flow requirements.

Table 3.1 Volume and Flows for Determining State of the Cedar River

Month	Reservoir Storage (ac-ft)	3 Month Moving Average Flow (Ac ft / mo)		
		Sub-Normal	Critical	Extreme
October	15700	2705	2214	1722
November	14000	3689	2618	2142
December	10000	6641	4735	3997
January	15500	8362	6641	6641
February	16000	5498	4665	4665
March	17000	5964	4919	4919
April	17500	9521	6903	6903
May	19400	13835	10453	9592
June	19400	14400	11306	10175
July	19500	10515	9223	5411
August	19400	5595	3874	2029
September	19400	2856	1845	1428

Although the Cedar River is the primary source of inflow to Lake Washington, rainfall, groundwater seepage, and several other streams provide substantial volumes of water. The amount of this local inflow is reduced by evaporative losses, especially in summer months. The net local inflow (inflow minus evaporation) provides about thirty percent of the yearly inflow to the lake but can be negative during the driest periods of the year.

Table 3.2 Cedar River Instream Release Requirements

Month	Required Release (acre feet per month)			
	Normal	Sub-Normal	Critical	Extreme
October	17524	11375	7994	7071
November	22017	18447	14876	13091
December	22751	19676	15372	13527
January	22751	19676	15372	13527
February	20549	17772	13885	12218
March	22751	19676	15372	13527
April	22017	19042	14876	13091
May	22751	19676	15372	13527
June	20232	17852	15471	8926
July	16602	10146	6764	4612
August	9039	7994	6764	4612
September	12615	8628	6546	4463

The highly permeable glacial deposits which underlie Masonry Dam form what is commonly known as the glacial moraine aquifer. Seepage losses from Masonry Pool to the aquifer range from 38 to 380 cfs, depending on the surface elevation of water held in storage. The capacity of the aquifer is not well quantified, although a water budget performed for the SWD [Chen, 1976] estimated it to be between 40,000 and 60,000 acre-feet. The aquifer acts as an underground storage reservoir, receiving water in one month and releasing it over the next several months.

Studies of the aquifer indicate that discharges are dependent on the elevation of the water level behind Masonry Dam and the level of water in nearby Rattlesnake Lake. These create a hydrostatic gradient that promotes or retards groundwater flow out of the moraine [URS, 1981]. Water is discharged from the aquifer to four sources; surface water which returns to the Cedar via Canyon Creek, groundwater which returns to the Cedar, surface water which is lost to the Snoqualmie via Boxely Creek, and groundwater which is lost to the Snoqualmie [USGS, 1989]. On average, about 80% of the seepage to the moraine is believed to return to the Cedar River over the next several months. If the flow into the aquifer exceeds its capacity, the aquifer is believed to spill the excess into the Snoqualmie system.

Seattle's second major water source is the Tolt River, located approximately 25 miles northeast of the city. The Tolt River drains an area of approximately 50 square miles. Elevations in the basin range from 5900 feet in the Cascades, to 600 feet near its juncture with the Snoqualmie River at Carnation. Precipitation and streamflow patterns are similar to those in the Cedar watershed, although the land area, and therefore the total volume of runoff, is reduced. Again, the most significant runoff events occur in the late fall and the early spring.

The Tolt River is composed of two forks which join near Shelton to form the Main Stem Tolt River. Currently, the only flow regulation is by an earth fill Dam on the South Fork Tolt. This dam rises to an elevation of 1,765 feet and provides 57,000 acre feet of storage, of which 56,000 acre feet is active. The rule curve under which the South Fork Tolt Reservoir is operated is shown in Figure 3.4. Above the dam the South Fork extends for about 12 miles, with a watershed that covers approximately 19 square miles. The SWD owns 28.6% of the land in the South Fork Tolt watershed. Much of the remainder is owned by private companies and managed for timber resources. The waters of the Tolt are subject to high turbidity during the rainy winter months. For short periods of time during the year, Tolt water is not available for municipal supply due to this turbidity.

The flow in the Tolt River serves fewer competing purposes than that of the Cedar. Provision of fisheries habitat and municipal water supply are the primary demands which must be satisfied. The City has water right permits on the South Fork Tolt for 150 cfs. Unlike the Cedar, however, instream flows on the Tolt have a higher priority than the SWD's water rights. If the inflow to the reservoir is insufficient for downstream fisheries needs, stored water is released to augment the natural flow. There are two levels of instream releases, with natural inflow used as the switching criteria. If inflow

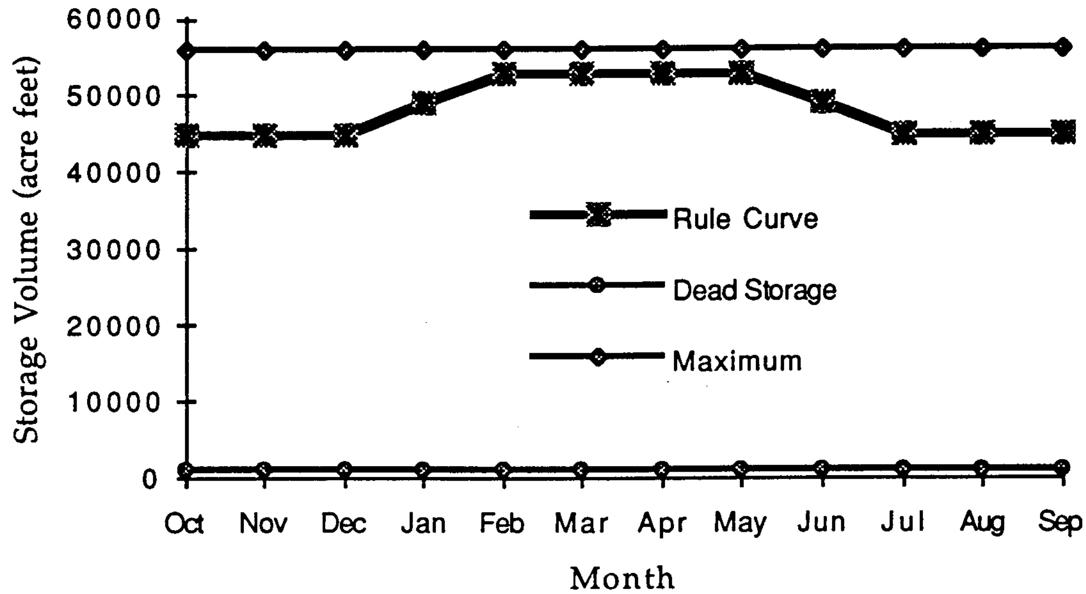

Figure 3.4 Tolt Reservoir Rule Curve

is below the normal instream requirement, the critical requirements are applied.

Municipal diversions from the Tolt River are made at the reservoir. Diverted water is channeled into the Tolt Regulating Basin and then transmitted to the distribution system via Tolt River Pipeline No. 1. This pipeline has a maximum capacity of 110 MGD. The SWD holds water right permits on the South Fork Tolt totaling 150 MGD. Current supplies from the Tolt diversion average about 60 MGD.

The primary demand that the SWD must meet is municipal water supply. Current water demand averages about 150 MGD, divided almost equally between single family residential and other usage (including apartments, businesses, and industry). Typically, summer water demands, such as lawn watering and car washing, cause a significant increase in water use during July and August. Demand in these months can be up to 1.5 times the yearly average. Most of the peak results from increased residential use, with minor increases in non-residential use. These non-essential uses of water are the primary reason that the system is currently stressed. The SWD is encouraging

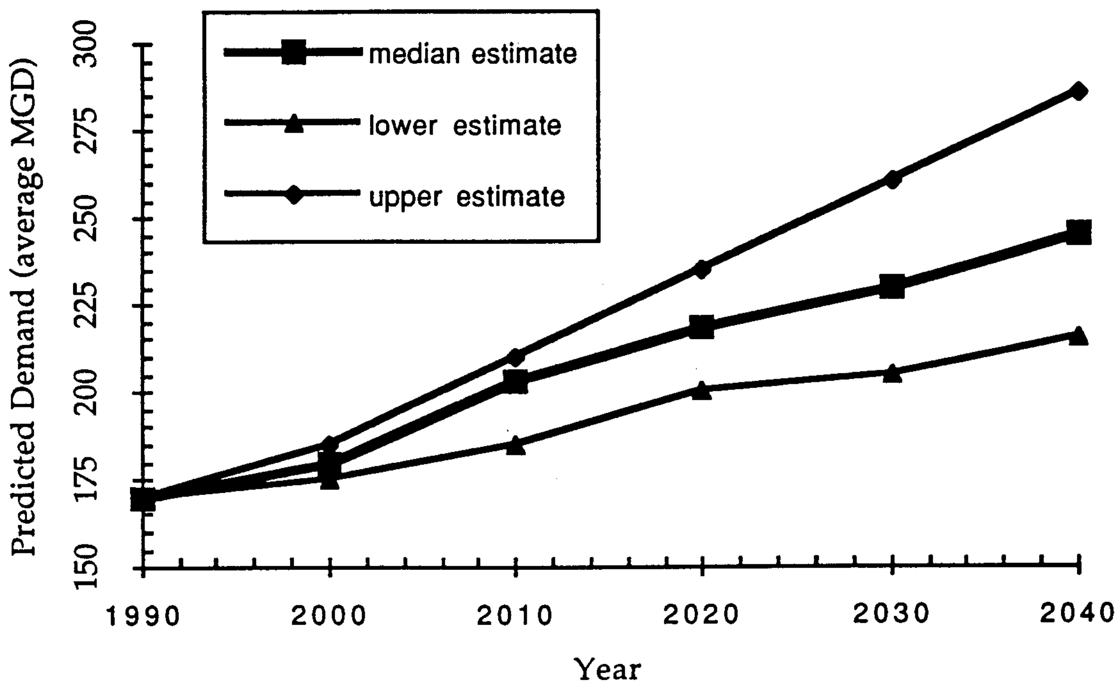

Figure 3.5 Projected Seattle Demands for 1990-2040

conservation programs aimed at reducing these peak demands. It remains to be seen what effect these programs will have.

The SWD has projected water use through the year 2035 based on economic and population forecasts, as well as a number of other factors. Figure 3.5 shows the high, medium and low estimates for future water demand. Seattle estimates of yield indicate that the current system configuration is capable of providing 173 MGD in 98% of the years. According to these estimates, new sources or alternative means of meeting demands will need to be developed immediately to maintain adequate reliability.

The SWD is reviewing several options for increasing the "firm yield" of the system. These include development of a diversion on the North Fork Tolt, increasing the pipeline capacity in Tolt Pipeline No. 1, and permanent pumping facilities to access the dead storage in Chester Morse Lake. The latter two options are the most likely to be implemented in the near future, as the high cost and environmental impacts of the North Fork diversion will require extensive review.

Tacoma

The Tacoma Water Division supplies drinking water to nearly 215,000 people living in the City of Tacoma and the surrounding area. Additionally, the Water Division provides water to Simpson Tacoma Kraft, a major industrial client. This system is supplied through conjunctive use of surface water from the Green/Duwamish River basin and groundwater from numerous wells. The Green River has its headwaters in the Cascades about 60 miles east of Tacoma and drains into Puget Sound at Elliott Bay south of Seattle (see Figure 3.1). The water from the Green River is of moderate quality. and treatment consists of screening, disinfection, and sedimentation in McMillin Reservoir. The numerous wells used by the Water Division are distributed throughout the service area. These wells provide high quality water which only requires chlorination.

The Green River stretches 60 miles, from its headwaters just south of the Cedar River basin, to the City of Tukwilla. Below Tukwilla, the Green's name changes to the Duwamish, and it continues another 11.5 miles to its mouth. The total drainage area above Tukwilla is 483 square miles. The basin supports mixed land uses, including logging in the upper reaches, farming in the flatlands between Auburn and Tukwilla, patchy residential development throughout, and heavy industry below Tukwilla.

Flow in the Green River is regulated by Howard A. Hanson Dam, located 53 miles above Tukwilla at river mile 64.5. The drainage area above Howard Hanson is approximately 220 square miles of primarily forested, steep sloping land. Access to the upper watershed is limited to maintain water quality. Howard Hanson Reservoir is operated by the Corps of Engineers (COE) for flood control and low-flow augmentation and has a maximum capacity of 107,000 acre-feet. The rule curve showing this operation is presented in Figure 3.6. From November through March, the reservoir is emptied to maximize flood storage. Beginning in April. the storage is increased to provide water for low-flow augmentation in the summer and early fall. In the congressional mandate that authorized construction of Howard Hanson Dam, irrigation and municipal and industrial supply were included as

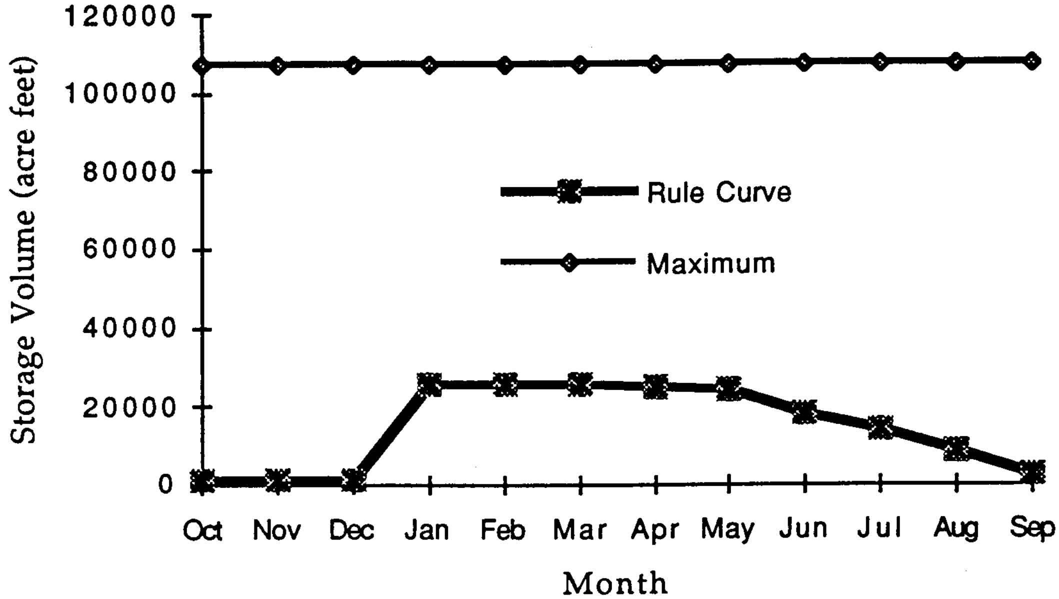

Figure 3.6 Howard Hanson Reservoir Rule Curve

purposes of the reservoir. Presently, however, the irrigation and M&I supply features have not been developed.

The Howard Hanson Dam and Reservoir has impacted the flow regime of the Green River in several ways. First, during a flood event, peak flow has been reduced to 12,000 cfs (24,000 acre feet/day). This means that flood releases are spread over days or weeks, providing an opportunity to utilize this flow in a conjunctive use operation. Also, during the spring runoff, reservoir refilling diverts about 25,000 acre-feet, moderating the flow below the dam. Finally, during the dry summer months, the storage in Howard Hanson can sustain 110 cfs for 115 days, even if there is no inflow to the reservoir. The overall impact of Howard Hanson Dam is year-round moderation of flow in the Green River. Since the dam's construction in 1961, releases have not exceeded 11,000 cfs nor been below 110 cfs.

The Tacoma Water Division maintains a diversion structure on the Green River near the town of Palmer, about thirty miles east of Tacoma. Diversions from the Green are transmitted to McMillin Reservoir via gravity flow Pipeline No. 1, which has a capacity of 72 MGD. During periods of the year when turbidity is high, the diversion from the Green is supplemented by pumping from the North Fork Well-field. Currently, there are six wells in the North Fork field with a total production capability of 72 MGD. The North Fork Wells and the Green River diversion are both governed by Tacoma's primary water right, so there is no added benefit to system capacity from concurrent operation. Historically, diversions from the Green River have averaged about 67 MGD or 85% of the total supply.

The City of Tacoma has a senior water right permit to approximately 112 cfs on the Green River. This permit is conditional only on the flow in the river, and has priority over all other uses. The Water Division has also registered a claim under the Water Rights Registration Act for 400 cfs based on postings in 1906 and 1908. The DOE estimates that it will require between 10 and 15 years to adjudicate water rights on the Green River.

To meet existing needs, the Department of Ecology has granted the Water Division an additional 100 cfs water right permit to be used in conjunction with Pipeline No. 5, which will be built in the near future. The second water right, however, is conditional on the maintenance of instream flow of 110 cfs. Refilling Howard Hanson Reservoir to the desired level for low-flow augmentation also has priority over the second water right. Therefore, the greatest benefit from the second water right will be realized when natural flow on the Green is high, typically from November through June. This will reduce the need to pump groundwater, allowing natural recharge to occur.

There are many competing uses for Green/Duwamish River flow. The primary purpose of low-flow augmentation by Howard Hanson Reservoir is protection of fisheries habitat. The Green and its tributaries provide about 105 miles of spawning and rearing water below the Palmer diversion. This diversion acts as a barrier to fish passage. Chinook, coho, and chum salmon and steelhead trout provide substantial economic, recreational and tribal cultural benefits. Both the volume and the timing of Green River flow are critical to the maintenance of these important fisheries. The instream flow requirement of 110 cfs has priority over all other uses except Tacoma's primary diversion.

Another major use of the flow, particularly below Tukwilla, is the assimilation of wastes. The Duwamish flows through a heavily industrialized sector which contributes significant pollutant loadings, both in runoff during storms and through direct outfalls. Without sufficient flow, the assimilative capacity of the river could easily be exceeded resulting in further contamination of Elliott Bay. From its mouth to five miles upstream, the Duwamish is dredged to allow passage of oceangoing vessels. Commercial navigation utilizing this stretch of river is an important industry on which much of the region relies. Adequate flow is necessary to accommodate this use.

When the water diverted from the Green River is insufficient to meet the needs of the Tacoma service area, additional supply is drawn from a number of wells. The most important of these by far is the South Tacoma Well-field which has 13 wells with a total pumping capacity of 48 MGD. Previous analysis indicates that the annual natural recharge of the South Tacoma aquifer is 12,000 acre feet [Tacoma Water System Plan, 1987]. The maximum capacity of the aquifer is estimated to be 42,000 acre-feet, about three and one-half times the annual recharge rate.

Additional wells and their capacity include Prairie Ridge Spring (0.8 MGD), Dash Point Wells (1.3 MGD), University Place Wells (3.8 MGD), Gravity Wells (8.0 MGD), Portland Avenue Well (1.8 MGD), and Tide Flats Well No. 1 (1.1 MGD). Analysis conducted by the Water Division ascertained that there is hydraulic continuity between the wells, and their use is limited by natural recharge rates. In total, the Water Division uses 22 wells to meet approximately 15% of the annual demand. These wells are primarily used to meet peak demands during the summer and early fall or to offset Green River supplies during periods of high turbidity.

Because natural recharge rates limit the yield provided by Tacoma's wells, artificial recharge by injection pumping or some other method would appear to offer the potential for increasing the system's yield. Unfortunately, the EPA has prevented Tacoma from attempting this due to the proximity of the aquifers to several Superfund sites. The EPA has invested considerable time and expense in monitoring groundwater pollutant plumes at these sites, and their fear is that pressurized recharge will alter flow patterns and nullify their previous efforts.

The Tacoma Water Division provides water to municipal and industrial users. Current demand averages about 77 MGD, with approximately 40 MGD being used for residential or commercial purposes and another 7 MGD lost to leakage. Typical use patterns for residential use are similar to those of Seattle, with high summer demands and moderate winter needs. However, the Tacoma system differs significantly from the Seattle system in that a single industrial user, Simpson Tacoma Kraft, consumes almost 40% of their supply, or 30 MGD. Simpson's usage is nearly constant throughout the year, thus moderating the overall peaking of Tacoma's demand. The high percentage of industrial use also has implications on economic impacts should curtailment be necessary. When the worst drought in recent history occurred in 1987, Tacoma was able to curtail supplies to Simpson and maintain sufficient allocations to other users.

The Water Division has projected water use through the year 2040 based on economic and population forecasts by the Puget Sound Council of Governments (PSCOG). Table 3.3 shows the minimum demand projection for the Tacoma service area. This projection does not include any new wholesalers, although Tacoma expects to become more involved in meeting the water needs of nearby communities. Tacoma's ability to meet these needs are currently constrained by the capacity of Pipeline No. 1, the lack of a pipeline for transporting the second Green River diversion, and the natural recharge rate of the city's aquifers. Without substantial system upgrades or an alternative operating plan, the city will have trouble meeting demands on an annual basis as early as the year 2000.

The city is currently in the process of obtaining permits to construct Pipeline No. 5. They hope to construct this line as early as 1995. This will provide a means to utilize the second Green River diversion. This may delay the need for further development for several years, but won't eliminate it. It is unlikely that the EPA will allow artificial recharge of the South Tacoma aquifer at any time in the near future. Therefore, the possibility of drawing more water from the city's wells appears to be slim.

Combined System

The combined system envisioned in this thesis incorporates all elements of the two individual systems. By constructing a pipeline that can transfer sufficient quantities of water between the systems, their complimentary characteristics can be utilized to produce significant gains in yield and reliability. Chapter 4 of this thesis documents the conceptual evolution and expected benefits of an intertied system. Here, it is sufficient to note that the physical and economic feasibility of an intertie has been considered previously, and the results of those studies offer promise that a cooperatively managed system is possible [Tacoma Water System Plan IV, 1987].

In their 1987 study, Charles Howard and Associates found the most favorable means of connecting the systems was via an intertie between Tacoma's proposed Pipeline No. 5 and Seattle's diversion structure at Landsburg [Tacoma Water System Plan IV, 1987]. This was referred to as the High Level Intertie option. The study found that a 32 inch diameter pipe would be capable of delivering 20 MGD to Tacoma, using about 500 kW of pumping power to overcome 57 feet of static head. The same line could be used to carry 40 MGD from Tacoma to Seattle by gravity feed.

Of the options studied by Howard, the high level intertie was found to be the most economical. The total length of pipe would be approximately 29,800 feet, at a cost of $ 200.00/foot. Including permitting, right-of-way, and present value of annual operating costs, the total expected cost of this option would be about $ 7.4 million. Two alternative options discussed by Howard had total costs closer to $ 10 million. Obviously, the sizing and total cost of an intertie depend on the amount of water which it will transmit. These earlier estimates, however, will facilitate preliminary comparisons between the expected costs and benefits of an intertie with those of other source development options.

It should be noted that the feasibility of an intertie between Seattle and Tacoma is conditional on the construction of Tacoma's Pipeline No. 5. Without this line in operation, the intertie would need to connect directly to the Tacoma diversion structure near Palmer or to Pipeline No. 1, much further south. Although this is physically possible, the added costs of additional pipe, right-of-way acquisition, and pumping power would make this alternative economically inferior. As Pipeline No. 5 is currently in the permitting process, with construction expected in 1995, this does not appear to present a problem.

Chapter IV
Perspective of the Intertie

In this chapter, the historic precedent for consideration of an intertie is presented, as well as documentation of the evolution that this concept has undergone. Recently, local utilities have recommended that the full range of impacts on system performance of an intertie be studied. The various reasons that such an intertie would be expected to improve the reliability of these water resource systems are explained. Following is a discussion of the choice of appropriate measures of system performance, before and after the implementation of an intertie. Finally, consideration is given to an alternative system performance measure (economic loss functions). The difficulties involved in formulating accurate loss functions and practical barriers to their implementation are presented.

The Intertie Concept

The possibility of interconnecting and jointly operating the water supply systems of Seattle and Tacoma has long been considered as a means of relieving the stress that growth has placed on these systems. Initially, the concept was that either utility could transmit limited supplies to the other in times of emergency [SWD, 1977]. The impact of catastrophic events such as transmission line or mechanical failures could be mitigated through the use of this alternative source. An intertie was also envisioned as a method by which supply capacities could be balanced and development of new sources could be economically incremented [CH2M Hill, 1974].

This idea evolved, and in 1980 the long range water supply plans of each municipality depicted the intertie as a means of sending substantial quantities of water to the other over a period of years. The timing and direction of transfers would be determined by the chronology of system expansions. The concept was that as new sources were brought on line by either utility, the excess capacity in the initial years of operation could be marketed to help recover development costs and to delay future source developments [Tacoma Water Division, 1980; Seattle Water Department, 1980]. The utilities expected to achieve significant savings through improved planning and coordinated system expansions.

More recently, the complimentary characteristics of the two systems have received greater attention, and the possibilities for conjunctive operation have begun to be explored [Tacoma Water System Plan IV, 1987]. The 1987 analysis quantified the expected costs of connecting the two systems via a new transmission line, and documented one scenario by which the transfer of water could prove mutually beneficial. This analysis, however, only considered excess water available from the systems on a daily basis. It did not consider the additional benefits that could be derived from a comprehensive management policy for the entire intertied system.

Tacoma's Water System Plan for 1987 recommended continued review of the potential benefits of a Seattle-Tacoma intertie and regional supply transmission system [Tacoma Water System Plan, 1987]. Specifically, the plan called for evaluation of the impact on system reliability that such an intertie could offer. To maximize the benefits offered by an intertie, the regional water supply system will require a coordinated operating plan. Because the prevailing political climate is resistant to change, significant advantages of this type of operation are a necessary condition for its implementation.

As was noted earlier, the original intent of an intertie was to improve the performance of the systems during emergencies. Later, it was seen as a means for diverting excess water between the utilities to offset future expansion. These benefits will still be available if the current intertie design is completed, but they are no longer viewed as the only advantages. Increased reliability and improved system performance on an annual basis are now of equal or greater concern. A cooperatively managed system is better equipped to respond to annual low-flow periods.

However, the primary threat to system reliability is drought. Therefore, the maximum benefit that could be offered by an intertie would be improved performance during drought events. Different flow scenarios stress the system in different ways. An appropriate operating policy must be flexible enough to respond to each of these threats and straightforward enough to have relevance for real time operations. This is the focus of the analysis completed in this thesis.

