EVALUATION OF
WATER DISTRIBUTION SYSTEM MONITORING
USING STOCHASTIC DYNAMIC MODELING

by

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(ABSTRACT)

A stochastic dynamic constituent transport model was developed, capable of simulating the operation of a water distribution system containing pumps and storage tanks, and subject to random demands and contaminant inputs. Long term operation of a hypothetical small town water supply system containing one pump station and one storage tank was simulated while the system was subjected to external contaminant inputs. Repeated simulations were made under different regimes of external contamination applied to the tank, the pump station and at system nodes, and internal contamination representing biofilm effects based on assumed relationships between flow velocities and biofilm cell detachment.

Seven sampling plans representing regulatory requirements and industry practice were applied during the simulation to evaluate their ability to detect the contamination under a presence/absence criterion.

The simulations were able to identify contamination patterns and provide information useful in the definition of sampling plans. Time of sampling was found to be as important as location. This was true both within the monitoring period, and particularly within the diurnal cycle of demand. Spreading samples over
different days within the monitoring period rather than sampling all on one day, always improved contaminant detection. Detection by plans based on fixed times and locations were very sensitive to those times and locations.

There was no best plan suitable for all situations tested. The better sampling plans were those that captured the temporal and spatial contamination patterns present in the system. No consistent advantage was noted from sampling in proportion to population served or in locating sampling nodes systematically instead of randomly. The location and timing of sampling for most plans could be improved with the knowledge of actual contamination patterns and timing provided by the model.

The presence of a storage tank was found to have a strong influence on hydraulic patterns and the location and timing of contamination reaching different parts of the system.
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INTRODUCTION

Background

Beginning in 1914, standards for drinking water covering chemical, physical and bacteriological parameters have been progressively introduced (Montgomery, 1985). Additional standards are added periodically as new pollutants and their associated risks become known and as reliable detection techniques are developed. Compliance with the standards in many cases requires sampling of the distribution system at regulated locations and intervals.

The National Primary Drinking Water Regulations (NPDWR) drawn up by the Environmental Protection Agency (EPA) under the Safe Drinking Water Act (SWDA) (EPA, 1989a, b) require each public water system to monitor for bacteria of the total coliform group. Each public water system must take a regulated minimum number of samples per month according to population served. Samples must be taken in accordance with a written sampling plan which is subject to State review and revision, and the State must establish a process which ensures the adequacy of the sampling plan for each system. Despite the considerable amount of practical experience that has been accumulated since regulatory sampling was introduced, there is comparatively little formal guidance as to how such a sampling plan should be formulated, or objective research evaluating the numerous alternative plans that could be devised within the requirements of the regulations.

Small communities generally have limited resources to pay for sampling, and may sample only at the minimum regulated level. Sampling is an expensive and labor intensive activity, and it is therefore desirable that whatever sampling is
performed is as effective as possible. The NPDWR may require notification to the State authority in the event of a contaminant level above regulated levels, or failure to sample, possible resulting in 'boil water' notices or undue public concern which may be counter productive. Sampling is not solely directed at detection of contaminants. It may be used for operational purposes such as to confirm maintenance of a disinfection residual, or to trace the path of water from different sources through the system using an indicator constituent such as fluoride. The benefits of more effective water distribution sampling therefore go beyond considerations of disease control.

The movement and concentration of constituents is obviously tied closely to movement and mixing of water in the system. Dynamic, or extended period simulation models are able to provide good estimates of varying hydraulic conditions in municipal systems, and can be tied to constituent transport models (Grayman and Clark, 1990). Appropriate growth or die-off kinetics can be applied to the constituent concentration to represent chemical or physical reactions or bacterial growth and die-off. The further addition of a sampling module to such a model makes possible the simulated operation of a water distribution system subject to contamination events, and the simulated application of a sampling plan and an evaluation of its effectiveness.

The use of a computer model is indicated because an attempt to compare sampling plans using field data must contend with a number of disadvantages. Contamination levels which the sampling plans are intended to detect occur sporadically and with long periods of good water quality between them so that it
may be several years before enough useful data are collected. Additionally, a very large number of samples is required to cover even a small selection of possible sampling plans at a level high enough to demonstrate a significant advantage of one plan over another. A computer modeling approach allows long term simulations of system operation and the application of numerous different sampling plans, providing a rapid indication of potential improvements without the expense of field work. A record of simulated concentrations in all parts of the system is also available so that unsampled areas of contamination can be identified and included in reruns. A re-evaluation of missed opportunities is not possible in the field.

Objectives

The overall objective of this research was to assess the relative effectiveness of different sampling plans for detecting contaminants in the water distribution systems of small communities. The results will be applicable to the definition and justification of the written sampling plans which are required under the SDWA. The research was directed primarily towards regulated microbiological contaminants, specifically total coliform group bacteria, although the results should be applicable to a wide range of microbial, chemical and physical constituents of regulatory or general operational interest.

The research was applied to a hypothetical small distribution system with characteristics typical of real systems, serving a community of 5000 people and required to take six samples per month. The results are expected to offer general guidelines applicable to similar distribution systems and a procedure for the evaluation of other types and sizes of systems.
The major steps to be taken to meet this objective were:

1) development of a non-conservative constituent transport model for the distribution system,
2) determination from the literature of reasonable input data describing the physical and hydraulic characteristics of small distribution systems, the factors that affect flow variations, the location, frequency and severity of contamination events and dynamics of contaminant persistence,
3) identification of typical sampling plans used by water distribution system operators and the rationale used to define them,
4) use of the Monte Carlo method to generate random input values from the typical data mentioned above, and random sampling times and locations for some sampling plans,
5) simulation of distribution system operation over a long time period with contemporaneous application of the sampling plans to be evaluated,
6) statistical comparison of the different plans based on a measure of effectiveness,
7) determination of the sensitivities of the effectiveness parameter to changes in selected input variables.

Two sets of simulations were made. The first used a constituent representing external contamination by unspecified reactive compounds, but with parameters based upon typical coliform contamination events. The second used a constituent representing the internal release of bacteria from biofilms and sediments.
LITERATURE REVIEW

DISTRIBUTION NETWORK CONSTITUENT TRANSPORT MODELS

Introduction

A constituent transport model is made up of two major parts:-

a. The determination of flows as a function of the physical layout and characteristics of the pipework system and time variable forcing functions.

b. The determination of constituent transport by these flows, with due regard to flow dependant effects such as mixing and dilution.

A number of approximations and assumptions are inherent in the modeling of constituent transport in hydraulic systems (Fowler and Jones (1991). These include the lumping of demands at nodes, complete mixing at nodes and tanks, and the acceptability of limited skeletonization and of equations describing flow and head loss relationships in pipes.

Hydraulic models

Hydraulic Network Solution

The prerequisite for a constituent transport model is the solution of the hydraulic network equations to determine flows and other relevant hydraulic characteristics. Solution techniques for these equations require very large amounts of repetitive calculation for their solution. Several approaches were developed in the late 1960s and early 1970s using digital computers to apply standard matrix procedures to solve these equations, with those described by Martin and Peters
(1963), Shamir and Howard (1968), Epp and Fowler (1970) and Wood and Charles (1972) being among the best known.

**Network Equations**

A network can be fully described by a set of simultaneous equations based upon accepted hydraulic relationships between flow and pressure loss in pipes flowing full, such as the Hazen-Williams and D'Arcy Weisbach equations (Jeppson, 1977). The most fundamentally sound method for computing head losses in closed pipes over a wide flow range is the D'Arcy Weisbach equation (Jeppson, 1977), although the Hazen-Williams formula is traditionally favored by engineers for the calculation of head losses in water distributions systems. Holub (1988), basing his conclusions on results of an uncertainty study, recommended use of the Hazen-Williams equation, and also pointed out its traditional use in supply water engineering and the availability of more extensive information on Hazen-Williams coefficients. The Hazen-Williams equation is also computationally simpler since the D'Arcy Weisbach equation requires additional calculation to update friction factors which are themselves a function of flow (Jeppson, 1977; Walski *et al.*, 1990).

Epp and Fowler (1970), Jepson (1977), and Wood and Rayes (1981) reviewed the derivation of equations used in the solution of pipe networks. The following account is drawn from their reviews.

The analysis of flow in pipe networks is based on continuity and energy principles. Respectively, these require that flow into any point in the network must equal flow out of that point, and that energy losses round a closed loop of the network must be zero. A pipe network typically consists of P pipes joined at N
nodes. For such a network, N continuity equations can be derived as a linear combination of the outflows at that node equated to zero. Thus:

\[ \Sigma (\text{flows out of node}) = 0 \]

Inflows are conventionally expressed as negative outflows, and demands and source flows are included as in- or outflows as appropriate.

A looped network containing L non-overlapping loops, provides a further L loop head loss equations, one for each loop, of the form:

\[ \Sigma (\text{head losses around loop}) = 0 \]

The head losses, measured with regard to direction of flow, are those in the pipes forming the loop. Head loss is measured in a constant direction, conventionally clockwise around the loop. A non-overlapping loop is one that does not contain another loop within it.

Equations based upon the above approach can be devised to account for additional unknowns such as flows into and out of tanks and reservoirs and in branches not forming parts of loops, pressure changes across pumps and control devices, and the effects of other commonly occurring pipe network features. All the above equations rely on hydraulic relationships that establish head loss as a function of flow rate, fluid properties, pipe size and the condition of the pipe wall (Davis and Jeppson, 1979). By substitution among the equations, either heads or flows can be eliminated providing a solution for the remaining parameter.

Three separate systems of equations, derived from the continuity and loop head loss equations and defining flows and head losses in a pipe network system
have been identified. They are variously named by different authors and are therefore redefined here:

1. The $Q$ equations. These are a combination of the continuity and loop head loss equations written in terms of flow. The flow rates in each pipe are the unknowns.

2. The $H$ equations. These consist of the continuity equations written in terms of differences in pressure between nodes at the ends of each pipe. The pressures or hydraulic grade line levels at each node are the unknowns.

3. The $\Delta Q$ equations. These consist of the loop head loss equations written in terms of pipe flows. The corrective flow rates around independent loops of the network required to reduce each loop head loss to zero are the unknowns.

**Equation Solution Methods**

Except for the continuity equations, the hydraulic expressions derived from the above principles and describing the pressure and flow in pipe networks are non-linear algebraic equations which cannot be solved directly (Wood and Rayes, 1981). Two basic approaches to their solution are available. Both are relaxation methods in which an initial guess is improved iteratively.

The first is the application of Newton's method (or the Newton-Raphson method), in which a reasonably good initial estimate of the solution is required. A variation of this method is the Hardy-Cross method in which the simultaneous equations are solved one at a time (Jeppson, 1977). Newton's method has been used to solve the $H$ equations (Martin and Peters, 1963; Shamir and Howard, 1968).
and the $\Delta Q$ equations (Epp and Fowler, 1970). A number of modifications to the basic Newton's method are described by Lam and Wolla (1972), Rao and Bree (1977), Lemieux (1972) and Chuang et al. (1987).

The second is linearization, followed by application of a simultaneous linear equation solution technique. Suitable methods include Gaussian elimination and Lower/Upper decomposition (Press et al., 1986). This approach has been applied to the $Q$ equations (Wood and Charles, 1972) and to the $H$ equations (Shamir and Howard, 1968). However, Wood and Rayes (1981) report a number of problems encountered in the solution of the linearized $H$ equations. Jeppson (1977) suggests that linearization could be applied to both the $\Delta Q$ and the $H$ equations, but advises against it without stating any reason. Nevertheless, Walski et al. (1990) describes a linearization of the $H$ equations, and does not admit to any problems.

**Comparison of Solution Methods**

A review of algorithms for pipe network analysis was presented by Wood and Rayes (1981). The Hardy Cross method is inefficient for computerized solution which is capable of solving a complete system of equations simultaneously, and suffers from a number of convergence problems (Martin and Peters, 1963; Epp and Fowler, 1970; Jeppson, 1977; Wood and Rayes, 1981).

Simultaneous solution of the equation set offers the prospect of much faster convergence to a solution (Epp and Fowler, 1970). They considered using Newton's method for simultaneous solution of both the $H$ equations and the $\Delta Q$ equations. They considered the $H$ equations to be easy to set up since loops do not have to be defined, initial flows can be chosen arbitrarily, and the coefficient matrix can be
made symmetric which almost halves the amount of computer storage required. However, they noted a number of practical problems that lead to slow convergence. Wood and Rayes (1981) cited the work of various researchers who reported convergence problems in solving the H equations. However, Chun (1985) and Chuang (1987) subsequently used a modified Newton's method to solve the H equations.

Alternatively, Epp and Fowler (1970) solved the ΔQ equations using Newton’s method, which requires definition of the loops and an initial estimate of flows in each pipe such that continuity conditions are satisfied. The coefficient matrix can be made not only symmetric but banded and positive definite, requiring less computer storage and allowing for very efficient computer code, such as the Cholesky decomposition, in its solution. To take advantage of the reduced memory requirement, Epp and Fowler (1970) proposed algorithms by which the computer would estimate initial flows, construct loops from initial data and number them so as to obtain a minimum bandwidth to the matrix of coefficients. For small systems, the ΔQ equations are fewer in number than the H equations, allowing a smaller coefficient matrix, but this advantage disappears for large systems (Walski et al., 1990).

Wood and Charles (1972) used linearization of the loop head loss equations in their solution of the Q equations. A particular advantage claimed for a solution using the linearized Q equations is that no prior calculations are necessary to obtain an approximate set of initial flows. They obtained starting values by assuming an initial flow equal to unity (without regard to the units being used) and without any
attempt to define flow in the most likely direction along the pipe. They also stated that a single iteration of the linear theory method requires substantially more time to run than an iteration using one of the other methods, but this difference is made up by the rapid convergence of the method. Walski et al. (1990) linearized the H equations, and initialized the solution with initial flows of unity as did Lemieux (1972), thereby avoiding a need for initial flow estimation.

All these methods rely on a convergence criterion to indicate attainment of a sufficiently accurate solution. Wood and Rayes (1981) point out that a criterion for convergence based on achieving a sufficiently small change in the solution does not guarantee that the solution is correct since convergence to the wrong solution will satisfy such a criterion. This appears to have been the source of significant problems in solving the H equations, particularly if some pipes have very low flows. Over-relaxation methods for improving convergence rates have been used (Jeppson, 1977; Walski et al., 1990)

The coefficient matrices of the equations to be solved are square and of size \( n \times n \), where \( n \) is the number of equations. The amount of computer storage required is potentially equal to the number of coefficients, and solution time is roughly proportional to the cube of the matrix size (Press et al., 1986). However, all the sets of equations are sparse, and there is considerable potential for economy (Epp and Fowler, 1970; Rao and Bree, 1977).

Despite the apparent advantages or disadvantages of various approaches to network solution, all of them, apart from the Hardy-Cross method have found a permanent place in established commercial models (Epp and Fowler, 1970; Wood
and Charles, 1972; Liou and Kroon, 1987; Walski et al., 1990). Clearly, acceptable algorithms have been devised which overcome the various disadvantages and make use of the particular advantages of each method.

**Non-Steady State Hydraulic Modeling**

A hydraulic solution provides an instantaneous value for flows and pressures. In real systems, boundary conditions are constantly changing, most of them as continuous variables. However, the changes are usually sufficiently gradual that changes may be considered constant over specified time intervals so that an approximation to the ideal situation is achieved through the solution of a sequence of steady state conditions. New boundary conditions for the next steady state may be derived from the previous steady state (Grayman et al., 1988). This approach has been variously termed quasi-dynamic modeling, (Males et al., 1988) and extended period simulation (Rao and Bree, 1977). The latter describe this approach as 'the development of an integration scheme to link several static solutions, each representing an interval of time, to changes in water levels in the tanks, in pumping schedules, and in demand patterns'. The length of each time interval is dependant upon the amount of variation in the boundary conditions and the degree of accuracy required of the result. Typically, time intervals of one hour are used (Rao and Bree, 1977).

**Constituent Transport Models**

**Introduction**

A waterborne constituent can represent many water quality parameters such as the concentration of a chemical, a measure of its physical condition such as
temperature, or an attribute such as cost of production or age (Clark and Males, 1985). Three processes which affect the transport of a constituent through a network are advection, reaction (decay or growth), and mixing (Liou and Kroon, 1987); these authors consider longitudinal diffusion to be small relative to bulk advection and therefore neglect it. A distribution pipe network has characteristics predominantly similar to a plug flow reactor with occasional flow reversals (Characklis, 1988).

The Mixing Equation

Two basic assumptions, those of complete mixing and mass conservation at a node, are used to formulate the mixing, or mass balance equation at each node (Males et al., 1985). This equation provides an expression for the unknown concentration leaving the node as a linear combination of the known flows and concentrations of each stream entering the node. Studies on a two source system showed the complete mixing assumption to be reasonable (Metzger, 1985). A distribution system containing N nodes provides N linear mixing equations in the unknown concentrations which can be solved simultaneously by standard methods (Males et al., 1985). The coefficient matrix for these equations is sparse, and can therefore take advantage of sparse solution methods.

Steady State Constituent Transport Modeling

Algorithms employing the mixing equation for confluent flows can be overlaid on a steady state hydraulic model of a water distribution system to produce a steady state constituent transport model. These algorithms do not require time-based tracking of a contaminant through the system, but rely on the assumption
of equilibrium conditions to calculate concentrations. The 'Solver' algorithm described by Males et al. (1985), applies the above simultaneous solution of node concentrations. It was used to model the cost of water delivered to various parts of the distribution system of the Village of New Vienna in Ohio, and to the town of Tampa, Florida, to model the percentage of water from two different sources of different quality reaching various parts of the distribution system. Another approach, termed a 'marching-out' solution (Grayman and Clark, 1990) was described by Clark et al. (1988), in which, given a steady state hydraulic solution, the nodes are ordered to enable sequential calculation of node concentrations using previously calculated concentrations at upstream nodes. A similar approach in which the pipes are ordered is described by Liou and Kroon (1987).

The 'marching out' algorithm was used to investigate trihalomethane and hardness spatial variation in the North Penn Water Distribution System (Males et al., 1988). In this simulation a twenty four hour period was represented by three sets of supply/demand conditions, one from 6:00 am to 12:00 noon, one from 12:00 noon to 12:00 midnight, and one from 12:00 midnight to 6:00 am. The results were compared to the results of sampling at strategic points within the distribution system at four hour intervals over a period of thirty two hours, and were considered to be useful as an initial indication of trends in constituent transport. The use of separate simulations subject to sequential sets of boundary conditions was termed 'sequential steady state modeling' by these authors. They pointed out that the sequential steady state method does not account for the reservoir of constituent remaining in the pipe network from the previous steady state solution. In other
words, it is necessary to ensure that the time interval between sequential steady states is sufficient for the entire pipe system to be flushed with new water before the end of the steady state period in order for calculated constituent concentrations to be valid.

**Dynamic Constituent Transport Modeling**

An algorithm which tracks the progress of a constituent over short time-steps without requiring establishment of equilibrium conditions was described by Grayman et al. (1988). It can therefore be linked to an extended-period hydraulic model which provides new hydraulic conditions at much shorter time periods. This algorithm divides each steady state time period of the extended period simulation into an integer number of computational time steps of length $\Delta t$, which is kept constant throughout the simulation. For each time period, each link is divided into an integer number of sublinks by evenly spaced subnodes such that travel time between subnodes is approximately equal to $\Delta t$. Constituent concentrations at nodes and subnodes are calculated iteratively at successive time steps and used to track constituent transport. When steady state conditions change at the beginning of the next time period, travel time, and hence the distance between sub-nodes, may vary. The new in-pipe starting conditions are derived from the previous ending conditions, using linear interpolation if the number of subnodes has changed. Subsequent constituent transport then depends upon the new steady state conditions. Concentrations of chloroform, hardness and trihalomethane in the North Penn distribution system were modeled using this approach with close
agreement between predicted and observed results, but also some anomalous behavior (Grayman et al., 1988).

Another algorithm is described by Liou and Kroon (1987) in which each pipe is divided into volume elements, one for each constituent concentration in the pipe. Volume elements may be of variable length reflecting the length of pipe over which concentration remains constant, and providing a continuous concentration profile. As the simulation proceeds, mixing at nodes produces increasing numbers of different concentrations, and hence volume elements. The volume of an element formed by new water entering a pipe is matched by deleting an equal volume of water in elements leaving the downstream end of that pipe. The tracking of elements is performed over variable time steps corresponding to the lengths of the various elements. A new time step begins whenever a change in concentration due to mixing at a node occurs, such as when the front end of a new volume element with a different concentration first arrives at a node, or when flows in the network change due to a change in externally applied conditions resulting in mixing of the incoming concentrations in different proportions. At this time, a new volume element with the new concentration will be created in each pipe leaving the node. A consolidation of elements, selected so as to cause the least possible loss of accuracy, is performed whenever the number of elements in a pipe exceeds a predefined limit.

In both the algorithms previously described, representation of constituent growth or decay is proposed by the application of an exponential reaction process to the concentration at each time step.
The relative merits these algorithms are discussed by Kroon and Liou (1990) and Grayman et al. (1990). An advantage of the first approach is the fixed time interval, $\Delta t$, which allows the user some control over the resolution of the solution, and prevents excessive computation times which might occur if $\Delta t$ were defined within the computational procedure. Some approximation and loss of accuracy is accepted to achieve integer numbers of evenly spaced subnodes. Also, the representation of a continuous concentration profile by point concentrations at nodes and subnodes gives no indication of concentrations at intermediate points other than by inference or interpolation. When flow rates change and the spacing of sub-nodes in a pipe changes, interpolation may not provide an accurate description of the new concentrations. Advantages of the second approach are the accurate representation of mass in each element and the fact that a change in flow rate does not change this representation. Also, a change in external conditions such as demands, or concentrations in inflows, can be allowed at any time, not just at fixed time periods. However, the length of each computational time step is generally controlled by the shortest time it takes the downstream element in any pipe to enter a junction. Large systems with many pipes may contain large numbers of elements, requiring frequent consolidation with attendant loss of resolution and accuracy, and slow execution. Availability of computer storage to store information at a high resolution and extended run-times may be limiting factors for large systems (Ferriera, 1991).
Data Requirements

Basic Requirements

Input data in the form of pipe sizes, demands, supply capacity and so on are required as input to the flow model. For real systems this information may be derived from a consultant study, examination of records and plans, and survey information, while metering and house counts can provide information on demands (Clark and Males, 1985). Additional elective data for particular purposes may also be used.

One of the more difficult inputs to obtain is roughness data for pipes. Holub (1988) discusses the determination and uncertainty of this parameter at length. Pipe diameters shown in records and on drawings are often nominal and rarely is any attempt made to correct them to actual internal diameters. He considers this practice to be defensible since determinations of roughness parameters made using field measurements of flows and pressure losses generally use nominal diameters, and the same nominal diameters would then be appropriate for modeling.

Connectivity

Pipe networks are usually defined as a system of links representing pipes, and nodes representing points where pipes join or at the location of a specific feature such as a valve or at points of demand (Clark and Males, 1986).

Water Demand

Water demand data have three components; quantity, time, and location. These cannot usually be considered separately. Temporal variations in consumption rates occur over hourly, daily, weekly and seasonal cycles, and from place to place
(Steel and McGhee, 1979). A curve showing hourly consumption variations for a limited area of a city may show a characteristic shape with fluctuations greater for smaller communities and over shorter time periods. Flow chart records of total system and subsystem demands, pumping rates, tank levels and pressures are routinely maintained by many water distributors. The American Water Works Association (AWWA) (1989) describes methods for developing demand curves for a specific distribution area from measured data.

