Manual of Design for Slow Sand Filtration

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Manual of Design for Slow Sand Filtration
Manual of Design for Slow Sand Filtration

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Foreword

The AWWA Research Foundation is a nonprofit corporation that is dedicated to the implementation of a research effort to help local utilities respond to regulatory requirements and traditional high-priority concerns of the industry. The research agenda is developed through a process of grass-roots consultation with members, utility subscribers, and working professionals. Under the umbrella of a Five-Year Plan, the Research Advisory Council prioritizes the suggested projects based upon current and future needs, applicability, and past work; the recommendations are forwarded to the Board of Trustees for final selection.

This publication is a result of one of those sponsored studies, and it is hoped that its findings will be applied in communities throughout the world. The following report serves not only as a means of communicating the results of the water industry's centralized research program but also as a tool to enlist the further support of the nonmember utilities and individuals.

Projects are managed closely from their inception to the final report by the Foundation's staff and large cadre of volunteers who willingly contribute their time and expertise. The Foundation serves a planning and management function and awards contracts to other institutions such as water utilities, universities, and engineering firms. The funding for this research effort comes primarily from the Subscription Program, through which water utilities subscribe to the research program and make an annual payment proportionate to the volume of water they deliver. The program offers a cost-effective and fair method for funding research in the public interest.

A broad spectrum of water supply issues is addressed by the Foundation's research agenda: resources, treatment and operations, distribution and storage, water quality and analysis, toxicology, economics, and management. The ultimate purpose of the coordinated effort is to assist local water suppliers to provide the highest possible quality of water economically and reliably. The true benefits are realized when the results are
implemented at the utility level. The Foundation's Trustees are pleased to offer this publication as a contribution toward that end.

Slow sand filtration is one of the technologies especially appropriate for small communities. This manual of design is intended to serve their special needs. At the same time, the manual does not restrict the application of slow sand technology to small communities; if more applications are found, the purposes of the manual will be fulfilled to an even greater extent.

Richard P. McHugh
Chairman, Board of Trustees
AWWA Research Foundation

James F. Manwaring, P.E.
Executive Director
AWWA Research Foundation

Preface

This manual of design was commissioned by the AWWA Research Foundation for the purpose of encouraging the use of slow sand filtration technology. With the resurgence of interest in "slow sand" and with the insights that have resulted from research conducted over the past 10 years, the missing link in the application of this technology is a compendium of guidelines for the consulting engineer. For small communities, slow sand filtration seems especially appropriate. Annual operating costs are low and the process is "passive," meaning that operator intervention is minimal.

Slow sand filtration has been available as a technology since 1829, when the first filter was installed in London by the Chelsea Water Company. Subsequently, slow sand was adopted widely in Europe, but it did not gain much foothold in the United States. By the turn of the century, the new "mechanical filtration," which we now call rapid rate filtration, had become the technology of the future. One reason for the transition to rapid rate was that the water conditions for which filtration was needed had high turbidity, which would translate to short run times with slow sand filtration. In locales in which slow sand filters were constructed, however, they were eminently successful. The Denver, CO, 152 mil L/d (million liter per day), or 40 mgd (million gallon per day), Kassler Plant, placed in operation in 1906, is a case in point.

Two events have caused increased interest in slow sand technology. First, the 1974 Safe Drinking Water Act mandated drinking water standards that were applicable to all community water supplies. Large communities can comply with the standards more easily than small communities, which typically have less money available for capital projects and for operation. Their plant operators usually work only part time, since they must handle several kinds of tasks within the community, and they have little time to master the skills needed to operate a plant. With the law in place, something had to be done to help the small communities, and the industry took up the task in a number of ways. One of its research responses was to investigate slow sand filtration. The second event to cause increased interest in slow sand was the recognition, in the late 1970s, of the problem of the Giardia lamblia cyst, which was most often associated with small
community water systems. The focus of much slow sand research was on whether slow sand could remove Giardia cysts.

The genesis of the current renaissance in slow sand was in the Environmental Protection Agency (EPA) research program. In 1980, the Drinking Water Research Division of EPA, Cincinnati, began in-house research on slow sand. This research led to funding at Colorado State University for the investigation of Giardia cyst removal and the roles of operating variables in slow sand filtration, at Dusfrene-Henry Consulting Engineers in Vermont for the study of Giardia cyst removal by a full-scale plant, at Syracuse University in New York for research on operating costs, and at Iowa State University to look at the cold weather performance of slow sand filters. Then in 1982, researchers at Utah State University looked at sand size and removal performance. In addition, in 1985, the AWWA Research Foundation sponsored work at Colorado State University that emphasized the performance of full-scale filters and used the new filter at Empire, CO, as a case study. The Empire filter was one of the outcomes of the earlier research activity at Colorado State. At the same time, Health and Welfare Canada sponsored research conducted by Dayton & Knight, Ltd., Vancouver, BC, on the new filter at 100 Mile House, BC, another "child" of the 1980s' research. This activity spurred the interest of the Idaho Health Department, which, in cooperation with the University of Washington, studied the performance of several slow sand filters that were constructed in northern Idaho since the 1960s. Other filters were constructed in Oregon beginning in the 1960s. Then in the late 1980s research was conducted at the University of Colorado and at the University of New Hampshire. From this foregoing effort came the vision of the AWWA Research Foundation that the findings of the research and the lessons of practice needed to be pulled together in a "how to" design manual.