Expected Intertie Benefits

There are numerous reasons why an intertie would be expected to enhance the reliability and yield of the water supply systems of Seattle and Tacoma. First, these systems have supply system characteristics and demand patterns that compliment each other well. Also, the streamflow regimes, particularly with the influence of Howard Hanson Dam, are different enough to offer potential benefits through proper timing of withdrawals. Additionally, the diversity and flexibility gained by cooperative operation will create a more robust system which should improve drought management.

The Seattle supply system functions as a run of river operation during the wet winter and early spring months. From about June through November the system relies on water stored in its two reservoirs, Chester Morse Lake and the South Fork Tolt. The Tacoma system has a run-of-river component in the Green River diversion, coupled with groundwater pumping capabilities to meet peak demands. The focus of an intertie would be to conjunctively use the storage in the Seattle system, with the firm yield and secondary diversion of the Green River, reserving aquifer storage to meet expected shortfalls in extreme year events.

Although the Seattle system relies on stored water to meet demands during much of the year, over year storage is effectively limited, due to small active storage capacities. One indicator of this is the ratio of active storage to mean annual flow which is 0.09 for the Cedar River, while on the Tolt, it is 0.45. Limited pipeline capacity also restricts the ability of the system to utilize much of the active storage on the Tolt. This combination of factors can cause water retained in storage in the Cedar to be depleted in a few months if inflow is inadequate. Coupling this high flow system with a sufficient active storage capacity, such as the South Tacoma Aquifer, would allow conjunctive use of the sources. In months when flow on the Cedar is high, demands would be met from this excess, and the aquifer would be allowed to naturally recharge. When flow is low or drawdown is substantial, the aquifer would be pumped to augment supply.

Tacoma's secondary diversion on the Green River would also play an important role in a cooperative use system. Water from the secondary diversion is available in most months with the exception of July, August, and sometimes September. If stored water from the Seattle system were available to supplement Tacoma's diversions in these months, it is highly likely that Tacoma could provide additional water the remainder of the year, particularly in October and November when Seattle reservoir storage is traditionally at its lowest. In the few years when Green River flow is inadequate to allow the second diversion, pumping from Tacoma's wells would be substituted.

In addition to the complementary nature of sources mentioned above, the variation in M&I demands on the two systems should offer some benefit when an intertie is in place. Tacoma's demand does not exhibit peaks of the same magnitude as Seattle's, primarily due to the stable demand of Simpson Tacoma Kraft. If the systems are intertied, the combined system will benefit from this large, near constant demand and summer peaks will be moderated. Shifting more of the total demand from summer months to the remainder of the year will allow current reservoir storage to operate more effectively.

Another potential benefit of an intertied system relates to instream flow. If the system is managed effectively to maximize reliability of the water supply, there is a greater chance that instream releases will not be restricted. During periods of high flow on any of the rivers, the excess will be utilized to meet the demand of the combined service area. If flow is lower or reservoir drawdown begins, pumping from storage will minimize diversions from rivers, leaving more water in storage. On the Cedar River, instream release is based partly on reservoir level. Therefore, pumping from aquifer storage will help maintain high instream releases. On the Tolt River, water retained in storage is used to augment natural flow for instream needs, so diminished reservoir releases will improve availability for instream release. Finally, if the South Tacoma aquifer is pumped early in the summer, but inflow to the reservoirs is adequate, rights to the additional stored water can be negotiated for release to increase instream benefits.

This leads to one of the critical issues that must be addressed by any operating plan - specifically when and how much to pump from the South Tacoma aquifer. Ideally, the aquifer would only be used to supplement other sources in years of extreme need. If perfect foresight for summer and fall flows were available in the spring, the optimal level of aquifer release could be determined. Unfortunately, this type of predictive capability is not possible with any degree of accuracy. Therefore, withdrawals must be based on other indicators, such as the level of storage in the reservoirs and the time of year (i.e., the expected future inflow and demand). This process will lead to some unnecessary releases and possibly reduced reliability, but if an operating policy is carefully designed, these adverse consequences can be minimized.

Performance Measures

Every water resource system is unique, and, therefore, no single measure of reliability during drought is sufficient for all. Firm yield offers a measure of the volume of water that can be withdrawn under specific conditions, but it does not communicate the full impact of failures should demand exceed this amount. More descriptive measures of performance are necessary to fully convey the consequences of using any operating policy within a particular system configuration. The impacts of system failures are conditional on the frequency of failure, the length of each sojourn into failure, and the magnitude of each failure that occurs. As discussed in Chapter 2, reliability, resiliency, and vulnerability attempt to transform these effects into quantifiable criteria.

A broad range of factors influences the applicability of these criteria to specific systems. Features of the supply system, consumer attributes, and demand patterns are just a few of these. Some impacts, such as lost revenue from the sale of water, are linear with respect to failure volume. Others are distinctly non-linear, such as the compounded economic effects that would accompany plant closures due to insufficient water supply. Instream demands are subject to different failure characteristics than M&I needs. For instance, beyond some threshold level, the viability of biological populations may be threatened. Also, the loss of fisheries habitat cannot be fully quantified until the fish spawned in a failure year return as adults. The timing of failures may also affect their impact with winter shortfalls being more detrimental than those that occur in summer.

To alleviate some of the difficulty involved in quantifying failures, this study concentrated on a range of performance indicators. The models developed here enable these criteria to be studied individually, and the impact of alternative operating plans are understood. Retaining the performance measures as separate entities and utilizing the negotiation features built into these models (discussed in Chapter 5), can facilitate trade-offs among competing interests.

The specific evaluation criteria used in this study were the following:

Reliability - (1) the number of months in which water supply was sufficient to meet demands divided by the total number of months in the historic record, (2) the probability that the system could supply normal instream releases in any period.

Resiliency - the probability that the system rebounds from failure in the next month after a shortfall occurs.

Vulnerability - (1) the average magnitude of failure that occurs for a given demand level, (2) the largest shortfall that occurred during the simulation run.

In some cases, a number of factors affecting system performance can be grouped into an economic loss function for valuing releases other than the target demand. However, accurate description of this loss curve is a complex procedure. It is highly dependent on consumers' demand elasticity. For instance, municipal water supplies can typically achieve up to 10% savings in summer by voluntary outdoor use restrictions. This curtailment can usually be accomplished for minimal cost. Industrial demands, on the other hand, are far less flexible. Any significant restriction on Simpson Tacoma Kraft's water supply (40% of Tacoma's total demand) can lead to plant closures and layoffs with serious economic repercussions. In either case, if the magnitude of the shortfall becomes too great, costly system failure is unavoidable as larger shortfalls cause disproportionately larger losses. The economic loss function must account for the demand elasticity, as well as the non-linear nature of failure costs.

Accurate economic loss functions also need to consider the length of time that a system is in a failure state. Voluntary curtailments may be sufficient for a short time, but droughts of several months or years in length might require mandatory restrictions. The losses due to a short term failure are probably very different from the costs of a longer term one. The time of year when a failure occurs also affects the expected cost. Different months have different demand elasticities and, therefore, different loss functions.

The inclusion of varying points of view is another concern when developing economic loss functions. Competing interest groups may value intangibles differently, and uniting these views into a single indicator may not be possible. Instream flow benefits from recreational, cultural, and scenic uses of rivers are liable to be particularly contentious. It is questionable if the utilities, the tribes, the environmentalists, and the DOE will ever reach consensus on appropriate values for these benefits. Without this consensus, economic loss functions are not likely to gain wide acceptance as a means of evaluating performance.

If appropriate economic loss functions are developed for a system, the consequences of alternative operating plans can be explored through the use of a simulation model and this single criteria. As is evident from the discussion above, however, developing adequate measures of economic loss is a difficult task. Therefore, rather than attempting to include all of the impacts of drought into a single measure, it is often advisable to look at the system's performance on several indicator criteria. In this study, experiments were conducted using both the discrete criteria and an economic loss function to gauge performance. The reason for this is discussed further in Chapter 6.

Chapter V
Model Construction

This study reports on the use of simulation models to analyze the effects of an intertie on the performance of water supply systems in Seattle and Tacoma. This chapter describes the construction of models using STELLA II™. Next, the specific characteristics of the models built for this study are listed. The physical components which replicate system features, and the decision rules which form an operating policy are described to communicate the essential characteristics of these models. Also, the specific evaluation criteria that were developed are discussed. Finally, comments about the modeling effort expended and possible applications of these models in management of the system are included.

Using STELLA

The simulation models developed using STELLA II™ were composed of a number of modules, each describing a particular component of the actual systems. These included the Tolt River, Cedar River, Green River, Cedar Moraine, Lake Washington, South Tacoma Well-field, and other Tacoma Wells. Modules were also included to predict future inflow, determine the Cedar instream flow requirement, calculate allocation targets for the Seattle reservoirs, allocate water from the Tacoma wells, and to compute failure characteristics.

STELLA provides four tools with which a system can be graphically represented. These are Stocks, Flows, Converters, and Connections. Stocks can take on four identities, including reservoirs, queues, conveyors, and ovens. These identities are self explanatory in regard to their functions. In these models, only reservoirs were used. Reservoirs represent a point at which items, in this case water, can be accumulated. Flows are used to describe rate of flow between reservoirs. Converters are used to store parameters, constants, or graphical functions which are used in computations. Mathematical processes, such as conversions, can also be performed using converters. Connections are used to show logical relations between Stocks, Flows, and Converters.

The process of building a model in STELLA begins with recognizing essential physical features of the module being modeled. In the case of these systems, the key reservoirs and all potential inflows and outflows are identified and drawn using the appropriate symbols. Flows that transfer water from one reservoir to another, for instance the seepage from Masonry Pool to the Cedar Moraine Aquifer, are connected to both of these (see Figure 5.1).

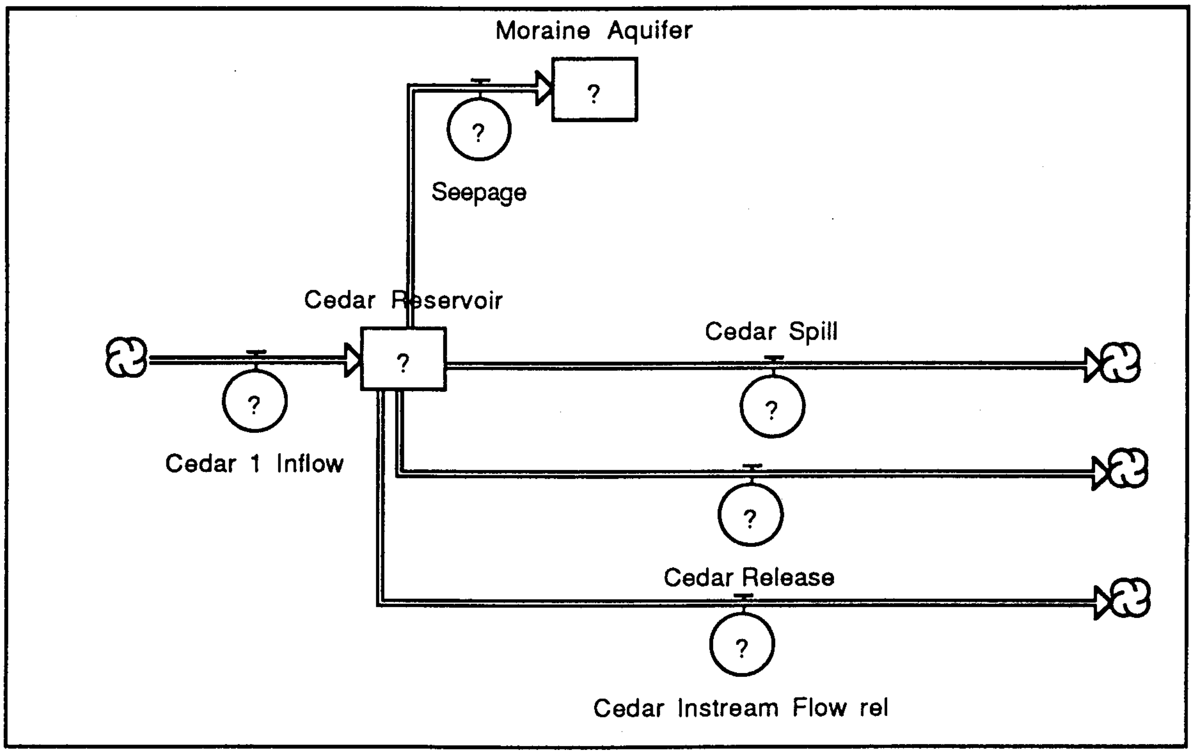

Figure 5.1 Identification of Key Elements in the Cedar Module

When a picture such as this is drawn within STELLA, an equation for reservoir storage as a function of time is automatically compiled. As an example, in the stock named Cedar Reservoir, the following equation has been compiled by the program:

Cedar_Reservoir(t) = Cedar_Reservoir(t - dt) + (Cedar_1_Inflow - Seepage - Cedar_Instream_Flow_Rel - Cedar_Release - Cedar_Spill) * dt

In this manner, continuity within any element of the model is guaranteed. The user merely defines the inflows and outflows related to a specific reservoir, and STELLA defines the storage function. The question marks within the elements on the diagram above indicate that these quantities have not been defined. In the case of the Reservoir, the only information required by STELLA is the initial storage volume.

After identifying physical components of the system, the user begins to establish the logical relationships between them. This typically requires additional information to be added in the form of converters. For instance, the inflow data are in cubic feet per second, but the reservoir storage is in acre-feet, so a conversion is required. This conversion is dependent on the number of days in the month in which the flow occurs, as well as an algebraic operation. Other quantities, such as evaporation from reservoirs or the Cedar seepage losses, may require additional information including storage to volume curves or surface area to volume curves. This information can be input in a graphical or algebraic format that other components can access as necessary.

Logical relationships between Stocks, Flows, and Converters are indicated by Connections (see Figure 5.2). Then equations are developed for each element of the model based on mathematical processes, and the values of other elements. These equations consist of algebraic and trigonometric manipulations of the input variables and can be conditional or singular functions. Appropriate constraints and bounds can also be included in the computation of these values.

If a logical connection exists between two elements, the equation of the dependent element must include a reference to the first. Likewise, no other element can be referenced within an equation unless a logical connection has been defined between the elements. This feature forces the modular to consider the logic governing any process before establishing equations to approximate it. It also prevents errors of omission and aids in the debugging process.

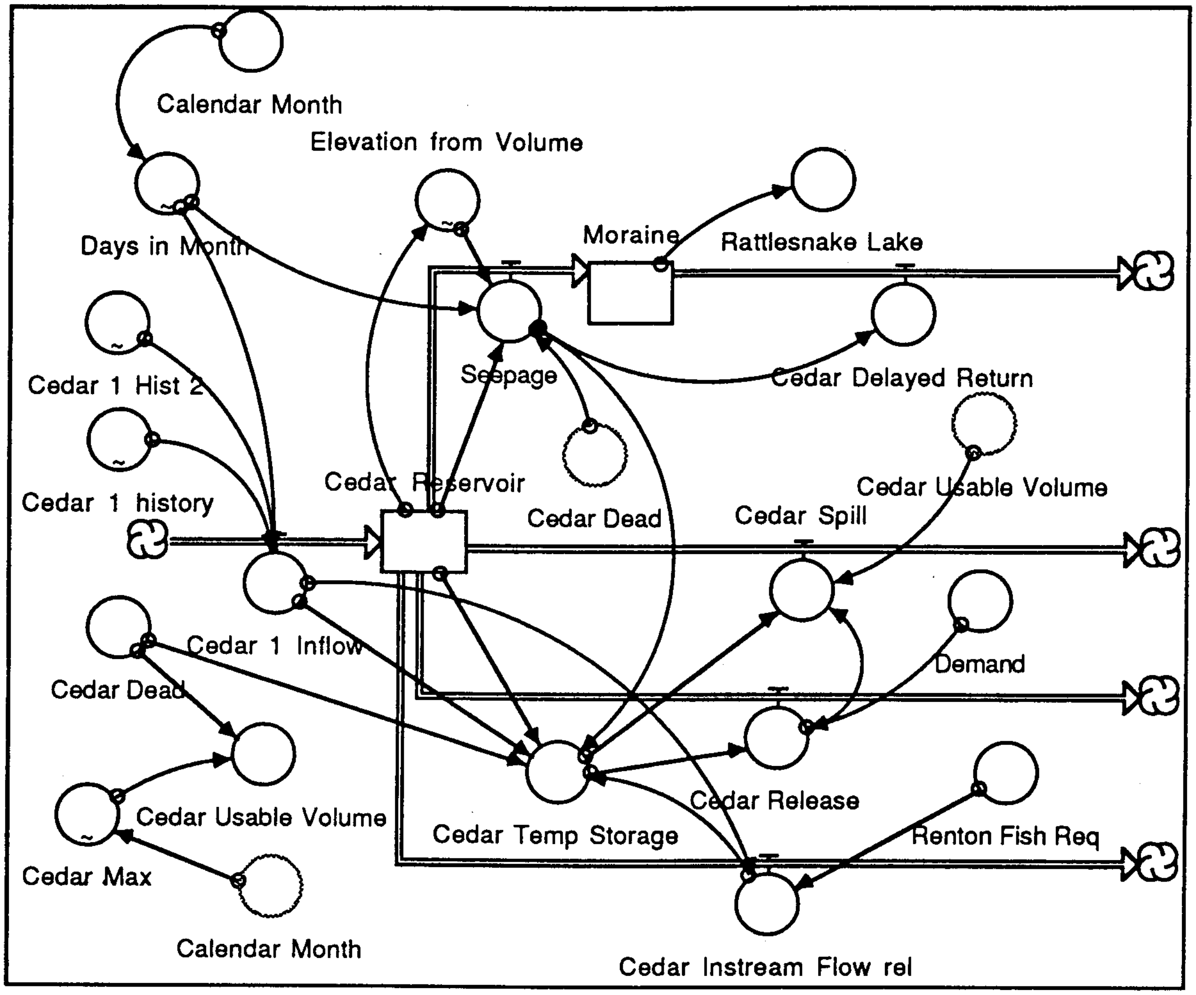

Figure 5.2 Interrelationships within the Cedar Module

Figure 5.2 depicts a simplified version of the Cedar module with the interrelationships between various elements shown by connectors. When working in STELLA, the user may view the equation that governs an inflow or outflow by double clicking with the mouse on the chosen element. When building the model, documentation can be provided to allow the user to understand the process which is being described. This documentation is available to the user at any time during the application of the model. For example, if the person using the model questioned why "Seepage" was connected to "Elevation from Volume", double clicking on "Seepage" would show the following:

Seepage = if Cedar_Reservoir <= Cedar_Dead then 0 else (35.47+0.023039*((Elevation_from_Volume/10)-1500)^2.3035) *86400/43560*Days_in_Month

DOCUMENT: Seepage from Masonry Pool is governed by the surface elevation of water behind Masonry Dam. This water provides head which forces seepage through the gravely glacial till which underlies the dam and into the Cedar Moraine Aquifer. In this simulation, the equation used to calculate the amount of this seepage is from the USGS memo to the SWD dated February 23, 1989. The USGS's "restricted non-linear model" was chosen because that is what the SWD normally uses.

Documentation should address any questions which might arise. Because the user of the model can see the logical connections made between elements, and can look at the equations which govern the simulation, and can read the documentation provided by the builder, the model is no longer a "black box" from which answers magically emanate. Instead, it becomes a tool, with its logic open to inspection, to allow the user to make investigations and modifications, as necessary.

Construction of System Modules

As was noted earlier, modules were developed for all of the major physical components of the water supply systems of Seattle and Tacoma. The complete formulation can be seen in Appendix A. The equations and documentation are provided in the Appendix B. An explanation of the complex interactions and constraints under which each element of the models operates would serve little purpose here. Instead, each of the major modules, its important constraints, and its relationship to the body of the model in which it appears is explained briefly below:

Cedar System - For simulation purposes, the primary element of the Cedar River system is flow regulation by the reservoirs. In the models developed for this study, Masonry Pool and Chester Morse Lake were treated as a single reservoir, with the storage capacity and seepage characteristics of the combination of reservoirs. Inflow to the single reservoir is computed from historic data provided by the SWD. In a refill cycle, Chester Morse Lake is allowed to refill to the level of the Crib Dam before spilling water into Masonry Pool, where it is subject to large seepage losses. Likewise, during a drawdown cycle, the storage in Masonry Pool is utilized first, retaining as much water as possible behind the Crib Dam where seepage is minimal. When inflow to the reservoir is high and releases are low, the Crib Dam is inundated and a single reservoir results, all of which is subject to seepage. The minimum storage in Masonry Pool, and, therefore, the minimum seepage, is constrained by elevation of the service spillway. Seepage losses are calculated using the USGS equations and the water surface elevation of storage in Masonry Pool at the beginning of each period.

After adding inflow to the reservoir and subtracting seepage to the moraine, the remaining volume is allocated as follows. Instream flow receives the highest priority to natural inflow, although stored water is not used for meeting this demand. The required release for instream flow augmentation at Renton is computed in a sub-module based on Department of Ecology (DOE) requirements and the inflow in the lowest reach of the Cedar. If, after making the instream release, there is sufficient water remaining to meet the target demand, then that amount is released, otherwise, the total available water is released. Finally, if there is water remaining, spillage release is calculated. The active capacity of the reservoir is calculated on a monthly basis by subtracting the dead storage from the maximum volume for the current month as specified by the rule curve. If the total volume after releases is above the active capacity, the amount above capacity is spilled, and the reservoir begins the next month full.

Cedar Moraine Aquifer - The Cedar Moraine Aquifer receives its inflow from the Masonry Pool as described above. Outflows are computed using regression equations developed for the Seattle Water Department [URS, 1981]. Several outflows are dependent on the surface elevation of Rattlesnake Lake, while others depend on the level of Masonry Pool. Water surface elevation of Rattlesnake Lake is, in turn, is a function of storage in the aquifer. The theoretical basis for this is that head gradients between Rattlesnake Lake and the moraine act to drive the flow of groundwater. Some of the outflow returns to the Cedar River, while some is lost to the Snoqualmie River. Also, the aquifer has a storage limit above which water is lost to the Snoqualmie system. Returns from the aquifer enter the Cedar system above Renton and also above the diversion.

Lake Washington - Water in the Cedar River, except what is diverted to meet municipal demands, flows into Lake Washington. Additionally, the lake's local inflow and evaporation are computed using the historic record supplied by the SWD. Releases from the lake are made for the fish ladder and lockages based on historic trends. Flow through the salt drain is calculated as a function of lake elevation, using the equation developed by Snyder [1983].

Spillage from the lake is computed based on the operating rule curve supplied by the Corps [Loren Jangaard, personal communication, 1991]. The minimum lake surface elevation, as set by congressional mandate, is 20 feet. If the level of Lake Washington drops below this, releases are made from Masonry Pool to regain this level. In the simulation models developed for this study, periods of insufficient lake elevation were noted as failures, but no additional Cedar River releases were made. The Corps has some flexibility in regard to operations of the locks and salt drain, and, if necessary, can limit outflows from the lake. If the number or magnitude of lake failures exceeds the ability of the COE to reduce releases, alternative operating plans will need to be developed.

Tolt System - The Tolt system is composed of three forks of the Tolt River, with historical inflow data available for each. Currently, only the South Fork is utilized for water supply. The North Fork and Main Stem are included in the models in the event that analysis of further development options is desired. The South Fork flows into the reservoir where its releases are controlled. On the Tolt system, instream flow demands have priority to all water, including reservoir storage, therefore, instream releases are made before any other allocation. These releases are based on normal and critical demand levels set by the DOE. Remaining water is next allocated to M&I releases based on a target demand but constrained by storage in the reservoir and the capacity of the South Fork Tolt pipeline. If water remains in storage, spillage is calculated similarly to the manner it was in the Cedar system.

Green System - Tacoma's diversion on the Green River is subject to inflows and regulation at Howard A. Hanson Dam. If inflows are above 112 cfs, then this amount is passed through the reservoir and diverted at the Headworks by Tacoma. If inflows are below 112 cfs, then Tacoma has a senior right to the total natural flow. As flow increases, regulation is accomplished by the Corps based on Howard Hanson's rule curve for instream flow augmentation. The minimum constant release for instream concerns is 110 cfs. If water remains after allocating Tacoma's primary water right, releasing instream flow, and raising Howard Hanson Reservoir to the desired level, then Tacoma's secondary water right is met to whatever extent possible. Finally, any water remaining in storage above the desired level is spilled to prepare the reservoir for flood control in the following period.

Tacoma's ability to utilize Green River water is dependent on the pipelines in place from the headworks to the distribution system. Currently, only a single pipeline with a capacity of 72 MGD is available. In the models, Pipeline No. 5 can be added by changing the value in a single converter. Because the construction of this pipeline appears imminent, and an intertie is not feasible without it, the bulk of the analysis done herein was based on the expanded system.

South Tacoma Wells - The well-field operated by the Water Division in South Tacoma was modeled as an underground reservoir which received 12,000 acre-feet/year of inflow due to natural recharge. This inflow was distributed throughout the year, with winter months having a greater proportion of the total recharge. The maximum storage in the aquifer was set at 42,000 acre-feet, and any recharge above this level was presumed to be exfiltrated.

Releases from the well-field to the M&I supply were made as required to augment other supplies. The maximum release in any month was constrained by the volume in storage and the pumping capacity. The current pumping capacity is about 48 MGD early in the season, dropping to about 43

MGD later in the year. The intent of the operating plan used in the intertied model was to retain the storage in the South Tacoma aquifer as an emergency supply for use during critical events.

Other Tacoma Wells - The seven additional wells in the Tacoma system were labeled as Gravity Wells in the models, named for the two largest wells. The total capacity of all these wells is 16.7 MGD. There is some debate over whether continuity between these wells and the South Tacoma aquifer is a limiting factor in their production. As currently structured, the models treat these wells as a discrete supply capable of producing a constant amount of water. This decision was based on conversations with Tacoma Water Division personnel [Craig Gibson, personal communication, 1991].

If the user wishes to alter the models to simulate continuity between the sources, the pumping rate for the South Tacoma Wells can be increased, and the Gravity Wells pump rate can be decreased, accordingly. At the limit, the Gravity Wells pump rate would be reduced to 0 MGD and the South Tacoma Wells increased to 65 MGD. This would have the effect of withdrawing all of the water out of the same aquifer, implying 100% continuity between the wells.