Typical data on system daily average, maximum day and per capita demand are readily available for many real water systems (AWWA, 1981) on an annual basis and example diurnal system demand curves at hourly resolution are available in many standard texts. Shawcross (1985) gives factors to calculate actual flow from mean flow on an hourly basis for twelve zones of the distribution system in Austin, Texas. Typical maximum hourly consumption is normally in the range of from 2 to 7 times average hourly demand on an average day (Montgomery, 1985) or 1.5 times average hourly demand on a particular day (Steel and McGhee, 1979). Maximum day demand is normally from 1.5 to 3.5 times average day demand (Montgomery, 1985). Minimum consumption rates will vary from 25 to 50% of the daily average (Steel and McGhee, 1979) or 0.2-0.6 times peak day hourly demand (AWWA, 1989).

Zone or nodal demands are necessarily site specific; in the absence of site data, typical nodal or zonal flows can be derived from system flows by application of a factor based on area, number of consumers or suchlike. For hydraulic modeling, overall system demand is usually assumed to occur at nodes (Clark and
Males, 1986) and must be realistically allocated to individual nodes. Several approaches are available (Casario, 1980; Walski, 1983; Shawcross, 1985; Hirrel, 1986), and may be used according to the availability of data and accuracy required. Large users may often be assigned to a specific node. Smaller users and unaccounted for demands may variously be apportioned according to the proximity of individual consumers to a node, the area around a node or the pipe lengths attached to a node. Hirrel (1986) used the weighted rolling average of consumption for each (metered) customer account and the location of each consumption record on a geographic grid to assign demands to the appropriate node.

Monitoring devices and estimates of consumption at nodes provide information on conditions at discrete points within a distribution system. System demand predictions may be accurate, but estimating individual nodal demands is more difficult. Bargelia and Hainsworth (1989) note that limitations of cost and practicability may result in there being insufficient measurements to define the state of the system, and therefore predictions, referred to as pseudo-measurements, of consumptions, nodal pressures or other data, are made. They also note that input measurements are usually considered deterministic in nature, but are in fact subject to various errors.

Unaccounted for water is the difference between total demand and metered consumption (AWWA, 1989), and is made up of leakage and losses from mains breaks, hydrant flushing, fire fighting demands, system maintenance and operation, unauthorized use and unmetered, or inaccurately metered, consumption. Unaccounted for water can amount to from 4 to 30 per cent of total system demand,
with 10 to 15 percent a typical figure. If apportioned throughout the system, unaccounted for water will create a demand at every node.

**Skeletonization**

A hydraulic layout may be simplified to reduce it to a manageable size and complexity, it is common practice to include in the model only those considered necessary to obtain an adequate representation of the complete system, a process known as skeletonization. In grid-like systems, flows and pressures calculated from skeletons containing pipe substitutions with as few as 20% of the actual pipes compared well with results using all pipes (Metzger, 1985). In modelling the Cheshire system of the South Central Connecticut Water Authority (SCCWA), Clark *et al*. (1991) skeletonized the system to include less than 10% of system pipes and nodes, but found that fluoride tracer studies showed good agreement between the modeled tracer concentrations and field measurements. Smaller diameter pipelines and those perpendicular to the general flow direction can be deleted, although this is less acceptable near large system inflows and outflows (Walski, 1983). Localized flows and pressures may be calculated using a localized model of all pipes with boundary conditions derived from the skeleton (Metzger, 1985).

Hunt (1988) found that the degree of system skeletonization had little effect on system hydraulic pressures, but that predictive modeling of chlorine residuals subject to a first order decay rate was significantly affected, with predicted residuals decreasing with less skeletonization. Less skeletonization apparently increased model hydraulic travel time, with resulting increased time for decay to take place. The effect of skeletonization on modeled residual at a particular point in a system
was also found by Hunt to be specific to the location of that point and the degree of skeletonization proximate to it. Hunt (1988) also noted that flow velocities were reduced when more pipes were included in the model, and when demand loads were more evenly distributed over the increased number of junctions. Thus any effect dependant upon flow, such as biofilm erosion and reinoculation of the bulk water, may require a less skeletonized system for acceptable accuracy.

The use of one equivalent pipe to replace two or more parallel pipes is frequently suggested (Jeppson, 1977). This is acceptable when solving for flows and head losses only, but is less acceptable when tracking water constituents since no single pipe can represent the travel times of two pipes with different travel times.

After a continuous stream of constituent is introduced at a point in a system, the effect of mixing and dilution of water reaching downstream points by parallel paths with different travel times can result in a gradual build up of concentration over time. Skeletonization can either increase or decrease the arrival of both intermediate and final equilibrium concentrations (Ferriera et al., 1991). This is the result of the conflicting effects of the removal of short cut routes to downstream points and increased velocities through a smaller number of available pipes. The effect becomes more marked with increased levels of skeletonization.

**System Reliability**

Established reliability engineering techniques may be applied to the estimation of mean time between events (MTBE), mean time to failure (MTTF) and mean time to repair (MTTR) for piping and pumping components and are discussed by several authors in the context of water distribution reliability (Shamir and
Howard, 1981; Mays, 1985). The MTTF is the expected value of the time to failure as defined by the probability density function of the reliability. The MTTR is similarly defined by its probability function, and the mean time between failures, MTBF, is the sum of MTTF and MTTR and also equals the mean time between repairs (Mays 1985).

The time to failure of a component is a random variable and may be assumed to follow a gamma distribution with mean (mean time to failure) \( \mu \) and shape parameter \( k \) (Lewis and Orav, 1989). If \( k = 1 \), this reduces to an exponential distribution which is often assumed to be the case (Cullinane, 1985).

Shamir and Howard (1981) quote Damelin et al. who found the time between failures, \( T \), (not including repair time) to be exponentially distributed. They define the probability distribution function by:

\[
 f_T(t) = \lambda e^{(-\lambda t)} \quad t \geq 0
\]

where \( \lambda = 1/\text{MTBF} \). They also quote Damelin et al. who found the repair duration to be log-normally distributed according to

\[
 f_M(m) = \frac{1}{\sqrt{2\pi \sigma_M}} e^{\frac{(M-m)}{2\sigma_M}}
\]

for \( m \geq 0 \), where \( m = \log (\text{repair duration, D}) \). \( M \) and \( \sigma_M \) are the mean and standard deviation respectively of the logarithms of the repair durations.

For water mains, regression equations have been developed expressing break rate in terms of the age and material of the main, surrounding soil type and other pertinent characteristics (Goodrich et al., 1985). If the MTTF (that is, the
reciprocal of the failure rate) is known, the time to the next failure can be simulated using a Monte Carlo method, and thus their effects incorporated in a time dependant model. This approach can be used to simulate the MTBE for contamination events which result from various system failures. Cullinane (1985) and Shamir and Howard (1981) list typical values of MTBF and MTTR for pumps and associated equipment.

**Stochastic Simulation and Monte Carlo Methods**

Monte Carlo methods may be generally applied to mathematical procedures that employ some element of chance (Cheney and Kincaid, 1985) and can be applied to the sampling of random variates from a probability distribution. Where stochastic factors control at least part of a system and are defined by some known probability distribution, inputs may be sampled from those distributions using the Monte Carlo method. For a water distribution system, a set of input values can therefore be constructed using a combination of the fixed input variables, and probable values sampled from the variables whose distribution of feasible values is known. Repeated sampling from a probability distribution using a Monte Carlo procedure is expected to generate a synthetic set of values which conform to the original distribution.

Monte Carlo simulations are often used to evaluate the response of a natural system to anticipated changes or proposed modes of operation (Loucks *et al.*, 1981; Cullinane, 1985; Preston *et al.*, 1989). For example, synthetic data series which represented typical water quality distributions were used by Valiela and Whitfield
(1989) to compare different monitoring strategies for meeting water quality objectives.
DISTRIBUTION SYSTEM CONTAMINATION

Contamination Events

Contamination can be introduced from an external source into a distribution system in three distinct areas of the system; the source where water first enters the network, the network system itself, and storage locations attached to the network.

Contamination entering from the source is often the result of inadequate treatment of some form, resulting from a breakdown in, or contamination of, the treatment plant itself (Wierenga, 1985; Burn, 1989) or inability to cope with unusually poor source water quality (Short, 1988; Stukel et al., 1990), or a combination of such events (Hopkins et al., 1986). Characklis (1988) cites penetration of the disinfection process as a source of bacterial inoculation. On occasions, water may by-pass the disinfection process (Wierenga, 1985). Contamination events described in such cases tend to be of low concentration, but long duration, that is from several hours to several weeks.

Events which introduce contamination into the distribution system can occur through backsiphon and cross-connection events in which an external source of contaminated water enters the system (AWWA, 1979; Craun, 1981; Brazos et al., 1985; USEPA, 1989), and through leaks and air valves during periods of negative pipeline pressures (AWWA, 1979; Wierenga, 1985). They can involve introduction of contaminated water from sumps and flooded manholes, backflow from tankers carrying unclean water or more dangerous substances such as heating fuels and antifreeze. These are likely to be high concentration events of short duration lasting from a few seconds to a few hours.
Contamination may enter distribution system storage tanks and tanks from many sources (Geldreich, 1900). These include atmospheric dust carried in through ventilation cowls, surface water or roof drainage entering through cracks and structural faults, ingress of animals such as birds or rodents, and deliberate acts of vandalism. Tank liner materials, joint sealants and wood construction material may harbor bacteria or provide organics that support growth (Geldreich, 1900; Seidler et al., 1977).

Bacterial Contamination

Waterborne Disease Reporting and Indicators.

The EPA and Center for Disease Control publish reported data and investigate disease outbreaks. In the period 1946 to 1974, 68 disease outbreaks in the United States were associated with distribution system deficiencies other than storage (AWWA, 1979), of which twenty were caused by contaminated surface or groundwater entering the system. Between 1981 and 1988, 248 waterborne disease outbreaks occurred nationally involving about 1950 cases of illness, of which 79% were associated with community and non-community systems (Craig, 1990). The remainder were associated with individual systems and recreational water sources. Failure to report disease symptoms to physicians, and a lack of communication between responsible agencies have contributed to under-reporting of waterborne disease outbreaks, and were cited (EPA, 1989b) as justification for more stringent total coliform rules.

The EPA considers the density of the coliform indicator group in water to be correlated with the presence of disease agents which may cause diarrhoea,
cramps, nausea, possibly jaundice, and any associated headaches and fatigue (EPA, 1989b). Many coliforms are of intestinal origin, but many are also found living freely in the environment, so that their presence indicates the possibility, but not the certainty, of fecal pollution.

The NPDWR require public notification of coliform violations. A public notice prepared by the EPA for issue in the event of a coliform violation (EPA, 1989b) states:

'Total coliforms are common in the environment and are generally not harmful themselves. The presence of these bacteria in drinking water, however, generally is a result of a problem with water treatment or the pipes which distribute the water, and indicates that the water may be contaminated with organisms that can cause disease.'

Definition of Terms

The Final Rule of the National Primary Drinking Water Regulations under the Safe Drinking Water Act (EPA, 1989b) which took effect in 1990 refers to 'total coliforms' as a related group of bacteria which includes 'fecal coliforms and Escherichia coli'. (The EPA allows testing for the presence of fecal coliforms, or E. coli in lieu of fecal coliforms, and routinely refers to them as quoted above, although E.coli are fecal coliforms). Coliforms are defined by the American Public Health Association (APHA) (1989) as a group of bacteria sharing the same specific metabolic and morphological characteristics. The definition is operational, not taxonomic (Brock and Madigan, 1988), so that the coliform group includes various organisms from different taxonomic groups.

Fecal coliforms are a sub-group of coliforms specifically adapted to survival in the gut of warm bloodied animals (Gaudy and Gaudy, 1980) and are characterized
by their specific metabolic response to defined test culture conditions (APHA, 1989). Their presence is much stronger evidence of pollution by fecal material from human or other warm blooded animals. A reference to coliforms in the context of water quality includes all bacteria in the coliform group and is not limited to fecal coliforms. The definitions of coliforms and fecal coliforms do not include any reference to the source of coliforms or their capacity to cause disease.

Characklis (1988) defined two primary mechanisms, breakthrough and growth, which result in the appearance of bacteria in distribution systems. He defined breakthrough as "the increase in bacterial numbers in the distribution system resulting from viable bacteria passing through the disinfection process", and growth as "the increase in viable bacterial numbers in the distribution system resulting from bacterial growth downstream of the disinfection process". He distinguished these mechanisms by defining breakthrough as a transport process which moves existing cells into the system, whereas growth is a conversion of substrate into new cells.

Characklis defined an episode as "an occurrence of "excessive" viable bacteria in the distribution system", with bacterial numbers being in excess when they exceed the drinking water standards. He pointed out that breakthrough and growth are both causes of coliform occurrence in the distribution system, but do not necessarily result in an episode.

Bacteria occur in the distribution system in association with biofilms and as planktonic cells. Characklis refers to biofilms as "microbial cells immobilized at the pipe surface or on a particle". He defined planktonic cells as "viable bacterial cells"
suspended in the water phase'. Transfer of cells from the biofilm to the water phase is achieved primarily through erosion, defined as 'the release of microscopic segments of biofilm into the bulk water', and sloughing, defined as 'the detachment of macroscopic patches of biofilm'; transfer of cells from the water to the biofilm occurs by adsorption of planktonic cells to the biofilm.

Coliforms initially enter a distribution system by breakthrough or some other contamination event. Analysis of historical data for a large treatment and distribution system indicated that either growth or breakthrough may result in coliform episodes (Goodman and Cunnane, 1988). Immobilization occurs within the matrix of extracellular polymers, in tubercles and sediments, and by combinations of these and unknown processes. Bacteria also attach to the surfaces of invertebrates that inhabit distribution systems (Levy et al., 1986).

Biofilm Growth

Characklis (1988) identifies biofilm growth as:

'the net result of at least three processes which include:

1. Transport (to) and adsorption of cells at the pipe wall,
2. Cell reproduction and product formation,
3. Detachment of biofilm components from the pipe wall by erosion or sloughing.'

After initial transport and adsorption of cells to the substrate, increased biofilm thickness occurs primarily by growth (van der Wende and Characklis, 1988). Bacteria in distribution systems are predominantly heterotrophic using organic carbon as the primary energy source, with biofilm growth being supported by the flux of nutrients transported by pipe flows, and organic carbon being the principal
substrate (Characklis, 1988). Total organic carbon (TOC) concentrations in water distribution systems of about 0.3-5.4 milligrams per litre (mg/L) (McFeters and Camper, 1988) have been recorded. Assimilable organic carbon (AOC) was found to be about 4-20% of TOC, with resultant AOC of about 0.05 to 1 mg/L. While concentration of substrate may be low, the flux, or loading rate of substrate may be sufficient to support a considerable growth of bacteria; in pilot plant tests higher flow rates (but at constant wall shear) tended to increase biofilm growth rates (van der Wende and Characklis, 1988). The assimilable portion of total organic carbon may be increased by ozonation (Geldreich et al., 1988). Bacteria growing within treatment filters may remove organic carbon from filtrate, while chlorine in water applied to filters or in backwash water can reduce biological activity in the filter, and hence carbon removals, allowing passage of larger concentrations of TOC into the distribution system and greater biofilm growth (Van der Kooij, 1985; van der Wende and Characklis, 1988). LeChevalier (1990) showed that removal of assimilable carbon 'occurred very quickly, within a short distance from the treatment plant', reflecting the increased nutrient flux originating there and the higher bacterial growth that resulted.

Low hydraulic flow rates suggested limitation of an unidentified growth factor for pilot plant biofilm heterotrophs (van der Wende and Characklis, 1988). Higher flow rates removed this limitation, and with unchlorinated feed water, greater biofilm growth occurred near the inlet to the system where loading rates would be highest. The nature of this growth factor was not discussed. McFeters and Camper (1988) suggested that the limiting nutrient may not be assimilable
carbon and also noted that distribution system bacterial isolates were capable of
growth in distilled water supplemented with mineral salts.

Pilot plant tests in the absence of residual chlorine suggested biofilm growth
rates of about 0.14 to 0.05 day\(^{-1}\) (van der Wende and Characklis, 1988) with total
counts of bacteria recovered on heterotrophic plate counts of up to 7.8\times10^{10} colony
forming units (cfu) per square metre (m\(^2\)).

**Planktonic Cell Growth**

In pilot tests, planktonic cells, including coliforms, generally showed little
growth compared to biofilms and could die-off, even in the absence of a chlorine
residual (van der Wende and Characklis, 1988), suggesting that no planktonic cells
would be found in the absence of a biofilm. However, large increases in total
bacterial populations and heterotrophic plate count (HPC) (Brazos and O'Connor,
1987) at growth rates of about 5 per day, and of coliforms (McFeters and Camper
1988) in dechlorinated water without supplemental organic carbon, often after a lag
period of 24 to 48 hours, have been reported. Brazos and O'Connor concluded that
bacterial populations in water samples (planktonic cells) were correlated with
concentrations in the treatment plant clear well, suggesting that 'treatment
effectiveness alone controls bacterial populations in the distribution system'. In this
case, breakthrough, not the biofilm, was the source of planktonic cells.

**Factors Affecting Growth**

**Combined Environmental Factors**

Goodman and Cunane (1988) applied several statistical analysis procedures
to one year of historical data of the South Central Connecticut Regional Water
Authority, considering pH, total and free chlorine, turbidity, temperature and true color in raw, treated and distributed water as possible predictors of Membrane Filter Total Coliform (MFTC) counts. Data used were weekly averages since simple regression on daily values had no useful predictive ability. In summarizing these statistical analyses, (Characklis, 1988) states the following conclusions:

'Sufficient chlorination is generally effective in suppressing distribution system coliforms. There are three "environmental" conditions which increase the risk of coliform outbreaks if chlorination is insufficient:

1. High pH generally decreases the disinfection effectiveness of chlorine.

2. High turbidity which may be an index to conditions of high surface runoff, associated perhaps with increased nutrient supply.

3. High coliform counts in the treated effluent which suggests coliform breakthrough possibly a result of coliform growth in the treatment plant or in sampling lines.'

The three environmental conditions identified by Goodman and Cunane were groups of variables identified by multiple regression on principal components. Their best predictive model which used these three components was only able to account for 32% of the variability in coliform numbers. Generally weaker or conflicting relationships were identified in the other procedures, all of which were considered unsatisfactory. It is unfortunate that flow was not included as a possible predictive variable.

Goshko et al. (1983) collected water samples from seven community water supplies and attempted to establish relationships between standard plate counts (SPC) and coliform occurrence, turbidity, free and total chlorine residual, and
temperature. Some significant correlations were found, but these were not found consistently, and were sometimes positive and sometimes negative. Goodman and Cunnane (1988) found a similar lack of correlation to apply to coliforms. Similarly inconclusive results were found by Lowther and Moser (1984) who were unable to correlate individual environmental variables with coliform densities during a prolonged coliform episode, attributed primarily to biofilm erosion.

Donlan and Pipes (1988) showed that the population of attached organisms on cast iron test surfaces in a distribution system was directly related to water temperature and suspended microbial population and indirectly to chloramine (total less free chlorine) concentration and maximum velocity.

It should be noted that predictive ability and correlation do not necessarily indicate cause and effect.

**Disinfectants**

Chlorine has routinely been used in the treatment and distribution of potable water as a disinfectant for the purpose of inactivating pathogenic bacteria, but not with the intention of sterilization which would affect all organisms (Gaudy and Gaudy, 1980). The efficiency of chlorine as a disinfectant is therefore measured primarily in terms of its ability to reduce numbers of pathogens, or specific indicators of pathogens such as coliforms. However, chlorine also affects the growth of non-coliform organisms.

Lippy (1986) cites statistics for waterborne disease outbreaks in the US for the period 1946 to 1980 that suggest that deficiencies in chlorine disinfection practices were the primary, or a major contributing cause of nearly half the
outbreaks and Characklis (1988) indicated that distribution system coliforms are generally suppressed by chlorination.

There is substantial evidence that planktonic coliforms numbers may not, under some circumstances, be reduced by chlorine. Reilly and Kippin (1983) in studying two distribution systems found that while standard plate counts were significantly reduced, coliform counts were not affected by a free chlorine residual of up to 1.0 mg/L. Goshko et al. (1983) found that coliforms can occur in drinking water at free chlorine concentrations above 0.2 mg/L, and found no relationship between coliform occurrence and free chlorine concentrations below 0.2 mg/L. These results are substantiated by Wierenga (1985), who discussed high coliform concentrations in Grand Rapids distribution systems when the free chlorine residual was between 0.6 and 1.0 mg/L throughout the distribution system. Brazos and O’Connor (1987) have demonstrated survival of bacteria for up to four days in chloramine concentrations exceeding 1.0 mg/L. Lowther and Moser (1984) reported the need to maintain at least 6.0 mg/L free residual chlorine to ensure coliform free samples from the Seymour, IN, distribution system. Similar reports can be found in the technical literature. Reilly and Kippin (1983) suggest that there may be a ‘threshold of effective disinfection’ which must be reached before the disinfection effect is significant, and in comparing two distribution systems, conclude that this threshold may vary from system to system.

Van der Wende and Characklis (1988) reported that the rate of biofilm accumulation is reduced by chlorine concentrations as low as 0.1 mg/L. However, chlorine concentrations up to 0.8 mg/L, while inhibiting biofilm growth and
reducing biofilm thickness, were unable to prevent it altogether. At the higher chlorine concentrations, heterotrophic bacterial growth occurred only in the protective biofilm environment, but ultimate planktonic cell numbers at these chlorine concentrations were unaffected.

Microbial diversity in the biofilm was reduced in the presence of chlorine (van der Wende and Characklis, 1988) which appeared to select for chlorine resistant organisms. Ridgway and Olson (1982) noted that the presence of either free or combined chlorine may select for chlorine resistant organisms which survived up to 10 mg/L of chlorine for two minutes, but that more sensitive species, including Klebsiella, are still readily killed at concentrations of 1.0 mg/L or less. From 75 to 90% of organisms in a pilot system containing 0.8 mg/L of chlorine residual were Pseudomonas, and these were found to be producing larger amounts of extracellular polymers (van der Wende and Characklis, 1988). The effect of chlorine on the biofilm is reduced by the high chlorine demand of the extracellular polymers and the reduced the rate of diffusion of chlorine into the biofilm. In the absence of chlorine, organisms were selected which probably had the fastest growth rates and which were acclimated to survival in the prevailing oligotrophic conditions.

Cells acclimated to slow growth in oligotrophic conditions, and which tend to be older, may be more resistant to disinfectants than younger rapidly growing cells typical of richer media, particularly at higher disinfectant concentrations (Gaudy and Gaudy, 1980; Brazos and O’Connor, 1987). The slow growth of cells in water depleted in TOC by its removal during filtration may therefore enhance
resistance to subsequent chlorination, allowing them to survive until absorbed into a biofilm where they can recover.

The bactericidal effect of chlorine is pH dependant (Gaudy and Gaudy, 1980). Hypochlorous acid is the more potent disinfectant so that lower pH values may be expected to enhance the disinfecting ability of chlorine. A pH removed from the value for optimum cell growth can also decrease a cell's viability and increase the effectiveness of a chemical disinfectant. Wierenga (1985) documents the addition of lime to increase pH levels in treated water to pH 8.3 with concurrent raising of free chlorine residuals to 4 mg/L. A reduction in coliform recoveries followed, although it is admitted that no cause and effect relationship with either the lime or chlorine concentration was established. Martin et al. (1986) concluded that in tuberculated pipe, the localized chlorine demand of the tubercle material rendered the chlorine less effective in controlling coliforms than an increase in pH by lime addition.