This manual also builds upon the lore of the past. The earliest reference work used was the 1913 book of Allen Hazen. The World Health Organization (WHO) book of Huisman and Wood (1974) and the WHO-sponsored works of the International Reference Center in The Hague, such as Technical Paper No. 24 by Visscher et al. (1987), form the more recent platform. But the manual also has its own identity as a contemporary work in that the research of the 1980s has been assimilated.

The investigations and studies since 1980 have expanded our knowledge about slow sand as a process, and the plants in operation have added an experience component. Thus, while past knowledge is utilized, the 1980s' research on slow sand filtration is the focus of this manual. Even with the recent research, however, many questions remain. Thus, slow sand design still requires considerable judgment on the part of design engineers, who must utilize their full range of knowledge of engineering for successful outcomes.
The project team that developed this manual included many of the persons involved in slow sand filtration in North America. The topics to be covered were reviewed by the team, the project officer, and the Project Advisory Committee (PAC) during an all-day meeting, September 7, 1988. Drafts were then developed by the principal investigator, and chapters were written by team members and reviewed by the team and the PAC. The method permitted a coordinated writing style and yet utilized an extensive knowledge base. The approach was to question all premises of the past. Team members were urged to respect tradition but not be captured by it. As a rule, the lore of the past was assimilated, not displaced, and new findings were incorporated to create the basis for guidelines for present practice. In this manual, the examples and other references to operating plants usually refer to the slow sand filter at Empire, CO, started in January 1985, the facility at the Village of 100 Mile House, BC, which started operation in November 1985, or the plant at Moricetown, BC, which started operation in 1988.

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Martin J. Allen, AWWA Research Foundation, Project Officer
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Kim Fox, Drinking Water Research Division, EPA, Cincinnati, Project Advisory Committee
How to Use This Manual

The purpose of this manual is to provide design guidance for the engineer who may wish to consider recommending slow sand filtration to a client. The assumption is made that the reader has no prior familiarity with slow sand filtration. Another assumption is that the reader is a graduate civil engineer in general practice. However, these assumptions do not preclude the use of the manual by the city official who may wish an introduction to and orientation concerning slow sand filters, nor by the operator who may wish additional depth of understanding or who may wish to have a reference at hand. The text, diagrams, tables, and most of the figures should be clear to persons having an interest in the subject. The operator, the city official, and the engineer can peruse the manual and select sections of interest. An engineering background would be useful for understanding the theory and design examples, but the manual has a tiered design to accommodate readers having a range of interests and needs.

As a part of the tiered design, each chapter starts with a basic orientation and progresses to provide a rationale on the particular topic of interest, examples on how to apply a particular principle or guideline, and the "bottom line" conclusion. Some engineers may wish to skim the theory and carefully read only the conclusions. The goal was to provide a comprehensive, in-depth manual that would be easy to use. To facilitate this goal, the headings provide a means to discern topics, and the example problems are presented in a smaller size type so that they can be identified easily.

Thus, the text may contain more information than is desired by some users. For example, the reader needs to study only the conclusion to Chapter 3 to learn that problems with air binding can be avoided by maintaining the effluent weir crest at the level of the top of the sand bed. If the reader is interested in the rationale behind this rule, he or she may read the discussion of gas precipitation. Since the side headings are clearly labeled, such detail can be omitted easily. For those who wish to have the added insight, the option is available.

Example problems are also used as a means to accomplish the tiering for different levels of interest. The example problems provide explicit illustrations on how to apply the principles and guidelines discussed. They also illustrate the calculation steps.
The project team, the Project Advisory Committee, and the project officer deliberated at some length over the question of whom the manual should be addressed toward. An initial idea was that the manual could provide a means for small communities to construct their own slow sand filters. However, this idea was superseded when it was realized that the construction of even a small filter for only a few persons is still an engineering job. Despite the simplicity of the slow sand filter, an engineer is required for its construction, both for the reason of substantive knowledge and because an engineer's seal is required for a building permit. The next question was, to what kind of engineer should the manual be addressed? We had in mind the general civil engineer who knows the water system problems of small communities. Although there is not a single individual who would characterize such a hypothetical role, the manual addresses a range of concerns that might be relevant to the different firms and persons who might have use for such guidance. The end result is that the manual is intended to make slow sand technology available to engineers and communities that may wish to consider its use.