Development of Policy Modules

In addition to modeling physical components of the system, modules were included to derive an appropriate operating policy. When numerous sources are available from which to withdraw water, the prioritization of withdrawals is a complex process. Optimum allocation of releases between reservoirs minimizes spill, which is of no value to the system. Withdrawals from aquifer storage should be limited to retain this water for critical periods, yet, due to pumping limitations, must begin early enough in a drawdown cycle to be effective. The operations and policy modules developed for these models included a demand module, a space rule module, a system state module, a module to enhance the use of the Tolt system, and a module to determine well releases.

Demand Module - To calculate the demand on the system, a module is included which does the following:

(1) Multiplies base demand times the monthly variation
(2) Converts from MGD to acre-feet per month
(3) Multiplies the total by the M&I demand factor
(4) Subtracts excess flow in the Cedar from the total demand (flow from the aquifer return and Cedar 2 inflow above that required to meet the Renton instream need)
(5) Allocates this demand by the following priorities:
    1. Tacoma primary diversion
    2. Tacoma secondary diversion
    3. Tacoma Gravity Wells
    4. Seattle Tolt system
    5. Remaining sources (Cedar and South Tacoma Aquifer)

System State Module - A number of decisions as to the proper allocations of water from alternative sources were dependent on the current condition of the system. For modeling purposes, eight states were chosen to characterize the status of the combined system, with regard to meeting demands. Table 5.1 lists these states and qualitatively describes the criteria used to establish the state of the system. A complete quantitative definition of the descriptors used in the model can be found in the documentation for the system state converter (See appendix A).

Table 5.1 System States and Indicator Criteria

State	Description	Indicator Criteria
0	Spilling	Both Tolt and Cedar reservoirs spilling
1	Critical I	Tolt and Cedar reservoirs severely drawn down
2	Dump Tolt	Tolt reservoir high and predicted inflow low
3	Use Tolt	Tolt reservoir high in August, September
4	Critical II	Cedar being drawn down in August, September
5	Use Cedar	Tolt reservoir low and Cedar reservoir high
6	Use all Cedar	Cedar will refill in next month
7	Other	Any situation not described above

Space Rule Module - When the Seattle reservoirs are in the process of refilling, releases are allocated according to the space rule. This procedure attempts to minimize the potential loss of water due to spillage, by equalizing the freeboard in each of the reservoirs. To make this computation, some estimate of the future inflow to each of the reservoirs must be made. In the models, this prediction is made in a sub-module, which utilizes the historic data and looks ahead three months, subtracting required instream releases from historic inflows to determine the expected usable inflow to each of the reservoirs. If the exact instream release requirement was known, this process would provide perfect prediction of usable flow. Unfortunately, instream releases are dependent on reservoir levels, as well as flow, and are not known until reaching the month in which the release is to be made. Also, the seepage loss from the Cedar reservoir is dependent on reservoir level and was not accounted for in the usable flow prediction. Although disregarding these complexities in the computation of available water means the models are using less than perfect prediction, it was not thought to present a problem. In any realistic operating plan, the ability to optimize use of stored water will be limited by predictive capabilities.

Tolt Release Module - Releases from the Tolt Reservoir ordinarily would be governed by targets established with the space rule. Due to pipeline capacity constraints, however, this does not provide an efficient method of operation. Use of the space rule alone often allocates water from the Cedar before utilizing Tolt storage because of the larger inflows on the Cedar. When drawdown begins and inflows are low, the water from Masonry Pool/Chester Morse can be exhausted in a few months if demands are high. Then when allocations from the Tolt are desired, the pipeline capacity limits its use to 110 MGD, less than half of the peak demand. If this is the case, the system can fail, even though water is available in the Tolt Reservoir.

To prevent this occurrence, the Tolt release module modifies the space rule whenever early season demands begin to drawdown the Cedar storage. By releasing small quantities of water throughout the drawdown cycle, the storage in the Cedar is retained at a high level, and the potential for failure is greatly reduced. Initiating the trigger was determined to be a function of the state of the system. Desired releases from the South Fork Tolt reservoir are listed in Table 5.2. It must be noted that these releases are subject to numerous constraints, such as transmission capacities and total water available. The releases are all also conditional on the amount of demand left to be met when the release rule is invoked. A complete listing of the constrained release rules can be found in the models in the Seattle Tolt Use Converter (See Appendix A).

Table 5.2 System Allocations for Eight Indicator States

State	Tolt Release [a]	South Tacoma Aquifer Release [a]
0	Space rule allocation	0
1	Tolt storage	Pumping capacity
2	Tolt storage	Pumping capacity
3	Tolt storage	Demand minus Cedar storage
4	Tolt storage	Pumping capacity
5	1/3 Tolt capacity	Demand minus Cedar storage
6	Minimum possible release	Demand minus Cedar storage
7	1/2 Tolt capacity	Pumping capacity

a: These releases are conditional to demand, transmission and capacity constraints

Well-field Release Module - As noted earlier, the South Tacoma Well-field is the most secure source of water available and can provide the most utility in times of drought. However, as in the case of the Tolt Reservoir, capacity constraints prevent the full use of this source in any single period. Because monthly pumping capabilities are quite low in comparison to peak demands (60 MGD compared to 300^{+} MGD), effective use of this source must begin well before failure is imminent. Optimum utilization of the Well-field initiates pumping the moment drawdown of other reservoirs begins in years when other sources will not be sufficient to meet demands. Typically, this would mean pumping at full capacity from June through October in about one year out of ten. In other years, limited withdrawals during summer peaks would adequately safeguard the system. Finally, in some years no pumping whatsoever is required, as natural flow is sufficient to meet all demands.

The difficulty for modeling and real time operation is how to establish appropriate indicators of the potential for system failure and, thereby, the need for pumping groundwater. Without predictions of flow far into the future, complete understanding of the Cedar Moraine, and precise estimates of instream release requirements, it is impossible to accurately project system response beyond the present time period. Therefore, some other method of triggering well-field withdrawals was required. As in the Tolt Release Module, the amount of water pumped from the aquifer was determined based on the state of the entire system, identified in the System State Module. Table 5.2 defines the amount of releases that were made for each of the eight states that were chosen to characterize the system in the model.

Evaluation Modules

The final stage of model development was to provide indicators of the performance of the systems under various scenarios. To do this, a number of evaluation elements were selected. These included converters to quantify resiliency, several measures of reliability, and two indicators of the vulnerability of the individual systems and the combined system. Explicitly computing these measures for each run of the model allows comparisons of the implications of alternative operating policies, without introducing the uncertainty of a generalized economic loss function. The specific evaluation converters used in this study were as follows:

M&I Reliability - the number of months in which water supply was sufficient to meet demands, divided by the total number of months in the historic record

Green Fish Failures - the number of months that flow in the Green River below the Tacoma diversion was less than 110 cfs.

Lake Washington Failures - the number of months that the level of Lake Washington dropped below 20 feet.

of failures shows the system reliability if 297.9 MGD were demanded. It is important to note that the current combined firm yield of these two systems is only 259.5 MGD. Therefore, the intertied system could provide an increase in system firm yield and still offer 30% more water for instream purposes. It is also noteworthy that the system could provide 297.9 MGD at a reliability level of 98%, even with increased instream release requirements.

Figure 6.1 shows graphically how M&I yield will be affected over a wide range of instream requirements. Several conclusions can be inferred from this figure. First, the curve is linear in the range near the current instream requirements. This happens because the first failure, which occurs in August 1941, is caused by limited storage and very low flows across the system. Because the system moves from a reservoir full state to a failure state in three months, the amount of water produced is not dependent on the choice of management plan. The limiting factor is the total amount of water that can be withdrawn, therefore, varying the instream requirement merely redistributes the water between instream and diversionary purposes.

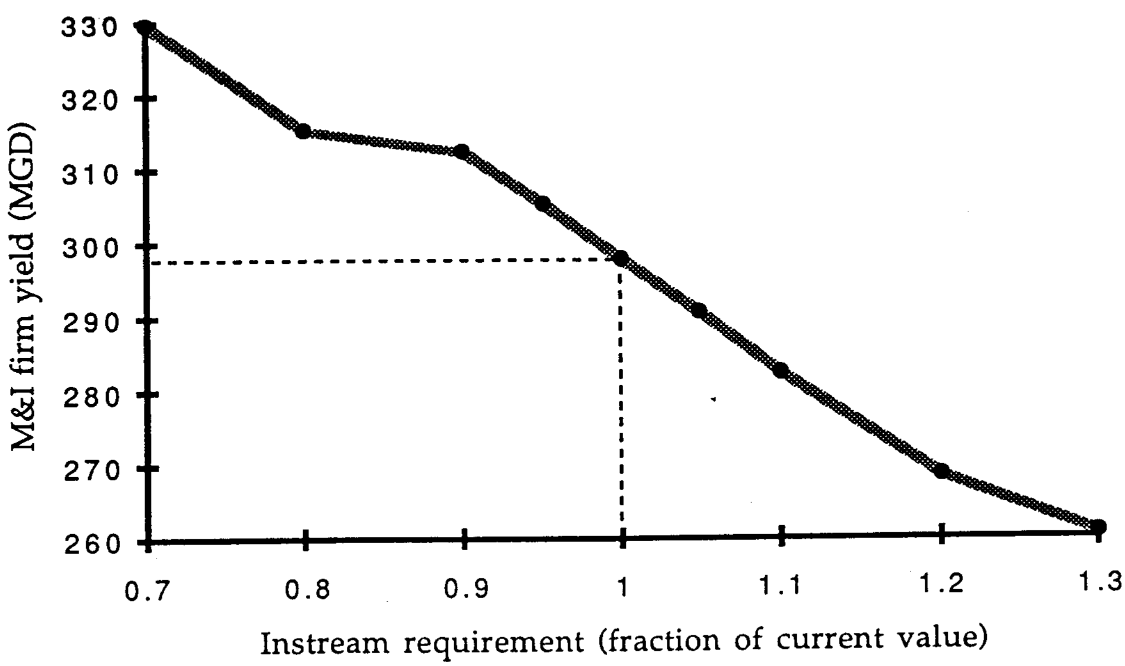

Figure 6.1 Effects of Instream Requirements on M&I Firm Yield

It is also obvious from the figure that decreasing instream requirements to between 80% and 90% of current levels provides almost no increase in yield. On the other hand, increasing requirements above 120% of current levels has less effect on firm yield than increases in the range of 90-120%. At these levels of instream releases, events other than the summer drought of 1941 become the limiting factor in firm yield. Some of these events have longer durations, and management plays a more important role in system performance.

Because the influence of the summer 1941 event was so pronounced in the results of this study, further analysis was performed. Using the combined system model, the firm yield at 98% reliability was determined over a similar range of instream requirements. Considering the system's performance at this reliability level gives weight to a wider range of inflow and demand scenarios and lessens the impact of any single event. This should provide more insight into the normal response of the system to increased demands. Figure 6.2 shows the expected system performance when instream requirements vary between 50% and 150% of current levels. The linear fit was established by connecting the upper and lower points on the curve. The solid dots represent simulated yields at various instream flow demands.

As demonstrated in Figure 6.2, the relationship between instream flow requirements and 98% reliable yield is generally linear. However, there is some variation from this rule, particularly when instream releases are increased slightly. As shown in the figure, if the instream requirement is increased by 5%, there is almost no loss of reliable yield. The impact that this small increase would have on instream users was not studied. Nevertheless, the preliminary results indicated that this increase could be achieved with a negligible 0.20% decrease in yield.

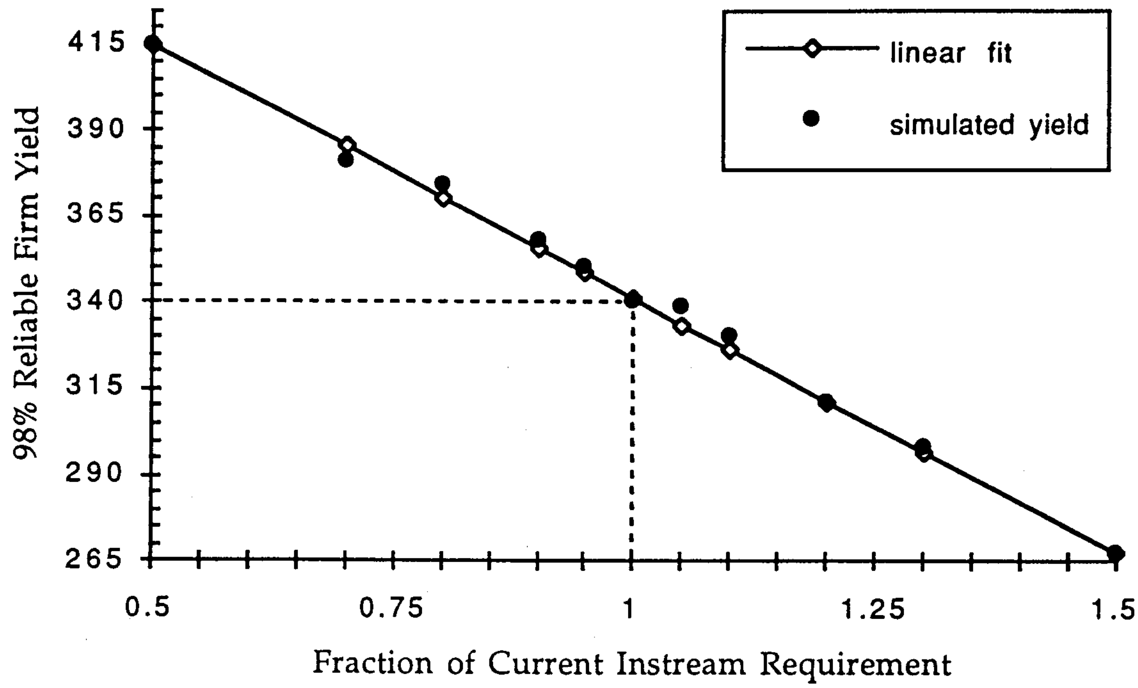

Figure 6.2 Effects of Instream Requirements on 98% Reliable Yield

Clearly, more work would be required to quantify the benefits of increased instream releases before proceeding with the modeling efforts. If new target levels for instream flow were established, the model could be used to study the effects on reliability and other key indicators. The instream requirements for the Cedar system have been studied extensively, with current levels having been negotiated between the DOE and the SWD. It is possible that these requirements are adequate and need no further modification. In contrast, the instream requirements in the Tolt and Green river systems have received less analysis and might necessitate significant changes. The combined system model developed in this study can easily be adapted to look at many of these questions.

Lockage Releases

The third experiment was performed to examine the impact on system performance of potential increases in releases through the Chittenden Locks. As demonstrated earlier, an intertie could provide substantial benefits to system reliability. This section looked at the effect the intertie would have on maintenance of adequate levels in Lake Washington. Using the Seattle system model and the intertied model, the maximum increase in lockage releases that could be accommodated without failure was computed. In this case, failure was defined as any month in which the surface elevation of the lake dropped below 20 feet.

One difficulty when comparing the performance across systems is the selection of demands that allow equitable comparison. If a uniform increase is applied to both systems, the intertied network is certain to be more robust because of its greater flexibility. If an expected future demand level is chosen, the combined system will be burdened with significant increases due to Tacoma's needs, while Seattle increases are more moderate. For this portion of the study, where M&I reliability was not the primary concern, equity was achieved by similarly stressing each system. To do this, each system was subjected to the demand that caused it to be 98% reliable for M&I supply.

The current Seattle system supplied 200.5 MGD at 98% reliability, while the intertied system delivered 341.0 MGD. Operating with these demands, the monthly lockage releases were then increased until failure occurred. The Corps of Engineers follows a rule curve to set releases from Lake Washington through flood spillways. The rule curve varies between 20 feet in winter, to 22 feet during summer months. At levels outside this range, there is the potential for damage to structures, such as the floating bridges. Therefore, a failure was defined as any month in which the lake surface elevation fell below 20 feet.

The Corps has the potential to alter operation of the salt drain or locks if single, intermittent failures are expected. Consecutive months of low inflow and/or frequent shortfalls would lead to greater operational difficulties.

Table 6.4 Combined System Response, 1990 - 2040

Year	Dem. (MGD)	Reliability (%)	Resiliency (%)	Lake/River Failures	Shortfall (Acre Feet)	
					Average	Maximum
1990	263	100	100	0	0	0
2000	314	99.6	100	0	2036	5641
2010	368	94.4	60	0	5873	22440
2020	423	84.8	44.9	0	9581	40308
2030	478	76.5	34.3	1	14286	49305
2040	537	70.1	29.8	1	20131	59644

instream requirements and providing flood control, has not been investigated.

After studying the responses of the individual systems, the combined system was simulated over the same demand range. Table 6.4 shows the results of these tests. Based on these findings, it appears that a combined system could adequately serve the region's water needs through the next decade and improve system reliability at any demand level. Firm yield of the combined system, using the proposed operating policy. was computed as 297.9 MGD. This represents a 14.5 % increase over the sum of the individual yields. For the predicted year 2000 regional demand of 314 MGD, the combined system could be expected to fail three times in 60 years (99.6% reliability).

Using the methodology designed by Hirsch [1977], the maximum potential gains of joint operation were also calculated. This was done by simulating a single reservoir which received all inflow of the South Fork Tolt and Upper Cedar rivers. The reservoir's capacity was set to the combined size of the two reservoirs, using the same rule curves. After meeting instream release requirements, the maximum M&I yield which could be supplied was determined. This supply was added to the Green River diversion and Tacoma's groundwater withdrawals. Simulating over the same period, the potential yield was calculated as 314 MGD, an 18.2% improvement over independent operation. A more detailed description of this method can be found in Hirsch [1977]. It should be noted this provides only a rough estimate of potential yield, as several features of these systems cannot be incorporated in the simplified model. These include the Tolt pipeline restriction, the Masonry Pool seepage losses, and the exact nature of Cedar instream release requirements. Regardless, the estimate produced was satisfactory for the purposes of this study.

In some sense, it is difficult to compare the performance of the intertied system to the individual systems. Because expected regional demand increases are primarily due to Tacoma's expanding market, the Seattle system appears more reliable than the combined network. Also, because the individual networks are only attempting to supply a small portion of the total demand, their vulnerability may seem to be improved. However, when appropriate measures are summed, the combined system performs better than the individual systems at most demand levels. This can be seen in Table 6.5. The one exception is Lake Washington failures. In the combined system, the lake level dropped below 20 feet for one month, while in the Seattle system it never fell to this level. Considering the huge demand increases that the combined system attempts to meet, this was not considered to be an important statistic.

Table 6.5 Percentage Improvement with Coordinated Management

Year	Dem. (MGD)	Reliability (%)[a]	Resiliency (%)[a]	Lake Wash Failures[b]	Shortfall (% improve)	
					Average[c]	Max[c]
1990	263	0.4	n.c.[d]	n.c.	100.0	100.0
2000	314	6.69	79.8	n.c.	62.5	52.2
2010	368	12.0	44.0	n.c.	38.0	6.6
2020	423	74.8	75.1	n.c.	4.1	-7.7
2030	478	59.6	37.2	+1	1.0	-5.1
2040	537	53.9	36.1	+1	-3.2	0.5

a: ((combined reliability-avg. individual reliability)/avg. ind. reliability)*100
b: change in total periods of lake level below 20 feet
c: ((total individual shortfall-combined shortfall)/total ind. shortfall)*100
d: no change

Considering these summary statistics and the increase in firm yield, it is apparent that the potential system intertie is a promising option. A quick comparison with some other expansion alternatives is shown in Table 6.6. The data in this table have been summarized from previous reports and are not intended to yield definitive conclusions. However, it is useful for providing a point of reference as to the magnitude of benefits and costs that an intertied system might deliver. The potential benefits of an intertie, in combination with other expansion alternatives, has not been studied. Certain options, such as development of Howard Hanson storage, would complement an intertie quite well. Total gains from combining several such alternatives could greatly exceed the summation of the parts.

The Tacoma Water Division is currently reviewing approaches aimed at meeting escalating demands. The intertie and storage at Howard Hanson are two of the most promising development alternatives. The model created for this study offers a great deal of flexibility for analyzing the range of impacts that these options would have. In the sections that follow, a few of the potential applications of the simulation model are presented.

Table 6.6 Comparison of Expansion Alternatives

Expansion Alternative	**Yield Increase (MGD)**	**Cost (mil $)**	**Yield/ Cost (GPD/$)**
Intertied system/coordinated management	39	7	5.57
Comprehensive showerhead program	5	6	0.83
Full marginal cost summer rates	4	3	1.25
Added Howard Hanson Capacity	65	unknown	n.a.
North Fork Tolt Diversion	48	55	0.87
Tolt Well-field	5	9	0.56
Cedar High Dam	40	94	0.42
Morse Lake Permanent Pumping	44	59	0.75
North Fork Snoqualmie River	90	468	0.19

Instream Flow Requirements

In the second experiment, the combined system model constructed within STELLA was used to examine the impact that an intertie may have on the provision of instream flow. In constructing the model, an operating plan was developed to allow the system to function efficiently for the provision of M&I supply. Instream release requirements were incorporated as constraints. The previous section detailed the influence that an intertie would have on reliability. In summary, these results indicate the following:

Combined System Yield (individual operation)	= 259.5 MGD
Combined System Yield (maximum yield model)	= 314.0 MGD
Combined System Yield (full model w/oper. rules)	= 297.9 MGD

However, this analysis does not provide any insight into the effects that an intertie will have on instream flow. In this section, a parametric study is performed showing the impact that altering instream flow requirements may have on supply. System safe yield is used to assess these impacts.

There are several reasons that cooperative operation improves system performance and increases the availability of water. Gains from increased operational flexibility and improved management, as well as synergistic gains from combining diverse supply systems, make better use of the same volume. Also, water which might be spilled without benefit in one system can often be used to augment supplies in the other, thus maximizing utility. With proper coordinated management, the same M&I demands can be met, while still leaving a surplus of water. The additional water can be utilized for instream or diversionary purposes. This analysis is intended to offer insight into the optimal distribution of this surplus.

Utilization of available water invariably involves trade-offs. Water withdrawn at Palmer or Landsburg for residential consumption provides no instream benefit. Likewise, if reservoir releases are made to maintain the level of Lake Washington, storage is decreased and reliability may be sacrificed. Pareto optimality asserts that if the utility provided to any user can be increased without harming another user, the allocation is not optimal. In the systems being studied herein, if instream flow can be increased without reducing M&I reliability, it is makes sense to provide more flow.

To perform this analysis, the results of the first section were used to provide a starting point. Under the present system configuration, and with the current DOE required instream releases, the combined system was able to supply 297.9 MGD without a single period of failure. Instream flow requirements were met in each period below Howard Hanson, the South Fork Tolt Reservoir, and at Renton. Next, the required level of releases at these three points were altered by a constant multiplier to gauge the effect on M&I supply. Table 6.7 shows the change in firm yield over the data history if new instream requirements are in place.

Table 6.7 Effects of Instream releases on M&I firm yield

Instream Req. (fraction)	M&I Yield (MGD) and % gain	Reliability if 297.9 MGD is demanded	
		Total Shortfall	No. of Failures
0.70	329.6 (10.6%)	0	0 (100%)
0.80	315.3 (5.8%)	0	0 (100%)
0.90	312.7 (5.0%)	0	0 (100%)
0.95	305.4 (2.5%)	0	0 (100%)
1.0	297.9 (n.a.)	0	0 (100%
1.05	290.9 (-2.3%)	2596	1 (99.9%)
1.1	282.8 (-5.0%)	5900	2 (99.7%)
1.2	268.6 (-9.8%)	7857	8 (98.9%)
1.3	260.8 (-12.4%)	14800	14 (98.0%)

In Table 6.7, the Instream Requirement is the fraction of the current instream flow requirements that were provided in that run of the model. M&I Firm Yield is the amount of water that can be diverted over the course of the data without failure. The percentage increase or decrease, as compared to current supply, is shown parenthetically next to the yield. Total Shortfall and Number of Failures refer to the municipal supply shortfalls at various instream requirements if an attempt is made to supply 297.9 MGD (the intertied system firm yield). The number in parentheses next to the number

Resiliency - the probability that the system rebounded from failure in the next month after a shortfall occurs.

Average Shortfall- the average magnitude of failure that occurs for a given demand level

Maximum Shortfall - the largest shortfall that occurred during the simulation period at any demand level.

Clearly, there are a number of ways to quantify any of the criteria discussed above. For instance, instream flow reliability or the ability to maintain adequate surface level in Lake Washington could be reported as a percentage of time, rather than a count of the number of occasions that these constraints were violated. Some users may also be more concerned with the average or maximum length of sojourn into failure, rather than the definition of resiliency used above. Fortunately, within STELLA, these changes could be made in minutes, and alternative definitions of performance measures can be studied.

Comparisons were also facilitated by the addition of several model features. On page one of the model (see Appendix A), it can be seen that a number of additional converters were included to allow exploration within the simulation. These included factors for adjusting instream release requirements, releases due to lockages, criteria for switching instream regimes on the Cedar, maximum storage in both reservoirs, and rule curves on each of the reservoirs. Elements were also included to simulate climate change by altering the historic records on all inflows.

Comments about the Models

It is worth noting that the three models used in this study were developed within STELLA over a period of less than two months. The graphical modeling environment and easy display of results allowed rapid construction and greatly simplified the debugging process. These features also made comparisons of alternative representations of system components and sensitivity analysis to determine key elements feasible. As the development process was progressing, a number of questions arose as to the appropriate level of detail to model. Much of this uncertainty was resolved by modeling several variations of the element in question and comparing its effect on the performance of the overall system.

Two features, in particular, required extensive preliminary analysis to model correctly. These were the glacial moraine aquifer below Masonry Dam and the South Tacoma Well-field. The glacial moraine involves numerous complex interactions between surface and subsurface features. Several studies have been performed to quantify the releases from Masonry Pool to the aquifer and the subsequent return flow to the Cedar River. Unfortunately, these studies have failed to produce conclusive results applicable to modeling. For the South Tacoma Wells, the primary question was the extent that continuity between the sources affects total availability of water. Again, extensive analysis has been conducted to increase understanding about this feature, but, as in the case of the moraine aquifer, there is no clear conclusion.