In modeling chlorine concentrations, Hunt (1988) assumed the decay or consumption rate of chlorine within the water phase of a distribution system to be first order with respect to chlorine concentration. Hunt characterizes other reactants as 'solute, particulates and the pipe walls'. Model calibration against measured chlorine residuals in one distribution system indicated a best value first order reaction rate of 0.00155 per minute. A conclusion of this calibration was that the chlorine demand of the pipe wall was greater than that of the water. Hunt admits that the rate expression for the reaction of chlorine with biofilms may not be first order. From initial concentrations of 0.2 and 0.8 mg/L chlorine,
approximately linear, but different, rates of removal were demonstrated in a pilot distribution system (van der Wende and Characklis, 1988). Characklis (1988) suggests that 'chlorine concentration will probably decrease faster as the water velocity decreases and water passes through smaller pipes'. Smaller pipes have a higher surface-area to volume ratio. Brazos and O'Connor (1987) stated that chlorine residuals in the Jefferson City distribution system reflected initial concentrations and were largely unaffected by the chlorine demand of pipe surfaces. Where chlorine depletion did occur, it was generally at extreme points in the pipe system.

Hunt (1988) notes that the chlorine decay rate is site specific within a distribution system, and recommends specification of decay rates on a pipe-by-pipe basis. Pipes with different diameters will have a different specific surface and hence effective biofilm 'concentration', and the nature of the biofilm may not be uniform due to the effects noted previously.

When modeling chlorine propagation in a distribution system, Hunt (1988) indicated that chlorine deficient regions were most probably caused by extremely low flow rates corresponding to long hydraulic residence times. Chlorine was consumed by prolonged contact with the biofilm, without replenishment from new water still containing a chlorine residual. Martin et al. (1982) report that Klebsiella Pneumoniae, a typical coliform found in water distribution systems, was gradually inactivated when suspended in distilled water with a free chlorine residual of 2.5 mg/L. The addition of ground pipe tubercle material prolonged survival of the bacteria and may have consumed the chlorine oxidative capacity, removing its toxic
effect. However, these authors apparently did not distinguish between survival of the original planktonic bacteria, and their replacement from the tubercle material which had not been sterilized. Attachment, bacterial encapsulation, biofilm age, and previous growth conditions were found to increase resistance to disinfection, and were multiplicative in their effect (LeChevallier et al., 1988). Alternatively, the bacteria may have adsorbed to the material which afforded protection from the disinfectant. Lowther and Moser (1984) recovered coliforms in water samples from a distribution system containing up to 6 mg/L chlorine residual. These same coliforms were rapidly killed in suspension by 1 mg/L residual chlorine, but the kill rate was significantly reduced when the same bacteria were grown as a biofilm on a glass slide. Ridgway and Olson (1982) concluded that cell clumps and bacteria attached to suspended particles in the water had an enhanced resistance to chlorine.

Other Factors

Pipe age. Nagy and Olson (1987) demonstrated an apparent logarithmic correlation between the age of a pipe and bacterial density on pipe walls. They alternatively suggested a correlation of age with pipe material.

Temporal patterns. Analysis of historical data revealed a 'day of the week' pattern of MFTC positive tests in distribution water, except in winter months (Goodman and Cunnane, 1988). There was no correspondence with raw and treated effluent numbers, so the higher numbers were probably a manifestation of growth rather than breakthrough. Since the effect was restricted to certain days of the week, an association with other seasonal effects such as temperature, which would
be expected to show a seasonal rather than weekly pattern, is confused. No reports of an attempt to correlate diurnal flow variations with coliforms was found. Generally, sample data are reported on a daily basis and do not allow resolution of variations with shorter frequencies.

**Temperature.** Statistical analysis of historical data showed temperature to be generally unrelated to MFTC weekly average positives (Goodman and Cunnane, 1988). The seasonal effect noted previously may be related to temperature because a warmer summer temperature could encourage higher growth in the biofilm, and higher summer flows could result in greater erosion of the biofilm. At higher temperatures, a faster chlorine reaction rate with established biofilm organic material in the upstream sections of pipe might quickly reduce chlorine residuals so that as waterborne coliforms are swept into downstream sections, the reduced residual is unable to control them. On the other hand, low river water temperatures were associated with decreased cell removals during treatment, attributed to the inability of chlorine to kill bacteria whose metabolic rate was reduced by the low temperature (Brazos and O'Connor 1987). These cells then entered the distribution system, increasing coliform counts.

**Turbidity.** Increased turbidity of raw, treated effluent and distribution water was found statistically to be a predictor of higher MFTC weekly average positives (Goodman and Cunnane, 1988), although simple regression on daily values had no predictive value. Turbidity may be caused by bacteria or particulate matter that harbors bacteria within it, and is a general indicator of poor water quality (Hunt, 1988), although turbidities of less than 0.2 nephelometric turbidity units (NTU)
should not raise concerns over bacteriological quality. High turbidity in raw water may be an indication that treatment processes will be overloaded, resulting in breakthrough. High turbidity in treated water may confirm that this has happened. Turbidity reductions during treatment may be much greater than concurrent coliform reductions (Brazos and O'Connor, 1987). The NPDWR (EPA, 1989b) require that surface water sources and groundwater sources under the influence of surface water sources collect a coliform sample near the first service connection in a distribution system if the source water turbidity exceeds 1 NTU.

Unidentified factors. Statistical analysis of coliform outbreaks in several distribution systems suggested the influence of some unidentified external factor that caused simultaneous coliform outbreaks in unconnected systems (Goodman and Cunnane, 1988). This finding applied to both surface and groundwater supplied systems. Simultaneous variations in demand were suggested as a possible explanation, which points to an association with flow rates.

Maul et al. (1986) partitioned a distribution system into four 'zones', each made up of sub-zones of similar bacterial density; the sub-zones of each zone were not contiguous. The bacterial densities within each zone apparently varied over time somewhat independently of the other zones. No other parameters were presented or considered in this context.

Lowther and Moser (1984) noted the generally 'infrequent', and 'sporadic' nature of coliform recoveries from distribution systems and the difficulty of associating an episode with its cause. LeChevallier (1990) reiterated this point in a comprehensive review of coliform regrowth in drinking water, and observed that
despite the abundant evidence confirming the existence of coliforms in distribution systems, there is little information about their occurrence at drinking water interfaces.

**Water-Biofilm Cell Transfers and Velocity Effects**

At steady state, biofilm growth is balanced by biofilm cell detachment and reinoculation of the water (Characklis, 1988). In pilot plant experiments, the major source of planktonic cells was continuous and uniform erosion of the biofilm surface (van der Wende and Characklis, 1988). A threshold velocity of from 1.7 to 2.1 m/s (5.6 to 7.0 ft/s) was identified below which biofilm detachment rate was constant, but above which it increased exponentially. An alternative conclusion was that significant additional detachment occurs at flow rates approximately twice the historical value. It was acknowledged that this threshold value may depend upon the past history of the biofilm and previous shear levels at which a balance between detachment and growth was established. Flow variations from 0.2 to 1.0 ft/s were found by McFeters and Camper (1988) to have little effect on the type and number of bacteria in the biofilm. Sloughing also occurred periodically as a result of sudden increases in flow velocity.

Sudden increases in chlorine concentration could cause complete biofilm detachment, and sloughing may also occur due to unexplained processes internal to the biofilm in the absence of chlorine (van der Wende and Characklis 1988). Erosion rates and corresponding planktonic cell concentrations were also increased by greater biofilm thickness.
Two years of intensive investigation of the source of coliforms in the Seymour, Indiana, distribution system considered over thirty environmental and operating variables, but found no correlation with any single measured environmental or operating condition, with one exception (Lowther and Moser, 1984). The exception was a positive correlation between coliform numbers and changes in pumping regimes caused by plant shut down and pump changes, suggesting that variations in flow velocity and direction were involved. This was combined with the concerted effect of several other variables which increased the corrosivity of the water, enabling erosion of the biofilm.

In modeling chlorine concentrations subject to assumed first order decay rates in a hydraulic system, Hunt (1988) found that of several variables including decay rate and initial dose, chlorine residuals were most sensitive to global flow rates. Any dependance of coliform concentration on chlorine residual therefore implies a correlation between coliform concentration and flow rates.

**Observed Coliform Densities**

Evidence for the variability of bacterial counts is found in the data of Pipes and Christian (1984) who enumerated three counts too numerous to count (TNTC), in their case defined as 'in excess of 80 per 100 mL'. Further dilution indicated bacterial counts of 1600, 330 and 2700 per 100 mL in a data set of 26 observations of which 16 had values of less than 10 per 100 mL. They presented tables of coliform sample concentrations together with their true mean and true variances. Ignoring one set of what appeared to be unusually clean samples, the mean counts generally ranged from about 0.1 to greater than 10, and the variances from about
0.5 to several hundred, with a few extreme values of both mean and variance. For simulation of typical contaminated water quality, a true mean of 1.0 and a variance of 20 are reasonable values. Hudson et al. (1983) report similar data in an apparent case of breakthrough where 13 out of 27 system wide samples had less than 4 coliforms per 100 mL, while the average exceeded 200 coliforms per 100 mL.

During coliform 'episodes' attributed to regrowth, 'general bacterial densities' were 'less than 10 colonies per mL', (which is equivalent to 1000 colonies per 100 mL) (Lowther and Moser, 1984). In another case, the peak mean coliform concentration was 12 coliforms per 100 mL (Characklis, 1988). Out of 17 sample coliform counts at the beginning of an episode (Wierenga, 1985), eleven had a count of zero coliforms per 100 mL, one each a count of 1 and 2, and six a count exceeding 4, of which three exceeded 80 coliforms per 100 mL.

Concentrations in contaminated source water and in water entering through cross-connections and leaks may also be highly variable. In two contaminated wells, one contained in excess of 4900 coliforms per 100 mL and another from 1 to 16 coliforms per 100 mL (Craun, 1981). Coliform bacteria were 'almost totally absent' from both the influents and effluents of four large open treated water reservoirs in California, Washington State and New Jersey (AWWA, 1983) but were present in concentrations of up to several hundred per 100 mL in water from contaminated redwood storage tanks (Nagy and Olson, 1985).
DISTRIBUTION SYSTEM SAMPLING

Total Coliform Regulations

Requirements

The National Primary Drinking Water Regulations (NPDWR) were amended by a Final Rule relating to total coliforms, which was promulgated in June 1989, (EPA, 1989b) and became effective on 31 December 1990.

The NPDWR set a non-enforceable Maximum Contaminant Level Goal (MCLG) of zero for total coliform bacteria. They also set an enforceable Maximum Contaminant Level (MCL), with compliance based on the presence/absence (P-A) test and a compliance period of one month. In summary, The Final Rule states:

• Compliance is based on presence/absence of total coliforms in sample, rather than on an estimate of coliform density.

• MCL for systems analyzing at least 40 samples/month: no more than 5.0 percent of the monthly samples may be total coliform positive.

• MCL for systems analyzing fewer than 40 samples/month: no more than 1 sample/month may be total coliform positive.

• A public water system must demonstrate compliance with the MCL for total coliforms each month it is required to monitor.'

One reason for acceptance of the P-A concept is the difficulty of calculating average coliform densities (EPA, 1989b), and the relative ease of positive detection as opposed to enumeration of bacteria.

Non-enforceable MCLGs for other microbiological contaminants were promulgated to take effect at the same time as the regulations covering total coliforms (EPA, 1989a). However, the enforceable standards for bacterial quality refer only to total coliforms and, after a positive total coliform test, to fecal
coliforms (or *E. coli* in lieu of fecal coliforms). From a regulatory point of view, therefore, only total coliforms, fecal coliforms or *E. coli* can be in excess. *Pseudomonas*, for example, cannot be in excess (Characklis, 1988).

**Compliance**

Compliance with the regulations is determined by testing samples taken from the distribution system. The NPDWR require public water systems serving 25 or more persons to be monitored monthly, and state the minimum number of routine samples to be taken each month depending on the population served. The number of samples ranges from 1 sample for systems with less than 1,000 consumers to 480 samples for systems serving over 3,960,000. At least three repeat samples must also be taken for every sample testing positive for total coliforms.

The regulations require public water systems to collect total coliform samples at sites which are representative of water throughout the distribution system, and in accordance with a written sampling plan which is subject to State review and revision. No greater importance is placed on samples from any particular part of the system except for repeat samples. References throughout the Regulations and introductory explanation are always to the system, and the intention is clearly to establish compliance of the system as a whole, and not individual parts of the system separately. Thus, any set of samples that does not comply with the regulations places the whole system in non-compliance, even if the offending samples are all located in one part.

The regulations state that a public water system 'must collect samples at regular time intervals throughout the month' (EPA, 1989b), except for some smaller
groundwater systems which may collect all required samples on a single day if they are taken from different sites. There is no specific statement defining the interval between sampling within the monitoring period. The 'Explanation of Final Provisions' published with, but not forming part of the regulations, says that in reviewing the sample siting plan States should 'determine whether the system should collect samples on a regular basis throughout the month, or whether it is acceptable to collect some or all required samples at the same time' (EPA, 1989b). Repeat samples are required after each positive result, and are not part of the initial sampling plan, but are counted in determining compliance. Repeat samples must be taken at service connections upstream and downstream of locations giving a positive result to determine whether the positive sample is the result of systemic contamination or is localized at the sampling point. Special purpose samples, such as those taken after pipe repair are not counted as part of the required samples.

Coliform Detection

Coliforms are detected in water samples using defined bacteriological tests (American Public Health Association, 1989). Depending on the test used and its interpretation, results may be used to enumerate bacteria per sample unit, or to indicate their presence as a P-A test. If the test provides an enumeration, a count of one or more bacteria is interpreted as a positive result in a P-A test.

Sampling Distribution Systems

Overall Considerations

For a water distribution system, sample space is limited to the lines corresponding to the water pipes, and may be further practically limited to specific
points on those lines, such as the locations of accessible faucets. Additionally, the coliform regulations limit sampling to detecting the presence of the contaminant of interest, so that parameter estimation of the contaminant concentration is not a consideration.

A number of sampling designs are described by Gilbert (1987). None of them precisely matches the above needs, but they provide an insight into suitable approaches and can be applied to sampling along lines. Some sampling approaches clearly require a prior knowledge of the population. The highly non-homogeneous nature of coliform contamination in both time and space precludes the use of a sampling plan that relies for efficiency or accuracy on homogeneity of the population. In this respect, a haphazard sampling plan is not acceptable.

**Sampling Designs**

**Judgement Sampling**

Gilbert defines judgement sampling as 'subjective selection of population units', that appear to be representative of the population, without the use of prior knowledge or expert opinion. Such an approach requires 'homogeneity and a completely assessable target population so that sample bias is not a problem'. Actual knowledge of even highly sampled and studied water distributions systems are not 'completely assessable', and are unlikely candidates for judgement sampling.

**Probability Sampling**

Several forms of probability sampling are described by Gilbert. Simple random sampling is the simplest approach, but contamination trends or patterns may be better detected using another probability sampling approach. Gilbert states
that Stratified random sampling may be 'useful when a heterogeneous population can be broken down into parts that are internally homogeneous'. The work of Maul et al. (1991), cited previously, has shown that a structured heterogeneity can sometimes be identified within a distribution system. However, this conclusion was reached only after a substantial field monitoring effort, and such effort is not practicable under normal circumstances, particularly for small water systems seeking to reduce monitoring costs.

Cluster Sampling

When members of a population cluster in naturally forming groups, such as schools of fish, cluster sampling may be appropriate. According to Gilbert, some clusters are randomly selected for sampling, and then every member of those clusters is measured. Stukel et al. (1987) applied a 'spatial cluster' sampling plan, in which the cluster consisted of 'five samples collected from different locations at one time per month'. The 'naturally forming groups' appear to have been based on the regular incremental distance of the sampling points from the source. Maul et al. (1990), basing his work on the methods of Anderson et al. (1984) used a 'nonhierarchical nearest-centroid clustering method' to allocate sampling stations to distribution system zones of similar bacterial density. However, the sampling plan they proposed was based on random selection from points on a spatial grid within each zone.

Systematic Sampling

Systematic sampling involves the measurement of systematically chosen members of the population, such as every n'th node on a list of distribution nodes.
It has the disadvantage that a complete list of population members is required (Hinkle et al., 1988). Every potential sample site must be identified according to relevant criteria, enumerated, and ordered, and a clear definition of the potential sample population is required that does not bias the sampling in an unacceptable manner. For a distribution system, a number of different lists could be compiled, all of which will bias the sampling in some way. Such a list might be of the individual pipes or nodes, or a list of pipe sections serving equal numbers of consumers or containing equal volumes of water, or covering equal linear distances. This last option would, for example, tend to bias sampling towards taking more samples per consumer in lowly populated areas, where consumers are more widely spread out along pipes, and conversely, the first option would bias sampling towards taking more samples in densely populated areas. The measure itself must also remain constant, and as pointed out by Geldreich (1990), a measure such as water consumption can vary seasonally and geographically, implying a need to vary the sampling plan also. He notes a number of similar disadvantages with different approaches of this kind. The requirement to monitor uniformly throughout the system does not specify what measure of uniformity is to be used, and the above measures are but three of many possibilities. In fact, proposals have been made to allocate sampling effort based on each of these, and other similar considerations (AWWA 1985, 1987, 1991; Geldreich, 1988).

**Search Sampling**

Search sampling is described by Gilbert as being useful 'when historical information, site knowledge, or prior samples indicate where the object of the
search may be found’. The search may be directed to areas in the distribution system with a record of bad samples, low velocities, old and leaky pipework and similar indications of poor water quality. This approach has some of the disadvantages of ‘quota’ sampling, as described by Weisberg and Bowen (1977), in which samplers are given the goal of taking a certain number of samples from specific population groups. The result is that, while keeping to the guidelines, samplers tend to sample population members that are cheap or convenient to sample. Thus, told to sample from certain areas of the distribution system, a sampler may choose the nearest house thereby excluding peripheral areas. The AWWA recommendations cited later use a search sampling approach to identify sampling points where it is thought that coliform occurrence is most prevalent.

**Sampling for Acute vs. Chronic Contamination**

Valiela and Whitfield (1989), in considering compliance with water quality objectives in general, consider the frequency of sampling to depend upon whether chronic or acute violations are to be detected, upon the statistical distribution of the variable being monitored, and the durations of the violation. Acute violations depend more upon the frequency, magnitude and duration of the contamination, whereas chronic violations are more associated with long term averages. In considering contaminant sampling of water distribution systems in general, Gomez-Taylor (1987) suggests that compliance with MCL limits can be determined either by averaging all results within the system or by determining compliance within sections of the system. The latter is preferred in order to avoid masking of an acute contamination event by averaging. However, it implies that part of a system

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could be in violation of the regulations, while another part would not. This possibility is excluded from the current coliform regulations, which would consider the whole system to be in violation.

The types of illnesses listed in the NPDWR as being associated with coliform violations suggest a concern with acute coliform concentrations which cause immediate disease outbreaks. However, they also state that identification of intermittent (chronic) coliform violations across sampling periods is important, though not presently regulated in the form of a contaminant level such as a maximum long term average. The requirement that all violations will eventually be detected implies that violations are chronic and will remain detectable long enough for the sampling plan to find them.

An average coliform density considerably less than one, however calculated, is implied by the requirement for complete absence of coliforms in all but 5% of samples. Even an average considerably below unity could be considered an 'acute' limit because of the possibility of an extreme count if certain proposed probability distributions for coliforms apply (Pipes et al., 1977; Christian and Pipes, 1983). It must also be remembered that the total coliform count is an indicator of possible contaminations by pathogens, and a small increase in coliform numbers may be associated with a disproportionate increase in other organisms.

The NPDWR imply that a 'violation' applies to the whole month in which the regulations are exceeded. Thus, one sample which results from transient, possibly gross, contamination may not trigger 'non-compliance' if repeat samples are negative. A second positive sample, quite unrelated to the first in either time or
location may, when counted with the first, place the system in violation. Such a scheme appears to be more directed towards detection of chronic contamination, that is, a tendency for repeated or persistent contamination.

**Fixed Frequency and Exceedance Driven Sampling**

Valiela and Whitfield (1989) consider fixed frequency sampling (FFS) as the most appropriate approach to the establishment of long term averages and detection of chronic violations. They also examine exceedance driven sampling (EDS) which starts out as FFS but 'responds to measured exceedances with more frequent sampling'. The NPDWR require a fixed number of 'base' samples taken every month at regular intervals, with additional samples to be taken in the case of a suspected or confirmed violation. This is clearly an EDS scheme in principle.

FFS and EDS control the frequency of sampling, but not the location. A similar principle to EDS could be applied with regard to location by taking more samples at the place where the exceedance occurred. In fact, the NPDWR do just this, with the slight variation that samples have to be taken within specified distances upstream and downstream of the initial violated sample for the purpose of detecting the direction of travel, or the source, of the contamination.

An extension of these principles would adjust the fixed frequency of sampling so that those locations and recurring periods (e.g. weekly and seasonal patterns) with more frequent violations would receive increased fixed frequency sampling. If the total number of samples is held constant, this would be at the expense of times and locations with less frequent violations on record.
The cyclical nature of coliform occurrence argues against a systematic sampling system if long term averages are to be determined (Ward and Loftis, 1986). A random sampling scheme would be more suitable for this purpose. However, if the purpose of sampling is to detect exceedances, then systematic sampling at those times judged most likely to include an exceedance is likely to be more effective. However, if all contamination is eventually to be detected, no part of the system or time of the day, week or month should be excluded from the certainty of being sampled 'eventually'.

LeChevalier commented in a workshop discussion moderated by Geldreich (1988) that a sampling plan should be defined to identify the source of bacteria, whether from breakthrough, cross-connections or biofilms etc, and that each would probably require a different sampling plan, but did not propose any sampling methods to achieve this. In the same discussion, Jones suggested retaining some regular sites to establish an historical data-base, although there was some general agreement that a large degree of randomization was desirable.

**Previous Research on Sampling Distribution Systems**

Much has been written on the sampling of water distribution systems, but most of that material relies on search sampling techniques, that is, heuristic knowledge of where contamination is most likely to occur, rather than objective studies of where it can most efficiently be detected. The expertise on which sampling plans are often based draws upon an understanding of the underlying processes such as microbiology and hydraulics, together with practical experience of where, among those points that have previously been chosen for sampling, most
contaminated samples tend to be found. This knowledge has been extrapolated to provide heuristic guidelines for general application, as embodied in the NPDWR and recommendations of the AWWA (1991). At the same time, the NPDWR require that samples be taken 'at sites representative of water throughout the distribution system', so that reliance on heuristic or historical information is not absolute. Such reliance would result in the use of sampling plans less likely to detect atypical contamination events. There is comparatively little published scientific work that considers the relative merits of alternative sampling plans on the basis of actual, or simulated sample results.

Stukel et al. (1987) examined three sampling plans for fifteen small communities of less than 1000 people. These plans were sampling once weekly from the same site, once monthly from the same site, and five simultaneous samples from different sites once monthly. They compared their sampling plans to each other rather than to an external standard, using the concept of relative sensitivity. This was defined as a conditional probability, evaluated for two plans, plan x and plan y, as:

\[ R_{xy} = \frac{\text{Total detections by plans} \ x \ \text{and} \ y}{\text{detections by plan} \ y \ \text{alone}}. \]

They showed that increased frequency of sampling, that is, weekly rather than monthly, or sampling monthly from five sites simultaneously was able to find more contamination incidents than monthly samples taken at a single site. They point out that it is easy to compare two plans on the basis of the number of contamination events detected, but that two plans may disagree entirely as to the location of those events. Locations sampled far from the source detected about 88%
of contamination detected close to the source, but samples taken at the site closest to the source detected only about 60% of contamination detected at the farthest site. Of the three designs, they concluded that 'no one sampling design dominated the other two'. In eight of the fifteen systems studied, no contamination was detected by any sampling scheme. Bailey et al. (1984) state that generally the number of samples required to show compliance is much larger than the number needed to show non-compliance.