In the manual, the metric system is the primary system of measurement used, but English units are also provided. Metric units were chosen because the system is accepted by the AWWA, because our Canadian partners use the metric system and the trend in the United States is toward metric, and because the metric system is easy to use. It is recommended that the engineer develop the slow sand filter design in metric units and convert back to English units only for the purpose of dealing with contractors. Chances for mistakes are minimized with the use of metric units.
Acknowledgments

The project team appreciates the involvement of the Project Advisory Committee, which included James R. Boydston of the Oregon Health Department, Robert Krill of the Wisconsin Department of Natural Resources, and Kim Fox of the Drinking Water Research Division, EPA. The project officer, Martin J. Allen of the AWWA Research Foundation, coordinated the project and facilitated the administration and the development of content. Deborah J. Lynes was the project editor. She coordinated the final production, which included copy editing, review of copy editing by this editor, and camera-ready word processing.

We also appreciate the reviews of Gordon Mills, of the Village of 100 Mile House, BC, and Don Kerven, of Empire, CO, who examined the chapter on operation. Lloyd Slezak of Dayton & Knight, Ltd., Vancouver, BC, provided a critical reading of the second draft of the manuscript. We gratefully acknowledge that the material for Chapter 7 was adapted from a paper presented by R. Robin Collins, T. Taylor Eighmy, and James P. Malley at the 1990 AWWA Annual Conference in Cincinnati. Photographs of the Empire, CO, slow sand filter were provided by Burt F. Leautaud, Versar Architects and Engineers, Inc., Greeley, CO. Photographs of the 100 Mile House filter were provided by Dayton & Knight, Ltd., Vancouver, BC, and the photograph of the Village of 100 Mile House was provided by that village.

While the reviews and suggestions have improved this document, the principal investigator and the project team accept responsibility for its content.
EXECUTIVE SUMMARY

PURPOSE OF THE MANUAL

This manual was commissioned by the AWWA Research Foundation for the purpose of encouraging the greater use of slow sand filtration technology. Its purpose is to provide design guidance for the engineer who may wish to consider recommending slow sand filtration to a client. With the resurgence of interest in "slow sand" and with the insights that have resulted from research conducted over the past 10 years, the missing link in the application of this technology is a compendium of guidelines for the consulting engineer. For small communities, slow sand filtration seems especially appropriate. Annual operating costs are low and the process is "passive," meaning that operator intervention is minimal.

The assumption is made that the reader has no prior familiarity with slow sand filtration. Another assumption is that the reader is a graduate civil engineer in general practice. However, these assumptions do not preclude the use of the manual by the city official who may wish an introduction to and orientation concerning slow sand filters, nor by the operator who may wish additional depth of understanding or who may wish to have a reference at hand. The text, diagrams, tables, and most of the figures should be clear to persons having an interest in the subject. The operator, the city official, and the engineer can peruse the manual and select sections of interest. An engineering background would be useful for understanding the theory and design examples, but the manual has a tiered design to accommodate readers having a range of interests and needs.

DEVELOPMENT OF THE MANUAL

This manual builds upon the lore of the past but has its own identity as a contemporary work in that the research of the 1980s has been assimilated. The investigations and studies since 1980 have expanded our knowledge about slow sand as a process, and the plants in operation have added an experience component. Thus,
while past knowledge is utilized, the 1980s' research on slow sand filtration is the focus of this manual.

The project team that developed this manual included many of the persons involved in slow sand filtration in North America. The topics to be covered were reviewed by the team, the project officer, and the Project Advisory Committee (PAC) during an all-day meeting, September 7, 1988. Drafts were then developed by the principal investigator, and chapters were written by team members and reviewed by the team and the PAC. The method permitted a coordinated writing style and yet utilized an extensive knowledge base. The approach was to question all premises of the past. Team members were urged to respect tradition but not be captured by it. As a rule, the lore of the past was assimilated, not displaced, and new findings were incorporated to create the basis for present practice. The manual starts with a discussion of the state of the art and ends with guidelines for practice.

MANUAL CONTENT

Slow sand filtration has been available as a technology since 1829, when the first filter was installed in London for the Chelsea Water Company. Subsequently, slow sand was adopted widely in Europe, but it did not gain much foothold in the United States. By the turn of the century, the new "mechanical filtration," which we now call rapid rate filtration, had become the technology of the future. Two events have caused recent interest in the slow sand technology. First, the 1974 Safe Drinking Water Act mandated drinking water standards that were applicable to all community water supplies. Second, the problem of the *Giardia lamblia* cyst was recognized in the late 1970s and was found to be most often associated with small community water systems, which gave special urgency to finding water filtration solutions for small communities.