To overcome this uncertainty, several representations of each of these features were modeled. Analysis was then performed to determine if the performance of the overall system model was sensitive to the alternatives. Interestingly, the firm yield of the Seattle system was virtually unaffected by the choice of equations to govern seepage and return. For this reason, the USGS equations that are typically used by the SWD to characterize the aquifer, were chosen for these models. It was felt that in the absence of any clear consequence of a particular choice of equations, it was most appropriate to follow the SWD's general preference.

In the Tacoma well-fields, just the opposite was found. The performance of the Tacoma system was highly dependent on the manner in which the well-fields were modeled. If the Gravity and other wells were hydraulically isolated from the South Tacoma Aquifer, the yield and reliability of the system were improved. This was because the additional wells acted as a constant supply of water when continuity did not affect their output. That left all of the water in the aquifer to meet peak demands. In this case, the judgment of Tacoma Water Division personnel was relied upon, and potential continuity between the wells was ignored. Although this decision could have been reached without the use of the model, the comparison was still a valuable exercise. The experience gained in this study indicates that further analysis of the interdependence of the wells is essential for accurate estimation of system performance.

The same features in STELLA that enabled rapid model development also lead to versatility in the final product. Updating the models is a simple procedure that can be performed graphically. Because all logical links are shown by connectors, and equations are readily accessible and written in a language that is easy to understand, earlier modeling assumptions are clearly defined. This enables the user to change components as necessary and leave the remainder as it is.

As mentioned previously, a number of converters were included to facilitate future analysis. These tools can be utilized for a number of purposes. In this study, the effect that altering instream flow requirements had on municipal and industrial firm yield was analyzed. Consideration was also given to the impact that lockage releases have on maintenance of the surface elevation of Lake Washington. Other analyses could include study of the impact that total storage has on M&I reliability, the influence that climate change might have on system performance, or the effect of increased M&I diversions on instream flow.

Whenever changes are made, the effects on important measures can be readily quantified using STELLA's tabular or graphical output features. Rather than exporting data to some other package for analysis, these models provide immediate feedback. Such feedback is extremely valuable when using models as negotiating tools and in making meaningful trade-off studies. Competing interests for scarce resources can suggest a potential operating policy and instantly know the effect that it will have. Also, if there are questions as to the models' representation of physical features, the user can use sensitivity analysis to examine any pertinent details.

Chapter VI
Experimental Results

After identifying important characteristics of the Seattle and Tacoma systems (Chapter 4) and constructing models within STELLA to simulate these networks (chapter 5), a number of experiments were completed to gauge their performance. These included:

- parametric study of reliability, resiliency, and vulnerability at alternative demand levels
- analysis of the influence that instream flow requirements have on system firm yield
- study of the impact that potential increases in lockage release will have on maintenance of adequate Lake Washington water levels
- development of an alternative operating plan, including hedging, to minimize expected losses while maintaining reliability

The parametric studies demonstrate the versatility of simulation models for analyzing alternative future scenarios. These studies also show the applicability of the models created here for providing analysis critical to any decision making process. The development of an improved operating plan highlights the models' suitability for system analysis and operational management.

Parametric analysis of each systems performance with increasing demands was first considered with respect to each of the criteria developed in Chapter 5. As mentioned previously, an economic loss function could be used as a singular measure of system performance. However, accurate description of economic losses due to water supply failures is a cumbersome and highly subjective task. The SWD has funded previous studies to develop loss functions for its water service area, with limited success. There remains a great deal of uncertainty as to the accuracy of these estimates and there practical application. Loss functions have not been developed for the Tacoma service area. Therefore, the development of regional economic loss functions would require substantial effort, and might not even be feasible.

Retaining discrete indicators of performance facilitates comparisons between systems and provides a basis for discussion, without relying on subjective economic decisions. Also, the development of accurate loss functions was far beyond the scope of this study. In the final experiment, an improved operating plan was developed using a generic economic loss function to gauge performance. This function was not intended as an accurate descriptor of the performance of these systems. Instead it was used as an index by which alternative plans could be measured with respect to municipal supply. The result of this experiment is cited to establish the feasibility of such a modeling effort, rather than as proof of a particular hypothesis. Such hypothesis testing would require consideration of specific economic and political information that was not used in this study.

The individual experiments performed to establish the usefulness of simulation modeling for water resources questions are described in the sections that follow. Each of the studies made use of one or more of the models developed herein, with only minor alterations as noted. The full documentation for these models is reproduced in Appendix B. Each of the following sections provide details about the procedure and the experimental results. Again, these results are intended to show the flexibility of the models, rather than provide definitive answers to specific questions.

Parametric Study of System Performance

The first experiment conducted in this study involved a comparison of the performance of the combined system versus the individual systems. Using the historic streamflow data from water years 1929 through 1989, each of the three models was run, and pertinent information was recorded. Although the operating rules discussed in Chapter 4 were instituted to improve system performance, no attempt was made to optimize it with respect to any single criteria. To the extent possible, these models replicated the configurations and operational policies of the current water supplies of Seattle and Tacoma.

This experiment was intended to demonstrate the feasibility of system intertie for helping to meet escalating regional water demands. The municipalities' estimated demands for the years 1990 through 2040 are listed in Table 6.1. Several facts are worth noting. First, the SWD foresees a slow, steady increase in demands through the study period. This indicates that performance of the existing water system will gradually diminish. This effect should be evident in reduced reliability and increased vulnerability, although the system should still be adequate in most years.

The Tacoma Water Division's prediction of future demands is very different. They expect to become more involved in the regional needs of South King County and western Pierce County. Therefore, their rate of increase will be much faster than the SWD's. In fact, the Water Division forecasts that demands will rise more than 300% over the next 50 years. This type of escalation can quickly render a supply system inadequate. When demands reach a certain level, it is likely that failure will occur even in winter months. Also, without significant storage capacity, the system will fail at lower demands during high peak months. Both of these effects were apparent in the runs performed for this study.

Table 6.1 Predicted System Demands, 1990 - 2040

Year	Seattle Demand (MGD)	Tacoma Demand (MGD)	Total Demand (MGD)
1990	170	93	263
2000	180	134	314
2010	203	165	368
2020	218	205	423
2030	230	248	478
2040	245	292	537

The Seattle system also performed as expected. Firm yield (1 failure) was found to be 170 MGD, which is approximately the same as other models have shown. These include the study by Palmer [1986] and the current modeling effort being performed by Hydrocomp Inc [1992]. Interestingly, this is the same amount as the current average demand, indicating that the system is nearing the point where failures would be expected during dry years. The results of the model runs for future demand levels are shown in Table 6.2.

Table 6.2 Seattle System Response, 1990 - 2040

Year	Dem. (MGD)	Reliability (%)	Resiliency (%)	LK Wash. Failures	Shortfall (Acre Feet)	
					Average	Maximum
1990	170	99.4	100	0	1403	2913
2000	180	98.7	55.6	0	3067	8046
2010	203	97.5	50	0	4555	12547
2020	218	96.0	51.7	0	5119	17865
2030	230	93.9	50	0	5595	25212
2040	245	91.1	43.8	0	6467	32754

Although reliability remains fairly high through 2010, the vulnerability of the system, as shown by the average and maximum shortfall, is sharply increased. This could cause significant economic losses, should these shortfalls occur. Recalling that the definition of resiliency used in this study is the inverse of the average sojourn into failure, another interesting result is noted. Presently, the system would be expected to return to a satisfactory state in the month following failure. By 2040, sojourns into failure would be expected to last almost 2.5 months.

The firm yield of Tacoma's water supply was determined by simulating existing conditions with the addition of Pipeline No. 5. Using historic inflow data from 1913 through 1991, it was found that the system could supply 103.5 MGD without failure and without mining the groundwater supplies. Until the addition of Pipeline No. 5, the firm yield is closer to 93 MGD, but this study assumed that construction of this line was imminent (see Chapter 4). Hydraulic independence between the South Tacoma Aquifer and the other groundwater sources was also assumed.

The response of the Tacoma system to expected increases in demand is shown in Table 6.3. Over the next decade, demands are expected to increase rapidly, and the firm yield of the system will be surpassed. Unlike the Seattle system, where supply is primarily a function of hydrology and storage capacity, the yield of the Tacoma system is governed by water rights and groundwater

Table 6.3 Tacoma System Response, 1990 - 2040

Year	Dem. (MGD)	Reliability (%)	Resiliency (%)	Green River Failures	Shortfall (Acre Feet)	
					Average	Maximum
1990	93.5	100	100	0	0	0
2000	134	88	55.6	0	2331	3738
2010	165	69.8	33.3	0	4917	11484
2020	205	1	~ 0	0	4875	16374
2030	248	~ 0	~ 0	0	8833	21676
2040	292	~ 0	~ 0	0	13007	27202

pumping capabilities. The second Green River diversion introduces some hydrologic uncertainty, but this water is rarely available during summer peaks and will be used primarily to reduce groundwater pumping during spring and fall months. Therefore, the greatest risk of system failure is due to mechanical or transmission difficulties. If mechanical failures are disregarded, the system is 100% reliable until its firm yield is reached.

If demands are increased beyond the firm yield, the reliability of the system drops precipitously. If the escalation in demands continues, a level is reached at which failures occur each month. Table 6.3 shows this level to be an average daily demand somewhere between 165 and 205 MGD, occurring between the years 2010 and 2020. Obviously, system improvements would be required long before that point in time. In fact, to operate the system at 98% reliability will require improvements before demands reach 104 MGD this decade.

The vulnerability of the Tacoma system increases with demand, as shown by the maximum shortfall. Resiliency oscillates somewhat as failures become more commonplace, but falls to zero (actually it becomes undefined) when reliability drops to zero. Of note, is the fact that the system never fails to provide a minimum of 110 cfs for Green River instream demands. This indicates that the rule curve and operational policy applied at Howard Hanson Dam by the COE is more than adequate for instream needs. Whether an alternative policy could improve M&I reliability, while still meeting

Figure 6.3 shows the response of each of the systems to increased lockage demands while delivering the same municipal diversions. The Seattle system suffers its first lake failure at an increase of only 7.5% above current levels. The intertied system, on the other hand, can provide adequate water to the lake even if lockage releases rise by 60%.

Monthly lockage demands are subject to substantial variability, but this fact was not incorporated into the models. If this variability is considered, the current Seattle system is already susceptible to failure under the right scenario of inflow and demands. A closer look at Figure 6.3 reveals more about the differences in the response of the systems. In the Seattle system, when lockage releases rise above 125% of current levels, the number of failures is nearly a linear function of lockages. If lockage releases increase approximately 75%, there would be an average of one failure every year. Proportionate increases in failures would accompany any increase in lockages. This would not be an acceptable situation.

For the intertied system, the graph of failures versus lockage releases has a

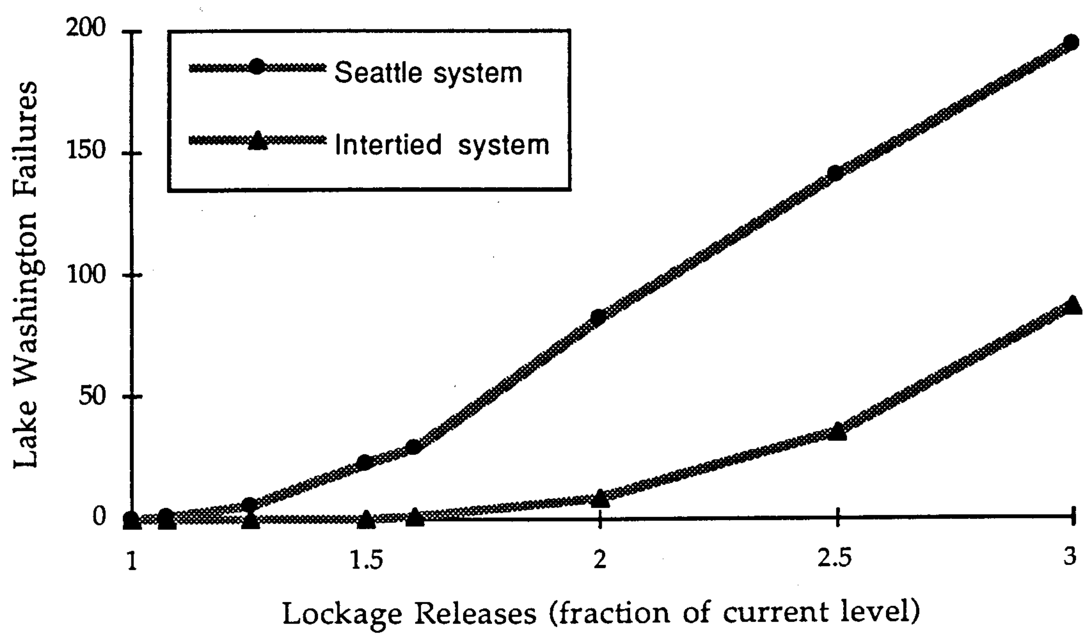

Figure 6.3 Lake Washington Response to Increased Lockage Releases

parabolic shape. Only occasional failures occur for releases more than double the current levels. Also, the slope of the graph is very gentle when lockages remain below 250% of present releases. It is unlikely that lockages will ever reach this level. Clearly, the intertied system would be extremely beneficial to the Corps efforts to maintain adequate levels in the lake. Even considering the uncertainty associated with future lockages and the variability of monthly demands, an intertied system would be diverse enough for any realistic scenario.

More important than these specific findings about potential benefits of an intertie, is the flexibility of the models demonstrated by this example. With a few simple modifications (converters were included to allow increasing lockage releases), the models were able to provide insight into a complex question. The data presented above were computed in minutes. Similarly, many other questions could be explored with little or no modification to the models. Changes are made to the graphical representation of the models, and output is generated in either tabular of graphical format. This type of modeling allows the user to concentrate on the formulation of appropriate questions, rather than construction of models and data manipulation.

Alternative Operating Plan

The final experiment performed exhibits the combined system model's applicability and flexibility which can be used to improve system management. The preceding sections utilized simple operating rules based on current conditions and historic trends to determine releases. These rules aimed to optimize the spatial distribution of releases among the various sources. No attempt was made at balancing intertemporal trade-offs to minimize losses. Disregarding intertemporal trade-offs essentially treats economic losses as a linear function of shortfall volume. This is typically not the case. Often, a policy which accepts small shortfalls in the current period to prevent the possibility of larger losses at a later time, is more cost effective. This style of operation is termed hedging, because current supplies are reserved to hedge against future losses [Loucks, 1982; Bayazit, 1990].

The actual benefits that a hedging policy can provide are dependent on a number of factors which influence the shape of the economic loss function. This study did not attempt to derive actual loss functions for the Seattle and Tacoma systems. Instead, a generic loss function was chosen to serve as a dimensionless index to potential benefits of improved operation. For simplicity, losses were assumed to be zero if supply was greater than 95% of demand. If shortfalls exceeded 5% of the target demand, losses were assumed to be proportional to the square of the difference between actual releases and target releases. Mathematically, this function was defined as follows:

$$\text{Loss} = 0 \qquad \text{for } x_t \geq 0.95\,T_t$$

$$\text{Loss} = 100\left[\frac{(0.95T_t - x_t)}{0.95T_t}\right]^2 \qquad \text{for } x_t < 0.95\,T_t$$

where: x_t = actual release in period t
T_t = target release in period t

The multiplicative factor of 100 was applied to losses to make the results easier to work with. Small shortfalls were assumed to cause no losses. This takes into account emergency curtailment programs which should be able to achieve savings of up to 5% in summer months. These programs are aimed at non-essential uses of water, such as lawn watering and car washing, which can be reduced without cost. Winter water curtailments are not as easy to institute, but failures during winter months are exceptionally rare.

It should be noted that this loss function does account for damages due to flood flows. The time step used in this study was inappropriate for consideration of flood flows. Daily or, preferably, hourly data would be required to assess losses from flooding. Reliable data of this type are not available for these basins. Even if it were, monthly data are better suited to the consideration of drought events. The operation of each reservoir in the model followed a rule curve which was established to minimize flooding. These rule curves were not altered during the course of this study. Therefore, ignoring losses from flooding should not affect these results.

For this experiment, the earlier definition of reliability was modified to reflect the curtailment savings. A failure was defined as any period in which water supply did not surpass 95% of the target demand. Because winter failures were so rare, the months between January and May were excluded from the calculation of reliability. Therefore, reliability was defined as the number of months over the simulation in which supply was less than 95% of demand, divided by the number of years, times seven (the months between June and December). Mathematically:

$$\text{Reliability} = \alpha = \frac{(\text{\# of months with } x_t < 0.95\ T_t)}{(\text{\# of years})(7)}$$

Interestingly, the 98% reliable yield produced with this definition was nearly identical to the earlier result. In this case, the intertied system supplied an average yield of 340 MGD. The definitions of vulnerability and resiliency were not altered. Therefore, the average shortfall was computed using all failures, even those which were within 5% of the target demand.

The goal of this experiment was to define an operating plan which reduced the cost of shortfalls, without substantially diminishing reliability. Baseline data was collected by running the model under the management policy discussed in Chapter 4. The cost index for the 60 year simulation period was 223.65 when the system target demand was 340 MGD. The average shortfall was 3362 acre-feet, with a maximum of 13,642 acre-feet.

In developing a hedging plan, both the availability of information and its certainty were considered. In the model, the amount of water currently available from each source can be determined exactly. In reality, there is uncertainty about quantities in aquifer storage and future recharge and inflow rates. To be applicable, an operating plan must make use of data which are known, or can be approximated during actual operations of the system. Therefore, optimization based on perfect information about future inflows is not realistic. There is also uncertainty about demands, which further complicates the development of an effective hedging policy.

The volume in reservoir storage at any time is one quantity that is known exactly. Therefore, a simple hedging policy might institute restrictions whenever reservoir storages dropped below a certain level. By varying the critical level as a function of calendar month, the average expected inflow and demands would also become an integral part of the hedge rule. Several runs of the model were made to estimate the effectiveness of a hedging policy based on these simple considerations. Table 6.8 compares the system's performance with and without hedging.

Table 6.8 Effects of Simple Hedge on System Performance

Criteria	Without Hedge	With Hedge
Cost Index	223.65	220.72
Reliability	98.3	98.5
Resiliency	85.7	83.3
Average Shortfall	3,362	1,146
Maximum Shortfall	13,642	10,917

Although the maximum failure was reduced substantially, the estimated losses under this simple hedging rule were only reduced moderately. The reason for this is that the policy was so conservative that it forced hedging in many years when the system would have had sufficient water to meet demands. Also, in the years when hedging was needed, the reductions were not adequate to minimize the eventual shortfall. A more discriminating hedging rule is necessary to improve on the economic savings. One possibility is to include a prediction of future inflows when deciding whether or not to hedge.

There is a always uncertainty involved when predicting future inflows, but short term predictions are often quite good. Baseflow and snowmelt provide a significant percentage of total flow during low flow periods, and these contributions are easier to estimate than storm runoff. If accurate predictions can be made for one or two months duration during the summer and early fall, an improved hedging rule can be developed. The ability of

Table 6.9 Effects of Improved Hedge on System Performance

Criteria	Without Hedge	With Hedge
Cost Index	223.65	192.74
Reliability	98.3	98.3
Resiliency	85.7	85.7
Average Shortfall	3,362	1,146
Maximum Shortfall	13,642	10,917

municipalities to make accurate predictions was an issue that was beyond the scope of this study. In this experiment, actual inflows for the two months following a decision period were used in place of predictions. Future research could be directed to determine the effect that less than perfect predictions would have on these results.

Several hedging rules were designed based on the current state of the Cedar and Tolt Reservoirs, the month, and the two-month inflow predictions. These were tested and refined using the model's built-in sensitivity analysis. The amount to hedge and the time to institute hedging was adjusted until the results were satisfactory. The final policy offered substantial advantages over a non-hedging policy. Table 6.9 summarizes the findings of this experiment. The cost index was lowered by 14%, while retaining the same level of reliability. If the cost index corresponded to millions of dollars, the hedging program would provide a cost savings of 30 million over non-hedged operation.

These results do not necessarily reflect an optimal hedging policy. More complex rules could provide substantially better results. Predicting increases in demands during dry periods would enhance the performance of a hedging rule. Also, modifications to available water that consider leakage to the aquifer and decreases in instream requirements would improve the characterization of the state of the system. Each of these improvements would introduce further complexity and some additional uncertainty into the hedging policy. Future study could address the difficulties of enhancing the hedging plan and expected gains derived from a more complex formulation.

Chapter VII
Summary and Conclusions

This chapter presents a summary of the goals and accomplishments of the present research. Conclusions are drawn about the applicability of this work towards improving management of existing water resources in western Washington. Simplifications and assumptions that facilitated this study are also presented. This discussion is followed by recommendations for future research which could use this work as a basis for exploration.

Summary

One goal of this thesis was to study the impact of system regionalization on water resource reliability. Another objective was to gain an improved understanding of the role of physical performance measures in water resource management. A third consideration was to document the ability of a graphical-interface driven model to improve decision making processes. The specific systems chosen for this study were the water supply networks of Seattle and Tacoma in western Washington. The primary tool used to accomplish these goals were computer models built within STELLA II™ simulation software.

The general characteristics of the systems considered in this research were described in Chapter 3. Of particular note were complex features, such as the Cedar moraine aquifer, about which there is a great deal of uncertainty. This uncertainty necessitated various assumptions and simplifications in the models. Details about the development of the models were discussed in Chapter 5, and full documentation is included in Appendix B.

The models developed were used to investigate the theory of physical performance measures and their practical application to a regional water resource system for western Washington. A multi-faceted definition of reliability was introduced, and user specific performance measures were developed. Firm yield, the measure of system dependability most commonly used by municipalities, was also computed. The potential impact on these performance measures of a regionally intertied and jointly operated water supply network for Seattle and Tacoma was analyzed.

The simulation models constructed for this study offer great flexibility which can aid in the evaluation of the benefits of an intertie or other system modifications. The models allowed comparison of the performance of a jointly managed system with that of the independently operated systems. A number of criteria were considered, including firm water yield and probability of failure at increased demand levels. The effect that the chosen operational policy had on specific performance measures was also documented. As discussed in Chapter 5, the models' output can be tailored by the user to address specific concerns.

Chapter 6 presented the results of a number of experiments which were designed to demonstrate the utility of the models. These included an examination of the implications of operational decisions on competing interests and a comparison of the systems' ability to meet potential instream demand increases. The studies conducted in this research were intended to illustrate the applicability of the current modeling approach to real problems. In some cases, more detailed information would be required to allow definitive conclusions. These are addressed below as issues for future research.

Conclusions

Regionalization of the water supply systems of western Washington is one of the most cost effective approaches to increasing the available water supply. If the municipalities of Seattle and Tacoma were to intertie their supply networks and operate the combined system jointly, a potential increase in firm yield of 39 MGD could be achieved. The cost of such an intertie would be substantially less than other source development options that would provide this large an increase in yield. Also, because an intertie would make more efficient use of existing supplies of water, the adverse environmental effects associated with many other options would be eliminated.

In addition to increases in firm yield, an intertied system would provide greater reliability and operational flexibility. Improved reliability would be characterized by less frequent shortfalls of shorter duration and smaller magnitude. If combined with conservation and curtailment programs, it might make it possible to delay new source development for many years. Greater flexibility would improve performance during times of drought when under-utilized sources (such as the South Tacoma aquifer) could be tapped to provide essential water. It would also be beneficial in the event of emergencies (such as the Tolt pipeline blowout) when the supplies of one system could be transferred to meet needs of the other.

The water saved by more efficient use of existing sources could provide numerous benefits. In addition to meeting escalating regional demands, instream uses of water could receive larger allocations, thus enhancing the biological and physical processes mentioned earlier. An equitable distribution of excess water would need to be determined. The results of the preliminary analyses conducted for this study, in conjunction with further refinements to the models, would be useful for making this determination.

It is necessary to consider various measures of system performance, rather than a single attribute, such as firm yield, to make proper decisions. The concept of reliability which considers frequency, magnitude, and length of failures gives substantially greater insight into system performance than any single measure. As shown by the improved operating plan developed in the final experiment of the previous chapter, reliance on a single attribute can lead to poor management choices. Considering each of these criteria independently allows operators to fully understand the implications of any operational policy.

The simulation models developed for this study are a promising alternative to current decision making tools. Graphical model construction allows anyone to use these models with only a minimal of instruction. They are highly realistic, simple to modify, and easy to understand. Assumptions of the modeler have been fully documented to answer any questions that users might have. Logical interdependencies between elements of the model are clearly represented, once again facilitating the users' understanding. These features are very different from typical batch processing models, where the user is left to wonder what assumptions the builder made about the system.

These models also have the potential to answer a wide range of questions about the response of the system to changes expected in the future. The studies performed in the previous chapter are just a few examples of the power that this tool offers. Interactive negotiation, rapid sensitivity analysis, and alternative scenario testing are three areas in which these models perform well. Tabular and graphical output formats facilitate their use in a number of situations where data manipulation in an external package would be cumbersome. These features also speed the debugging process, allowing the modeler's concentration to be shifted to understanding the system being simulated rather than writing complex computer code.

When all of their features are considered, the models developed for this study are well suited to many tasks involved in water resource management. Benefit/cost analysis, public communication, rapid system prototyping, and management training are all potential uses. It should be noted, however, that some of the assumptions made for the models will require verification. Wherever possible, accepted equations governing specific elements of the model, such as the Cedar Moraine, were used in the models. As the inflows and outflows from this feature become better understood, the models will require further modification. The same comment holds true for the hydraulic continuity among Tacoma's well fields.

Other simplifications that were used in this study might not be appropriate for all situations. The simplified economic loss function used in development of an operating policy would not be useful when comparing benefits of instream versus diversionary uses of water. This type of cost assessment would require accurate estimates of the marginal value of each end use. Accurate economic valuations were beyond the scope of this study, but could easily be incorporated into the model.

As with any model, the results that are derived are only as valid as the approximations and simplifications used in constructing the model. The level of information utilized in modifying these models should be consistent with the purpose of the study. Consideration should be given as to which features require further clarification and which ones are already adequately modeled. With this sort of planning and the base models constructed for this thesis, the potential for study of the region's water resource system is unlimited.

Directions for Future Research

There are a number of questions which arise as a logical extension of this research. Several have already been addressed at a preliminary level, while others were beyond the scope of this thesis. This section highlights a few areas of concern and outlines pertinent questions for each. Three main areas for which this thesis might provide a basis for further study include improved water resource management for instream purposes, resource development benefit/cost studies, and improved communication between municipalities and the public.