When investigating metals contamination, Hulsmann (1987) states that scatter in samples from different locations is greater than scatter in successive samples from the same location, and concludes that more information is obtained from sampling at various points distributed over the service area than from repeated sampling at the same location. Particular attention should be given to high risk areas such as old cast iron pipes, dead ends and ring mains with low velocities and extended residence times.

Maul et al. (1986) address the problem of allocating a fixed number of samples between zones of a distribution system where each zone has been found to have a different bacterial density probability distribution. They show that where a negative binomial distribution applies, the number of samples required to meet a prespecified probability that the system is in compliance increases with the mean density, and is inversely related to the dispersion within each zone.

The effect of flow direction and contamination incidence within the distribution was considered by Lee et al. (1991) using steady state simulations of a contaminant transport model applied to the Flint, Michigan and Cheshire,
Connecticut water supplies. They considered the pathways taken by water from its source to a particular sampling point. They then inferred the quality of water at upstream points on those paths from the water quality at the sampling point. The amount of information that can be inferred about an upstream node was considered to be related to the proportion of water reaching the sampling node that passed through the upstream node. Using this concept, which they named 'coverage', it was suggested that sampling nodes can be selected that will increase the amount of information gathered.

Current Sampling Practice for Water Distribution Systems

The AWWA Committee on Bacteriological Sampling Frequency in Distribution Systems (1985) reviewed industry practice in 1985. There was found to be no consensus, with much variation in the timing and location of sampling, while maintaining overall compliance with the regulations. Most samples were taken early in the day and early in the week. The Committee commented that this was probably advantageous since it tended to coincide with peak demands, suggesting that bad samples tend to coincide with high flows. Few samples were taken at weekends or at night and often samples were taken all within one week during the month. Many utilities routinely sampled water leaving the treatment plant. Without considering size of system, about one third of utilities sampled at fixed points, 47% sampled at both fixed and variable points and 16% used variable locations. Few, if any, used random locations. About one fifth of utilities took less than the required number of samples, but the generally poor record of smaller utilities in complying with monitoring requirements suggests that they are unlikely
to take additional samples voluntarily. However, about half of all utilities took more than the minimum number of samples, indicating the potential for improved sampling plans in many cases.

Donner (1987) described how Seattle chose a mix of distribution mains and house plumbing to sample. The AWWA Organisms in Water Committee (1987) recommended that consideration be given to the length of the distribution system, with samples taken both at fixed representative sites and at variable sites so that all parts are sampled. The Committee and Donner take pressure zones into account when locating sampling points. Geldreich et al. (1988) advocated sampling close to the treatment plant every day, especially where a surface water is used with minimal treatment. Considerations such as these imply some knowledge of the processes affecting coliform occurrence.

The AWWA (1991) guide to microbiological sampling states:

'Sampling points should be

* distributed uniformly throughout the system;
* representative of each different water source entering the system;
* representative of each pressure zone;
* situated so that storage tanks can be sampled during times of high demand;
* located in relative proportion to the number of people served by each water source if there is more than one source; and
* sampled at times when the water source can be determined since it is possible that excessive demand in one part of the distribution system can cause water to be brought into that area from other parts of the system and perhaps other sources.'
They accept the taking of water samples at public buildings, as well as at private residences and special sampling stations.

Maul et al. (1991) suggest several monitoring approaches. 1) A 'standard driven' approach using regulatory guidelines without regard to conditions prevailing in the system; 2) Use of existing information on system configuration and bacterial occurrence to interpret, or compromise with, the regulations to advantage; 3) use of historical data and a preliminary study to optimize a fixed sampling plan, without regard to regulations, and 4) a continuous sampling and evaluation effort that adjusts sampling levels to reflect current conditions in the system.

**Measures of Effectiveness**

Sampling schemes 'require agreement upon a rational basis for sampling which justifies the effort involved in relation to the information obtained' (Bailey et al., 1984). Within a regulatory framework, an objective, rather than subjective basis is required. One approach to the comparison of sampling strategies is the determination of a measure of effectiveness for the proposed sampling plan. In addition, sampling schemes must be feasible and practicable, both of which can be expressed in terms of the quantity of resources required to execute the proposed sampling plan and which usually results in an upper limit to the number of samples that can be taken. Some measure of efficiency may be incorporated in the measure of effectiveness.

A measure of effectiveness is an objective and quantitative measure which can be used for comparison purposes (Chamberlain et al., 1974; Ossenbruggen, 1983). In many situations, more than one measure of effectiveness can be
identified, but these may be unlike in kind. In the case of distribution sampling, measures of effectiveness to improve violation detection or alternatively to reduce the cost of sample collection are unlike since they are expressed in different units. Such measures are difficult to combine into a meaningful single measure, and must usually be satisfied as separate objectives. Ossenbruggen suggests that in such cases each objective can be met independently, and a final choice made using subjective methods.

Chamberlain et al. (1974) distinguish between spatial and temporal considerations. They rate the location of a sampling point on the basis of the probability that the water quality standard will be violated at that point, with more information being obtainable at a high probability location. This probability can be expressed in terms of the mean and standard variation of the quality probability distribution and the standard threshold value, all of which must be known.

A quantitative measure of the temporal effectiveness of a sampling scheme used to detect violations would always increase with sampling frequency (Beckers et al., 1972), and does not yield a preferred frequency. Such a measure can be used as a 'measure of goodness' which, combined with a cost effectiveness measure, can yield an optimal sampling frequency.

A measure of effectiveness for sampling frequency proposed by Beckers et al. (1972), is denoted by $M$, the expected proportion of violations detected, and is defined by the expression,
\[ M = \frac{\text{Expected number of violations detected}}{\text{Expected number of violations}} \]

Chamberlain and Beckers (1974) also expressed \( M \) as a function of the sampling frequency, expected interval between violations, and the duration of the violation. This presumes an ability to define and estimate expected violations and their durations.

Casey et al. (1983) derive two measures of effectiveness for random sampling schemes in which the sampling interval is exponentially distributed and the duration of a violation is known. The first, \( M' \), is for cases in which it is known when successive violated samples are taken from the same violation so that multiple counts may be discarded, and is defined by,

\[ M' = \frac{\text{Expected duration of a violation}}{\text{expected duration of a violation} + \text{expected sampling interval}} \]

A value of \( M' \) approaching unity implies a high frequency of sampling (short sampling interval), with a large number of duplicate samples from the same violation being discarded. However, only when \( M' = \text{unity} \) are all violations detected.

The second, \( M^* \), is for cases in which it is not possible to tell whether two (or more) successive samples are taken from the same or different violations, so that possible multiple counts of the same violation are allowed for. \( M^* \) is defined by
\[ M'' = \frac{\text{Expected duration of a violation}}{\text{expected sampling interval}} \]

which they show can also be expressed as

\[ M'' = \frac{\text{Expected number of violated samples}}{\text{expected number of violations}} \]

If statistic \( M'' \) is set equal to unity, then knowing the expected duration of a violation, an expected sampling interval is defined, and is equal to the expected duration of a violation. While use of the \( M'' \) statistic can ensure that the expected total number of violations can be detected, there is no assurance that each individual violation will be detected.

Casey et al. (1983) point out that their expressions do not include the (expected) time between violations from which it can be inferred that the number of non-violated samples, which would be taken between violation periods and whose number would increase as sampling frequency increases, does not affect \( M, M' \) or \( M'' \). Thus the total number of samples is not a parameter on which the measure depends, and availability of unlimited sampling resources is assumed. If efficiency is defined as some function of the amount of information obtained per sample, then the cost, or efficiency is not expressed by \( M, M' \) or \( M'' \) as defined above.
MATERIALS AND METHODS

Overview - Stochastic Simulation with Sampling

The operation of a hypothetical distribution system subject to stochastic demands and several types of random and deterministic contamination was simulated for periods of twenty years. Random contamination events included the location, duration and severity of contamination inputs from external sources. Some internal contamination sources representing biofilm effects were linked to the simulated hydraulic characteristics of the distribution system. Seven sampling plans were applied to simulate the taking of samples according to typical and proposed sampling plans, and were interpreted in a manner representing compliance with sampling regulations applied to coliform monitoring. The comparative effectiveness of the sampling plans was evaluated using a multiple comparison method to detect significant differences between plans. A sensitivity analysis was performed on some of the contamination parameters and on one aspect of system layout.

Model Development

Constituent Transport Model

A constituent transport model was constructed by converting the hydraulic model WATER (Municipal Hydraulics, 1990) to extended period simulation mode, and using it to provide hydraulic input to a suite of new constituent tracking subroutines named TRAK. Additional subroutines were provided to apply the sampling plans and provide stochastic inputs.
Stochastic Data Generation

Stochastic inputs were generated using the Monte Carlo technique. The random number generator used is based on the recursion

\[ IX = 16807 \times IX \mod (2^{31} - 1) \]

described by Law and Kelton (1991). Separate random number streams were used for each stochastic variable.

Hypothetical Distribution System

System Layout

The hypothetical distribution system is shown in Figure 1. The basic layout, referred to as 'Layout 1', has a pump station at P as the only input, and a storage tank at T with a capacity of 12 hours average demand. A second layout with the pump station moved to the alternative position A is referred to as Layout 2. Both layouts are otherwise identical. The system serves a community of 5000 people. Demand for the hypothetical system was set at 100 gal/cap/day (379 L/cap/day). The central area, shown hatched on Figure 1, and comprising twenty of the forty demand nodes was designated a high demand area consuming two thirds of daily demand, with the outlying twenty nodes consuming the remaining third of total demand. The numbers of the six nodes used for fixed location sampling are shown on Figure 1, and are referred to again in the discussion of Figures 27 and 29.
Figure 1. Hypothetical distribution system layout

- P - pump station, layout 1.
- A - pump station, layout 2.
- T - tank
- 4 inch pipe
- 6 inch pipe
- 8 inch pipe
- High demand area
- Fixed sampling point

Legend:
The system was considered to consist of five supply zones each containing an approximately equal number of nodes. The central high demand area consists of two zones separated by the single link of the trunk main and each containing 10 nodes. The 7 node grids to the east and west of the high demand area were each considered to be one zone, and the six nodes to the south of the high demand area made up the fifth zone.

These layouts are characteristic of typical published water distribution systems (Grayman and Clark, 1990; Kennedy et al., 1991; Lee et al., 1991). Local demand areas are often supplied by a pipe grid, with adjacent areas somewhat sparsely connected by trunk mains. Major sources are often located within, but not central to the area of demand. A trunk main usually extends from the major source through the major part of the demand area, and will generally also supply a storage tank located in the middle third of the demand area.

The hypothetical system contains 56 distribution pipes and two pipes connecting the pump station and storage tank to the system. Nominal pipe sizes were determined to provide average velocities in the range 0.1 to 1.0 ft/sec. An 8 inch trunk main connects the tank to the treatment plant and extends beyond the tank to the edge of the high demand area. Other pipes within the high demand area are all 6 inch diameter, and those in the outlying area are 4 inch diameter. All pipes are 500 feet long and were assigned a Hazen Williams C factor of 100. The pump station supplies water to the system according to a typical pump curve defined by a set of head/quantity data pairs in the input data set. A daily system demand curve was defined on an hourly basis and is shown in Figure 2.
System Operation

Stochastic System Demand

The network flow regime was calculated every hour when demand changed, and at interim times if the level in the tank changed more than 1 foot. At the start of each simulated hour the mean demand for the next hour was obtained from the demand curve and then randomized using a beta distribution with lower and upper bounds of 80% and 130% and a coefficient of variation of 0.0594, using algorithm BB from Cheng (1978). This distribution is approximately normal in shape, but the tails are curtailed at the bounds and it is slightly skewed to the right to reflect typical positively skewed demand variations. Use of the bounded beta distribution avoids having to truncate or discard extreme outlier values that might be obtained if using an unbounded distribution such as the normal. A lagging flow adjustment equal to the ratio of the randomized demand to mean demand for the previous hour was applied to smooth large variations between consecutive hourly demands. Weekly and seasonal variations were not included as part of the simulation. No two days in the twenty year simulation period had exactly the same demand pattern.
Figure 2. Hypothetical system daily demand curve
Tank and Pump Control

To prevent possible overflow of the tank, pump control was defined to turn the pumps off if tank water level came within 0.5 feet of top water level, but this occurred very rarely during the simulation. The pumps were turned on again after the tank level fell to three feet below tank top water level. The tank never emptied. On average, about 27 hydraulic calculations were performed every 24 hours, one for every hourly change in demand plus some interim solutions if the tank level changed more than one foot.

Initial Conditions

Operation of the system was started at midnight (00:00 am) on day one. The initial tank level was set at a level typical for this time of day, as determined from trial runs. A warm-up operation period of 24 hours was allowed before any hydraulic or contamination data from the simulations was saved for analysis.

Simulations Performed

Events representing externally introduced contamination at the tank, pump station and nodes, and resulting from various internal biofilm events were simulated in separate twenty year simulations. The parameters used are summarized in Table 1. Contamination was introduced at the location and mean interval stated by increasing the concentration of the water over any existing concentration by the amount stated. Each node contamination event was located at one of the forty system nodes (not including the pump station or tank nodes) chosen at random with replacement. For tank, pump station and node contamination, event times were randomly generated from an exponential distribution with the stated mean.
Simulation run-times were, with exceptions noted where applicable, 7302 days, that is 1043 weeks and one day for model 'warm up'. Warm up of the model substantially removed the effect of initial conditions on variables subject to stochastic inputs. For example, an initial tank water level must be specified, but subsequent levels will vary according to stochastic demands. Each simulation was applied to both Layouts 1 and 2, with a conservative contaminant unless noted otherwise. In addition to using two layouts, variations in some parameters were made to provide data for sensitivity analyses. These conditions are summarized in Table 1. For some sets of conditions, repeat simulations were made using different seeds for all random number streams.
Table 1. Parameter combinations used for distribution system simulations.

<table>
<thead>
<tr>
<th>Location and cause</th>
<th>Mean interval</th>
<th>Concentration increment</th>
<th>Duration hours</th>
<th>Decay rate per day</th>
</tr>
</thead>
<tbody>
<tr>
<td>Externally introduced contamination</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tank, sudden leakage</td>
<td>30 days</td>
<td>10 units</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>Pump Station</td>
<td>7 days</td>
<td>10 units</td>
<td>1</td>
<td>0, 0.5, 1.0, -2.0</td>
</tr>
<tr>
<td>7 days</td>
<td>10 units</td>
<td>4 hours</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>7 days</td>
<td>10 units</td>
<td>16 hours</td>
<td>0, 0.5, 1.0, -2.0</td>
<td></td>
</tr>
<tr>
<td>Random node</td>
<td>4 days</td>
<td>10 units</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>4 days</td>
<td>100 units</td>
<td>1 hour</td>
<td>0§</td>
<td></td>
</tr>
<tr>
<td>4 days</td>
<td>100 units</td>
<td>4, 16 hours</td>
<td>0¶</td>
<td></td>
</tr>
<tr>
<td>4 days</td>
<td>1,000 units</td>
<td>1 hour</td>
<td>0, 0.5, 1.0, -2.0</td>
<td></td>
</tr>
<tr>
<td>4 days</td>
<td>10,000 units</td>
<td>1 hour</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>Biofilm contamination</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stagnation</td>
<td>v</td>
<td>0.1, 0.5</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>Flow reversals</td>
<td>v</td>
<td>1</td>
<td>0, -1.0, -2.0</td>
<td></td>
</tr>
<tr>
<td>High velocity</td>
<td>v</td>
<td>1.0</td>
<td>1</td>
<td>0</td>
</tr>
</tbody>
</table>

Simulations were performed for both layouts except where noted.

§ Five duplicate simulations with different random number streams
¶ Five duplicate simulations with different random number streams, Layout 1 only.
V Timing depends on velocity conditions
Tank Contamination

Two regimes of tank contamination were tested, representing sudden contamination and slow leakage respectively. For sudden tank contamination, the concentration of all water in the tank was instantaneously increased by 10 units. For leakage contamination, the concentration of all water in the tank was increased by 0.1 units every hour for a period of 24 hours. A mean time between events of 30 days was used for both contamination types, with no reaction.

Pump Station Contamination

Pump station contamination was simulated with a mean time between events of 7 days, an input concentration of 10 units, and three contamination duration times of 1, 4, and 16 hours. For the 1 and 16 hour durations, simulations were also made at three decay rates of -0.5, -1.0 and -2.0 per day. For conditions of 1 hour contamination duration and no decay, five duplicate simulations were made using completely different streams of random numbers for the stochastic inputs. Contaminant was applied by raising the concentration of all water leaving the pump station during the event by 10 units, without regard to flow rate.

Node Contamination

Node contamination was simulated with a mean time between events of 4 days and generally 1 hour contamination duration time, and input concentrations of 10, 100, 1,000 and 10,000 units. For the simulations with 1,000 units concentration, simulations were made with decay rates of -0.5, -1.0 and -2.0 per day. For 100 units concentration and no decay, simulations were also made with 4 and 16 hour contamination durations. For conditions of 100 units concentration and no
decay, and for 1,000 units concentration and no decay for layout 1 only, five
duplicate simulations were made using completely different streams of random
numbers for the stochastic inputs. Contaminant was applied by increasing the
concentration of all water leaving the affected node by the contaminant
concentrations stated above, without regard to the flow rate.

**Biofilm Contamination**

Biofilm events were linked to individual pipe velocities indicating increases
in planktonic cell numbers under stagnant conditions and biofilm sloughing due to
flow reversals or unusually high velocities. This procedure is explained in more
detail in Appendix 5. The concentrations resulting from a biofilm event were
applied by increasing the concentration of all plugs in the affected pipe. The level
of contamination applied was adjusted according to the type of event and the pipe’s
distance from the pump station, the letter reflecting the frequent observation of
higher biofilm growth rates near the main source of assimilable carbon in the
system. Separate simulations were performed for high velocity, stagnation and flow
reversal effects with no decay. For layout 1 only, flow reversal effects were
simulated with decay rates of -1.0 and -2.0 per day.

**Sampling**

**Definition of a Violation**

A monitoring period was defined as a calendar week from midnight Sunday
to midnight Sunday. The contaminant was defined as being detectable if its
concentration was greater than or equal to 1.0 unit. This was therefore the
effective Maximum Contaminant Limit (MCL). The concentration at every node,
including the tank and pump station nodes, was checked every hour, and if it exceeded the MCL, the system was considered to be in actual violation for the current monitoring period. If a sampling plan detected a concentration exceeding the MCL at any time during a monitoring period then a violation was counted for that monitoring period under that sampling plan. If more than one such sample was detected by the same plan in the same monitoring period, then no additional violation was counted under that plan.

**Sampling Plans**

Seven sampling plans were defined each requiring six samples to be taken over a weekly monitoring period. The six fixed sampling nodes were chosen on the basis of 'engineering judgement' and were intended to reflect the AWWA's guidelines for choosing sampling points (AWWA, 1991); the six points are representative of the whole area, one point is within the first 20% of customers from the pump station, all five zones are sampled and water coming from the tank is sampled. The fixed sampling time of 9.00 am reflects the usual practice of taking samples early in the day. All samples were taken at the forty distribution nodes, or by dipping from the tank water. The nodes at the tank and at the pump station were never sampled. The seven plans are defined in Table 2.
Table 2. Sampling Plans

1. 6 fixed nodes sampled at 9:00 every Monday morning.

2. 6 fixed nodes samples at 9:00 every morning, one each day and in the same order, Monday through Saturday.

3. 6 randomly chosen nodes, one sampled at 9:00 every morning, Monday through Saturday. The node for sampling was picked at random with replacement from the 40 distribution nodes on each of the six days.

4. 5 randomly chosen nodes, one sampled at 9:00 every morning, Monday through Friday, picked as for plan 3. A sixth weekly sample was dipped from the tank at 8:30 on Saturday.

5. 4 random nodes chosen from the High demand areas and 2 random nodes chosen from the Low demand areas, sampled one per day at 9:00 every morning Monday through Saturday in the order H-L-H-H-L-H.

6. 1 node chosen at random and sampled at a random time distributed exponentially with a mean time between sampling of 7/6 days, giving an average sampling frequency of 6 samples per week.

7. 6 systematically chosen nodes, one sampled at 9:00 every morning Monday through Saturday. The 40 distribution nodes were sampled without repetition in systematic order defined so that no two successively sampled nodes would be in the same zone and at least four of the five zones would be sampled every week. When all forty nodes had been sampled, the same cycle was repeated.

NOTES
- The same six fixed sampling nodes were used for plans 1 and 2, and are identified on Figure 1.
- The regulations do not specify that water either entering, leaving, or in the tank is to be sampled as part of a sampling plan, and the tank node was therefore not included in any sampling plan. However, the adjacent node forming part of the system pipe grid was chosen as one of the fixed sampling nodes for plans 1 and 2, and the tank itself was included as a sampling point for plan 4.
- Plan 6 only seeks to take an average of six samples per week, and does not guarantee that exactly six samples will be taken per weekly monitoring period, and therefore does not strictly comply with the regulations.
Model Output

Data provided by the model included the following:

1. An hourly record of node concentrations in excess of the Maximum Contaminant Level (MCL),

2. A count for each node of the number of hours for which the concentration at that node exceeded the MCL,

3. A daily record of node concentrations exceeding the MCL each day,

4. A daily record for each sampling plan of whether that plan had detected a concentration in excess of the MCL during that day.

This data was processed run time and by a special purpose post-processor, to provide a violation occurrence and detection record based on a weekly monitoring period, a record of the number of violated weekly monitoring periods that were detected by each sampling plan, and other data presented later. Synoptic data on flow characteristics in each pipe during the simulation were also recorded.

Statistical Methods

Statistically significant differences between plans were identified using a multiple comparison procedure based on Friedman rank sums (Hollander and Wolfe, 1973) at a 90% confidence level 90%. The test was used to compare the sets of twenty values obtained for the number of violations detected per year by each sampling plan. A detailed explanation is contained in Appendix 7. Where no significant difference was detected, the proportion of violations detected over twenty years was used as a measure of effectiveness for comparison.

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RESULTS

CONSTITUENT TRANSPORT MODEL DEVELOPMENT

Introduction

The constituent transport model consists of two major modules;

1. The hydraulic module, WATER

2. The constituent tracking module, TRAK

The two modules are combined into one model, with a controlling main program named PIPE that calls the appropriate subroutines of each module as required. In many cases, it was expedient to code activities related to extended period simulation of the hydraulic module and constituent transport in the same section of code. References to major subroutines therefore appear in both the following sections, and the functions of each subroutine are summarized in APPENDICES 1 and 2.

The Hydraulic Module

Existing Steady State Model - WATER

The hydraulic module used was the steady state model WATER (1990), available commercially as a steady state hydraulic model. (A dynamic version of WATER named WATEX is available, but it was considered easier to customize the steady state version for the purposes of the study). The version of WATER used in this study was provided for research purposes by A. G. Fowler of Municipal Hydraulics. The user manual distributed with WATER gives full details of the input required. A brief description of each subroutine is included in Appendix 1.
Extended Period Simulation

Conversion

WATER was converted to extended period simulation mode by placing it under the control of a new calling program named PIPE, so that WATER becomes a subroutine of PIPE. Successive calls to WATER can then be made by PIPE which passes variables defining different boundary conditions to simulate dynamic conditions. After the very first solution at the start of a simulation, the boundary conditions for each call are defined by the results of the previous call to WATER, amended if necessary by re-definition of input variable values such as demands and the status of pumps and tanks. Since the basic layout of the system only needs to be defined once, the call to the subroutine which reads the data file was removed from WATER and called once only by PIPE at the beginning of a simulation. To simplify conversion and to facilitate changes and extension to the data file, the editor and some of the user interface sub-programs of WATER were removed. The model, WATER, is written in FORTRAN77 and all additional code was written to this standard also.

The length of the time interval between successive hydraulic solutions is variable and is controlled by the need to update boundary conditions when they depart significantly from the previously defined conditions. This can occur either when user defined changes, such as a change in system demand, are specified in the data file, or when the model senses a change in certain variables, such as tank water levels, that exceed a specified tolerance. The new boundary conditions are usually sufficiently similar to the previous boundary conditions that the previous flow
solution can be used as a set of estimated flows for the next solution. In these circumstances, this generally gives a better estimate of starting flows than the minimal spanning tree (MST) algorithm used in WATER, and faster convergence to a new solution, and obviates having to run the MST algorithm.

**Boundary Condition Updates**

In a long term simulation, specific events or changes to conditions may occur periodically or at known times in the future. Typical changes which are accommodated include variations in demand, periodic operation of pumps, and the start and end, or change in concentration of a constituent input. Two mutually exclusive options were provided to allow changes to be specified to take effect during the simulation, the chosen method being specified in the input data file. The first option reads in a limited number of changes from the data file. The second option generates changes at run-time according to the particular needs of the user. Since all the types of changes which future model users may wish to specify could not be anticipated, a section of easily amended code was provided in PIPE to facilitate extension for future needs.

The practicalities of keying in and storing large numbers of data changes as might be required for a simulation lasting many years effectively limits use of the first option to relatively short term simulations. The second option calls a subroutine UPDATA to change or generate input data at run-time. This subroutine must be customized by the user to meet particular requirements. The version used for this study specifies an overall demand factor at one hour intervals in accordance with a diurnal demand curve, and itself calls further subroutines to alter
contaminant inputs. Customization of source code for UPDATA can be complex, but allows great flexibility.

**Tank and Pump Control**

Changes in demands over time and changes of the flow into or out of a tank cause the tank water level and source pump delivery rates to change, in turn altering hydraulic conditions in the whole system. A suite of subroutines was written to account for changes in tank levels and constituent concentrations in tanks and to control source pumps. These subroutines provide the following facilities to the dynamic model:

- Maintain a record of tank constituent concentration updated at the end of each timestep.
- Maintain a record of tank water level updated at the end of each time step and invoke a new hydraulic solution when the level changes more than a preset amount.
- Close off a tank at specific times or when it is about to overflow or become empty, and invoke a new hydraulic solution.
- Reopen a tank at specific times or when hydraulic conditions allow a full tank to start emptying or an empty tank to start refilling, and invoke a new hydraulic solution.
- Turn a designated pump on or off when a tank level reaches a predesignated value, and invoke a new hydraulic solution.
- Turn pumps on or off at predesignated times as specified in the data file, and invoke a new hydraulic solution.
A new hydraulic solution is invoked because boundary conditions are changed sufficiently from previous conditions.

Tanks can be specified as having a common inlet-outlet pipe, or an inlet at top water level and an outlet at bottom water level. A pump can be linked to the water level in a tank, to be turned on when the level falls below a certain level, and off when it rises above a certain level.

The steady-state version of WATER received allows pumps and tanks to be entered in the data set in any order and represents tanks as pumps with flat pump curves. For contaminant transport modeling, it is helpful to be able to access tank data as a group distinct from pump data, and the data entry format of WATER was amended to account for this.

**Constituent Tracking Module**

A suite of subroutines was written to track a waterborne constituent through a distribution system, given the hydraulic conditions in the system. These subroutines form the constituent tracking module, named TRAK. Hydraulic input is obtained from WATER, but in principle, any hydraulic network module could be used with a suitable interface.

**Plugs and Mixing at Nodes**

The constituent tracking algorithm described by Liou and Kroon (1987) was used as the basis of this model. Inherent assumptions are those of no longitudinal dispersion (plug flow) and complete mixing at nodes.

The constituent profile along a pipe is represented as a step function by a series of elemental pipe sections, or plugs, each with a known length and
concentration. The concentration of each plug, multiplied by the volume of the plug, represents the mass of constituent in that plug. Initially, if the system is assumed clean at start-up, all pipes contain one plug of zero concentration. When a constituent is introduced at a node, either from an external source or from a pipe carrying flow into the node, that concentration is completely mixed with other influent concentrations according to the mixing equation. For a node with \( n \) inflows,

\[
C_{\text{new}} = \frac{\sum_{i=1}^{n} C_i Q_i}{\sum_{i=1}^{n} Q_i}
\]

where \( C_i \) and \( Q_i \) are the concentration and flow entering a node from pipe \( i \), and \( C_{\text{new}} \) is the mixed concentration in all pipes with flow leaving the node and defining the concentration of new plugs created in pipes flowing away from such nodes. As the constituent moves through the system, plugs in different pipes converge at downstream nodes, and the process is repeated. A one-to-one correspondence of plug length and concentration makes internal bookkeeping by the computer very simple. Effects of reaction, mixing and consolidation can then be obtained using linear combinations of these variables.

Whenever a plug first arrives at a node, the mixing equation is invoked to obtain the new mixed concentration of subsequent outflows from that node, but need not generally be invoked at other nodes. The mixed concentration calculated at a particular node is used for as long as no new concentration plug or variation in flow rate occurs to alter it. If system flows change, such as when pumping rates or
demand factors change, then the flow weighted average concentrations will be affected at all nodes, and the mixing equation must be applied to all nodes. A flag for each node is maintained indicating which nodes need to be remixed at the next iteration. Avoidance of unnecessary application of the mixing equation can speed program execution in a heavily contaminated system. This scheme can accept flow reversals without loss of integrity, requiring only knowledge of the flow direction in order to identify which end of a pipe is the downstream end at a particular time. The maximum number of plugs in a pipe can be specified. This must be less than or equal to the size of the arrays declared to hold plug data.

**Timesteps**

Progress of the constituent occurs in a series of incremental timesteps applied globally to the system. Movement of constituent in each pipe is represented by a sequence of incremental movements, each corresponding to a timestep. Movement sizes are calculated for each pipe from the pipe flow rate and duration of the timestep.

The length of the next timestep is set to the shortest of several possible values. First, the time it would take the plug at the downstream end of each pipe to enter the downstream node completely is calculated for each pipe, and the shortest value is identified. This is then compared to the time it will take each tank to fill or empty and the time until the next data change. For non-conservative constituents, it is then compared to a default maximum value which corresponds to the time required for a maximum allowable percentage decay that can occur between timesteps. The shortest of these various times is used as the next
timestep. The maximum decay allowed between timesteps can be specified in the input data file, or if not specified, defaults to 1%.

Plug Consolidation

A larger number of plugs per pipe produces a step function which more closely resembles the continuous concentration profile. However, there is a practical limit imposed by the computational effort and computer memory required to track and store the length and concentration of large numbers of plugs. A tracking algorithm of the kind used in TRAK can result in excessive numbers of plugs. Liou and Kroon (1987) described a plug consolidation scheme which consolidates the shortest plug in a pipe with the adjacent plug of closest concentration. A similar scheme is used in TRAK which consolidates the two adjacent plugs of most nearly equal concentration, but any very short plug less than one foot in length will be preferentially consolidated, and end plugs will not be consolidated. The latter condition is imposed because short plugs are often formed at the ends of pipes during a short timestep just after a new mixed concentration has been defined at the upstream node. This plug will increase in length as more constituent at the same concentration enters the pipe. Consolidating this plug will change its concentration, forcing creation of a new plug on the next iteration, and requiring a repeat of the procedure. If no suitable short plug exists, then the two adjacent plugs with the most nearly equal concentrations, regardless of the plug length, are consolidated, but still avoiding end plugs.

The consolidated plug is given a concentration equal to the volume weighted average concentration of the two chosen plugs, which then form one plug with a
length equal to the combined length of the member plugs. In this way, strict conservation of mass is achieved, with the disadvantage of some dispersion of the constituent.

**Non-Conservative Constituents**

Reaction of a non-conservative waterborne constituent is a commonplace phenomenon. This text will talk generally in terms of decay, though it is equally applicable to reaction of any kind. The rate of decay is often a function of time and initial concentration; i.e.,

\[ \frac{dC}{dt} = f(t, C_0) \quad \text{or} \quad C_t = f(t, C_0) \]

An exponential reaction rate is frequently assumed and has the form:

\[ \frac{dC}{dt} = Kt \]

or, after integration,

\[ C_t = C_0 e^{Kt} \]

where

- \( C_0 \) is the initial concentration at time 0
- \( C_t \) is the concentration at time \( t \)
- \( K \) is a reaction constant, a positive sign signifying growth, and a negative sign signifying decay.

In the constituent transport model, decay is calculated at the end of each timestep, so that the value of \( t \) is the same as the timestep just completed.

For a relatively simple decay model, such as the exponential model, it is possible to calculate the ratio \( C_t/C_0 \), which applies during a particular timestep and which remains constant during that timestep. If \( K \) is a global value which does not
vary geographically within the physical boundary of the model, the ratio need only be calculated once for each timestep. If $K$ varies geographically, a different ratio applies to different pipes.

The ratio $C_i/C_o$ is applied to all concentrations in the model at each timestep, specifically, all plug concentrations, all mixed concentrations at nodes and all tank concentrations. Because plug and node concentrations are multiplied by the same factor, the mixing equation does not need to be invoke to determine the new mixed concentration derived from decaying plugs entering a node.

**External Reactive Constituent Sources**

A non-conservative constituent input at a node will decay as it travels down the pipe to form the first plug downstream from the node. Its actual average concentration, $C_{ave}$, after decay at the end of the timestep during which the plug was formed is then given by

$$C_{ave} = \frac{1}{AL} \int_0^L AC_o e^{\frac{xL}{V}} dx$$

$$= \frac{C_o}{Kt} [e^{\frac{xL}{V}} - 1]$$

where $A$ is the area of the pipe, $V$ is the flow velocity, $C_o$ is the entering concentration and $x$ is the distance along the plug, $K$ and $t$ are the decay rate constant and timestep, and the limit of integration, $L$, is the length of the plug. However, the model will record its average concentration as $C_n$, calculated from $C_i = C_o e^{(Kt)}$ as described previously. This results in an underestimate of the plug's

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true average concentration. An adjustment which increases the plug's concentration to the correct value is the ratio of $C_{\text{ave}}$ to $C_n$ such that

$$\text{Adjustment} = \frac{C_0 e^{K_t} - 1}{K_t}$$

$$\quad = \frac{1}{K_t} [1 - e^{-K_t}]$$

This adjustment is applied only to the portion of concentration resulting from the external source of constituent, and is not applied to constituent entering the node from other pipes. In subsequent timesteps, the plug is treated as any other 'existing' plug.

**Purging Low Concentrations**

Through the effects of dilution and decay, the concentration in a plug may become numerically very small, and the computational effort required for its continued tracking is wasted. To speed up model execution, a 'detection limit' for the constituent can be specified. The model then calculates a threshold concentration equal to 10% of the detection limit. Tracking of concentrations below this threshold value is unproductive, and they are therefore set equal to zero. There is a removal of constituent mass by this procedure, but when that mass is represented by very low concentrations, then that loss is not significant.

Three approaches for implementing this procedure are available:
1. Set the concentrations at a node to zero when a new mixed concentration less than the threshold value is calculated at that node.

2. Set low concentrations at nodes to zero when a new decayed concentration less than the threshold value is calculated at that node.

3. Set low plug concentrations to zero when a new decayed plug concentration less than the threshold value is calculated for that plug.

By testing some small systems with the different options, it was found that the first method generally offered a small decrease in run time of the order of 7%. This was used in all simulations.

**Model Verification**

**Introduction**

Model verification ensures that the model performs the various functions of data handling and process simulation in the manner intended by the model builder. It assumes, and does not confirm, that those processes themselves are correctly defined.

**Hydraulics**

It was assumed that the hydraulic portion of the model was already substantially verified through long term use as a commercial model. However, conversion to extended period simulation mode might have introduced some errors in the hydraulic solution process. The data set and hydraulic solution of the distribution system for Newcastle, PA, as obtained by others using the commercial hydraulics program, LIQSS (developed by Stoner Associates, Carlisle, PA) was provided by personnel of the American Water Works Service Company. The
extended period model derived from WATER was set up to provide a steady state solution to the Newcastle system. The two steady state solutions were compared, and were found to be substantially identical.

Once the correct operation of the hydraulic module had been verified, operation in extended period simulation mode was verified as part of the constituent transport verification described in the following sections.

**Constituent Transport**

The simple pipe network shown in Figure 3 was set up to verify the model's representation of concentration profiles along pipes, and of node concentrations. It consisted of three pipes in series. A flow of 0.5 ft³/s, with a constituent concentration of 100 units was introduced at node 1. The first pipe was of 10 in diameter and 6000 ft length, with a travel time of 109 min. The other two pipes were 14.142 in diameter and 5000 and 7000 ft long respectively, their diameters being chosen so that the flow velocity in them would be half that in the first pipe.

**Pipe concentration profiles**

A record of the plug disposition after 109 minutes was preserved, showing the plugs created up to the time of first arrival of the constituent at node 2. The simulation was repeated with differing values for the default maximum percent decay allowed per timestep, and the number of plugs allowed per pipe. A decay rate of -10 per day was used. This system was simulated with a maximum of 25 plugs per pipe and default maximum decay values of 10%, 5% and 1%. The concentration profiles produced are shown in Figures 4a, 4b and 5a respectively. The last case was
repeated with only 10 plugs per pipe maximum, and the resulting profile is shown in Figure 5b.

**Node concentrations**

Three more simulations of this system were performed with a decay rate of -5.0 per day, a PCDKDT value of 5.0%, and with 10, 20 and 25 plugs. The simulations were allowed to run for 550 minutes of simulated time, that is, about five travel times for the first pipe, to allow formation of a typical plug pattern unaffected by initial conditions. The concentrations of plugs passing nodes 2, 3 and 4 were monitored for a further 150 minutes (time 550 to 700 minutes) to examine the variability of concentrations that would be sampled at those nodes. These concentration are plotted in Figure 6a, 6b and 6c.

**Time period of simulation**

The simulation relied on stochastic inputs for some variables, and therefore the results were expected to be variable over short time periods. Over long time periods, the effectiveness measure of a particular sampling plan provided by the simulation should become more stable as it tends towards its actual value. The length of time required for reasonable stability was determined from a plot of cumulative percentage detections of each plan. The plot for simulation no. 62 is shown on Figure 7.
Figure 3. Simple pipe system for model verification
Figure 4. Plug profiles obtained with 10% and 5% maximum decay between timesteps and 25 plugs.

a. maximum decay = 10%
b. maximum decay = 5%
Figure 5. Effect of varying number of plugs with a 1% maximum percent decay between timesteps.

a. 25 plugs per pipe
b. 10 plugs per pipe
Figure 6. Observed concentrations at three nodes of a simple pipe system during passage of contamination using different maximum numbers of plugs per pipe.
   a. 10 plugs, b. 20 plugs, c. 25 plugs.
Figure 7. Cumulative percentage detections of violations from 1 to 20 years for each of the 7 sampling plans, simulation no 62.
SIMULATION RESULTS

Flow velocities

This section describes the results of simulating operation of the hypothetical small town water supply described previously, and illustrated in Figure 1. The position of the tank is marked by 'T' and of the pump station by 'P'.

The frequency of flow velocity effects associated with unusually high velocity are illustrated in Figure 8. The frequencies shown are those of high velocity conditions causing sloughing events, not of high velocities themselves. The former are obtained after filtering actual velocities through the algorithm, described in Appendix 5, which takes account of recent velocity and biofilm history in defining an event occurrence. A general trend may be seen for areas downstream of the tank, that is, on the side away from the pump station, to be more prone to high velocity sloughing events. Additionally, peripheral areas, including some dead ends exhibit this effect. The flow in dead end pipes is solely dependent upon the demand at the dead end node, and is not affected by tank water levels and the response of the pump station to system pressures.

The frequency of flow reversals is illustrated on Figure 9. For this figure, a flow reversal is counted as two changes of direction, since reversals always occur in pairs, with the predominant flow direction being restored after a period of reversed flow. In other respects, these frequencies are raw data, unamended in any way. The frequency ranges are intended to give an indication of whether reversals are an everyday occurrence associated with the diurnal demand pattern, or whether they are more the result of variations in the demand pattern which occur at lesser
frequency. The flow reversals indicated for the pipe between the pump station and tank on layout 2 occurred when the tank filled and the pump was temporarily switched off.

Stagnation condition frequencies are plotted on Figure 10. A pipe must have a low velocity for 12 hours before stagnation conditions are established, and these frequencies do not count those qualifying hours. Thus a frequency of 50% indicates that the pipe flow is usually low, with perhaps a short daily high flow that interrupts those conditions and requires a further 12 hour qualifying period, so that only the remaining 12 hours (50%) of the day count as stagnation hours. Dead ends do not show stagnation because the demands specified for the nodes at the ends of the dead ends do not result in a velocity below 0.1 ft/s. Note that occurrence of stagnation is almost symmetrical with respect to the position of the pump station in the two layouts.
Figure 8. Frequency of high pipe velocity, Layouts 1 and 2.
Figure 9. Pipe flow reversal frequencies, Layouts 1 and 2.
Figure 10. Frequency of pipe stagnation conditions, Layouts 1 and 2.
Simulations with externally applied contaminant

Contamination patterns

In the following figures are shown the overall patterns of system contamination detected at nodes and resulting from external contaminant inputs of the kind specified in the figure titles. The frequency of occurrence of a contaminant level above the MCL of 1.0 units was calculated as a percentage of all hourly measurements in a twenty year simulation, and the area of the filled circle at each node represents this frequency. The outer open circle represents an area equivalent to 5% or 20%, as indicated, of total simulation hours. These figures therefore represent for how long each node was contaminated during the simulation, not the concentration of contaminant. These contamination patterns give only a spatial representation of contamination frequency, and do not take account of temporal variations.

Tank contamination events

The contaminant frequency pattern for high level instantaneous tank contamination events is shown on Figure 11, and for a 24 hour low concentration leakage input on Figure 12. Note the influence that pump station position has on these patterns.
Figure 11. High concentration TANK contamination events, Layouts 1 and 2. Frequency of violating concentrations at nodes as percent of total simulation time. Instantaneous contaminant input of 10 units 30 days mean intervals without decay.
Figure 12. TANK leakage contamination events, Layout 1 and 2. Resulting frequency of violating concentrations at nodes as percent of total simulation time. Hourly contaminant input of 0.1 for 1 day at 30 days mean intervals without decay.
Pump station contamination events

Figure 13 illustrates contamination frequency patterns from transient, or short term, contamination at the pump station for both layouts. Figure 14 shows contamination frequency patterns in layout 1 resulting from prolonged pump station contamination events, with and without an applied decay rate. The trends towards higher contamination frequencies downstream of the tank at long input contamination durations (Figure 14a), and for this higher frequency to be reduced under conditions of decay (Figure 14b) was noted for both layouts 1 and 2. Decay had little effect on frequencies resulting from shorter input contamination durations.

Node contamination events

Figure 15 shows contamination frequency patterns resulting from contamination inputs at nodes at a concentration of 1000 units. There is a clear increase in contamination frequency on the side of the tank downstream from the pump station. When decay was applied to these simulations at -0.5, -1.0 and -2.0 per day, the overall frequencies of contamination were reduced, but the pattern was substantially unchanged. The pattern of contamination frequencies for these input conditions, but with a decay rate of -2.0 per day, are shown on Figure 16.

Patterns resulting from decreased contaminant input concentrations for layout 1 are shown on Figure 17 (note that the scale circles are 5%). The difference in contamination frequency from upstream to downstream of the tank is much reduced at the lowest input concentration, and a similar effect was seen on layout 2.
The effect of increasing the duration of contaminant input is shown on Figure 18, (scale circles of 20%). When viewed with Figure 17b, these indicate an increasing trend towards high frequencies downstream of the tank as duration increases.

**Sample Plan Evaluation for externally applied contaminants**

The statistical evaluation of each plan as worst, poor, good or best, the number of violations detected by each plan, and number of actual violations under each regime of external contaminant input are presented in tables 3, 4, and 5. Note that there are 1042 weeks in twenty years, and that some simulations showed the system to be permanently in violation, even though few, or even no, detections were recorded in some cases. Where no entry is made for statistical ranking, no statistical difference was found. The footnotes to Table 3 apply also to Tables 4 and 5.
Figure 13. Transient PUMP STATION contamination events, Layouts 1 and 2. Frequency of violating concentrations at nodes as percent of total simulation time. Contaminant input of 10 units for 1 hour duration at 7 days mean intervals without decay.
Figure 14. Long duration PUMP STATION contamination events, Layout 1. Frequency of violating concentrations at nodes as percent of total simulation time. Contaminant input of 10 units for 16 hour duration at 7 days mean intervals.

a. No decay.  b. decay of -2/day.
Figure 15. Transient high level NODE contamination events, Layouts 1 and 2. Frequency of violating concentrations at nodes as percent of total simulation time. Contaminant input of 1000 units for 1 hour duration at 4 days mean intervals without decay.
Figure 16. Transient high level NODE contamination events with decay, Layouts 1 and 2. Frequency of violating concentrations at nodes as percent of total simulation time. Contaminant input of 1000 units for 1 hour duration at 4 days mean intervals, with decay rate of -2/day.
Figure 17. Transient NODE contamination events at different input concentrations, Layout 1. Frequency of violating concentrations at nodes as percent of total simulation time. Contaminant input for 1 hour duration at 4 days mean intervals without decay.
a. input concentration = 10 units.
b. input concentration = 100 units.
Figure 18. Transient and prolonged NODE contamination events, Layout 1. Frequency of violating concentrations at nodes as percent of total simulation time. Contaminant input of 100 units for 4 and 16 hour durations at 4 days mean intervals without decay. a. duration = 4 hours. b. duration = 16 hours.
Table 3. Actual and detected numbers of violations, and sample plan evaluation for tank contamination events.

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<th>statistical ranking by plan</th>
<th>Actual number of detections by sampling plan</th>
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<td>w w w B w W</td>
<td>13 29 29 87 22 4 24 290</td>
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</table>

x Sudden contaminant input.
y Slow leakage contaminant input over 24 hours
B BEST plan. Significantly better than the most number of other plans
b GOOD plan. Significantly better than at least one other plan and not significantly worse than any other plan
w POOR plan. Significantly worse than at least one other plan
W WORST plan. Significantly worse than the most number of other plans
Table 4. Actual and detected numbers of violations and sample evaluation for pump station contamination events.

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Notes: All input concentrations at pump station were 10 units
Table 5. Actual and detected numbers of violations, and sample evaluation for node contamination events.

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</table>
Sensitivity Analysis

Sensitivity analyses results are shown on Figures 19 through 25. The percentage of violations detected is plotted for each sampling plan. Where applicable, results for layouts 1 and 2 are shown on the same figure.

Tank Contamination

Figure 19 shows percentage detections for two regimes of contamination introduced at the tank for layouts 1 and 2. Detection of slow leakage contamination was lower than for instantaneous contamination at the concentrations used. Plan 6 (sampling all nodes at random times and locations) was inferior in both layouts.