In Chapter 6, analysis was performed on the effect that increased instream requirements would have on M&I reliability. That investigation used current requirements as a starting point and computed reductions in firm yield at alternative instream release levels. It provided some insight into the trade-offs involved when balancing instream and diversionary allocations. Further research should concentrate on the full range of impacts that management of water resources for municipal supply has on instream concerns. Some of the primary questions are:

Is maximization of reliability in M&I supply detrimental to fisheries interests?

Can the system be operated in some other manner to limit these problems?

Can water right policy be adjusted to provide a "win-win" situation between instream and M&I uses?

The relative importance of water resource problems, and the extent to which solutions are needed, often rely on economic considerations. In Chapter 6, the potential of an intertie for increasing M&I supply was documented. The cost of an intertie was also compared to other source development options. With rapid growth and escalating demands on the current water infrastructure, no single option will suffice for meeting expected needs into the early part of the next century. The models developed in this study can be adapted quite easily to incorporate new sources and analyze their full impact on the systems. Some of the primary questions are:

What is the estimated firm yield of any new source?

Does this source offer substantial synergistic gains due to compatibility with current supplies?

Are the benefits of the new source primarily increases in yield, or is reliability improved as well?

Would the new source increase the robustness and flexibility of the system?

Communicating the significance of water resource problems is an especially difficult issue in a humid climate which receives abundant rainfall. There is a limit to the amount of information that affected parties can understand. Therefore, it is essential to provide clear, concise illustrations of the extent of problems. The models developed for this study offer a variety of output formats and can be interfaced with other tools, such as spreadsheets and graphics packages, to increase their utility. The nature of the models allow specific questions of competing interest groups to be addressed. Through this process, with relatively simple modifications to the models, significant insight can be gained. These tools can be used by municipalities to create public information packets and to improve their own understanding of the intricacies of their systems. Some particular aids which might be developed include:

Charts which communicate seasonal and long term system deficiencies.

Graphs of system performance measures as a function of time, showing points at which criteria fall below an acceptable level.

Comparisons of allocations among users and the effect that increased allocations to any user has on system-wide response.

Real time operation studies which facilitate formulation of a tangible decision process, rather than unplanned, reactive policies.

Negotiation games with which users are forced to comprehend the ramifications of specific actions and formalize trade-offs.

Proper management of water resources in a period of rapidly escalating demand is a challenging issue faced by many municipalities. Well designed tools and a better understanding of system complexities will improve management decisions. Developing adequate criteria for gauging system performance is another important step. Also, fully assessing the trade-offs involved in operational decisions and determining how different users will be affected will lead to better informed compromises. This thesis has addressed each of these topics to varying degrees. The models developed for this research are flexible enough to be adapted to many issues, and the analysis that was performed provides a basis for further work.

References

Abbott, J.D., "A Systems Engineering Approach Applied to an Anoxic/Aerobic Oxidation Ditch Simulation Model", Masters of Science Thesis, University of Washington, 1992.

Acres Consulting Services Limited, "The Trent Basin: Plan of Operation", Report to the Department of Indian and Northern Affairs, December 1973.

Anderson, R.L., "Conflict Between Establishment of Instream Flows and Other Water Uses on Western Streams", *Water Resources Bulletin*, Vol. 18(1), Feb. 1982, pp. 61-65.

AWWA Water Utility Management Training Course, Manual M6, AWWA Inc., Denver, Colorado, 1959.

Bayazit M. and Unal, N. E., "Effects of Hedging on Reservoir Performance", *Water Resources Research*, Vol. 26(4), pp. 713-719, 1990.

Boumans, R. and Sklar, F.H., "A Polygon-Based Spatial (PBS) Model for Simulating Landscape Change", *Landscape Ecology*, Vol. 4(2/3), pp. 83-87, 1990.

Bower, Blair T., Hufschmidt, M.M., and Reedy, William W., "Operating Procedures: Their Role in the Design of Water Resource Systems by Simulation Analyses", from Maass, A., Hufschmidt, M.M., Dorfman, R., Thomas, H.A., Marglin, S.A., and Fair, G.M., Design of Water Resource Systems: New Techniques for Relating Economic Objectives, Engineering Analysis, and Governmental Planning, Harvard University Press, Cambridge, Massachusetts, pp. 443-458, 1962.

CH2M Hill, "Environmental Management for the Metropolitan Area: Part I, Water Resources", Final Report to the River Basin Coordinating Committee (RIBCO), 1974.

Chen, E.H., "A Brief Report on Seepage Through the Cedar Glacial Moraine During the 1976 Experimental Raising of Morse Lake", Draft Report for the Seattle Water Department, Seattle, Wa., 1976.

Collings, M.R., Smith, R.M., and Higgins, G.T., "The Hydrology of Four Streams in Western Washington as Related to Pacific Salmon Species", USGS Water Supply Paper, 1968, 1972.

Demello, Steven and Gardner, B.H., "A powerful Tool for Healthcare Management", *Healthcare Forum Journal*, pp.27-31, 1990.

Draper, S.H., Palmer, R.N., Lettemmeier, D.P. and Burges, S.J., "Water Resource Reliability Under Drought Conditions: The Seattle Water Supply System as a Case Study", Technical Report No. 72, Harris Hydraulics Laboratory, University of Washington, Seattle, Wa., 1981.

Economic and Engineering Services, Inc., "Tacoma Water Division, Howard Hanson NED Analysis", Draft Report to the Tacoma Water Division, 1991.

Erikson, Karol, "An Interactive Simulation Model for the Cedar/Tolt Water Supply System", Masters of Science Thesis, University of Washington, Seattle, Wa., 1982.

Evenson, D.E. and Mosely, J.C., "Simulation/Optimization Techniques for Multi-Basin Water Resources Planning", *Water Resources Bulletin*, Vol. 6(5), pp. 725-736, 1970.

Gibson, Craig, Tacoma Water Division, Personal Communication in Regard to the Hydraulic Continuity in the Tacoma Wells, December 15, 1991.

Grygier, Jan C., and Stedinger, Jery R., "Algorithms for Optimizing Hydropower System Operation", *Water Resources Research*, 21(1), pp. 1-10, 1987.

Hashimoto, Tsuyoshi, Stedinger, Jery E., and Loucks, Daniel P., "Reliability, Resiliency and Vulnerability Criteria for Water Resource System Performance Evaluation", *Water Resources Research*, 18(1), pp. 14-20, 1982.

Hashimoto, Tsuyoshi, Stedinger, Jery E., and Loucks, Daniel P., "Robustness of Water Resources Systems", *Water Resources Research*, 18(1), pp. 21-26, 1982.

Hirsch, Robert M., "A Comparison of Streamflow Record Extension Techniques", *Water Resources Research*, Vol. 18(4), pp. 1081-1088, 1982.

Hirsch, Robert M., Cohon, Jared L., ReVelle, Charles S., "Gains from Joint Operation of Multiple Reservoir Systems", *Water Resources Research*, 13(2), pp. 239-245, 1977.

Hirsch, Robert M., "Synthetic Hydrology and Water Supply Reliability", *Water Resources Research*, Vol. 15(6), pp. 1603-1615, 1979.

Jackson, Barbara Bund, "The use of Streamflow Models in Planning", *Water Resources Research*, Vol. 11(1), pp. 54-63, 1975.

Jangaard, Loren, US Army Corps Of Engineers, Personal Communication in Regard to the Operation of Howard Hanson Dam, October 18, 1991.

Jettmar, R.V. and Young, G.K., "Hydrologic Estimation and Economic Regret", *Water Resources Research*, Vol. 11(5), pp. 648-656, 1975.

Johnson, Sharon A., Stedinger, Jery R., and Stachus, Konstantin, "Heuristic Operating Policies for Reservoir System Simulation", *Water Resources Research*, Vol. 27(5), pp. 673-685, 1991.

Klemes, V., "Storage Mass-Curve Analysis in a Systems-Analytic Perspective", *Water Resources Research*, Vol. 15(2), pp. 359-370, 1979.

Loucks, D.P., and Dorfman, P., "An Evaluation of Some Linear Decision Rules in Chance Constrained Models for Reservoir Planning and Operation", Water Resources Research, Vol. 11(6), pp. 777-782, 1975.

Loucks, D.P., "Surface Water Quantity Models" from Biswas, A.K. Systems Approach To Water Management, McGraw Hill, New York, pp. 156-218, 1976.

Loucks, D.P., and Sigvaldson, O.T., "Multiple Reservoir Operation in North America" from Kaczmerick, Z., and Kindler, J., The Operation of Multiple Reservoir Systems, International Institute for Applied Systems Analysis, Laxenburg, Austria, 1982.

Mandelbroot, B.B. and Wallis, J.R., "Noah, Joseph, and Operational Hydrology", *Water Resources Research*, Vol. 4(5), pp. 909-916, 1972.

Mishalani, Nabeel R., Palmer, R.N., "Forecast Uncertainty In Reservoir Operation: A Case Study", Water Resources Series, Technical Report No. 107, University of Washington, Seattle, Wa., 1987.

Palmer, R.N., and Johnston, D.M., Completion Report for Optimization of Yield Analysis on Seattle Water Supply System, Memorandum to Jo. Rekhi and David Parkinson, SWD, March, 1984.

Ramsay, M.W., Grass, G.M., and Vallner, J.J., "Pharmacokinetic Simulations Using STELLA: Prediction of In Vivo Performance of Oral Dosages", *European Journal of Pharmaceutics and Biopharmaceutics*, Vol. 37(3), 1991.

Seattle Metropolitan Water Supply System, Summary, 1970.

Seattle Water Department, "Seattle Comprehensive Regional Water Plan (COMPLAN)", Prepared by Kramer, Chin, and Mayo, Seattle, Wa., 1980.

Seattle Water Department, "Water Resource Management Model of the Seattle Water Supply System, Main Report", Prepared by URS Co., December, 1981.

Snyder, Regina Marie, "The Effects of Instream Flow Requirements on Water Supply Reliability", Masters of Science Thesis, University of Washington, Seattle, Wa., 1983.

Stober, Q.J., and Graybill, J.P., "Effects of Discharge in the Cedar River On Sockeye Salmon Spawning Area", FRI, University of Washington, Seattle, Wa., 1974.

Stober, Q.J., Steward, C.R., and Winchell, F., "Tolt River Fisheries and Instream Flow Analysis", FRI, University of Washington, Seattle, Wa., 1983.

Tacoma Water Division, "Water System Plan, Volume I", City of Tacoma, Department of Public Utilities, Prepared by Economic and Engineering Services, July 1987.

Tacoma Water Division, "Water System Plan, Volume IV", City of Tacoma, Department of Public Utilities, Prepared By Charles Howard and Associates, July 1987.

U.S.G.S., William G. Sikonmia Memorandum to David Parkinson, Seattle Water Department, April 1989.

U.S. Statutes at Large, Vol. 96, Part I., US Government Printing Office, Washington D.C., p. 1046, 1976.

Yeh, William W-G, "Reservoir Management and Operations Models: A State-of-the-Art Review", *Water Resources Research,* Vol. 21(12), pp. 1797-1818, 1985.

Appendix A

STELLA Model

The figure below is a reduction of the complete STELLA model for the intertied system. The following nine pages show individual components of the model at a larger scale to facilitate the readers understanding the logic and assumptions used to replicate the system.

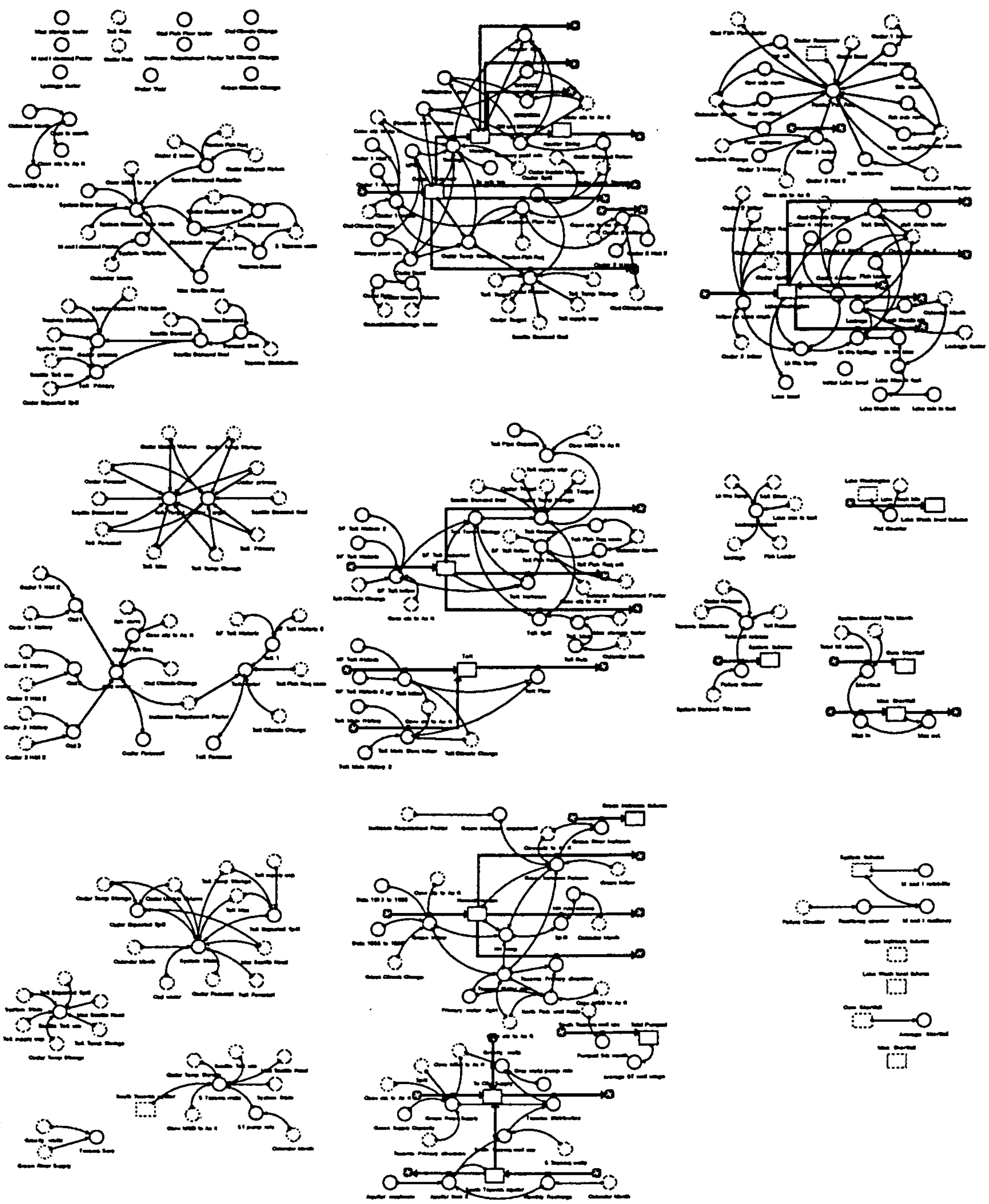

Figure A.1 Stella model of Seattle - Tacoma intertied system

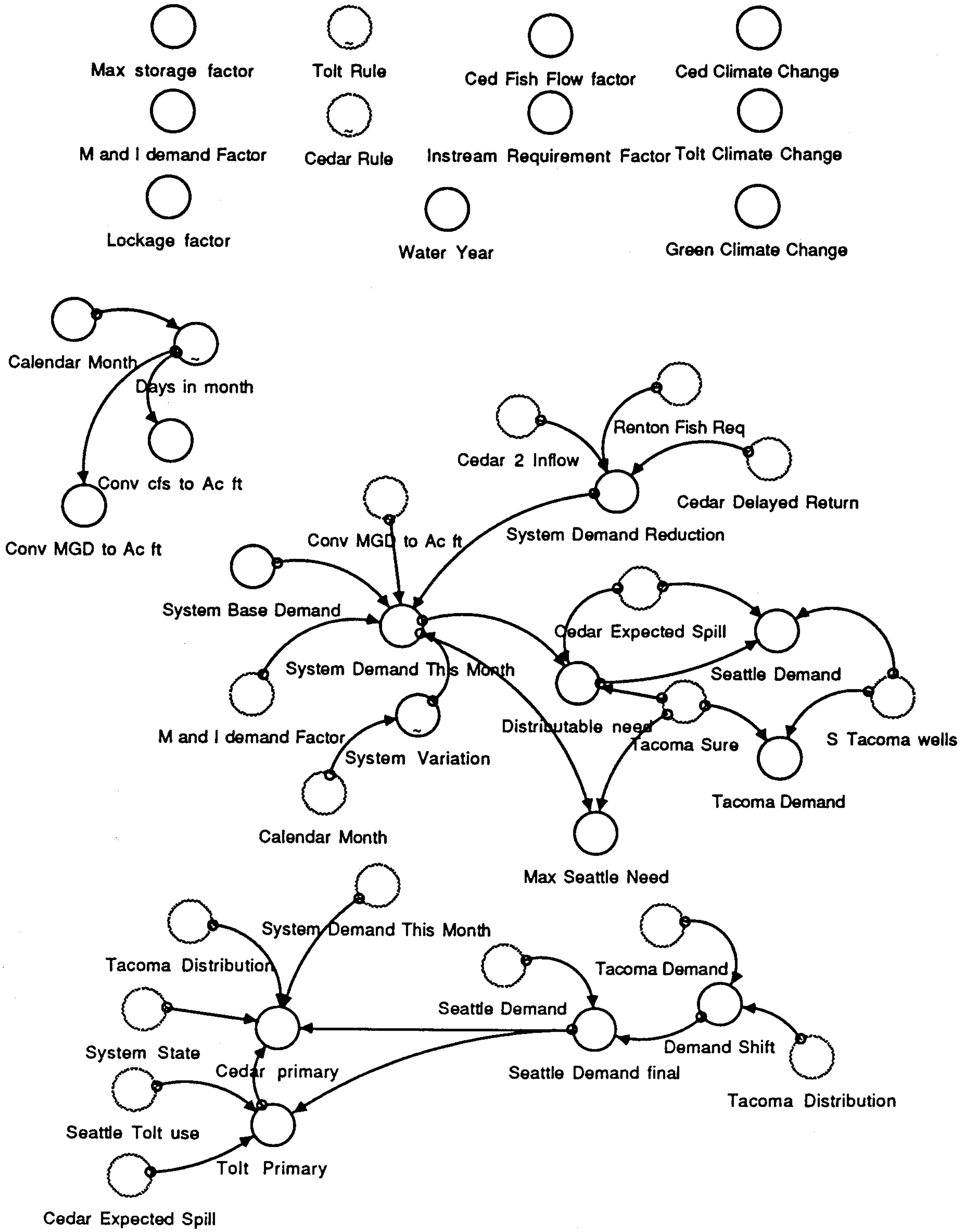

Figure A.2 Seattle - Tacoma intertied system (section 1)

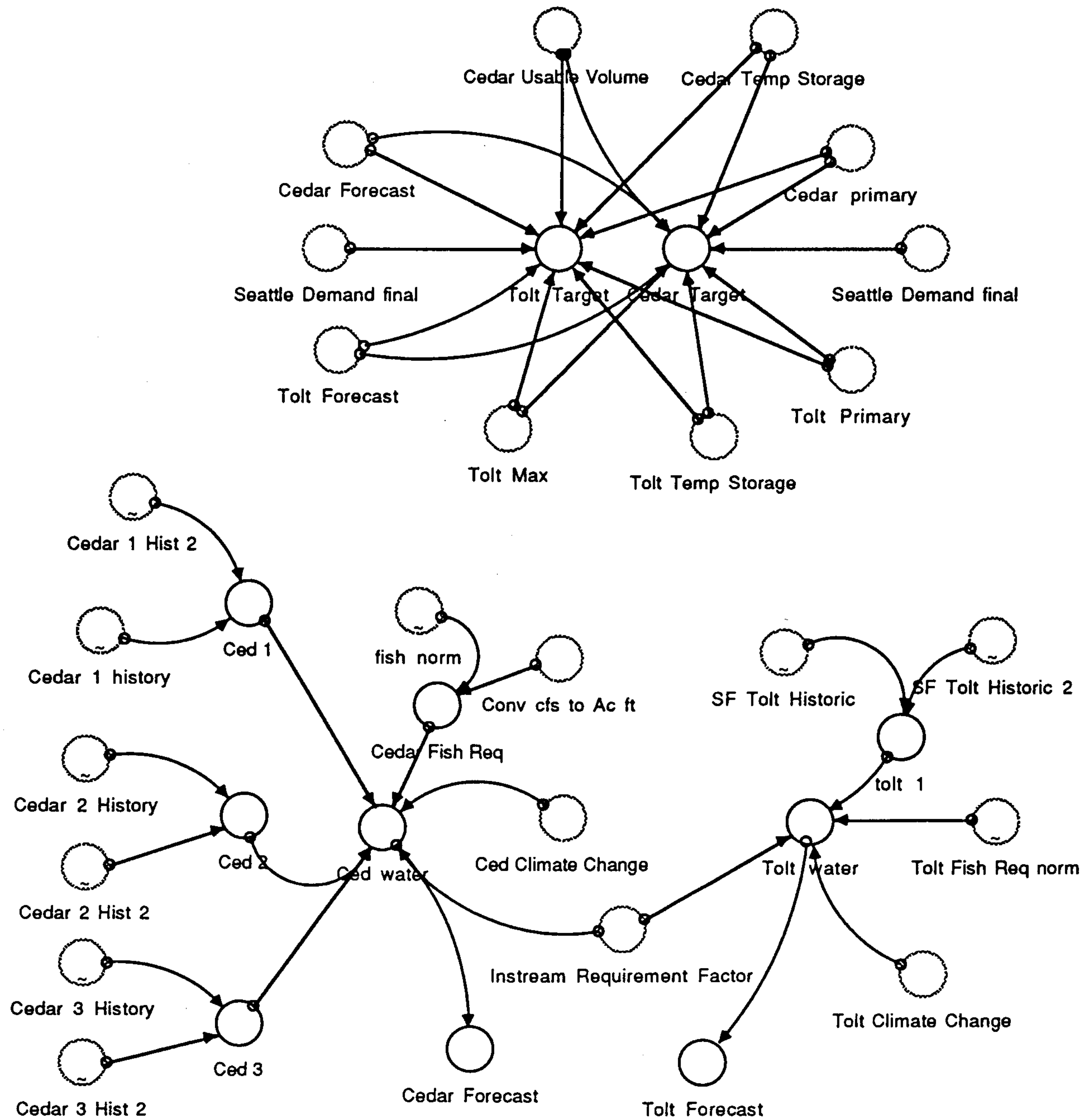

Figure A.3 Seattle - Tacoma intertied system (section 2)

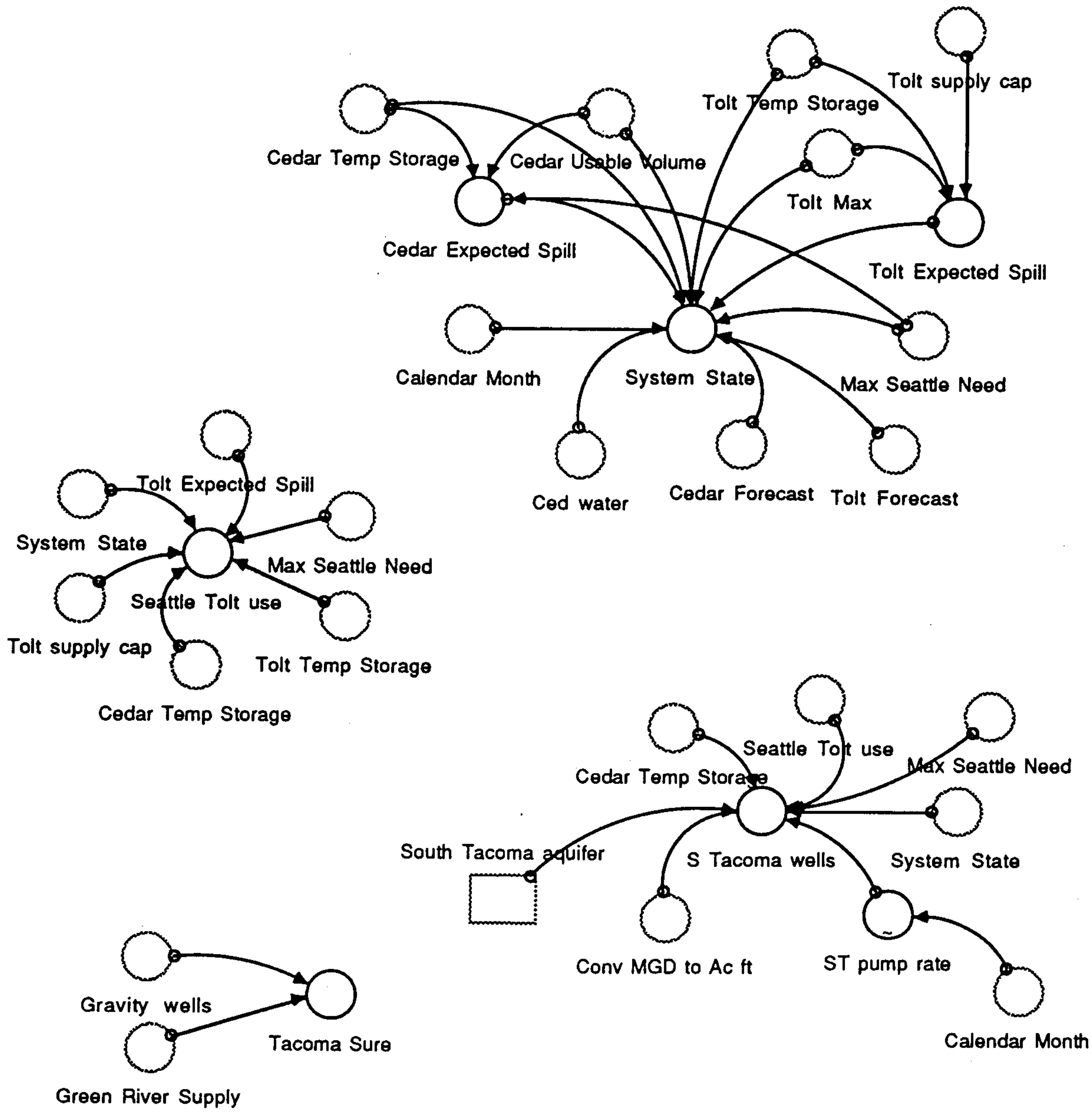

Figure A.4 Seattle - Tacoma intertied system (section 3)