Pump station contamination

Figure 20 shows the effect of increasing the duration of pump station contamination. Detection rates were comparable in both layouts, with plan 1 (totally fixed sampling) and plan 6 (totally random sampling) being inferior at higher duration. Detection rates increased with contaminant input duration in approximately direct proportion.

Figure 21 shows the effect of applying a decay factor to 1 hour duration pump station contamination. For layout 1, the general trend of data is towards reduced percentage detections with increasing decay, with totally random sampling (plan 6) being superior to any other plan. Layout 2 shows a similar but weaker trend, with totally fixed sampling (plan 1) being the worst plan. Figure 22 shows data for 16 hour duration pump station with increasing decay rates applied. Plan 1 is still the worst plan, and plan 6 also becomes inferior for both layouts.
Figure 19. Total detections by each sampling plan for two tank contamination regimes for layouts 1 and 2.

a. Layout 1.
b. Layout 2.
Figure 20. Effect of increasing duration of PUMP STATION contamination, Layouts 1 and 2. Contamination input of 10 units for 1, 4 and 16 hours duration at 7 day mean intervals without decay.
Figure 21. Effect of increasing decay rate of transient PUMP STATION contamination, Layouts 1 and 2. Contamination input of 10 units for 1 hour at 7 day mean intervals and decay rates of 0, -0.5, -1.0 and -2.0 /day.
Figure 22. Effect of increasing decay rate of prolonged PUMP STATION contamination, Layouts 1 and 2. Contamination input of 10 units for 16 hours at 7 day mean intervals and decay rates of 0, -0.5, -1.0 and -2.0 /day.
**Node contamination**

Figure 23 illustrates the effect of increasing duration of contaminant input at nodes. All detection percentages increase at slightly less than in direct proportion to duration. Plans 1 and 6 are inferior to other plans.

Figure 24 illustrates the effect of increasing decay rates on high level (1000 units) node contamination lasting 1 hour. Detection percentages are significantly reduced for both layouts, but the proportional reduction is much greater for layout 1. Plans 1 and 6 demonstrate inferior detection percentages compared to other plans.

Figure 25 shows the effect of increasing node contamination input concentration. A similarity can be noted with the decay result in Figure 24, except that detections increase. Proportional increases are much larger for layout 1 than layout 2, and plans 1 and 6 are inferior to other plans.
Figure 23. Effect of increasing duration of NODE contamination, Layout 1. Contamination input of 100 units for 1, 4 and 16 hours duration at 4 day mean intervals without decay.
Figure 24. Effect of increasing decay rate of transient high level NODE contamination, Layouts 1 and 2. Contamination input of 1000 units at 4 day mean intervals with decay rates of 0, -0.5, -1.0 and -2.0/day.
Figure 25. Effect of increasing concentration of NODE contamination, Layouts 1 and 2. Contamination inputs of 10, 100, 1000 and 10,000 units for 1 hour duration at 4 day mean intervals without decay.
Hourly variations in node contamination frequencies

The following figures show the average number of contaminated node, as a percentage of all nodes, that were contaminated at each hour during the twenty year simulations, and the percentage of times each of the fixed sampling nodes was contaminated at each hour during a twenty year simulation. For the latter case, the nodes are ordered in the legend according to their distance from the western side of the layout as shown in Figure 1, except for the southernmost sampling node which is listed last.

Pump station contamination

Figure 26 shows the percentage of all nodes contaminated at each hour for each layout during pump station contamination. Note the similarity of variation with time of day, and the association with the system demand curve, shown in Figure 2. Figure 27 shows the number of times each fixed sampling node was contaminated at each hour. Demand clearly has an effect on most nodes, but they are affected differently. The fixed sampling nodes on the side of the pump station away from the tank were less affected, and always had low contamination percentages.
Figure 26. Percentage of all nodes contaminated at each hour due to PUMP STATION contamination. Layouts 1 and 2. Contaminant input of 10 units for 1 hour at 7 day mean intervals.
Figure 27. Percentage of days that fixed sampling nodes were contaminated due to PUMP STATION contamination. Contaminant input of 10 units for 1 hour at 7 day mean intervals.
**Node contamination**

Figure 28 shows the percentage of all nodes contaminated at each hour for node contamination events. There is an apparent association with system demand morning and afternoon peaks for both layouts, but this is less apparent in the case of layout 2.

In Figure 29 the percentage of times fixed sampling nodes are contaminated during contamination inputs at nodes are plotted for layouts 1 and 2. Note a greater association with system demand for some nodes, and very little association for others.

For both node and pump station contamination, note also that time of sampling during the day will markedly affect detection of contamination at a particular fixed node sampling point.
Figure 28. Percentage of all nodes contaminated at each hour due to NODE contamination. Layouts 1 and 2. Contaminant input of 100 units for 1 hour at 4 day mean intervals.
Figure 29. Percentage of days that fixed sampling nodes were contaminated due to NODE contamination, Layouts 1 and 2. Contaminant input of 100 units for 1 hour at 4 day mean intervals.
Simulations with biofilm derived contamination

Contamination patterns

The magnitudes of contamination frequencies and violations due to biofilm events are dependent upon the assumed parameters used during modeling. These results should therefore be accepted as giving comparative data and indications of trends and patterns, rather than as absolute values. These figures may be read together with those presented previously showing system flow characteristics.

Biofilm sloughing

Figure 30 shows the contamination frequency pattern due to biofilm sloughing under unusually high velocity conditions, and in Figure 31 is shown the pattern resulting from flow reversals. There is a tendency for a pattern dependant on system layout as seen in previous non-biofilm contamination patterns, but with some additional high contamination frequency nodes.

Biofilm stagnation

In Figure 32 is shown the pattern resulting from stagnation conditions in pipes. Note the comparatively high frequencies at some nodes adjacent to other nodes with no frequency, and the general lack of symmetry compared to that in Figure 10 which indicated low velocity frequencies.
Figure 30. Biofilm sloughing effects due to high pipe velocities. Layouts 1 and 2.

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Figure 31. Biofilm sloughing effects due to flow reversals. Layouts 1 and 2.
Figure 32. Biofilm effects due to stagnant pipe conditions. Layouts 1 and 2.

133
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<th>Statistical ranking</th>
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<th>total violations</th>
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<td></td>
<td>45 53 47 41 36 44 50</td>
<td>1042</td>
</tr>
</tbody>
</table>

B: BEST plan. Significantly better than the most number of other plans
b: GOOD plan. Significantly better than at least one other plan and not significantly worse than any other plan
w: POOR plan. Significantly worse than at least one other plan
W: WORST plan. Significantly worse than the most number of other plans
Sampling effectiveness for biofilm events

High velocity events

Percentage detections for high velocity induced biofilm contamination for layouts 1 and 2 are shown on Figure 33a. Detections by fixed sampling plans are lower for layout 2 than layout 1, but for most random plans, they are very similar.

Flow reversal events

Figure 33b shows contamination frequencies for flow reversal induced sloughing. For layout 1, all plans are about equally good (no statistical difference between any of them was identified), but for layout 2 the more random plans are better. For layout 2, note that the fixed location sampling plans failed to detect any contamination at all.

Stagnation induced biofilm events

Figure 34 shows the result of contamination frequencies due to stagnation conditions. In both layouts, the totally random sampling of plan 6 was significantly better than all other plans which did about equally poorly. In layout 2, all plans detected about half as many violations as in layout 1.

Sensitivity analysis for biofilm contamination detections

Flow reversals

The sensitivity of flow reversal induced biofilm contamination to increasing decay rates is shown in Figure 35. The general trend is for detections to decrease somewhat, but the pattern is erratic.
Figure 33. Effect of different layouts on high level biofilm contamination due to abrupt flow variations.

a. High velocity induced sloughing.

b. Flow reversal induced sloughing.
Figure 34. Effect of different layouts on low level biofilm contamination due to stagnant pipe flows.
Figure 35. Effect of decay on flow reversal induced biofilm effects, Layout 1.
DISCUSSION

CONSTITUENT TRACKING

Plug Formation

Maximum Decay Between Timesteps

Figures 4a, 4b, and 5a show the plug concentration profile resulting from three simulations under identical conditions except for a variation in the maximum allowable percent decay per timestep (PCDKDT) between the formation of successive plugs. Figure 4a shows the concentration profile obtained when PCDKDT equals 10%. Although up to 24 plugs are available, only eight are used.

It is useful to consider the mechanisms that control the formation of plugs in order to specify appropriate model parameters to achieve adequate resolution without excessive computation. Referring to Figure 4a, the seven plugs nearest the downstream end are the same length of 835 feet, each representing the travel time of about 15 minutes during which 10% decay of the constituent occurs at the rate of -10 per day. The youngest plug, at the upstream end of the pipe, has a shorter length of 153 feet because this value of PCDKDT gives a timestep which is not evenly divisible into the travel time for the pipe. Had the simulation continued, the next new plug formed would have had a length equal to the shorter of times derived from the arrival of the penultimate plug at the downstream node, and the PCDKDT value controlling the formation of new plugs.

In this case, the two times are theoretically identical, since the length of the downstream plug was itself originally defined by PCDKDT, but round-off may result in a very small difference as calculated by the computer. If the time for the

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downstream plug to enter the node is the longer, then a very short residual length will be left in the pipe as a downstream plug. This short plug will control the length of the next timestep which will therefore also be very short, and will result in the concurrent formation of a very short plug at the upstream end, which will eventually cause the formation of more short plugs. The preferential removal of short plugs by consolidation will help to keep the problem in check.

Figure 4b shows the result of reducing PCDKDT to 5%. There are now fifteen plugs, each about half as long as those in Figure 4a, and producing a more acceptable concentration profile. Figure 5a shows the result of reducing PCDKDT to 1%. The plugs are now much reduced in length, offering the potential for a much smoother concentration profile. However, the number of plugs produced tends to exceed the number allowable for the pipe, invoking plug consolidation to keep within the allowable limit.

Consolidation occurs where absolute differences in concentrations between adjacent plugs are smallest, i.e., where the profile tends to flatten out, in this case at the downstream end of the pipe where the water is oldest. The difference between the concentration of a consolidated plug and its adjacent plugs tends to be larger than the difference between unconsolidated plugs, so that a consolidated plug will tend not to be consolidated a second time. This is an advantage of basing consolidation on comparative concentrations rather than on plug lengths. In this simulation, the next consolidation is between plugs further upstream. Thus consolidation gradually works its way up the pipe against the direction of flow, and in this case can be seen to have backed up to a point about 2100 feet from the
upstream end. The consolidated plugs now produce concentration steps that are not much smoother than those achieved with the 5% value of PCDKDT, and have been obtained at the expense of a substantial amount of additional computation. However, the result is no worse than that using a value of PCDKDT of 5%.

These results are for a single pipe under constant hydraulic conditions. When flow velocities and directions are changing in a pipe network, and when timesteps are controlled by the movement of plugs in other pipes, the tendency will be for timesteps to be short regardless of the value of PCDKDT which is, after all, a default value.

Allowable number of plugs

Figure 5a and b, shows the result of reducing the allowable number of plugs to ten while maintaining a PCDKDT value of 1%. In this case, even more consolidations have been required to maintain the smaller number of plugs, and have produced a non-uniform profile which is worse than that when PCDKDT had a larger value.

It appears therefore that the value of PCDKDT and of the allowable number of plugs per pipe should be compatible such that the number of plugs that would be formed for a given value of PCDKDT is about the same as the number of plugs allowed. However, the optimal values are likely to be very site specific, since the length and travel times for pipes may be highly variable in real systems, both geographically, and, in the case of travel times, temporally. Since the length of existing plugs and their arrival at nodes controls timesteps once the system is
contaminated, a large number of plugs per pipe will tend be formed regardless of
the value of PCDKDT.

Plug consolidation

Consolidation is a time-consuming and computationally intensive procedure,
which is a disadvantage of constituent representation using plugs. A larger
allowable number of plugs per pipe can be an advantage, since fewer consolidations
will be required to keep within the maximum number per pipe. Against this must
be weighed the additional memory requirements and computational effort required
to mix and decay larger plug numbers, and the shorter time steps resulting from
the shorter lengths of the plugs.

Concentrations at nodes

At the nodes, the error in the recorded concentration is equal to the
difference between the average plug concentration and the theoretical
concentration calculated by analysis. When a plug first arrives at a node, its average
concentration is higher than the theoretical value. As the back end of the plug
reaches the node, its average concentration (which has continued to decay as it
enters the node) is below the theoretical concentration. The maximum error is
very nearly proportional to the length of the downstream plug, so that shorter plugs
will generally give smaller errors in node concentrations. Observed concentrations
at nodes could be made more accurate by increasing the number of plugs.

Figure 6a, b, and c show the concentrations that would be observed by
continuous recording at nodes 2, 3 and 4 of the pipe system shown in Figure 3. The
saw-tooth profile illustrates the decaying concentration of the plug as it passes
through the node. Nodes 3 and 4 show longer saw-tooth patterns for two reasons. Firstly, the flow velocity downstream from node 2 is half that of the first pipe and plugs therefore take twice as long to pass through these nodes. Secondly, the last pipe is the longest, and therefore the average length of a plug must be longer than in the other pipes. This is achieved by consolidation of the shorter plugs formed in the upstream pipes. Since downstream plugs tend to be consolidated first, only three separate plugs in the 10 plug simulation (figure 6a) actually reach the downstream end of the system during the 250 minute period illustrated. With 20 plugs, the observed error in concentration is much reduced, but little advantage is obtained by using 25 plugs.

Model Verification

The constituent transport model can provide good representations of constituent transport processes and decay. The accuracy of the concentration profile desired should be considered in light of the accuracy with which the various model parameters are known, and the resources available to the modeler. Disproportionate accuracy in one area is not productive, and for routine modeling and system evaluation, a PCDKDT value of 5% or less, and about 10 plugs per pipe are probably adequate. Where higher accuracy is desired, PCDKDT can be set to minimize concentration differences between plugs to a level that is considered significant in a particular context. The allowable number of plugs per pipe can be set as high as computer memory and model run times allow.
Twenty Year Simulations

Figure 7 shows that reasonable stability of percentage detections by each plan is achieved within twenty years for a moderate level of contamination introduced at nodes. Other plots, not presented, for higher levels of contamination reached stability sooner, sometimes within five years. Plots for lower levels of contamination were fairly stable after twenty years, but the overall low level of detections resulted in poor separation of plots for each plan. In general, a twenty year simulation was adequate for the purposes of this study. It might be said that differences not apparent after twenty years are probably not useful differences anyway. A water distribution system's layout, demand pattern and other features will almost certainly change significantly within a period shorter than twenty years, requiring that the benefit of a plan be apparent in a shorter period of time, especially if its application requires additional resources over other plans.

SYSTEM HYDRAULICS

High velocities

Figure 8 indicates the frequency of unusually high velocities defined according to the scheme described in Appendix 5. Briefly stated, an unusually high velocity in a pipe is one which is much above the usual range of velocities for that pipe. Note that the criterion is based on relative velocity, not on absolute velocities, so that a pipe with low average flows may be just as subject to a high velocity event as a pipe with high average flows.
Velocity variations result from the underlying diurnal demand variation, the random variations to that demand, and the level of the tank which is a function of the demand variations over the previous day or so. The range of high to low demand over the day is about 5.3 on average (from peak demand factor of 1.7 to low demand factor of 0.32), but can be as much as 8.6 as a result of the randomization of demands (applying the upper beta bound of 1.3 to the high demand flow and the lower beta bound of 0.8 to the low demand flow). The pump flows vary somewhat as system pressures rise and fall, but are damped by the in- and out-flows at the tank which mainly compensate for variations in demand. Thus, pipes between the pump station and the tank tend to have comparatively constant flows, whereas pipes outside this area have more variable flows. Therefore pipes which supply only demand nodes and which are outside the damping influence of the pump station and tank area suffer fairly frequent high velocity events.

**Flow reversals**

The frequencies of flow reversals in each layout are shown in Figure 9. Flow reversals occur in pipes near the tank as the tank alternately fills and empties. Examination of the verbose model output, not presented here, showed that flow reversals near the pump station occur when unusually large swings in demand result in large differences in pressure between the pump and the tank, forcing water to take a very circuitous route from the pump to the tank. They also occur where alternative routes from a source to a demand node are available, so that small differences in pressures alternately favor one route over another. Another
cause of flow reversals is the diurnal demand variation that causes nodes in some areas to be supplied alternately from the tank and the pump station.

**Flow stagnation**

The frequencies of flow stagnation conditions are shown in Figure 10. These conditions occur in pipes where low flow conditions have prevailed for at least 12 hours. Pipes with a frequency of over 50% are in fact almost permanently stagnant. It is only the occasional flushing action of a high flow on a high demand day that reduces the figure to below 100%. The dead end pipes have a nodal demand at their extremities which is always high enough during peak demand hours to disturb the establishment of stagnant conditions.

It is noticeable that most of the stagnant pipes are oriented perpendicular to the general direction of flow away from the pump and tank, and are located in peripheral areas of the system where flows are expected to be lower than usual. However, there are a few pipes close to the pump station and tank, and this was an unexpected result. A stagnant pipe obviously does not carry much flow, and would be a good candidate for exclusion if the system is to be further skeletonized for solely hydraulic modeling. However, exclusion of a pipe which may store or be a source of contamination may not be acceptable for contaminant transport modeling.

One pipe, near the tank in layout 1, is subject to all three effects of high velocity, flow reversals and stagnation. This is explained by the fact that this pipe's proximity to the tank results in flow reversals, but, regardless of direction, these flows are usually small. Therefore the average flow in this pipe is small, so that
occasional excessive demand conditions can cause higher flows sufficient to qualify as high velocity events for that pipe, while the overall flow regime is still stagnant.

Stagnant areas in distribution systems are frequently considered to be the source of bacterial regrowth and to require special attention. The model has successfully identified these pipes, providing a means for operational personnel to isolate areas in real systems that may require additional sampling and flushing.

**SAMPLING PLAN EVALUATION**

**Introduction**

The sampling plans applied provide a range of random and deterministic alternatives that might actually be used by a utility constrained by practical considerations, together with some plans defined from a more theoretical viewpoint to assess general approaches to sampling, such as the degree of randomness in sample timing and location. Real sampling schemes can be quite flexible regarding the location of sampling. A larger constraint is the timing of sampling because of practicalities of access to premises and availability of staff outside normal working hours, and the need to deliver samples promptly to a laboratory for analysis.

Plan 1 which samples fixed nodes once weekly satisfies the minimum requirements of the regulations as applied to some small systems. Plan 2, which samples a fixed node every day, when compared to plan 1 shows the effect of spreading samples out over the monitoring period. Plans 3, 6 and 7, which sample different regimes of randomly chosen nodes every morning will, over the long term, sample all nodes with equal frequency. Differences in the detection rates shown by plans 3 or 7 compared to plan 2 should indicate, respectively, the effect of
sampling all nodes randomly or systematically compared to sampling representative fixed nodes. Any difference between plan 3, and plan 6 which samples at totally random times and locations, should be a result of randomizing sample timing for the latter. Any difference between plans 3 and 7 should be a result of the more disciplined sample location under plan 7 which separates consecutive samples by location. Any difference between plans 6 and 7 should also result from the separation of consecutive samples by location and the effect of random timing in plan 6 which could result in there being extended periods when no samples are taken. Over a long time span, random choice of sample node location should ensure that every node is represented, and random timing will avoid ‘time of day’ effects that may bias results when representative locations are sampled at fixed times. Plan 4 illustrates the effect of including tank dipping compared to plan 3, and plan 5 shows the effect of sampling in proportion to population compared to plan 3 which samples all nodes equally. For small systems, plans 1 and 2 are probably most representative of actual sampling plans used.

**Externally introduced contamination**

**Contamination patterns**

The sampling plans were applied in the context of specific regimes of contamination, and the resulting contamination patterns are the cumulative results of twenty years of sampling. They do not indicate the pattern of contamination that would result from an individual contamination event, whose timing, and in the case of node contamination, location, may result in a contamination pattern distinctly different from the average. Those contamination patterns presented
previously (Figures 11-18) which are not specific to one hour of the day, demonstrate which areas of the distribution system become contaminated most frequently over twenty years. These patterns do not contain any information regarding the time of contamination or its intensity, but do clearly identify major problem areas in a system, and indicate where poor quality water is most likely to be drawn by consumers. The contamination patterns indicate in several cases that the locations of most frequent contamination are not necessarily near the source of contamination.

Statistical comparisons

The non-parametric multiple comparison was found to order plans very similarly to the order observed on twenty year plots. The results for simulation number 62, presented in Table 5, on Figure 7 and in the explanation of the statistical procedure in Appendix 7 may be compared.

The multiple comparison test used to evaluate the sampling plans ranks each plan in order of goodness without regard to how much better one scheme is over another. The statistical rankings presented in tables 3, 4, 5 and 6 are based on this procedure. An ordering of plans based on the total number of detections by each plan over the twenty year simulation results in a similar, but not identical ranking. One unusually good year for a plan may increase its count of detections sufficiently to give it the appearance of being the best plan on the basis of total number of detections. But the statistical test accounts only for the rank of a plan, so that a very good year is no better than a year in which the plan is only just better than other plans. Simulation 692 in Table 6 offers an example, in that Plan 7 has more

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total detections than Plan 5, but on the basis of comparisons with all plans for each of twenty years, Plan 5 receives a higher rating.

The multiple comparison procedure is statistically defensible, while a comparison based on total detections is subjective. However, when read together, the total number of detections may be used to confirm the multiple comparison results, and to strengthen conclusions in cases where only limited statistical differences can be identified.

Once weekly-sampling

An initial review of the data in tables 3, 4, and 5 shows that once-weekly sampling as performed by plan 1 was the worst plan in most cases, and at best a poor scheme when ranked at all. In no case did it offer any indication of being an acceptable sampling plan compared to others tested, and no other plan demonstrated such a consistent result. The residence time of contamination in the pipe system is from about an hour, when contamination occurs near a peripheral node, to about a week for the heavier cases of contamination. Therefore, sampling only once a week allows much contamination to occur undetected during the unsampled periods. While the regulations state that samples should be spread out during the monitoring period, States have the final discretion to enforce this provision. The results of these simulations suggest that any sampling plan will be more effective if based on a requirement to spread samples out throughout the monitoring period.
Tank Contamination Events

Contamination patterns

For contamination introduced into the tank, either as a sudden large dose or as a slow input over 24 hours, for both layouts, the area of the system downstream of the tank with respect to the pump station is always the most frequently contaminated, and most of the area upstream of the tank is never contaminated (figures 11 and 12). The relative position of the tank and pump station therefore appears to be an important factor in the distribution of contaminant. The concentration at the node at the tank indicated the concentration of water entering and leaving the tank, not the concentration of water in the tank. This node is contaminated more frequently than any other node, (with a few exceptions for instantaneous tank contamination in layout 2), and for sampling spread throughout the day is the best node at which to detect contamination arising in the tank.

Sometimes contaminated water that previously left the tank will re-enter the tank when it starts to re-fill, although much of this water will have been mixed with new clean water from the pump station, and its concentration is much diluted. If sample timing is a factor, the best time to sample this node would be when the tank is emptying. Allowing for the travel time of contaminated water leaving the tank to reach other parts of the system, the best time to sample the other more frequently contaminated nodes is also while the tank is emptying, and possibly for a few hours afterwards.
Examination of the two layouts for each contamination regime, shows little change in the contamination pattern of layout 1, but much higher frequencies for the instantaneous input. The leakage contamination regime builds up the concentration in the tank very slowly over 24 hours. The larger variation in tank water level for layout 2 provides more rapid dilution, and frequently prevents the concentration from reaching the MCL. For layout 1, the smaller tank level variation is unable to dilute the contamination to the same degree.