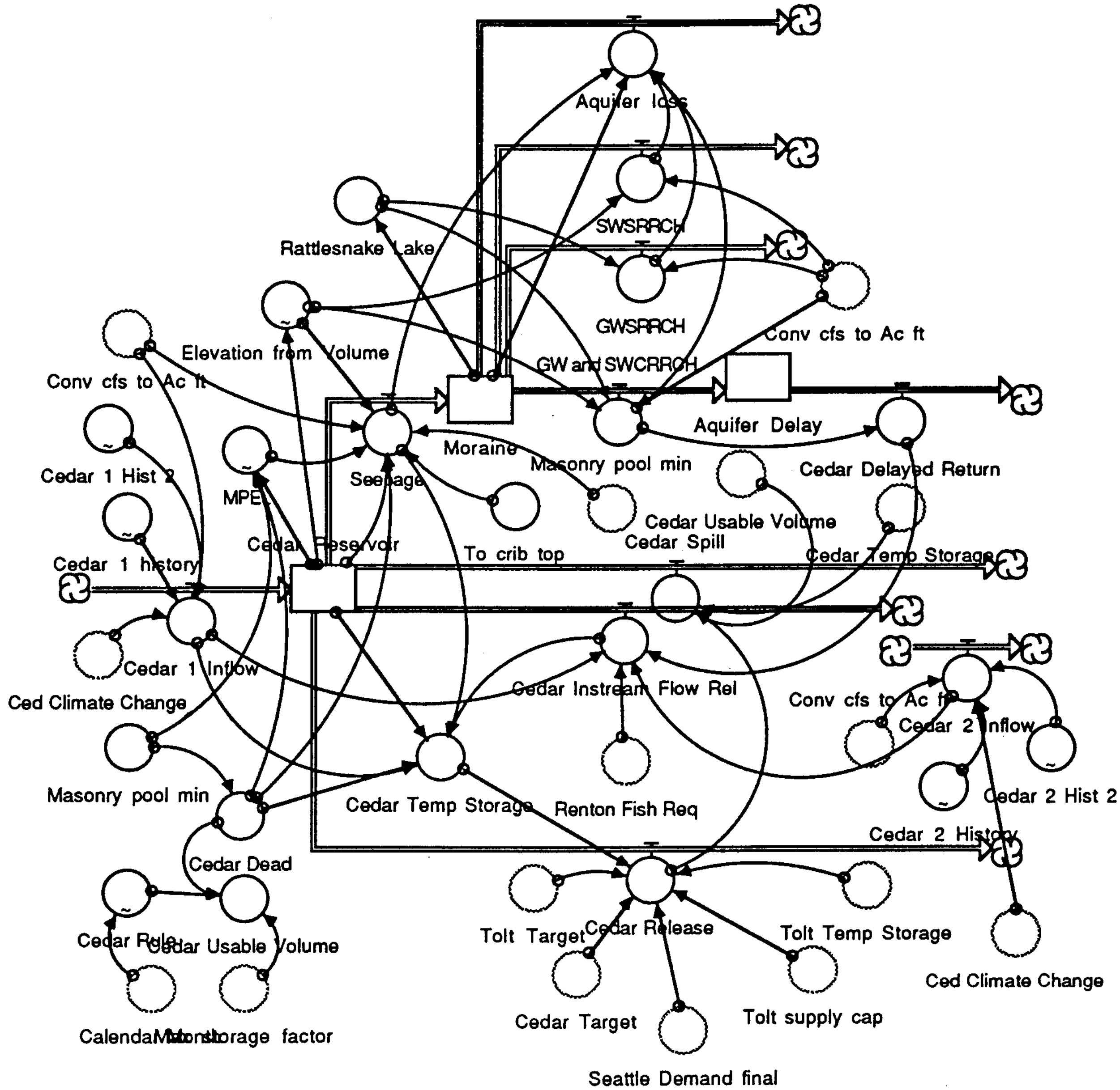

Figure A.5 Seattle - Tacoma intertied system (section 4)

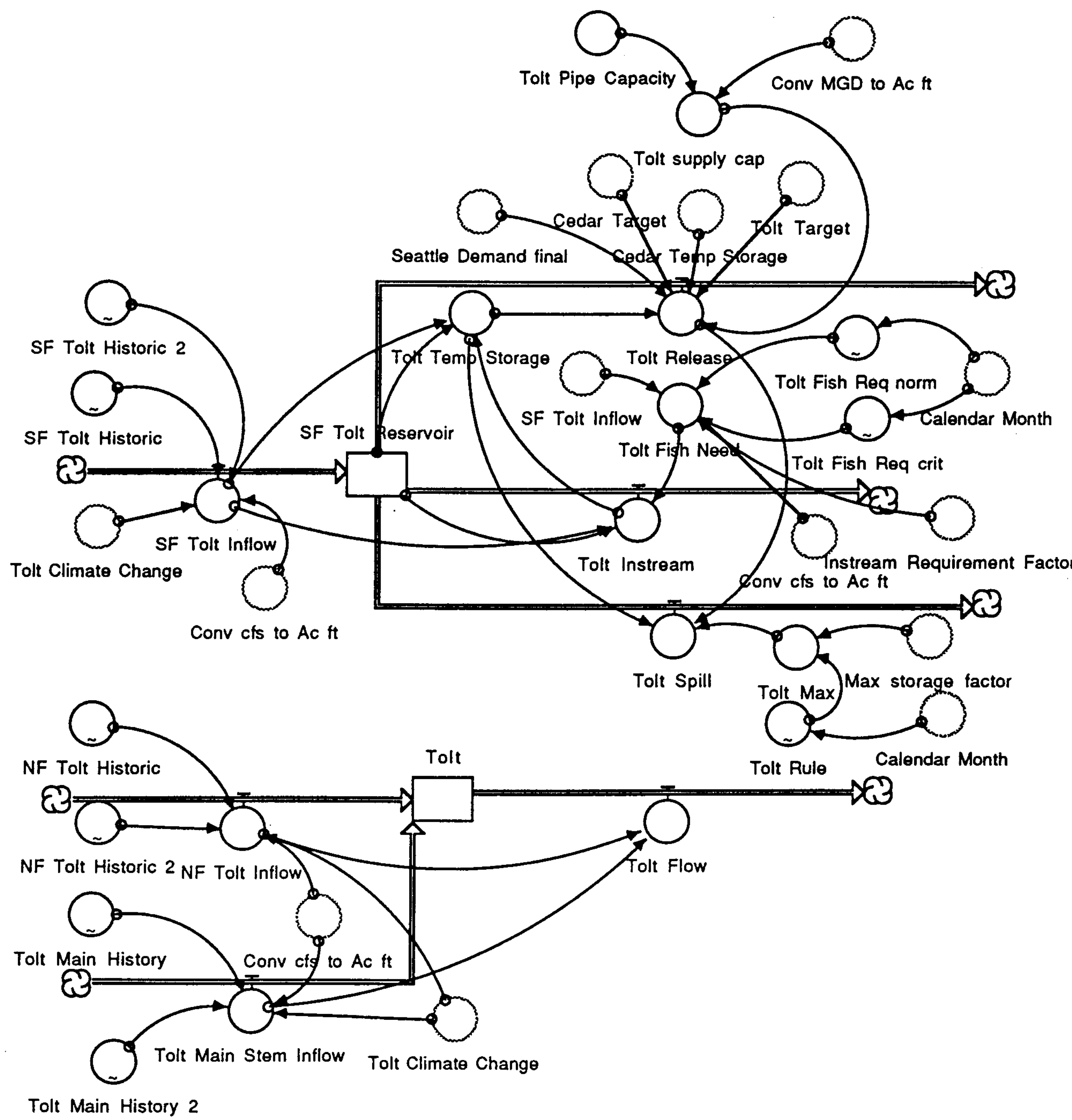

Figure A.6 Seattle - Tacoma intertied system (section 5)

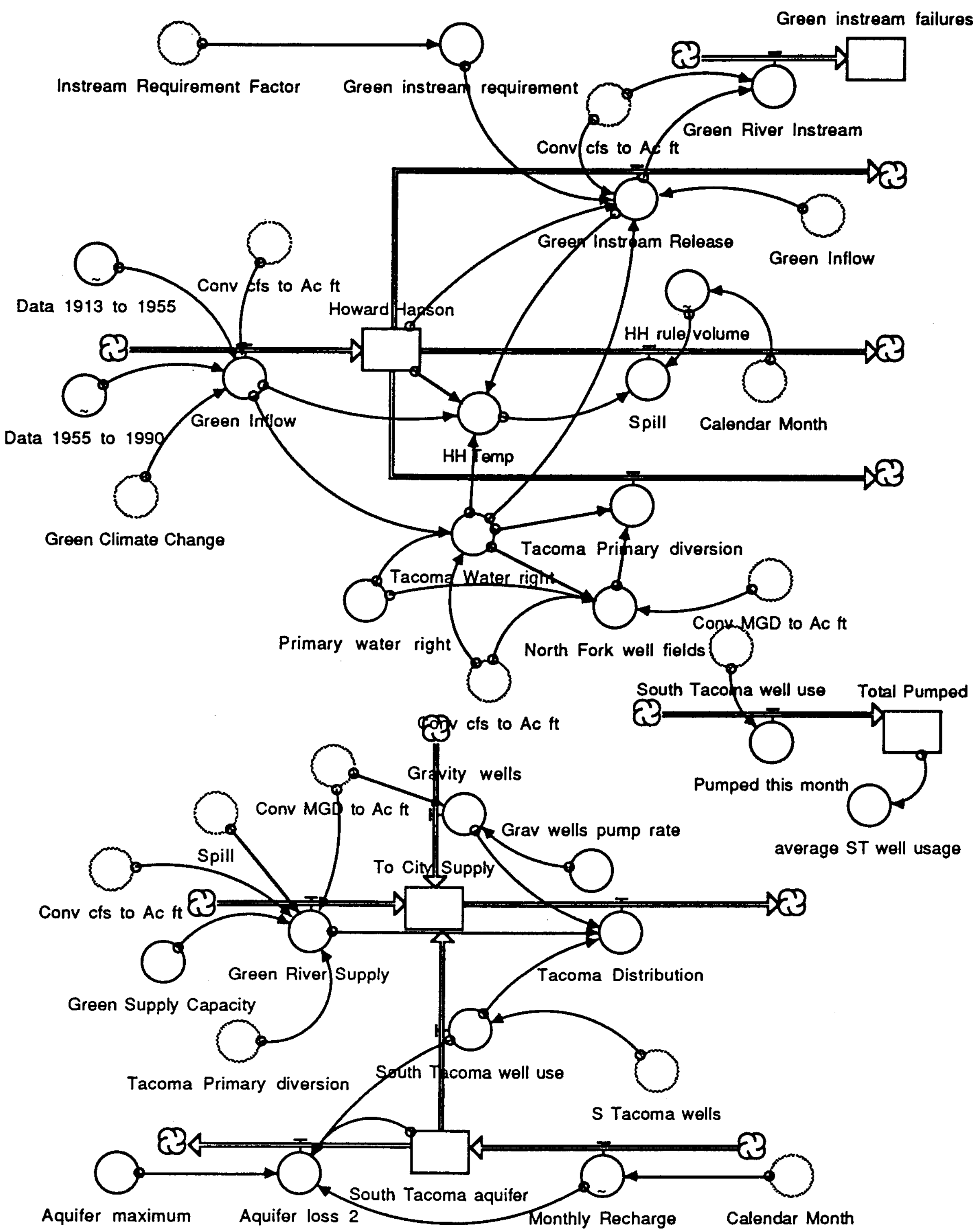

Figure A.7 Seattle - Tacoma intertied system (section 6)

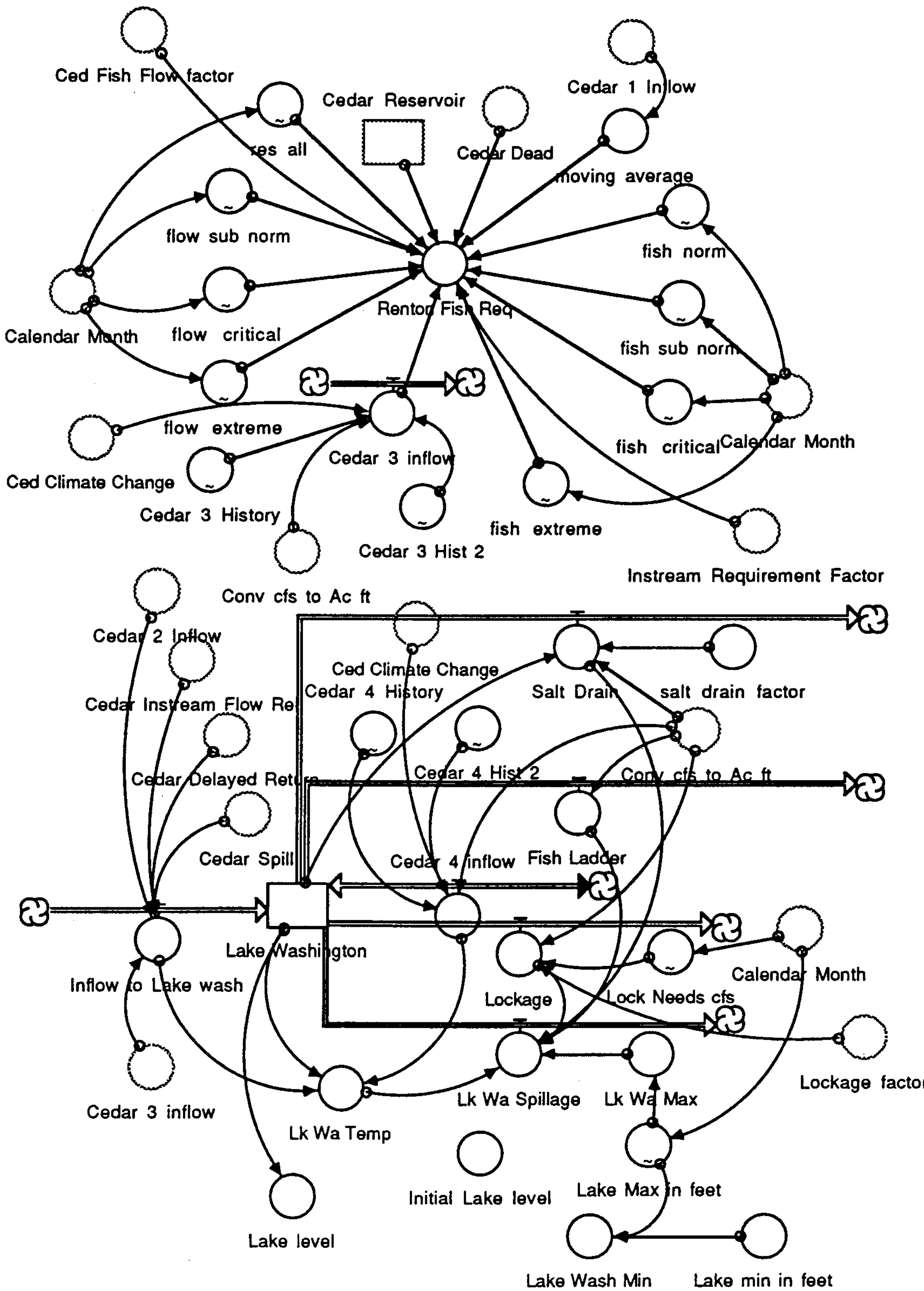

Figure A.8 Seattle - Tacoma intertied system (section 7)

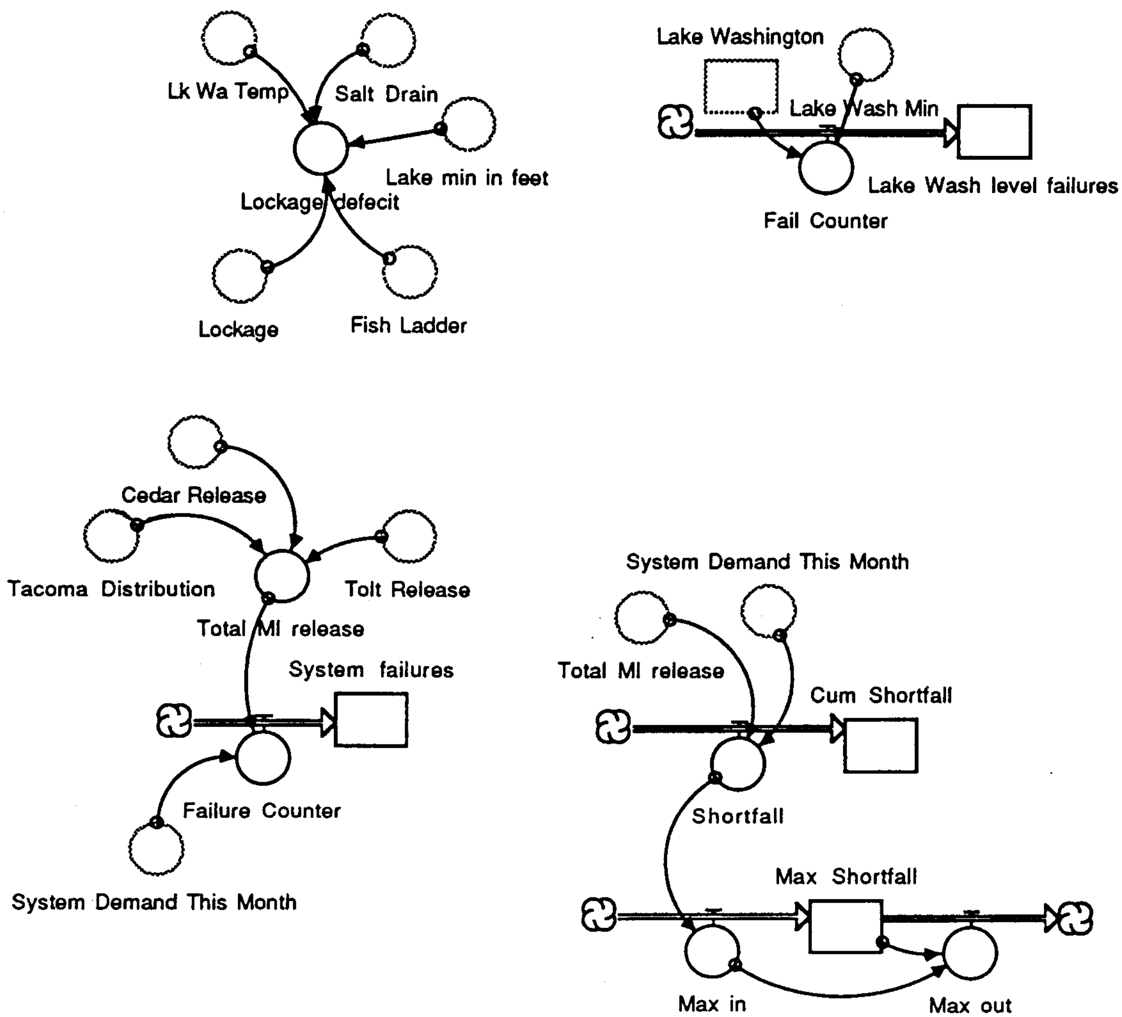

Figure A.9 Seattle - Tacoma intertied system (section 8)

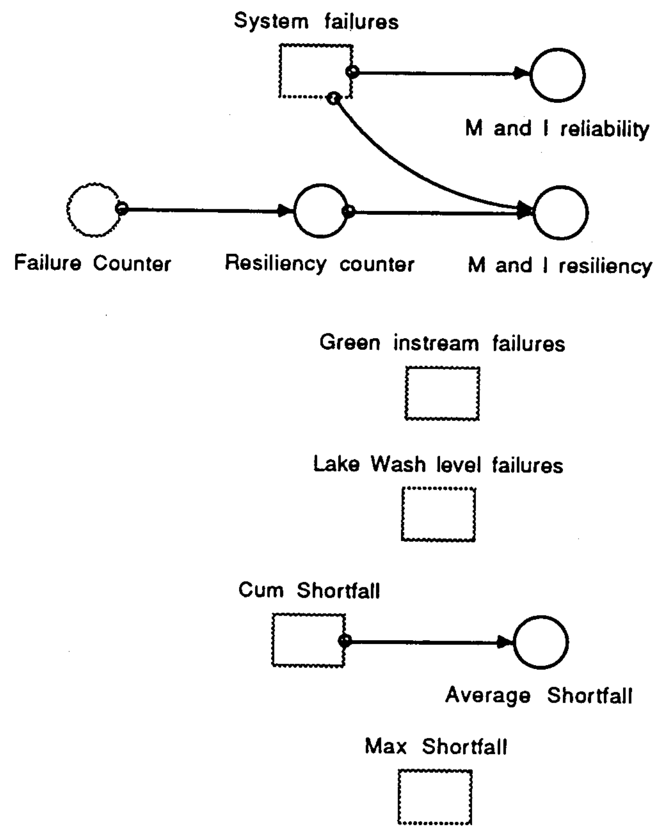

Figure A.10 Seattle - Tacoma intertied system (section 9)

Appendix B

Model Documentation and Equations

Aquifer_Delay(t) = Aquifer_Delay(t - dt) + (GW_and_SWCRRCH - Cedar_Delayed_Return) * dt

INIT Aquifer_Delay = 5000

INFLOWS:

GW_and_SWCRRCH = if .7*Elevation_from_Volume+ .3*DELAY(Elevation_from_Volume,1) > 15234.4 and DELAY(Rattlesnake_Lake,1) > 852 then ((1.95677*DELAY(Rattlesnake_Lake,1))-1638.28)*Conv_cfs_to_Ac_ft + (2.28+0.003825*(((.7*Elevation_from_Volume/10)+(.3*(DELAY(Elevation_from_Volume,1)/10))-1523.44)^2.5827))*Conv_cfs_to_Ac_ft else if DELAY(Rattlesnake_Lake,1) > 852 then ((1.95677*DELAY(Rattlesnake_Lake,1))-1638.28)*Conv_cfs_to_Ac_ft else 0

OUTFLOWS:

Cedar_Delayed_Return = GW_and_SWCRRCH

Cedar_Reservoir(t) = Cedar_Reservoir(t - dt) + (Cedar_1_Inflow - Seepage - Cedar_Instream_Flow_Rel - Cedar_Release - Cedar_Spill) * dt

INIT Cedar_Reservoir = Cedar_Rule

INFLOWS:

Cedar_1_Inflow = ((DELAY(Cedar_1_history,3)+DELAY(Cedar_1_Hist_2,3))*Conv_cfs_to_Ac_ft)*Ced_Climate_Change

OUTFLOWS:

Seepage = if Cedar_Reservoir <= Cedar_Dead+To_crib_top then min(Cedar_Reservoir-Cedar_Dead,40.01*Conv_cfs_to_Ac_ft) else if Cedar_Reservoir < Cedar_Dead+To_crib_top+(3800-Masonry_pool_min) then min(Cedar_Reservoir-Cedar_Dead,(35.47+0.023039*((MPEL/10)-1500)^2.3035)*Conv_cfs_to_Ac_ft) else min(Cedar_Reservoir-Cedar_Dead,(35.47+0.023039*((Elevation_from_Volume/10)-1500)^2.3035)*Conv_cfs_to_Ac_ft)

Cedar_Instream_Flow_Rel = MIN(Cedar_1_Inflow,Renton_Fish_Req-Cedar_Delayed_Return-Cedar_2_Inflow)

Cedar_Release = if Cedar_Target+Tolt_Target >= .9999*Seattle_Demand_final and Tolt_Temp_Storage >= Tolt_Target then min(Cedar_Target,Cedar_Temp_Storage) else if Cedar_Temp_Storage-Cedar_Target <= 0 and Tolt_Temp_Storage-Tolt_Target > 0 then max(min(Cedar_Temp_Storage,(Cedar_Target+Tolt_Target)-Tolt_supply_cap),min(Cedar_Temp_Storage,(Cedar_Target+Tolt_Target)-Tolt_Temp_Storage),0)else if Cedar_Temp_Storage > Cedar_Target and Tolt_Temp_Storage > Tolt_Target and Tolt_Target <= Tolt_supply_cap then Cedar_Target else if Cedar_Temp_Storage > Cedar_Target and Tolt_Temp_Storage < Tolt_Target and Tolt_Target <= Tolt_supply_cap then min(Cedar_Temp_Storage,Tolt_Target-Tolt_Temp_Storage+Cedar_Target) else if Cedar_Temp_Storage > Cedar_Target and Tolt_supply_cap <= Tolt_Target then min(Cedar_Temp_Storage,Cedar_Target+Tolt_Target-Tolt_supply_cap) else Cedar_Temp_Storage

Cedar_Spill = if Cedar_Temp_Storage-Cedar_Release > Cedar_Usable_Volume then Cedar_Temp_Storage-Cedar_Release-Cedar_Usable_Volume else 0

Cum_Shortfall(t) = Cum_Shortfall(t - dt) + (Shortfall) * dt

INIT Cum_Shortfall = 0

INFLOWS:

Shortfall = max(0,System_Demand_This_Month-Total_MI_release)

Green_instream_failures(t) = Green_instream_failures(t - dt) + (Green_River_Instream) * dt

INIT Green_instream_failures = 0

INFLOWS:

Green_River_Instream = if Green_Instream_Release < 110 * Conv_cfs_to_Ac_ft then 1 else 0

Howard_Hanson(t) = Howard_Hanson(t - dt) + (Green_Inflow - Tacoma_Primary_diversion - Spill - Green_Instream_Release) * dt

INIT Howard_Hanson = HH_rule_volume

INFLOWS:

Green_Inflow =
((DELAY(Data_1913_to_1955,3)+DELAY(Data_1955_to_1990,3))*Conv_cfs_to_Ac_ft)*Green_Climate_Change

OUTFLOWS:

Tacoma_Primary_diversion =
North_Fork_well_fields+Tacoma_Water_right

DOCUMENT: The Tacoma Primary diversion is the Green River diversion that is allowed under the initial 113 cfs water right granted to the City of Tacoma. Currently there is no switching mechanism for transfer between the water right diversion and the well fields. If some indicator could be chosen for turbidity, it would be possible to incorporate a switching mechanism between these two sources.