**Sampling plans**

Reference to Table 3 indicates that the overall best plan statistically for detecting tank contamination is plan 4, which takes all samples at 8.30 am and which includes a weekly sample taken directly from the tank. Plan 2, which also takes samples at 8.30 am, and which takes a fixed node sample from the system node nearest the tank once a week is also one of the better plans except for leakage contamination in layout 2. The choice of a fixed node sampling point near the tank is instrumental in the relative success of plan 2.

The other plans that sample random nodes in the morning when the tank is emptying show mixed results. Plan 6 which takes samples at totally random times and locations is consistently the worst plan. This is because a significant number of samples are taken when the tank is filling and no contamination can then enter the system. The somewhat better performance of plan 3 which samples random nodes at 8:30 am, supports the AWWA recommendation (AWWA, 1991) to sample near tanks when they are emptying.
The statistical results are generally borne out by the counts of total detections shown in table 3 and presented on Figure 19.

**Pump Station Contamination Events**

**Contamination patterns**

For contamination introduced at the pump station at a concentration of 10 units and duration of 1 hour, the typical regime presented on Figure 13 shows widespread contamination of the whole system. For both layouts, there is a tendency for increased frequency of contamination in the area of the system downstream of the pump station from the tank. There is also a tendency for peripheral nodes to be more frequently contaminated. This may be explained by the fact that flow velocities in peripheral areas tend to be slower, so that a contaminant plug takes longer to pass through a peripheral node, and also that contamination at points far from the contaminant source can receive contamination from the same event by different paths with different times of travel, dispersing the contaminant longitudinally and extending the node’s exposure time to that event.

When the duration of pump station contamination is increased, some contamination may enter the tank and subsequently produce contamination patterns with features of tank contamination. This is seen in Figure 14 which resembles a combination of the patterns for layout 1 on figures 13 and 29. For shorter duration contamination, a different explanation is offered in the next section.

When a decay rate is applied, the contaminant decays while stored in the tank, so that when it returns to the system it is more likely to be below the MCL
and is less frequently detected. This accounts for the reduced detection frequencies downstream of the tank in Figure 14b compared to 14a.

**Time variations**

In Figure 36 is shown a series of contamination patterns for layout 1 at three hour intervals throughout the day (Figure 13, layout 1 is the pattern for combined frequencies of the same contamination conditions). These are patterns for the specific hours shown averaged over a twenty year simulation of pump station contamination of 10 units applied for one hour at 7 day mean intervals.

At this level of contamination, as opposed to the higher levels discussed previously, the water in the tank is not usually raised above the MCL. This is because contamination at 10 units is diluted when it enters the tank to a level which is usually below the MCL. Therefore, while the tank is emptying, the tank node cannot become violated, while other nodes in the system may still be violated by new pump station contamination events.
Figure 36. Transient PUMP STATION contamination events, Layout 1. Frequency of violating concentrations at nodes as percent of total simulation time at 3 hour intervals during the day. Contaminant input of 10 units concentration for 1 hour duration at 7 days mean intervals without decay.
The higher frequencies of contamination noted in the east end of the system occur mostly when the tank is filling, and are a result of the dispersion noted previously. Although the tendency is for higher frequencies of contamination in the east end of the system as shown in Figure 13, the period of high system demand from 6:00 am to noon results in large outflows from the tank. Since tank water is generally of comparatively low concentration, the outflow displaces more highly contaminated water coming from the pump station. The tank is emptying for most of the last 18 hours of the day, hence the tank node reflects the concentration of water leaving the tank and appears to be uncontaminated. However, outflows in the afternoon are smaller, and the flushing action at that time is therefore less marked.

These effects are also illustrated on Figure 26 where it is seen to be more marked for layout 1 than layout 2. The variation in tank level is smaller in layout 1 than layout 2 so that in- out-flows for the tank in layout 1 are smaller than in layout 2. If the inflow is contaminated, the tank concentration is raised less, and is less able to contaminate the system on outflow, but if the tank is contaminated already and the inflow is clean, the tank contamination is diluted more slowly and so lasts longer. The higher level variation in layout 2 results in greater tank concentration initially, but faster dilution afterwards.

Sampling plans

The comparisons of sampling plans of contamination introduced at the pump station at a concentration of 10 units and duration of 1 hour were inconclusive. In order to improve the strength of conclusion, and also to see if the result was
reproducible, five duplicate simulations were made for both layouts. These are numbered 430 to 434 for layout 1 and 420-424 for layout 2 in Table 4. Results for layout 2 were substantially reproduced in each of the five simulations. Plan 1 was always the worst, and plan 6 was always the best, but with other plans showing a generally good or joint best rating. Plan 4 which samples the tank node, showed mixed results, and recorded a generally lower number of detections.

For layout 1, the results from the five separate 20 year simulations were combined to give a 100 year simulation, designated simulation number 430-4. Plan 6 is the best, and plan 1 the worst, with other plans still showing mixed results. Although a statistical difference exists to show that plan 6 is statistically significantly better than plan 1, the need for a 100 year simulation to demonstrate this facts suggests that this difference may not be of much practical value.

The random timing of plan 6 is its distinguishing feature. Contamination coming from the pump station may be diluted by clean water coming from the tank. Samples taken when the tank is emptying will tend to be more diluted in those areas served by the tank. Taking samples at random times transfers some of these samples to times when the tank is not diluting the contaminant, and resulting in higher detection rates. The average number of nodes contaminated at each hour during the day for layouts 1 and 2 subject to pump station contamination are shown on Figure 26. The early morning period from about 8 am to 12 noon, when the tank is emptying at the fastest rate, is probably a poor time to detect contamination. For samples taken during working hours, the afternoon would be a better sampling time. Figure 27 are shows the percentages of days that each of
the fixed nodes is contaminated at each hour. Some nodes, for example node 31 in layout 2, are the best nodes for sampling at some times of the day, and the worst nodes at other times. It is clearly not merely sufficient to choose good sampling locations, but to ensure that good sampling times are chosen too.

When the duration of a pump station contamination event is increased, the ranking of sampling plans changes. This is illustrated in Figure 20. A short duration pump station contamination event may or may not occur when the tank is filling, and even if it does, the short duration does not result in a large increase in tank contamination. Even if it is increased above the MCL, it is rapidly diluted when returned to the system, and is more diluted the next day when the tank fills again with clean water, assuming that a second contamination event does not occur at this time. When the duration is increased, the chance of an event coincident with tank filling is increased, as is the duration of contaminated water inflow, both of which lead to higher tank concentrations above the MCL. Plan 6 becomes the worst plan (with plan 1), and plans 2, 3, 4, 5 and 7, which sample when the tank is emptying, all become the better or best plans for both layouts. The overall rate of detection is significantly increased for all plans as event duration increases.

When a decay rate is applied to pump station contamination, Figure 31 shows a small overall decline in detection rates. This is to be expected, since the contaminant concentration is more rapidly reduced to a level below the MCL. Decay does not appreciably alter the identification of best and worst sampling plans.

No benefit over other plans was apparent in using plan 5 which preferentially samples areas of high demand.

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**Node Contamination Events**

**Contamination patterns**

Contamination patterns for 1 hour duration node contamination events at 1000 units input concentration for layouts 1 and 2 are presented on Figure 15. Note that node contamination is randomly applied over the whole system, but that in many cases the frequency of violating concentrations is concentrated in specific areas of the system. High frequencies of contamination are restricted mostly to areas downstream of the tank in a manner similar to tank contamination events. This indicates that under these conditions, a sufficient amount of contaminant enters the tank to produce a pattern similar to that observed for direct tank contamination. When contamination concentrations are reduced, the effect of interception of contaminant in the tank is retained for 100 units concentration inputs, but virtually disappears for the 10 units input, as illustrated in Figure 17 for layout 1 (note that scale circles on this figure are 5%). The effect is similar to the distinction between long and short duration pump station contamination. High node contamination concentrations are able to raise the concentration in the tank above the MCL, while low concentrations are not. The same effect is seen in Figure 18 when the duration of input contamination at 100 units concentration is increased to 4 and 16 hours.

The effect of decay is illustrated on Figure 16 which should be compared to Figure 15 for the case of no decay. The frequency of contamination downstream of the tank is reduced, reflecting the holding time in the tank during which decay continues.
Some peripheral nodes on the side of the system opposite to the tank also show a tendency for higher contamination frequencies. This is a manifestation of the 'coverage' effect noted by Lee *et al.*, (1991). Water that reaches peripheral nodes has passed through more intermediate nodes with the potential to pick up more contamination.

**Time variations**

On Figure 37 are shown contamination frequencies at three hours intervals throughout the day, resulting from node contamination of layout 1 at 100 units concentration at 4 day mean intervals. Node contamination occurs randomly, so contamination frequencies at individual nodes are, for most of the day, comparatively low. However, water takes many paths from the pump station to the tank and may become contaminated at any one of them while the tank is filling. The tank therefore frequently becomes contaminated, and can contaminate all the nodes it serves while it is emptying.
Figure 37. Transient NODE contamination events, Layout 1. Frequency of violating concentrations at nodes as percent of total simulation time at 3 hour intervals during the day. Contaminant input of 100 units for 1 hour duration at 4 days mean intervals without decay.
The effect is much less for layout 2 because the tank is much nearer the pump station, and the potential for contamination during travel is reduced. Some water suppliers use a dedicated trunk main to connect sources to storage tanks. This practice would help to prevent the widespread system contamination noted here.

Figure 28 shows the frequency of all nodes contaminated at each hour of the day for a 100 units node contamination of 1 hour duration. The early morning is seen to be the best sampling time, but even so, a difference of one hour could seriously affect sampling results. As for pump station contamination, the effect is more marked for layout 1 than layout 2. Figure 29 shows the frequency of contamination at the fixed node sampling points due to the above node contamination events. In layout 1, only those nodes within the sphere of influence of the tank are really worthwhile sampling points for this regime of contamination. For layout 2, the same comment applies.

Sampling Plans

For both layouts, the detection rates for lower levels of contamination do not provide enough data to identify statistically significant differences between plans. Examination of table 3 shows that neither plan 1 (early morning, fixed node, once-weekly sampling) nor plan 6 (totally random sampling) attains a good or best rating for node contamination for any of the contamination regimes tested. Plan 2, which samples fixed nodes at 9:00 am every day is consistently the best plan for both layouts under all contamination conditions. Plan 4, which samples random nodes in the early morning plus one sample dipped from the tank each week, is
consistently either the best or one of the better plans for layout 1, and for layout 2 where a statistically significant difference exists. Its effectiveness for layout 2 is reduced in the case of decay, for reasons previously discussed.

For layout 1, randomizing the location of sampled nodes for plan 3 reduces its effectiveness compared to plan 2, but has less effect on layout 2 where significant differences between plans exist. This may be because more nodes are contaminated on layout 2, offering a better chance that a randomly chosen node will fall in the violated area. The same effect is seen with plan 7, but no benefit of separating the location of consecutively sampled nodes is apparent.

The effect of increasing contamination duration for layout 1 (Figure 23) is to increase the percentage of detections without altering the relative effectiveness of sampling plans. For both layouts 1 and 2, applying an increasing decay rate (Figure 24) decreases detection rates, and increasing contamination concentration (Figure 25) increases detection rates without altering the relative effectiveness of the sampling plans. In both these cases, the sensitivity of layout 1 is about one and one half times that of layout 2.

No benefit over other plans was apparent in using plan 5 which preferentially samples areas of high demand.

**Biofilm effects**

**Introduction**

The attempt to model biofilm effects was simplified to relate those effects to assumed relationships between the hydraulic conditions in the system and detachment of cells from the biofilm. No attempt was made to simulate
relationships of various parameters such as disinfectant or assimilable carbon concentrations with biofilm growth kinetics, neither was any attempt made to account for physical parameters such as temperature. The results of this study must be viewed in light of this simplification.

The parameters describing coliform kinetics and their populations in water distribution systems are highly variable. For simplicity, no attempt was made to add any stochastic variation to these parameters. The values used in this study are typical of values found in the literature, but many of those values are themselves the result of investigation in response to problem situations or are pilot scale laboratory results, neither of which are necessarily typical of real distribution system values under normal conditions. The results of this study are therefore interpreted as being indicative of trends that result from certain types of biofilm effects. In this respect, contamination patterns and relative frequencies may be useful in identifying potential problem areas of a system. However, absolute numbers may not be useful until substantial further information is available for calibration of the model.

The results of sampling plans applied to biofilm effects are presented in Table 6.

**High Velocity Sloughing**

**Contamination patterns**

The frequencies of contamination from high velocity sloughing are shown on Figure 30, and Figure 8 shows the underlying hydraulic conditions. About 50% more pipes are subject to high velocity sloughing in layout 2 than in layout 1. In
both layouts, the frequency of contamination detection is largely coincident with the pipes with high frequency of high velocities. This suggests that for these simulations, contamination did not enter the tank in sufficient quantity to cause effects similar to tank contamination.

**Sampling plans**

Detection of high velocity biofilm effects is poor by any sampling scheme, and numbers of detections in layout 2 are much less than in layout 1, despite there being more pipes affected in layout 2. For layout 1, plan 1 is identified as the best and plan 6 as the worst, while for layout 2, no statistical differences exist. Examination of the total number of detections by each plan suggests that plan 6 is by far the worst plan for both layouts. High velocity effects could be expected when high velocities occur during the morning peak demand period, accounting for the better detections by plans that take samples at 9:00 am. At this time also, the tank is emptying, preventing any contamination from entering the tank.

**Flow Reversal Effects**

**Contamination patterns**

Contamination patterns for flow reversal effects are shown on Figure 31, and the underlying velocity effects are shown on Figure 9. The involvement of the tank in causing flow reversals results in the tank becoming contaminated. The patterns of contamination show similarities with those for tank contamination, with much of the contamination being in areas distant from the source of contamination. This is particularly clear in layout 1. However, some local effects close to pipes with frequent flow reversals are also seen.
Sampling Plans

Flow reversals occur principally as demand varies causing the tank to change from filling to emptying or *vice versa*. Reference to Figure 38 shows that this occurs at about 6 am and again at about 9 pm. The reversal at 9 pm is followed by a 9 hour period of tank filling during which time any contamination in the vicinity of the tank will enter the tank. This would be expected to be best detected by morning sampling when the tank is emptying, so that sample timing is important. This is the case with layout 1, if numbers of detections are considered in the absence of statistical significance, where randomly timed samples are least effective. For layout 2 however, the tank and pump are so close together, that very few pipes near the tank experience a flow reversal, and resulting tank contamination is not dominant. Local contamination of pipes elsewhere is more frequent, so that sampling location becomes more important than sample timing. For layout 2 therefore the better plans are those that sample all nodes rather than fixed nodes.

In layout 2, neither plan that relies on fixed node samples detects any violations. This points out the need for careful selection of fixed node sampling points, and the possibility that engineering judgement may not result in a good choice for some types of contamination.
Figure 38. Reservoir water levels during a typical diurnal cycle.
Stagnation effects

Contamination patterns

The contamination patterns for stagnation conditions are shown in Figure 32, and the underlying hydraulic conditions are shown in Figure 10. The contamination is detected mostly in the locality where the effect occurs. Some nodes adjacent to frequently stagnant pipes do not show a high frequency of contamination. This is because the low flow from the stagnant pipe is mixed with a high flow of clean water from an intersecting pipe at the node, so that the resulting mixed concentration is below the MCL. To detect this contamination, a node could be inserted in the middle of the pipe, but the locations of such pipes would have to be first identified in trial simulations.

Sampling Plans

Detectations of contamination due to stagnant conditions were generally low, but for each layout, plan 6 was clearly the best with statistical significance. It appears that the fixed nodes were again poor choices for detecting this type of contamination, and samples taken at random nodes but at fixed times offered little improvement. The nature of stagnation appears to be such that the contamination it causes can occur at any time, so that random timing is required.

Erosion

Erosion of the biofilm occurs as a more or less continuous process at normal velocities (van der Wende and Characklis, 1988). There does not appear to be any selective criterion that favors some pipes over others apart, possibly, from proximity to the pump station which may control nutrient and disinfectant fluxes. Initial
attempts to model this effect by adding a constant addition of contaminant to all pipes in the system resulted in a permanently contaminated system, which offered no method of evaluating sampling plans. An alternative approach, which limited the occurrence of erosion to periods of above average, but not extreme, velocities tended to favor those pipes with more variable flows as being more heavily contaminated, which was not a realistic representation of the process. In fact, erosion appears to result in 'episodic' contamination which occurs system-wide for prolonged periods, as noted by Lowther and Moser (1984). In that case, any sampling plan should detect it, since contamination would then not be dependant on timing or location of samples.

Concluding Comments

This study has evaluated several sampling plans chosen \textit{a priori} to be representative of various sampling options. A technique has been demonstrated that can evaluate these plans and indicate those which are superior or inferior under certain circumstances. It is possible that there are much better sampling plans available, and improved sampling plans might be identified using the information provided in a study of this kind. Formal optimization procedures may offer even better results for a specific set of inputs.

Recommendations offered by the water industry on the definition of sampling plans mostly emphasize the location of sampling points. Less emphasis is placed on the timing of samples or knowledge of the source and intensity of likely contamination, yet these are, from the results of this study, factors requiring equal consideration.
Low frequencies of contamination can still result in actual violation of the system from a strict regulatory perspective. Only one node need be contaminated at a level above the MCL for one hour per monitoring period for the model to place the whole system in violation. Such sparsely occurring contamination is rarely found by any sampling scheme. This explains why, for some biofilm and low level externally applied contamination, the detection rate is very low despite frequent, and in some cases a permanent, state of violation for the system.

The existence of a tank in the system has considerable effect on the disposition of contamination from any source. By storing contamination as the tank fills, it affects both the time and location at which contamination is most effectively detected. The position of the tank within the system, and relative to the pump station, is also a factor, as shown by the different results for the two layouts. Note that moving the tank from one side of the system to the other while the pump station position was fixed would have achieved the same results. Thus, changes in the location, and probably size, of both storage and sources is likely to alter the effectiveness of a sampling plan.

For high concentration and/or long duration events, the effect of the tank is to cause similarities in the long term patterns of contamination. Contamination detected system wide is most likely to have originated at the pump station. Contamination detected in the area served only by the tank may result from either tank contamination or node contamination. To distinguish between pump station and other contamination, it is necessary to sample system wide in order to establish the extent of contamination. Thus, the requirements of the regulations for
representative sampling are supported if some indication of the source of contamination is required, but they may not be sufficient if a relatively small number of fixed sampling points is accepted as being representative.

In the case of short duration low concentration events, widespread contamination of the system points to the pump station as the source, while limitation of contamination to the area served by the tank suggests the tank as the source of contamination. For low concentration contamination at nodes, the pattern of cumulative contamination covers the whole system. Thus interpretation of sampling results in an attempt to locate the source of contamination may require knowledge of the intensity of contamination in addition to the overall pattern, but this information is not available from presence/absence testing.

Identification of the location of an individual node contamination event was not investigated, but consideration of the number of pipes subject to flow reversals suggests that the recommended procedure of taking up- and down-stream samples may be misleading. This is especially true if repeat samples are taken at a different time of day when the flow regime is different from the time at which original contaminated samples were taken.

The choice of fixed nodes for sampling may be critical for detecting some types of contamination. Selection on the basis of 'engineering judgement' may not provide a good set of sampling points. It is quite likely that different engineers would select different fixed sampling points as being representative of the system, and with very different results regarding the level of contamination of the system.
Biofilm contamination patterns tend to be more patchy than those from externally applied contamination. This agrees with experience that records coliform detections as being erratic. The model can identify hot-spots related to certain hydraulic conditions, an if these are related to coliform events, the model offers a possible explanation and means to improve detection of coliform occurrence.

Overall conclusions on sampling approaches are that sampling on only one day in a monitoring period is always inferior to spreading samples out over time. The choice of fixed nodes as sampling points can be very instrumental in the success or failure of a sampling scheme. Well chosen fixed nodes that frequently intercept flows of contaminated water can be comparatively effective, but a good choice of fixed sampling nodes must be based on some informed knowledge. A storage tank can intercept a significant amount of contaminated water in a system, and influences the effectiveness of sampling plans. Sampling at fixed times when the tank is emptying can be very effective if overall contamination levels are high enough to contaminate the tank, allowing for dilution with clean water already in the tank. Randomly timed contamination events that do not contaminate the tank are more effectively detected with randomly timed sampling.

**Limitations and Future Work**

This study has used a small hypothetical distribution system. The results are considered indicative of the effects that could be investigated in real systems. However, real systems may be more complex and larger than the system used in this study. Temporal effects are likely to be different in larger systems, especially if the hydraulic retention time is much longer than a day, in which case the effect
of the diurnal demand may be damped out considerably. Application of this approach to a large system would extend its usefulness considerably. Larger systems will also require much greater computer resources to enable effective simulations to be completed in a reasonable time. Optimization of the model, particularly with regard to the amount of storage required for contaminant tracking may be worthwhile.

To provide better insight into biofilm effects, a re-evaluation of existing data, considering flow rate or system demand as a predictive variable for coliform data would be useful. Verification of the temporal patterns could be attempted by taking regular samples from a real system at intervals during the day over an extended time period of many days. It would be necessary to find a heavily contaminated system that will be able to demonstrate such a pattern, if it exists. Additional information is required to identify and verify the relationship of biofilms to easily modeled parameters such as flow.

A comparison between a formal optimization of sampling options and the approach shown here would be useful to see if simulation is preferable in some aspects. This may include the time taken to perform one or other procedure, and the robustness of the result when boundary conditions and stochastic inputs are changed.

**Summary**

This study has demonstrated that long term simulations of small water supply systems are feasible, and that much information on the hydraulics of the system and fate of contaminants can be obtained. Several typical and often
proposed sampling strategies have been tested and the ability to identify advantages of some sampling schemes over others has been demonstrated. The value of this approach lies in the fact that this type of information is, from a practical point of view, impossible to collect by field sampling. The amount of effort required to apply this kind of analysis using computer modeling is considerably less than that required to collect even a fraction of this information in the field.
CONCLUSIONS

This research has produced a number of important results. Foremost among these is that the time of sampling within the diurnal demand pattern of a small water distribution system may be as important as the location at which samples are taken. For small systems, contamination patterns move very rapidly, and the effectiveness of a sampling plan can change from hour to hour. Sampling was always more effective when spread throughout the monitoring period, but no sampling plans was consistently superior to other plans when tested with different contaminant inputs. Overall levels of violation detection were low, leading to the conclusion that many contamination events are not detected.

Another finding was that specific patterns of chronic contamination can be associated with different sources and with different intensities of contamination, offering the possibility of identifying the source and cause of contamination. For such an analysis, contamination concentration information is required, and a presence/absence test is not adequate for this purpose.

It was also found that the dynamic hydraulic model, without any constituent tracking function, can be used to characterize the long term velocity data for individual pipes in the system. Areas of flow reversal, stagnation and pipes subject to occasional very high velocities were identified in unexpected regions of the distribution system as well as regions expected to exhibit such characteristics. Identification of these areas may offer some insight into the location and cause of coliform contamination resulting from biofilm effects.
The following specific conclusions were made from the results of this study:

1. The model was capable of simulating 20 years of stochastic dynamic operation of a small water supply system containing a pump station and storage tank. Simulations could be performed at a time resolution of one hour, accounting for changes in tank levels and stochastic variations in demand from hour to hour.

2. The simulations could be performed on a desktop computer of the kind available to engineers in 1991, with acceptable run times of up to a few days.

3. During a simulation, the model could track constituents introduced under a number of different stochastic regimes. The output from constituent transport simulations indicated:
   - Chronic patterns of contamination
   - Variations in contamination frequency, both geographically and temporally within the diurnal demand cycle.