Spill = if HH_Temp>HH_rule_volume then HH_Temp-HH_rule_volume else 0

Green_Instream_Release = min
(Green_instream_requirement*Conv_cfs_to_Ac_ft,Howard_Hanson+Green_Inflow-Tacoma_Water_right)

Lake_Washington(t) = Lake_Washington(t - dt) + (Cedar_4_inflow_ + Inflow_to_Lake_wash - Lockage - Lk_Wa_Spillage - Fish_Ladder - Salt_Drain) * dt

INIT Lake_Washington = Initial_Lake_level*23200

INFLOWS:

Cedar_4_inflow_ =
((DELAY(Cedar_4_History,3)+DELAY(Cedar_4_Hist_2,3))*Conv_cfs_to_Ac_ft)*Ced_Climate_Change

Inflow_to_Lake_wash =
Cedar_Spill+Cedar_Delayed_Return+Cedar_Instream_Flow_Rel+Cedar_2_Inflow+Cedar_3_inflow

OUTFLOWS:

Lockage = (Lock_Needs_cfs*Conv_cfs_to_Ac_ft)*Lockage_factor

Lk_Wa_Spillage = if Lk_Wa_Temp-Fish_Ladder-Lockage-Salt_Drain > Lk_Wa_Max then Lk_Wa_Temp-Fish_Ladder-Lockage-Salt_Drain-Lk_Wa_Max else 0

Fish_Ladder = 47*Conv_cfs_to_Ac_ft

Salt_Drain = ((Lake_Washington/23200)*salt_drain_factor - 38.4)*Conv_cfs_to_Ac_ft

DOCUMENT: The flow through the salt drain is modeled as a function of the elevation in the lake. The constant (the final term in the equation below) was found to be -38.4 by the COE in a regression analysis of salt drain flows. URS found the constant to be -8.67.

Lake_Wash_level_failures(t) = Lake_Wash_level_failures(t - dt) + (Fail_Counter) * dt

INIT Lake_Wash_level_failures = 0

INFLOWS:

Fail_Counter = if Lake_Washington<=Lake_Wash_Min then 1 else 0

Max_Shortfall(t) = Max_Shortfall(t - dt) + (Max_in - Max_out) * dt

INIT Max_Shortfall = 0

INFLOWS:

Max_in = Shortfall

OUTFLOWS:

Max_out = min(Max_in,Max_Shortfall)

Moraine(t) = Moraine(t - dt) + (Seepage - SWSRRCH - GWSRRCH - GW_and_SWCRRCH - Aquifer_loss) * dt

INIT Moraine = 30000

INFLOWS:

Seepage = if Cedar_Reservoir <= Cedar_Dead+To_crib_top then min(Cedar_Reservoir-Cedar_Dead,40.01*Conv_cfs_to_Ac_ft) else if Cedar_Reservoir < Cedar_Dead+To_crib_top+(3800-Masonry_pool_min) then min(Cedar_Reservoir-Cedar_Dead,(35.47+0.023039*((MPEL/10)-1500)^2.3035)*Conv_cfs_to_Ac_ft) else min(Cedar_Reservoir-Cedar_Dead,(35.47+0.023039*((Elevation_from_Volume/10)-1500)^2.3035)*Conv_cfs_to_Ac_ft)

OUTFLOWS:

SWSRRCH = if (.5*Elevation_from_Volume/10)+(.5*DELAY(Elevation_from_Volume,1)/10)-1527.43 < 0 then 0 else (2.45+0.027836*((.5*Elevation_from_Volume/10)+(.5*DELAY(Elevation_from_Volume,1)/10)-1527.43)^2.2543)*Conv_cfs_to_Ac_ft

GWSRRCH = if DELAY(Rattlesnake_Lake,1) > 905.2 then (10+(11.334*(DELAY(Rattlesnake_Lake,1)-905.2)^1.5))*Conv_cfs_to_Ac_ft else 10*Conv_cfs_to_Ac_ft

GW_and_SWCRRCH = if .7*Elevation_from_Volume+ .3*DELAY(Elevation_from_Volume,1) > 15234.4 and DELAY(Rattlesnake_Lake,1) > 852 then ((1.95677*DELAY(Rattlesnake_Lake,1))-1638.28)*Conv_cfs_to_Ac_ft + (2.28+0.003825*(((.7*Elevation_from_Volume/10)+(.3*(DELAY(Elevation_from_Volume,1)/10))-1523.44)^2.5827))*Conv_cfs_to_Ac_ft else if DELAY(Rattlesnake_Lake,1) > 852 then ((1.95677*DELAY(Rattlesnake_Lake,1))-1638.28)*Conv_cfs_to_Ac_ft else 0

Aquifer_loss = if Moraine+Seepage -SWSRRCH-GWSRRCH-GW_and_SWCRRCH > 47000 then 47000 - Moraine+Seepage -SWSRRCH-GWSRRCH-GW_and_SWCRRCH else 0

SF_Tolt_Reservoir(t) = SF_Tolt_Reservoir(t - dt) + (SF_Tolt_Inflow - Tolt_Instream - Tolt_Spill - Tolt_Release) * dt

INIT SF_Tolt_Reservoir = Tolt_Max

INFLOWS:

SF_Tolt_Inflow = ((DELAY(SF_Tolt_Historic,3)+DELAY(SF_Tolt_Historic_2,3))*Conv_cfs_to_Ac_ft)*Tolt_Climate_Change

OUTFLOWS:

Tolt_Instream = if SF_Tolt_Inflow+SF_Tolt_Reservoir > Tolt_Fish_Need then Tolt_Fish_Need else SF_Tolt_Inflow+SF_Tolt_Reservoir

Tolt_Spill = if Tolt_Temp_Storage-Tolt_Release > Tolt_Max then Tolt_Temp_Storage-Tolt_Release-Tolt_Max else 0

Tolt_Release = if Cedar_Target+Tolt_Target >= .9999*Seattle_Demand_final and Cedar_Temp_Storage >= Cedar_Target then Tolt_Target else if Cedar_Temp_Storage-Cedar_Target <= 0 and Tolt_Temp_Storage-Tolt_Target > 0 then min(Tolt_Temp_Storage,(Cedar_Target+Tolt_Target),Tolt_supply_cap) else if Tolt_Temp_Storage > Tolt_Target and Tolt_Target <= Tolt_supply_cap and Cedar_Temp_Storage > Cedar_Target then Tolt_Target else if Tolt_Temp_Storage > Tolt_Target and Tolt_Target >= Tolt_supply_cap then Tolt_supply_cap else if Tolt_Temp_Storage > Tolt_Target and Cedar_Temp_Storage < Cedar_Target then min(Tolt_supply_cap,Tolt_Temp_Storage,Cedar_Target-Cedar_Temp_Storage+Tolt_Target) else min(Tolt_supply_cap,Tolt_Temp_Storage)

South_Tacoma_aquifer(t) = South_Tacoma_aquifer(t - dt) + (Monthly_Recharge - Aquifer_loss_2 - South_Tacoma_well_use) * dt

INIT South_Tacoma_aquifer = Aquifer_maximum/2

INFLOWS:

Monthly_Recharge = GRAPH(Calendar_Month)

(1, 1250) (2, 1250) (3, 1250) (4, 1250) (5, 1250) (6, 1000) (7, 500) (8, 500) (9, 500) (10, 750) (11, 1250) (12, 1250)

OUTFLOWS:

Aquifer_loss_2 = if South_Tacoma_aquifer+Monthly_Recharge-South_Tacoma_well_use > Aquifer_maximum then South_Tacoma_aquifer+Monthly_Recharge-South_Tacoma_well_use-Aquifer_maximum else 0

South_Tacoma_well_use = S_Tacoma_wells

System_failures(t) = System_failures(t - dt) + (Failure_Counter) * dt

INIT System_failures = 0

INFLOWS:

Failure_Counter = if Total_MI_release >= .9999*(System_Demand_This_Month) then 0 else 1

Tolt(t) = Tolt(t - dt) + (NF_Tolt_Inflow + Tolt_Main_Stem_Inflow - Tolt_Flow) * dt

INIT Tolt = 0

INFLOWS:

NF_Tolt_Inflow = ((DELAY(NF_Tolt_Historic,3)+DELAY(NF_Tolt_Historic_2,3))*Conv_cfs_to_Ac_ft)*Tolt_Climate_Change

Tolt_Main_Stem_Inflow = ((DELAY(Tolt_Main_History,3)+DELAY(Tolt_Main_History_2,3))*Conv_cfs_to_Ac_ft)*Tolt_Climate_Change

OUTFLOWS:

Tolt_Flow = NF_Tolt_Inflow+Tolt_Main_Stem_Inflow

Total_Pumped(t) = Total_Pumped(t - dt) + (Pumped_this_month) * dt

INIT Total_Pumped = 0

INFLOWS:

Pumped_this_month = South_Tacoma_well_use

To_City_Supply(t) = To_City_Supply(t - dt) + (Green_River_Supply + South_Tacoma_well_use + Gravity_wells - Tacoma_Distribution) * dt

INIT To_City_Supply = 0

INFLOWS:

Green_River_Supply = min(Tacoma_Primary_diversion+min(Spill,100*Conv_cfs_to_Ac_ft),Green_Supply_Capacity*Conv_MGD_to_Ac_ft)

DOCUMENT: The Green River supply is composed of three parts. The first is the direct diversion of Green River water allowed by the primary water right of 113 cfs. The second is supplemental water from the North Fork well field in the event that the Green River flow is insufficient. The third component of the Green River Supply is water withdrawn from the reservoir spillage as per the secondary water right of 100 cfs. This withdrawal is available only in times of low turbidity (here modeled as times when change in inflow is less than three fold and total inflow is higher than 40000 Acre feet). Additionally the total withdrawal from the Green system cannot exceed the Pipeline capacity which is currently 110 MGD.

South_Tacoma_well_use = S_Tacoma_wells

Gravity_wells = Grav_wells_pump_rate*Conv_MGD_to_Ac_ft

OUTFLOWS:

Tacoma_Distribution =
South_Tacoma_well_use+Green_River_Supply+Gravity_wells

Cedar_2_Inflow =
((DELAY(Cedar_2_History,3)+DELAY(Cedar_2_Hist_2,3))*Conv_cfs_to_Ac_ft)*Ced_Climate_Change

Cedar_3_inflow =
((DELAY(Cedar_3_History,3)+DELAY(Cedar_3_Hist_2,3))*Conv_cfs_to_Ac_ft)*Ced_Climate_Change

Aquifer_maximum = 42000

Average_Shortfall = if TIME=STARTTIME then Cum_Shortfall else Cum_Shortfall/(time - STARTTIME)

average_ST_well_usage = if time = starttime then 12000 else 12*(total_pumped/(time - starttime))

Calendar_Month = if mod(time,12) < 1e-10 then 12 else mod(time,12)

DOCUMENT: Month is the month in the calender year that the simulation is currently in.

Cedar_Dead = 36064+Masonry_pool_min

Cedar_Expected_Spill = max(0,min(Cedar_Temp_Storage-Cedar_Usable_Volume,Max_Seattle_Need))

Cedar_Fish_Req = (fish_norm/Conv_cfs_to_Ac_ft)

Cedar_Forecast =
DELAY(Ced_water,2)+DELAY(Ced_water,1)+DELAY(Ced_water,3)

Cedar_primary = if System_State = 5 then 0 else
min(max(0,Seattle_Demand_final-
Tolt_Primary),max(0,System_Demand_This_Month-Tolt_Primary-
Tacoma_Distribution))

Cedar_Target = if Cedar_primary+Tolt_Primary >=
.999*Seattle_Demand_final then Cedar_primary else
max(0,min(Seattle_Demand_final,Cedar_Temp_Storage+(Tolt_Max+Cedar_
Usable_Volume-Tolt_Temp_Storage-
Cedar_Temp_Storage+Seattle_Demand_final)*(Cedar_Forecast/(Tolt_Forec
ast+Cedar_Forecast))-Cedar_Usable_Volume))

Cedar_Temp_Storage = Cedar_Reservoir+Cedar_1_Inflow-Seepage-
Cedar_Dead-Cedar_Instream_Flow_Rel

Cedar_Usable_Volume = (Max_storage_factor*Cedar_Rule)-Cedar_Dead

Ced_1 = Cedar_1_Hist_2+Cedar_1_history

Ced_2 = Cedar_2_History+Cedar_2_Hist_2

Ced_3 = Cedar_3_History+Cedar_3_Hist_2

Ced_Climate_Change = 1

DOCUMENT: The Cedar Climate Change converter allows the user to increase or decrease the inflows on the Cedar system by a specified amount. All Cedar river historic flows will be multiplied by the value that is input here. A value of 1 will use the historic streamflow data provided by the Seattle Water Department.

Ced_Fish_Flow_factor = 1

DOCUMENT: This converter allows the user to change the level of flow in the Cedar River at which a new instream release regime will be utilized. The within year variation will remain the same but all of the switching levels for instream flows will be modified by the factor input here. A value of 1 will use the current switching criteria. Values less than 1 will require the system to provide higher levels of instream releases even under low flow scenarios. Values greater than 1 will allow switching to alternative release regimes at higher inflow levels.

Ced_water = max(0.01,(Ced_1+Ced_2+Ced_3)*Ced_Climate_Change-(Cedar_Fish_Req)*Instream_Requirement_Factor)

Conv_cfs_to_Ac_ft = Days_in_month*86400/43560

Conv_MGD_to_Ac_ft = 1000000/7.480530459/43560*Days_in_month

Demand_Shift = Tacoma_Demand-Tacoma_Distribution

Distributable_need = max(0,System_Demand_This_Month-(Cedar_Expected_Spill)-Tacoma_Sure)

Grav_wells_pump_rate = 16.7

DOCUMENT: This converter allows the user to input the maximum allowable pumping rate from the gravity and any additional wells which can be utilized to meet demands in the event of a shortfall. The rate should be input in sustainable MGD. (Prairie Ridge = 0.8 MGD, Gravity Wells = 8 MGD, Tide Flats = 1 MGD, Dash Point & University Place = ?)

Green_Climate_Change = 1

DOCUMENT: The Green Climate Change converter allows the user to increase or decrease the inflows on the Green system by a specified amount. All Green river historic flows will be multiplied by the value that is input here. A value of 1 will use the historic streamflow data provided by the Corp of Engineers.

Green_instream_requirement = 110*Instream_Requirement_Factor

DOCUMENT: The Corp of engineers operates the Howard Hanson reservoir for flood prevention. Its secondary purpose is to provide a minimum instream release throughout the year. The current level of this release is 110 cfs.

Green_Supply_Capacity = 72+72

HH_Temp = Howard_Hanson+Green_Inflow-Tacoma_Water_right-Green_Instream_Release

Initial_Lake_level = 21

DOCUMENT: This is the initial level of Lake Washington in feet above the datum. The typical range is between 20 and 22 feet.

Instream_Requirement_Factor = 1

DOCUMENT: This converter allows the user to change the level of instream flow release required in the Cedar and South Fork Tolt rivers. The within year variation will remain the same but all of the established levels for instream flows will be modified by the factor input here. A value of 1 will use the current requirements. Values less than 1 will allow the system to provide lower levels of instream releases. Values greater than 1 will nescessitate greater instream releases than are currently required.

Lake_level = Lake_Washington/23200

Lake_min_in_feet = 20

Lake_Wash_Min = min(Lake_min_in_feet*23200,(Lake_Max_in_feet)*23200)

Lk_Wa_Max = Lake_Max_in_feet*23200

Lk_Wa_Temp = Lake_Washington+Inflow_to_Lake_wash+Cedar_4_inflow_

Lockage_defecit = if Lk_Wa_Temp-Salt_Drain-Fish_Ladder - Lake_min_in_feet*23200 < Lockage then abs(Lk_Wa_Temp-Salt_Drain-Lockage-Fish_Ladder - Lake_min_in_feet*23200) else 0

Lockage_factor = 1

DOCUMENT: This converter allopws the user to increase the amount of lockage flow at the Chittendon locks on Lake Washington. The within year variation will remain the same with the total volume of water released each month being multiplied by this factor. A value of 1 will use the current average lockage rates.

Masonry_pool_min = 415

Max_Seattle_Need = max(0,System_Demand_This_Month-Tacoma_Sure)

Max_storage_factor = 1

DOCUMENT: The maximum storage factor is a variable that allows the user to increase the storage capacities of the Chester Morse Lake- Masonry Pool and the South Fork Tolt reservoirs. A value of 1 will use the current system configurations.

moving_average =
(DELAY(Cedar_1_Inflow,2)+DELAY(Cedar_1_Inflow,1)+Cedar_1_Inflow)/3

DOCUMENT: This converter calculates a moving average of the Cedar I inflows in the previous month and the current month for purposes of switching from normal instream flow requirements to some other regime.

M_and_I_demand_Factor = 1.148

DOCUMENT: This converter allows the user to increase the expected municipal demand by a fractional amount. The base demand that will be altered by this factor is 259.5 MGD.

M_and_I_reliability = if TIME=STARTTIME then (1-System_failures)*100 else (System_failures/(time - STARTTIME))*100

M_and_I_resiliency = if System_failures=0 then 1 else (Resiliency_counter/System_failures)*100

North_Fork_well_fields = if time < 1000 then 0 else if Tacoma_Water_right < Primary_water_right*Conv_cfs_to_Ac_ft then min(Primary_water_right*Conv_cfs_to_Ac_ft-Tacoma_Water_right,25*Conv_MGD_to_Ac_ft) else 0

DOCUMENT: This cell allows the North Fork well field to be utilized as an additional source of water when the available Green River source is not capable of meeting the primary water right. The maximum pumping rate on the well fields is set at 25 MGD as per the analysis done for the 1987 Water System Plan. As currently modeled there is no time at which the Green River direct diversion is taken out of service due to turbidity. (By setting this value to 0 we have eliminated its benefits in meeting demands.)

Primary_water_right = (111.4002994)

Rattlesnake_Lake = if Moraine < 50000 then 820.461+(0.00445776*(Moraine))-(7.21857E-8) *(Moraine)^2 + (3.5093E-13)*(Moraine)^3 else 907+.00015*(Moraine-50000)

Renton_Fish_Req = if (Cedar_Reservoir-Cedar_Dead) < res_all and moving_average < Ced_Fish_Flow_factor*flow_extreme then max(0,Instream_Requirement_Factor*fish_extreme-Cedar_3_inflow) else if (Cedar_Reservoir-Cedar_Dead) < res_all and moving_average < Ced_Fish_Flow_factor*flow_critical then max(0,Instream_Requirement_Factor*fish_critical-Cedar_3_inflow) else if (Cedar_Reservoir-Cedar_Dead) < res_all and moving_average < Ced_Fish_Flow_factor*flow_sub_norm then max(0,Instream_Requirement_Factor*fish_sub_norm-Cedar_3_inflow) else max(0,Instream_Requirement_Factor*fish_norm-Cedar_3_inflow)

Resiliency_counter = if Failure_Counter=0 and DELAY(Failure_Counter,1)=1 then 1 else 0

salt_drain_factor = 13.1

DOCUMENT: This converter allows the user to specify the additional losses through the salt drain as a function of Lake Washington elevation. These are only the losses above the lockage flows and fish ladder flows that are already incorporated in the model. The Corp of Engineers regression analysis showed thi sto be 13.1. The URS model [1980] found the rate to be 14.25.

Seattle_Demand = max(0,Distributable_need+Cedar_Expected_Spill-S_Tacoma_wells)

Seattle_Demand_final = Seattle_Demand+(Demand_Shift)

DOCUMENT: Demand is the monthly M&I requirement after reductions due to inflows below the dam and above the Landsberg diversion.

Seattle_Tolt_use = if System_State = 0 then Tolt_Expected_Spill else if System_State = 1 then min(Tolt_supply_cap,Tolt_Temp_Storage, Max_Seattle_Need) else if System_State = 2 then min(Tolt_supply_cap,Tolt_Temp_Storage,Max_Seattle_Need) else if System_State = 3 then min(Tolt_supply_cap,Tolt_Temp_Storage,Max_Seattle_Need) else if System_State = 4 then min(Tolt_supply_cap,Tolt_Temp_Storage,Max_Seattle_Need) else if System_State=5 then min(Tolt_supply_cap,Tolt_Temp_Storage,Max_Seattle_Need)/3 else if System_State = 6 then min(Tolt_supply_cap,Tolt_Temp_Storage,max(0,Max_Seattle_Need-

(Cedar_Temp_Storage))) else
min(Tolt_supply_cap,Tolt_Temp_Storage,Max_Seattle_Need/2)

System_Base_Demand = 259.5

DOCUMENT: This variable lists the average daily demand for the combined system in MGD. The Seattle system has a firm yield of 150 MGD (this is the average daily demand that can be satisfied [without failure] year round when multiplied by the appropriate monthly weighting factors). The system fails in 1953 at 156 MGD. The Tacoma system with the (secondary water right) has a firm yield of 103.5 MGD with its first failure occurring in 1987. Adding these two together gives a total separate system yield of 264 MGD.

System_Demand_Reduction =
max(Cedar_Delayed_Return+Cedar_2_Inflow-Renton_Fish_Req,0)

System_Demand_This_Month =
max(0,((System_Base_Demand)*M_and_I_demand_Factor*System_Variation*Conv_MGD_to_Ac_ft)-System_Demand_Reduction)

System_State = if Tolt_Expected_Spill+Cedar_Expected_Spill >= Max_Seattle_Need then 0 else if Cedar_Expected_Spill >= Max_Seattle_Need/2 then 0 else if Tolt_Temp_Storage < .45*Tolt_Max and Cedar_Temp_Storage < .85*Cedar_Usable_Volume then 1 else if Tolt_Temp_Storage > .9*Tolt_Max and Cedar_Forecast+Tolt_Forecast <= 200 then 2 else if Cedar_Forecast+Tolt_Forecast <= 1 then 2 else if Calendar_Month >=8 and Calendar_Month<= 9 and Cedar_Temp_Storage < DELAY(Cedar_Usable_Volume,11) then 4 else if Tolt_Temp_Storage >= .75*Tolt_Max then 3 else if Tolt_Temp_Storage < .5*Tolt_Max and Cedar_Temp_Storage >= .85*Cedar_Usable_Volume then 5 else if Cedar_Temp_Storage-Max_Seattle_Need + 60*DELAY(Ced_water,2) >= DELAY(Cedar_Usable_Volume,10) then 6 else if Calendar_Month>=3 and Calendar_Month<=5 and Cedar_Temp_Storage < Cedar_Usable_Volume then 1 else 7

S_Tacoma_wells = if System_State = 0 then 0 else if System_State = 1 then min(South_Tacoma_aquifer,Max_Seattle_Need,ST_pump_rate*Conv_MGD_to_Ac_ft) else if System_State = 2 then min(South_Tacoma_aquifer/3,max(0,Max_Seattle_Need-Seattle_Tolt_use),ST_pump_rate*Conv_MGD_to_Ac_ft) else if System_State = 3 then min(South_Tacoma_aquifer,max(0,Max_Seattle_Need-Cedar_Temp_Storage-Seattle_Tolt_use),ST_pump_rate*Conv_MGD_to_Ac_ft) else if

System_State = 4 and South_Tacoma_aquifer > 30000 then min(South_Tacoma_aquifer,max(0,(Max_Seattle_Need-Seattle_Tolt_use)),ST_pump_rate*Conv_MGD_to_Ac_ft) else if System_State = 4 then min(South_Tacoma_aquifer/3,max(0,(Max_Seattle_Need-Seattle_Tolt_use)/3,Max_Seattle_Need-Seattle_Tolt_use-Cedar_Temp_Storage),ST_pump_rate*Conv_MGD_to_Ac_ft) else if System_State = 5 then min(South_Tacoma_aquifer,max(0,Max_Seattle_Need-Cedar_Temp_Storage-Seattle_Tolt_use),ST_pump_rate*Conv_MGD_to_Ac_ft) else if System_State=6 then min(South_Tacoma_aquifer,max(0,Max_Seattle_Need-Cedar_Temp_Storage-Seattle_Tolt_use),ST_pump_rate*Conv_MGD_to_Ac_ft) else if System_State=7 then min(South_Tacoma_aquifer,max(0,Max_Seattle_Need-Seattle_Tolt_use),ST_pump_rate*Conv_MGD_to_Ac_ft) else 0

Tacoma_Demand = (Tacoma_Sure)+(S_Tacoma_wells)

Tacoma_Sure = Gravity_wells+Green_River_Supply

Tacoma_Water_right = min(Primary_water_right*Conv_cfs_to_Ac_ft,Green_Inflow)

tolt_1 = SF_Tolt_Historic_2+SF_Tolt_Historic

Tolt_Climate_Change = 1

DOCUMENT: The Tolt Climate Change converter allows the user to increase or decrease the inflows on the Tolt system by a specified amount. All Tolt river historic flows will be multiplied by the value that is input here. A value of 1 will use the historic streamflow data provided by the Seattle Water Department.

Tolt_Expected_Spill = max(0,min(Tolt_Temp_Storage-Tolt_Max,11,Tolt_supply_cap))

Tolt_Fish_Need = if SF_Tolt_Inflow >= (Tolt_Fish_Req_norm*Conv_cfs_to_Ac_ft)*Instream_Requirement_Factor then (Tolt_Fish_Req_norm*Conv_cfs_to_Ac_ft)*Instream_Requirement_Factor else (Tolt_Fish_Req_crit*Conv_cfs_to_Ac_ft)*Instream_Requirement_Factor

Tolt_Forecast =
DELAY(Tolt_water,2)+DELAY(Tolt_water,1)+DELAY(Tolt_water,3)

Tolt_Max = Tolt_Rule*Max_storage_factor

Tolt_Pipe_Capacity = 110

DOCUMENT: This is the capacity of the Tolt Supply pipeline in MGD (currently about 110 MGD). Alternatively this figure could represent the Seattle Water Department water right for the South Fork Tolt River Basin (currently 150 MGD).