4. The model could provide summary statistics of hydraulic characteristics of each pipe in the system. It could identify pipes subject to stagnant conditions, flow reversals and unusually high relative velocities.

5. The model could identify monitoring periods which, as a result of applied contamination, were either in or not in violation of regulations based on presence/absence sampling.

6. The model could apply a number of sampling plans and provide a complete record of MCL exceedances and detections by each sampling plan. This data could be processed to provide a record of violations and detections based
upon any chosen monitoring period, allowing comparison of sampling plan
effectiveness by formal statistical procedures.

The following conclusions are specific to the hypothetical distribution system
tested. Unless otherwise stated, they apply to both layouts.

**Hydraulics**

7. The presence of a storage tank and its position relative to the pump station
considerably affected the hydraulic characteristics of the system.

8. Pipes located generally between the pump station and the tank had flows
which were damped by the in- and out-flows at the tank. Pipes outside the
influence of this area were more subject to flow velocity variations controlled
by system demands. This effect was more marked when the pump station
and tank were widely separated.

9. Flow reversals occurred near the storage tank. They also occurred in
peripheral areas as system pressures changed.

10. The verbose model output showed flow paths between the pump station,
tank and demand nodes to be very circuitous at times, indicating a large
potential for mixing and spreading of contamination through the system.

11. Pipes subject to stagnation occurred not only in peripheral and low demand
areas, but also in high demand areas, sometimes close to the tank.

**Contamination patterns**

12. If contamination arose or was introduced into the tank, the area served by
the tank was particularly prone to repeated contamination.
13. Instantaneous and leakage input of contamination into the tank yielded the same contamination pattern in the system.

14. Tank storage provided dilution, and for decaying constituents, time for reaction, which may have reduced concentrations to a level below the MCL by the time water was returned to the system.

15. Contamination input at the pump station resulted in widespread system contamination. Peripheral nodes became contaminated more frequently.

16. Low concentration contamination input randomly at nodes was detected more frequently at peripheral nodes and nodes distant from system sources.

17. The pattern of contamination could indicate the source of contamination. Knowledge of detected concentrations would allow distinction between some contaminant inputs that produce similar contamination patterns.

**Contamination timing**

18. Contamination applied at random times resulted in patterns showing distinct temporal patterns within the diurnal demand cycle, under the influence of tank storage. Patterns were different for different constituent inputs. The following conclusions account for these differences:

- Contaminated water was released to the system during peak demand periods, and could only be detected at this time or shortly afterwards.

- Uncontaminated water in the tank could maintain an area of clean water in the system while the tank was emptying, even during a concurrent contamination event elsewhere in the system.
Conversely, contamination, once stored in the tank, could be re-released to contaminate the system for several days after the initial contamination event.

**Sampling Plans**

19. Sampling at fixed nodes at the same time on a single day in the monitoring period was always an inferior sampling strategy.

20. When contamination of the tank resulted from a contamination event of any kind, sampling directly from the tank was one of the better sampling strategies. Sampling from a node near the tank when the tank was emptying was also a superior strategy.

21. When contamination occurred system wide as a result of contamination at a continuous source (pump station), sampling at random times and locations was the best strategy. But if contamination entered the tank, the previous conclusion could prevail.

22. For randomly timed and located contamination inputs (contamination at nodes) of low concentration and short duration, all plans performed very poorly. Improvements at higher concentrations and durations were generally a result of contamination entering the tank.

23. No consistent advantage was noted in fixed time random node sampling by plans which took twice as many samples in the high demand area as in the low demand area.

24. No consistent advantage was noted in fixed time sampling by plans which sampled all nodes systematically as opposed to sampling random nodes.
A choice of fixed nodes based on engineering judgement may not provide the best sampling points. Interpretation of contamination patterns and timing provided by this model could have led to a better choice of fixed sampling locations.

**Biofilm effects**

These conclusions accept the simplifying assumptions of the algorithms used to model biofilm effects based upon assumed relationships between flow velocity and biofilm detachment. They should be interpreted accordingly.

Considerations of flow velocities could indicate areas of a distribution system likely to be affected by velocity induced sloughing, and contamination arising from the biofilm under stagnant conditions. Attempts to model erosion of the biofilm produced results that either overwhelmed the system or were not considered representative of the underlying cause.

Contamination patterns due to the biofilm effects modeled were generally coincident with the pattern of pipes affected. If contamination entered the tank at sufficient concentrations, a contamination pattern similar to that for tank contamination resulted.

Biofilm effects related to changes in flow velocity (unusually high velocities or flow reversals) were time dependant, being tied to the diurnal demand pattern. Resulting contamination is therefore even more dependant on the time of sampling than contamination resulting from randomly timed events.

Stagnation effects could develop at any time during the day, and random timing of samples was required for highest detections.
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APPENDIX 1

THE HYDRAULIC MODULE - WATER

The hydraulic module consists of the model WATER (Municipal Hydraulics, 1990), which is available commercially as a stand-alone steady state hydraulic model, and was provided for research purposes by A. G. Fowler of Municipal Hydraulics. (A pseudo-dynamic version of WATER named WATEX is available, but it was considered easier to customize the steady state version for the purposes of the study). The program is written in FORTRAN77. The user manual gives full details of the input required, and the following is only a brief description.

WATER can model networks with up to 1500 pipes and nodes and up to twenty-five sources configured as pumps and/or tanks. Other features, such as booster pumps, check valves and pressure control valves can also be modeled. A variety of input unit combinations is available. Before input to the model, all nodes in the network must be numbered uniquely (though not necessarily in any particular order or even consecutively) and then each pipe must be numbered similarly and defined in terms of the nodes it connects. Physical information about the layout, such as node elevation, pipe diameter, length and friction coefficient is also required. Either the Hazen-Williams or Manning equations can be specified for calculation of head losses.

Pump curves can be defined for both source and in-line booster pumps. A piecewise linear approximation is used as defined by nine sets of x-y coordinates for each curve. Identical parallel pumps can be specified for any pump location.
Water demands on the network are specified by demands at each node. An overall factor can be specified to be applied to all demands so that solutions can be obtained with different demand factors without re-specifying individual node demands. To aid in the initial solution, an estimate of the proportion of total demand supplied by each source, if there is more than one, can be provided. A convergence criterion for the iterative solution process is required. This is based on the size of the largest loop-flow correction at each iteration, scaled to the flow rate of the largest water source. An editor, which simplifies input file preparation and editing by prompting for the required information, is provided. The model also prompts for information controlling the format of output and printer specifications.

The solution method employed by WATER is solution of the ΔQ equations described previously. An initial approximate solution is obtained for a minimal spanning tree (MST) which is derived using Prim's algorithm (Prim, 1957). The model checks input data and processes them to set up the problem for computer solution, using algorithms described by Epp and Fowler (1970).
List of subroutines forming the hydraulic steady state model
WATER

MAIN program WATER controls calls to the following subroutines:-

Subroutine
AAA : Read and check data input file.

S0 : Define network layout in terms of active pipes connected to each node. Activate control valves according to solution from S6, and update the active pipe as required.

S1 : Determine the Minimal Spanning Tree to obtain a set of starting flows that satisfies demands and mass balance at all nodes. If a pipe is 'removed' by check valve action, add an equal and opposite flow to one loop containing that pipe to reduce flow in the pipe to zero maintaining flow balance.

S2 : Find natural set of loops.

S3 : Add in pseudo-loops formed from lines of pipes connecting reference node to other source nodes and pressure control valves.

S4 : Minimize the bandwidth of the solution matrix using neighbouring loops.

S5 : Renumber loops and find loops with no flow.

S6 : Use flows from previous solution to obtain heads at sources from pump curves. Compute matrix and call BAND. Compute loop flow corrections and check for convergence. Update flows in pipes and at source nodes and rerun S6 until convergence achieved. If close to convergence, check if any control valves are activated by current conditions, and set flag, otherwise proceed to full convergence.

BAND: A matrix solver, called by S6.

S7 : Calculate pressures. If control valve flag set, go back to S0 or S3 as required and re-solve. Otherwise, verify solution by checking mass balance at nodes and head loss around loops. Print out solution.
APPENDIX 2

List of subroutines in TRAK

PIPE: Main calling program controlling subroutines in TRAK and calls to WATER.

QMIX: Contains four subroutines.

QSUM: Calculates total inflow (without subtracting outflow) at each node.

XMIX: Calculates mixed concentration of incoming flows at a node.

DELTAT: Calculates next timestep increment, DELTAT.

DECAY: Applies a decay factor for the current timestep to all concentrations in the model.

RESVR: Contains four subroutines.

RESVR1: Compares the status of a tank with the hydraulic solution. E.g., if a tank is full and the hydraulic solution calculates an inflow, the tank is closed off, and WATER is recalled to obtain a new solution with that tank temporarily removed from the system.

RESVR2: Records incremental in/out-flows at tanks and recalculates associated constituent concentrations. Checks if tank level has changed sufficiently to require new hydraulic solution or whether tank has just filled or emptied, and if so opens/closes tank in/outlet as required and sets flag for PIPE to recall WATER.

RESVR3: Uses incremental values from RESVR3 to update tank status just before a new hydraulic solution.

SOURCE: Controls pumps by acting upon pump status changes defined by the data file or UPDATA or by changes in reservoir levels where these control pump activity.
PLUG: Contains three subroutines

PLUGIN: Defines initial plug lengths and concentrations.

PLUGWR: Prints out lengths and concentrations of plugs in each pipe.

PLUGUP: Creates new plugs as new concentrations flow into a pipe and deletes old plugs. Adjusts length of end plugs if inflowing concentration has not changed. Consolidates plugs.

ARRTIM: Detects arrival of specified concentration of constituent at nodes, saves data and prints table of arrival times at end of simulation.

RANV: Contains a selection of generators to produce random numbers from various probability distributions. Source code is from various references.

UPDATA: This is a customizable subroutine to provide updates to simulation boundary conditions as required. Data changes are provided either in initial data or by subroutines to UPDATA. In either case, the changes and their timings are stored in two arrays until needed. UPDATA manages these arrays, inserting new data as it is generated and discarding old data. This subroutine does not accept input variables other than initial changes provided in the data file; it is intended to be written and compiled to suit specific requirements.

SAMPLE: A customized subroutine to interrogate the model database for concentrations both for establishing a routine record, and to simulate application of sampling plans. Concentrations are compared to limits representing regulatory requirements. Violations of the regulations are identified and recorded.
APPENDIX 3

SAMPLE DATA SET USED IN SIMULATIONS

D1d.dat NB62 Node 100mg/L 8 4 day MTBE, No DK, 20yr run. 13-Nov-91 Compaq
- TYPICAL SMALL TOWN DATA SET 23 Aug 91

datafile input. Derived from d1c.dat. Resvr level raised.

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* Actual number plugs/pipes | Max runtime in minutes
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58 42

* Units specifiers

* PIPES DATA

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SOURCES and DEVICES
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  1    1    0    0    0    0

PUMP SOURCES
node parl npnt pcd off man nrcp wlof wlon
  41    1    7    0.9    19.5   17.0
0.000   .35000   .650    0.850    0.980    1.02   1.10    * XH - flows
 150.0  148.00  138.000  120.000  100.000    80.00   00.0    * YQ - heads

RESERVOIR SOURCES
node pcd depth twl area isactv irtype
  42    0.1  12.000  232.00  1666.00    1    1

* BOOSTERS
pipe parl npnt

* PRV
pipe downstream pressure

* CHECKVALVES:
pipe nos. (All on one line; 5 spaces each.)

* PSV:
data

TSTART ENDTIM(d) SUBDAT
  0.0    7301.0    1
APPENDIX 4
Sample contamination event record

TABLE A4a. Node contamination event start times for the first 110 days at Mean Time Between Events of 4 days.

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TABLE A4b.  Pump station contamination event start times for the first 230 days at Mean Time Between Events of 7 days.

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<td>229: 527.17</td>
<td>229: 587.17</td>
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TABLE A4c. Tank contamination event start times during the first 750 days at Mean Time Between Events of 30 days.

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<th>EVENT END day:minute</th>
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<td>714:1140.00</td>
</tr>
<tr>
<td>751: 900.00</td>
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</tr>
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APPENDIX 5

SIMULATING BIOFILM CONTAMINATION

GENERAL APPROACH

Three types of biofilm contamination events were simulated, using flow velocity conditions as an indicator of event occurrence. These were biofilm sloughing due to high velocities, sloughing due to flow reversals and migration of bacteria from the biofilm under stagnant conditions. The velocity in each pipe was examined every hour, and biofilm events were invoked according to the following procedures.

When an event occurred, the concentrations of all plugs in the affected pipe were increased by an amount appropriate to the nature of the event and the pipe's location in the system. A baseline incremental concentration was defined for each pipe reflecting the observation that biofilm growth tends to be greater near the source (a result of higher nutrient fluxes). The value of the baseline incremental concentration was 15.0 for all pipes within 3 pipe lengths of the pump station, 5.0 units for pipes within six pipe lengths of the pump station, and 2.0 units for all remaining pipes. A multiple of this concentration appropriate to the type of event was then used to increase plug concentrations.

SLOUGHING AT HIGH FLOW VELOCITY

The cause of velocity induced sloughing events was assumed to be an unusually high velocity. It was assumed that biofilms that evolve under highly variable velocity regimes are resistant to shear effects of velocities typical of that regime, and will require an exceptionally high velocity above the typical range to
cause sloughing. It is therefore not so much a high velocity per se that causes sloughing, but a velocity that is above the normal velocity range for the pipe. The algorithm that identifies these criteria is constructed using exponentially smoothed averages (Chatfield, 1984) of the following velocity data on a pipe-by-pipe basis:

1. The average pipe velocity, and

2. The difference between pipe velocity and average pipe velocity.

Calculation of the exponentially smoothed averages is described in Appendix 6. The averages are recalculated for each pipe every hour and employ a weighting factor, $\alpha$, of 0.981.

Potential sloughing is indicated when the difference between the current pipe velocity and the average difference calculated at 2 above exceeds 2.5 times the average difference for that pipe, provided that the pipe velocity is higher than twice the average pipe velocity. The latter requirement ensures that differences due to an unusually low velocity do not indicate sloughing, although such differences are included in calculation of the average difference. Additionally, to account for the possibility that the biofilm does not contain any coliforms, only 25% of potential sloughing events are actually used to generate contamination events.

Once a biofilm has sloughed off, it is assumed that a period of regrowth must elapse before a repeat event can occur in that pipe, unless another exceptionally high velocity event occurs with a velocity even higher than that causing the previous event. That is, the criterion must be made more restrictive, then gradually relaxed as the biofilm recovers. The algorithm achieves this by doubling the average difference when a sloughing event occurs. This parameter then no
longer represents the average difference but is used purely as a criterion for sloughing. The weighting in the averaging method gradually reduces the effect of doubling the average so that after ten days it is reduced to a value only 1% above its value before sloughing. If a second very high velocity occurs before ten days, it can cause a second sloughing event if it satisfies the above criteria, and the average difference at that time is again doubled to simulate the increased period required for the biofilm to recover from two sloughing events closely spaced in time.

Potential sloughing events were discarded with a 75% probability to reflect the possibility that either the event may not actually take place, or that the even if it did, the biofilm may not contain any coliforms at that time. When a high velocity sloughing event was confirmed, the concentration in the affected pipe was increased by an amount equal to the baseline concentration increment described previously.

**SLOUGHING AT FLOW REVERSALS**

A potential sloughing event was indicated for a pipe when the flow direction in the pipe reversed. Some pipes have frequent flow reversals, sometimes as many as four a day due to diurnal demand changes. It was assumed that such frequent reversals could not cause sloughing on every flow reversal, but that a period of biofilm recovery would be required between events. A waiting time of 6 days was therefore required before repeat sloughing events could occur in a pipe. Potential events were discarded with a 75% probability. If not discarded, the concentration in the pipe was increased by an amount equal to the baseline concentration increment.
BIOFILM EFFECTS UNDER STAGNANT CONDITIONS

Stagnant conditions were assumed to be indicated by continuous periods of low flow. Low flow was defined as any velocity below 0.1 ft/s, and stagnation conditions were defined as being established in a pipe when twelve consecutive hours of low flow in that pipe had occurred. Any flow higher than 0.1 ft/s was considered to have flushed the pipe, requiring a new twelve hour qualifying period. A 75% random discard of diffusion events was applied. When stagnation was indicated, an incremental concentration of 0.1 times the baseline concentration increment was initially applied. This gave inconclusive results, and the simulations were repeated using a factor of 0.5. No waiting period between events was used, since prolonged periods of diffusion are feasible.
APPENDIX 6

CALCULATION OF EXponentially SMOOTHED AVERAGE

The following is derived from a procedure described by Chatfield (1984). The exponentially smoothed average velocity for a pipe at time \( t \), \( V_{\text{av}} \), is defined as:

\[
V_t = C_0 V_t + C_1 V_{t-1} + C_2 V_{t-2} + C_3 V_{t-3} + \ldots
\]

where \( V_t \) is the velocity at time \( t \), and the \( C_i \) are weights. If the weights, \( C_i \), are defined so that they decrease geometrically, as given by

\[
C_i = \alpha(1-\alpha)^i \quad (i = 0,1,\ldots)
\]

where \( \alpha \) is a positive constant less than unity, then the sum of the weights is one, and greater weight is given to more recent values. The smoothed average may then be redefined as

\[
V_t = \alpha x_t + \alpha(1-\alpha)x_{t-1} + \alpha(1-\alpha)^2x_{t-2} + \ldots
\]

which can be rewritten as

\[
V_t = \alpha x_t + (1-\alpha)\bar{V}_{t-1}
\]

This method of update is computationally simple and economical of memory, requiring only that the previous average, \( \bar{V}_{t-1} \), be stored between iterations. The weight of a velocity when first included in the average is \( \alpha \). The value of \( \alpha \) is chosen so that the weight of a velocity after 10 days (or 240 hours) is reduced to 1% of \( \alpha \). Thus,

\[
(1-\alpha)^{240} = 0.01 \quad \text{so that} \quad \alpha = 0.981
\]

The influence of exceptional velocities is therefor effectively removed from the average within this time. A prolonged change in typical velocities, such as a seasonal trend with a period of about 10 days or longer will nevertheless be
reflected by a gradual change in the average. However, the ten day period is too long to be influenced by diurnal variations, so that the average remains fairly constant from day to day.

Strictly speaking, this method requires an infinite number of historical values. However, the weighting factor reduces the contribution of distant historical values sufficiently that their absence is not detrimental to the procedure.
APPENDIX 7

Distribution Free Multiple Comparisons Based on Friedman Rank Sums

Each sampling plan is to be compared with each other sampling plan in a multiple comparison procedure, so that an ordered ranking for those plans that are statistically different from other plans can be identified. This is done by comparing plans on a year by year basis, to obtain a total of twenty comparisons from a twenty year simulation. Sampling results within a single year are not independant, because the number of actual violations in a year tends to affect the number of detections by all plans for that year. The lack of independance indicates the use of a non-parametric test. A suitable test is the distribution free multiple conparison based on Friedman rank sums described by Hollander and Wolfe (1973).

The application of this procedure is briefly described and applied to a typical data set. The $X_{ij}$ represent the 140 detection data from the $k = 7$ plans over the $n = 20$ years, where subscript $i$ represents the year number and subscript $j$ represents the plan number. The seven $X_{ij}$ are first ranked from least to greatest within each year, with $r_{ij}$ representing the rank of the $j$th plan in the $i$th year. In the case of ties, ranks are averaged among ties. It should be noted that the ranking within years is identical whether actual violation numbers or proportions are used. The ranks are then summed for each plan over the twenty years so that

$$R_i = \sum_{j=1}^{20} r_{ij}$$
The twenty-one absolute differences between all pair combinations of plans, $|R_u - R_v|$, $u < v$, are then calculated. Decide at an experimentwise error rate of $\alpha$ that plan $u$ differs significantly from plan $v$ if $|R_u - R_v| \geq r(\alpha, n, k)$. Values of $r$ are tabulated for $n$ up to 15, so that appropriate values for $n$ of 20 are not available. A large sample approximation method is also presented by Hollander and Wolfe, where a significant difference exists if

$$|R_u - R_v| \geq q(\alpha, k, n) \left[ \frac{n(k)(k+1)}{12} \right]^{1/2}$$

where $q$ is the upper $\alpha$ percentile point of the range of $k$ independent $N(0,1)$ variables. Hollander and Wolfe give an example large sample approximation procedure where $n = 22$. For $n = 15$, $k = 7$, the first method gives a critical value of 15.95, and the second a critical value of 16.0, suggesting that values of $n$ above 15 can be usefully tested using the large sample approximation.

Raw data in the form of number of detections by each plan in each of the twenty years of a simulation are presented in TABLE A7-1, with the corresponding rankings shown in TABLE A7-2.

The test statistics calculated for each pairwise comparison are cross-tabulated in Table A7-3. Values of the test statistic exceeding the critical value for a 90% confidence level are emboldened.

A formal method for further interpretation of this test is not given by Hollander and Wolfe. The following procedure was used to categorize the goodness of each plan, as indicated by letters listed below:

B  BEST plan. Significantly better than the most number of other plans.
b GOOD plan. Significantly better than at least one other plan and not significantly worse than any other plan.

w POOR plan. Significantly worse than at least one other plan.

W WORST plan. Significantly worse than the most number of other plans.

Plans were first listed vertically in ascending order of total number of detections over twenty years. This was done simply to aid the application of the statistical tests, and does not affect the findings of the procedure. With reference to the tabulation of critical values, a line was drawn from the plans with the highest number of detections to any plan that was statistically different from it. All such plans are seen to have much lower numbers of detections. The same procedure was applied to the next plan, and so on, but only linking them to plans with smaller numbers of detections. e.g., Plan 7 is statistically different from plans 1, 6 and 4. The result is shown in Table A7-3b.
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Table A7-2 Plan Rankings for Simulation No. 62

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Σ ranks 58.5 95.0 77.0 109.0 76.0 53.5 91.0

213
Table A7-3A Critical Differences Between Plan Rankings for Simulation 62

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CRITICAL \(|R_u-R_v| \geq q_{\alpha} [n, k(k+1)/12] = 36.79\) at \(\alpha = 0.1\)
Critical differences in bold type.

Table A7-3B Ranking Based on Statistical Test Results for Simulation 62

W  6
w  1
5
3
b  7
b  2
B  5
VITA

Philip Edward James Jones was born on 19th July 1946, in Greenwich, UK. He graduated from St. Mary's Grammar School, Sidcup, in July 1965. He entered the service of London's Metropolitan Water Board as a Student Apprentice, attending The City University part time, from which he received a B.Sc. degree in Civil Engineering in 1970. He worked in the water supply industry in Britain for fifteen years, obtaining Chartered Engineer status in 1972, and holding posts of increasing responsibility in design, construction and facilities maintenance. From 1975 to 1981 he continued his career in East Africa, first with Voluntary Service Overseas, and then as a consultant working on rural water supply design and planning throughout Kenya and a Regional Water Master Plan in Tanzania. While in Kenya, he met and married the former Susan Kay Rozmus of Orlando, Florida on the 9th January 1981.

After moving to the United States, he obtained an M.Sc degree in Civil (Environmental) Engineering from Purdue University Indiana in June 1984. He held positions as a consultant and Director of the Research Center at Blue Plains wastewater treatment plant in Washington DC, obtaining a Professional Engineer's Licence from the State of Maryland in 1987. He entered the graduate program at Virginia Polytechnic and State University in Blacksburg, Virginia, in December 1987.

He intends to continue his career in research and consulting to the environmental industry worldwide.