Tolt_Primary = min(Seattle_Tolt_use,max(0,Seattle_Demand_final-Cedar_Expected_Spill))

Tolt_supply_cap = Tolt_Pipe_Capacity*Conv_MGD_to_Ac_ft

Tolt_Target = if Cedar_primary+Tolt_Primary >= .999*Seattle_Demand_final then Tolt_Primary else max(0,min(Seattle_Demand_final,Tolt_Temp_Storage+(Tolt_Max+Cedar_Usable_Volume-Tolt_Temp_Storage-Cedar_Temp_Storage+Seattle_Demand_final)*(Tolt_Forecast/(Tolt_Forecast+Cedar_Forecast))-Tolt_Max))

Tolt_Temp_Storage = SF_Tolt_Inflow+SF_Tolt_Reservoir-Tolt_Instream

Tolt_water = max(0.01,(tolt_1*Tolt_Climate_Change)-(Tolt_Fish_Req_norm*Instream_Requirement_Factor))

Total_MI_release = Tacoma_Distribution+Cedar_Release+Tolt_Release

To_crib_top = 19442

Water_Year = 1929+int(((time)-10)/12)

Cedar_1_history = GRAPH(time)

(7.00, 444), (8.00, 183), (9.00, 153), (10.0, 114), (11.0, 85.0), (12.0, 519), (13.0, 459), (14.0, 867), (15.0, 683), (16.0, 74.0), (17.0, 20.0), (18.0, 18.0), (19.0, 56.0), (20.0, 41.0), (21.0, 437), (22.0, 221), (23.0, 881), (24.0, 514), (25.0, 565), (26.0, 394), (27.0, 337), (28.0, 61.0), (29.0, 20.0), (30.0, 43.0), (31.0, 225), (32.0, 269), (33.0, 225), (34.0, 491), (35.0, 419), (36.0, 671), (37.0, 657), (38.0, 447), (39.0, 301), (40.0, 77.0), (41.0, 45.0), (42.0, 144), (43.0, 370), (44.0, 539), (45.0, 352), (46.0, 634), (47.0, 706), (48.0, 1298), (49.0, 954), (50.0, 802), (51.0, 702), (52.0, 233), (53.0, 15.0), (54.0, 123), (55.0, 306), (56.0, 1698), (57.0, 826), (58.0, 1025), (59.0, 229) ...

Cedar_1_Hist_2 = GRAPH(time)

(503, 0.00), (504, 614), (505, 724), (506, 554), (507, 125), (508, 55.0), (509, 138), (510, 266), (511, 562), (512, 409), (513, 950), (514, 1000), (515, 371), (516, 498), (517, 1204), (518, 901), (519, 580), (520, 120), (521, 124), (522, 297), (523, 797), (524, 599), (525, 737), (526, 1269), (527, 1385), (528, 641), (529, 1178), (530, 868), (531, 426), (532, 97.0), (533, 252), (534, 153), (535, 344), (536, 1083), (537, 557), (538, 224), (539, 331), (540, 399), (541, 546), (542, 352), (543, 126), (544, 49.0), (545, 60.0), (546, 167), (547, 475), (548, 770), (549, 1203), (550, 523), (551, 563), (552, 816), (553, 969), (554, 1501), (555, 617) ...

Cedar_2_History = GRAPH(time)

(7.00, 195), (8.00, 121), (9.00, 166), (10.0, 201), (11.0, 193), (12.0, 388), (13.0, 380), (14.0, 360), (15.0, 313), (16.0, 243), (17.0, 96.0), (18.0, 121), (19.0, 92.0), (20.0, 87.0), (21.0, 213), (22.0, 198), (23.0, 367), (24.0, 303), (25.0, 296), (26.0, 211), (27.0, 207), (28.0, 152), (29.0, 157), (30.0, 129), (31.0, 137), (32.0, 154), (33.0, 150), (34.0, 261), (35.0, 249), (36.0, 241), (37.0, 336), (38.0, 188), (39.0, 178), (40.0, 127), (41.0, 165), (42.0, 159), (43.0, 186), (44.0, 245), (45.0, 230), (46.0, 414), (47.0, 439), (48.0, 693), (49.0, 453), (50.0, 216), (51.0, 233), (52.0, 60.0), (53.0, 164), (54.0, 158), (55.0, 167), (56.0, 530), (57.0, 121), (58.0, 613), (59.0, 28.0) ...

Cedar_2_Hist_2 = GRAPH(time)

(503, 0.00), (504, 365), (505, 273), (506, 183), (507, 136), (508, 105), (509, 114), (510, 127), (511, 176), (512, 281), (513, 540), (514, 487), (515, 220), (516, 248), (517, 282), (518, 250), (519, 184), (520, 126), (521, 204), (522, 176), (523, 313), (524, 373), (525, 525), (526, 659), (527, 774), (528, 315), (529, 273), (530, 233), (531, 199), (532, 133), (533, 155), (534, 135), (535, 163), (536, 364), (537, 348), (538, 207), (539, 219), (540, 186), (541, 183), (542, 162), (543, 130), (544, 106), (545, 121), (546, 99.0), (547, 221), (548, 386), (549, 521), (550, 493), (551, 486), (552, 372), (553, 344), (554, 328), (555, 163) ...

Cedar_3_History = GRAPH(time)

(7.00, 88.0), (8.00, 64.0), (9.00, 84.0), (10.0, 99.0), (11.0, 102), (12.0, 281), (13.0, 132), (14.0, 172), (15.0, 127), (16.0, 110), (17.0, 37.0), (18.0, 52.0), (19.0, 59.0), (20.0, 53.0), (21.0, 180), (22.0, 114), (23.0, 282), (24.0, 159), (25.0, 199), (26.0, 92.0), (27.0, 84.0), (28.0, 58.0), (29.0, 43.0), (30.0, 50.0), (31.0, 164), (32.0, 118), (33.0, 120), (34.0, 218), (35.0, 173), (36.0, 230), (37.0, 218), (38.0, 96.0), (39.0, 91.0), (40.0, 59.0), (41.0, 46.0), (42.0, 107), (43.0, 175), (44.0, 180), (45.0, 170), (46.0, 193), (47.0, 230), (48.0, 318), (49.0, 323), (50.0, 132), (51.0, 127), (52.0, 95.0), (53.0, 58.0), (54.0, 63.0), (55.0, 215), (56.0, 384), (57.0, 256), (58.0, 260), (59.0, 101) ...

Cedar_3_Hist_2 = GRAPH(time)

(503, 0.00), (504, 213), (505, 136), (506, 75.0), (507, 62.0), (508, 52.0), (509, 65.0), (510, 74.0), (511, 125), (512, 317), (513, 341), (514, 236), (515, 280), (516, 198), (517, 132), (518, 118), (519, 93.0), (520, 57.0), (521, 77.0), (522, 111), (523, 182), (524, 262), (525, 310), (526, 355), (527, 338), (528, 221), (529, 133), (530, 123), (531, 106), (532, 65.0), (533, 77.0), (534, 62.0), (535, 97.0), (536, 287), (537, 224), (538, 141), (539, 148), (540, 119), (541, 99.0), (542, 92.0), (543, 64.0), (544, 47.0), (545, 54.0), (546, 72.0), (547, 201), (548, 322), (549, 319), (550, 287), (551, 273), (552, 201), (553, 152), (554, 124), (555, 99.0) ...

Cedar_4_History = GRAPH(time)

(7.00, 201), (8.00, 207), (9.00, 363), (10.0, 508), (11.0, 531), (12.0, 1318), (13.0, 464), (14.0, 434), (15.0, 170), (16.0, 73.0), (17.0, 13.0), (18.0, 89.0), (19.0, 136), (20.0, 170), (21.0, 776), (22.0, 595), (23.0, 1456), (24.0, 744), (25.0, 695), (26.0, 229), (27.0, 112), (28.0, 39.0), (29.0, 15.0), (30.0, 86.0), (31.0, 375), (32.0, 377), (33.0, 521), (34.0, 1129), (35.0, 891), (36.0, 1077), (37.0, 765), (38.0, 242), (39.0, 121), (40.0, 40.0), (41.0, 16.0), (42.0, 187), (43.0, 401), (44.0, 578), (45.0, 734), (46.0, 1008), (47.0, 1190), (48.0, 1488), (49.0, 1130), (50.0, 333), (51.0, 170), (52.0, 63.0), (53.0, 20.0), (54.0, 109), (55.0, 492), (56.0, 1232), (57.0, 1097), (58.0, 1353), (59.0, 522) ...

Cedar_4_Hist_2 = GRAPH(time)

(503, 0.00), (504, 635), (505, 188), (506, 9.00), (507, -3.00), (508, 12.0), (509, 96.0), (510, 269), (511, 470), (512, 1476), (513, 1778), (514, 1184), (515, 1619), (516, 713), (517, 232), (518, 73.0), (519, 121), (520, -52.0), (521, 142), (522, 142), (523, 749), (524, 1170), (525, 1365), (526, 1542), (527, 2540), (528, 258), (529, 282), (530, 245), (531, 108), (532, 75.0), (533, 260), (534, 134), (535, 372), (536, 2078), (537, 1650), (538, 631), (539, 593), (540, 299), (541, 227), (542, 218), (543, -13.0), (544, 49.0), (545, 115), (546, 209), (547, 1127), (548, 2660), (549, 2240), (550, 1893), (551, 1832), (552, 997), (553, 669), (554, 387), (555, 183) ...

Cedar_Rule = GRAPH(Calendar_Month+1)

(1.00, 62770), (2.00, 62770), (3.00, 62770), (4.00, 69756), (5.00, 77022), (6.00, 84565), (7.00, 84565), (8.00, 84565), (9.00, 84565), (10.0, 73354), (11.0, 62770), (12.0, 62770)

DOCUMENT: This node represents the maximum level of the Cedar Lake and Masonry Pool storage (adapted from WRMM model). By changing the value in the graphical function the user can modify the within year variations on the maximum volume of water to be held in storage. Currently, a flood pocket is established beginning in September, and the maximum amount of water is held in storage during the winter months.

Data_1913_to_1955 = GRAPH(time)

(-185, 630), (-184, 1120), (-183, 970), (-182, 1563), (-181, 1078), (-180, 1923), (-179, 1970), (-178, 1660), (-177, 700), (-176, 333), (-175, 150), (-174, 291), (-173, 357), (-172, 2048), (-171, 508), (-170, 446), (-169, 590), (-168, 885), (-167, 2145), (-166, 845), (-165, 292), (-164, 238), (-163, 163), (-162, 114), (-161, 270), (-160, 836), (-159, 1100), (-158, 502), (-157, 785), (-156, 2914), (-155, 2088), (-154, 1245), (-153, 1050), (-152, 1058), (-151, 288), (-150, 237), (-149, 178), (-148, 1066), (-147, 320), (-146, 670), (-145, 1946), (-144, 480), (-143, 990), (-142, 1760), (-141, 2100), (-140, 1300), (-139, 414), (-138, 182), (-137, 158), (-136, 188), (-135, 2380), (-134, 6028), (-133, 1570) ...

Data_1955_to_1990 = GRAPH(time)

(312, 0.00), (313, 1583), (314, 1649), (315, 3073), (316, 1399), (317, 493), (318, 230), (319, 841), (320, 2678), (321, 2791), (322, 1025), (323, 570), (324, 801), (325, 1842), (326, 2363), (327, 1908), (328, 809), (329, 232), (330, 170), (331, 222), (332, 997), (333, 3512), (334, 829), (335, 517), (336, 1488), (337, 2100), (338, 2012), (339, 580), (340, 309), (341, 210), (342, 153), (343, 146), (344, 524), (345, 1296), (346, 938), (347, 1448), (348, 799), (349, 801), (350, 1258), (351, 466), (352, 222), (353, 131), (354, 122), (355, 887), (356, 3327), (357, 3164), (358, 1762), (359, 1081), (360, 1182), (361, 2180), (362, 1838), (363, 1183), (364, 522) ...

Days_in_month = GRAPH(Calendar_Month)

(1.00, 31.0), (2.00, 28.0), (3.00, 31.0), (4.00, 30.0), (5.00, 31.0), (6.00, 30.0), (7.00, 31.0), (8.00, 31.0), (9.00, 30.0), (10.0, 31.0), (11.0, 30.0), (12.0, 31.0)

Elevation_from_Volume = GRAPH(Cedar_Reservoir)

(0.00, 15000), (2500, 15100), (5000, 15100), (7500, 15100), (10000, 15100), (12500, 15100), (15000, 15100), (17500, 15100), (20000, 15100), (22500, 15100), (25000, 15100), (27500, 15100), (30000, 15100), (32500, 15100), (35000, 15100), (37500, 15100), (40000, 15100), (42500, 15100), (45000, 15100), (47500, 15100), (50000, 15100), (52500, 15100), (55000, 15100), (57500, 15448), (60000, 15460), (62500, 15478), (65000, 15493), (67500, 15507), (70000, 15521), (72500, 15535), (75000, 15549), (77500, 15563), (80000, 15576), (82500, 15589), (85000, 15602), (87500, 15615), (90000, 15628), (92500, 15640), (95000, 15652), (97500, 15664), (100000, 15676), (102500, 15688), (105000, 15699), (107500, 15711), (110000, 15722)

fish_critical = GRAPH(Calendar_Month)

(1.00, 15372), (2.00, 13885), (3.00, 15372), (4.00, 14876), (5.00, 15372), (6.00, 15471), (7.00, 6764), (8.00, 6764), (9.00, 6546), (10.0, 7994), (11.0, 14876), (12.0, 15372), (13.0, 15372)

fish_extreme = GRAPH(Calendar_Month)

(1.00, 13527), (2.00, 12218), (3.00, 13527), (4.00, 13091), (5.00, 13527), (6.00, 8926), (7.00, 4612), (8.00, 4612), (9.00, 4463), (10.0, 7071), (11.0, 13091), (12.0, 13527), (13.0, 13527)

fish_norm = GRAPH(Calendar_Month)

(1.00, 22751), (2.00, 20549), (3.00, 22751), (4.00, 22017), (5.00, 22751), (6.00, 20232), (7.00, 16602), (8.00, 9039), (9.00, 12615), (10.0, 17524), (11.0, 22017), (12.0, 22751), (13.0, 22751)

fish_sub_norm = GRAPH(Calendar_Month)

(1.00, 19676), (2.00, 17772), (3.00, 19676), (4.00, 19042), (5.00, 19676), (6.00, 17852), (7.00, 10146), (8.00, 7994), (9.00, 8628), (10.0, 11375), (11.0, 18447), (12.0, 19676), (13.0, 19676)

flow_critical = GRAPH(Calendar_Month)

(1.00, 6641), (2.00, 4665), (3.00, 4919), (4.00, 6903), (5.00, 10453), (6.00, 11306), (7.00, 9223), (8.00, 3874), (9.00, 1845), (10.0, 2214), (11.0, 2618), (12.0, 4735), (13.0, 4735)

flow_extreme = GRAPH(Calendar_Month)

(1.00, 6641), (2.00, 4665), (3.00, 4919), (4.00, 6903), (5.00, 9592), (6.00, 10175), (7.00, 5411), (8.00, 2029), (9.00, 1403), (10.0, 1722), (11.0, 2142), (12.0, 3997), (13.0, 3997)

flow_sub_norm = GRAPH(Calendar_Month)

(1.00, 8362), (2.00, 5498), (3.00, 5964), (4.00, 9521), (5.00, 13835), (6.00, 14400), (7.00, 10515), (8.00, 5595), (9.00, 2856), (10.0, 2705), (11.0, 3689), (12.0, 6641), (13.0, 6641)

HH_rule_volume = GRAPH(Calendar_Month+1)

(1.00, 1588), (2.00, 1588), (3.00, 1588), (4.00, 25649), (5.00, 25649), (6.00, 25649), (7.00, 24887), (8.00, 24000), (9.00, 18362), (10.0, 14000), (11.0, 9000), (12.0, 2500)

Lake_Max_in_feet = GRAPH(Calendar_Month+1)

(1.00, 20.0), (2.00, 20.0), (3.00, 20.7), (4.00, 21.3), (5.00, 22.0), (6.00, 22.0), (7.00, 22.0), (8.00, 22.0), (9.00, 21.5), (10.0, 21.0), (11.0, 20.5), (12.0, 20.0)

Lock_Needs_cfs = GRAPH(Calendar_Month)

(1.00, 67.0), (2.00, 79.0), (3.00, 79.0), (4.00, 95.0), (5.00, 103), (6.00, 117), (7.00, 121), (8.00, 123), (9.00, 124), (10.0, 114), (11.0, 99.0), (12.0, 82.0), (13.0, 82.0)

MPEL = GRAPH(Cedar_Reservoir-Cedar_Dead+Masonry_pool_min)

(0.00, 15000), (271, 15069), (543, 15125), (814, 15173), (1086, 15214), (1357, 15248), (1629, 15278), (1900, 15304), (2171, 15329), (2443, 15352), (2714, 15374), (2986, 15394), (3257, 15414), (3529, 15434), (3800, 15454)

NF_Tolt_Historic = GRAPH(time)

(7.00, 350), (8.00, 269), (9.00, 185), (10.0, 138), (11.0, 85.0), (12.0, 479), (13.0, 371), (14.0, 525), (15.0, 451), (16.0, 193), (17.0, 106), (18.0, 63.0), (19.0, 88.0), (20.0, 97.0), (21.0, 292), (22.0, 257), (23.0, 563), (24.0, 377), (25.0, 306), (26.0, 261), (27.0, 271), (28.0, 100), (29.0, 58.0), (30.0, 77.0), (31.0, 234), (32.0, 130), (33.0, 222), (34.0, 463), (35.0, 289), (36.0, 526), (37.0, 353), (38.0, 265), (39.0, 293), (40.0, 113), (41.0, 58.0), (42.0, 150), (43.0, 246), (44.0, 265), (45.0, 337), (46.0, 426), (47.0, 453), (48.0, 778), (49.0, 608), (50.0, 532), (51.0, 490), (52.0, 277), (53.0, 133), (54.0, 103), (55.0, 255), (56.0, 657), (57.0, 672), (58.0, 541), (59.0, 189) ...

NF_Tolt_Historic_2 = GRAPH(time)

(503, 0.00), (504, 349), (505, 288), (506, 206), (507, 123), (508, 69.0), (509, 217), (510, 278), (511, 357), (512, 407), (513, 760), (514, 543), (515, 372), (516, 356), (517, 571), (518, 468), (519, 332), (520, 122), (521, 152), (522, 231), (523, 508), (524, 431), (525, 519), (526, 686), (527, 869), (528, 499), (529, 625), (530, 461), (531, 358), (532, 127), (533, 194), (534, 112), (535, 219), (536, 577), (537, 400), (538, 185), (539, 239), (540, 246), (541, 295), (542, 265), (543, 120), (544, 69.0), (545, 110), (546, 315), (547, 403), (548, 581), (549, 760), (550, 381), (551, 499), (552, 461), (553, 516), (554, 598), (555, 342) ...

res_all = GRAPH(Calendar_Month)

(1.00, 15500), (2.00, 16000), (3.00, 17000), (4.00, 17500), (5.00, 20000), (6.00, 20000), (7.00, 20000), (8.00, 20000), (9.00, 20000), (10.0, 15700), (11.0, 14000), (12.0, 10000), (13.0, 10000)

DOCUMENT: This converter contains information about critical through sub-normal reservoir levels for the current month for purposes of switching from normal instream flow requirements to some other regime.

SF_Tolt_Historic = GRAPH(time)

(7.00, 244), (8.00, 118), (9.00, 99.0), (10.0, 41.0), (11.0, 37.0), (12.0, 185), (13.0, 127), (14.0, 294), (15.0, 194), (16.0, 73.0), (17.0, 58.0), (18.0, 30.0), (19.0, 39.0), (20.0, 85.0), (21.0, 190), (22.0, 88.0), (23.0, 347), (24.0, 164), (25.0, 117), (26.0, 159), (27.0, 97.0), (28.0, 38.0), (29.0, 17.0), (30.0, 40.0), (31.0, 99.0), (32.0, 91.0), (33.0, 114), (34.0, 236), (35.0, 121), (36.0, 270), (37.0, 225), (38.0, 128), (39.0, 161), (40.0, 59.0), (41.0, 32.0), (42.0, 89.0), (43.0, 136), (44.0, 218), (45.0, 168), (46.0, 199), (47.0, 202), (48.0, 462), (49.0, 358), (50.0, 296), (51.0, 265), (52.0, 117), (53.0, 37.0), (54.0, 49.0), (55.0, 143), (56.0, 411), (57.0, 347), (58.0, 324), (59.0, 70.0) ...

SF_Tolt_Historic_2 = GRAPH(time)

(503, 0.00), (504, 203), (505, 154), (506, 117), (507, 61.0), (508, 34.0), (509, 123), (510, 103), (511, 202), (512, 191), (513, 434), (514, 299), (515, 181), (516, 170), (517, 336), (518, 248), (519, 185), (520, 45.0), (521, 75.0), (522, 140), (523, 295), (524, 224), (525, 293), (526, 388), (527, 461), (528, 270), (529, 334), (530, 236), (531, 182), (532, 36.0), (533, 123), (534, 53.0), (535, 131), (536, 376), (537, 205), (538, 82.0), (539, 128), (540, 142), (541, 163), (542, 148), (543, 58.0), (544, 18.0), (545, 68.0), (546, 196), (547, 242), (548, 319), (549, 442), (550, 165), (551, 279), (552, 256), (553, 320), (554, 354), (555, 189) ...

ST_pump_rate = GRAPH(Calendar_Month)

(1.00, 48.3), (2.00, 48.3), (3.00, 48.3), (4.00, 48.3), (5.00, 48.3), (6.00, 48.3), (7.00, 47.0), (8.00, 45.8), (9.00, 44.5), (10.0, 43.3), (11.0, 45.8), (12.0, 48.3)

DOCUMENT: This converter allows the user to input the maximum allowable pumping rate from the South Tacoma well field which can be utilized to meet demands in the event of a potential shortfall. The rate should be input in sustainable MGD. The graph shows that the sustainable MGD is a function of the time of year. More water can be withdrawn during winter months than during the summer and early fall when drawdown and low recharge limit withdrawals.

System_Variation = GRAPH(Calendar_Month)

(1.00, 0.85), (2.00, 0.85), (3.00, 0.85), (4.00, 0.85), (5.00, 1.00), (6.00, 1.20), (7.00, 1.45), (8.00, 1.40), (9.00, 1.00), (10.0, 0.85), (11.0, 0.85), (12.0, 0.85), (13.0, 0.85)

DOCUMENT: The within year variation converter contains information about the ratio of monthly demands to the average annual demand. This allows the model to more accurately simulate the Seattle demand pattern (ie high summer demands and low winter demands).

Tolt_Fish_Req_crit = GRAPH(Calendar_Month)

(1.00, 30.0), (2.00, 36.0), (3.00, 36.0), (4.00, 36.0), (5.00, 36.0), (6.00, 36.0), (7.00, 30.0), (8.00, 30.0), (9.00, 30.0), (10.0, 30.0), (11.0, 30.0), (12.0, 30.0), (13.0, 30.0)

Tolt_Fish_Req_norm = GRAPH(Calendar_Month)

(1.00, 45.0), (2.00, 50.0), (3.00, 53.0), (4.00, 53.0), (5.00, 53.0), (6.00, 53.0), (7.00, 53.0), (8.00, 53.0), (9.00, 53.0), (10.0, 53.0), (11.0, 45.0), (12.0, 45.0), (13.0, 45.0)

Tolt_Main_History = GRAPH(time)

(7.00, 97.0), (8.00, 43.0), (9.00, 66.0), (10.0, 63.0), (11.0, 45.0), (12.0, 223), (13.0, 87.0), (14.0, 78.0), (15.0, 128), (16.0, 44.0), (17.0, 28.0), (18.0, 9.00), (19.0, 43.0), (20.0, 97.0), (21.0, 134), (22.0, 54.0), (23.0, 369), (24.0, 32.0), (25.0, 174), (26.0, 99.0), (27.0, 37.0), (28.0, 13.0), (29.0, 6.00), (30.0, 6.00), (31.0, 137), (32.0, 143), (33.0, 93.0), (34.0, 83.0), (35.0, 111), (36.0, 39.0), (37.0, 106), (38.0, 31.0), (39.0, 41.0), (40.0, 20.0), (41.0, 4.00), (42.0, 60.0), (43.0, 96.0), (44.0, 137), (45.0, 88.0), (46.0, 141), (47.0, 317), (48.0, 51.0), (49.0, 158), (50.0, 137), (51.0, 6.00), (52.0, 26.0), (53.0, 21.0), (54.0, 30.0), (55.0, 62.0), (56.0, 358), (57.0, 414), (58.0, 280), (59.0, 141) ...

Tolt_Main_History_2 = GRAPH(time)

(503, 0.00), (504, 169), (505, 109), (506, 61.0), (507, 29.0), (508, 16.0), (509, 57.0), (510, 130), (511, 95.0), (512, 185), (513, 372), (514, 397), (515, 168), (516, 137), (517, 126), (518, 104), (519, 72.0), (520, 21.0), (521, 39.0), (522, 60.0), (523, 123), (524, 128), (525, 125), (526, 414), (527, 259), (528, 175), (529, 163), (530, 124), (531, 98.0), (532, 22.0), (533, 80.0), (534, 44.0), (535, 39.0), (536, 263), (537, 151), (538, 90.0), (539, 61.0), (540, 73.0), (541, 55.0), (542, 46.0), (543, 21.0), (544, 5.00), (545, 15.0), (546, 61.0), (547, 273), (548, 252), (549, 284), (550, 285), (551, 198), (552, 189), (553, 160), (554, 135), (555, 64.0) ...

Tolt_Rule = GRAPH(Calendar_Month+1)

(1.00, 45000), (2.00, 45000), (3.00, 45000), (4.00, 49000), (5.00, 53000), (6.00, 53000), (7.00, 53000), (8.00, 53000), (9.00, 49000), (10.0, 45000), (11.0, 45000), (12.0, 45000)

DOCUMENT: This converter represents the current Tolt reservoir rule curve as provided by the Seattle Water Department. By changing the value in the graphical function the user can modify the within year variations on the maximum volume of water to be held in storage. Currently, a flood pocket is established beginning in September, and the maximum amount of water is held in storage during the winter months.