State of the Art

James Simpson set forth the basic design for the slow sand filter in London in 1829. His design became, and still is, the standard for practice. In the United States and Canada, slow sand filtration has not been a favored technology, but interest is being revived due to a resurgence of research that started in about 1980. The research findings have both verified past observations and provided new information useful for contemporary practice. The guidelines of this manual incorporate these research findings and the recent experience of plants in operation. The guidelines also build upon past practice and incorporate such practice, except as superseded by the findings of the 1980s.
Slow sand filtration is especially suited for small communities because the technology is effective, has low operating costs, and is passive. Without question, communities having populations of 1,000 to 2,000 persons, or even as many as 5,000 persons, should find slow sand filtration to be an appropriate technology. For larger communities, however, the labor costs of processing the larger amounts of sand required would be greater than the cost of rapid rate filtration. The point of this crossover will depend upon circumstances. The city of Salem, OR, with a population of 107,000 (serving 135,000), uses slow sand filtration, as do West Hartford, CT, which serves a population of 300,000, and other cities, such as those listed by Slezak and Sims (1984).

Preferred turbidity levels are <10 nephelometric turbidity units (NTU). As an upper limit, 30–50 NTU has been mentioned as a rule of thumb, but the limit is more a matter of engineering judgment than an absolute level. Whether slow sand can be used for high-turbidity waters depends on the effluent turbidity and the length of run and on whether pretreatment may be acceptable. Color and volatile organics are other considerations; if the raw water concentrations are high, pilot plant work should be done to determine removals.

The slow sand filtration process is expected to remove such biological particles as cysts, oocysts, algae, bacteria, viruses, parasite eggs, nematode eggs, and amorphous organic debris at 2-log to 4-log levels when the filter bed is biologically mature. The foregoing applies to biological particles present in the influent flow and not to species that find ecological niches and grow within the filter bed as a part of the biofilm around the sand grains (contributing to the maturity of the sand bed). The biofilm will be sloughed from the filter and will appear in the effluent flow. Cartridge filter sampling using 1-μm (micrometer) pore size filters may be conducted to assess the performance of the filter. Also, measurements of total coliform bacteria, if present in the influent water in sufficient densities, is useful for assessing removal efficiencies. Removals should be 2-log to 4-log for a biologically mature filter. Mineral particles may, however, pass through a filter bed that is effective in removing biological particles. Therefore, turbidity removal may not be a useful indicator of performance. Nevertheless, the turbidity of the slow sand filter effluent should comply with regulatory requirements.

The design flow used to size the filter bed area should be the maximum daily flow projected to the end of the design period and should take into account the removal from use of one filter bed for scraping (unless adequate treated water storage is provided to compensate). The design period will depend upon local circumstances, but 20 years is common. Treated water storage should be sufficient to permit a steady flow of water
through the slow sand filter over any daily cycle, and should have sufficient additional storage for fire protection and emergencies.

DESIGN OF SLOW SAND FILTERS

The components of a slow sand filter include intake, pretreatment (if any), the filter box, piping, disinfection, and treated water storage. The layout is site-specific. The filter box should include two or more equal-sized cells, independently operated. The filter bed area is calculated as the maximum expected flow divided by the maximum permissible hydraulic loading rate with all but one cell in service. The depth of the filter box is the sum of the depths of the gravel support, sand bed, and freeboard plus the maximum water depth. The structure of the filter box should be designed by a civil engineer qualified in structural design, and the foundation should be designed by a qualified geotechnical civil engineer. A housing may be provided to cover the filter beds to prevent the water from freezing, to inhibit algae growth, and to protect the sand bed from wind-blown debris. The housing should also be equipped with ventilation and with access ports for entry and sand handling and transfer. Portable electric lighting that is in compliance with electric codes should be provided.

Provision must be made for filter-to-waste, drainage of headwater, backfilling of the sand bed with filtered water, and adjustment of flow to each filter. An overflow should be installed at the maximum headwater level. Taps should be installed for easy sampling at appropriate sample points. Incoming flow should be distributed around the filter bed at low velocity and into a headwater of ≥0.3 m (meters), or 1.0 ft (foot), depth above the sand bed to minimize sand bed erosion. The underdrain system should be designed using the "manifold" hydraulic principle, that is, the headloss within the main pipe should be small compared with the headloss through the orifices into the main pipe. If the manifold principle is maintained, the hydraulic loading rate across the filter bed should be uniform. The gravel support should be graded, with the smallest gravel size on the top and the largest on the bottom surrounding the underdrains. The design is empirical, and established rules should be followed.

Drainage of the headwater should be provided for by the influent distribution manifold system. Final lowering of the water surface to the desired level can be done through the underdrain system. The water level should be lowered to about 2–5 cm (centimeters), or 1–2 in. (inches), below the sand surface before the sand is scraped. After the sand bed is scraped, the filter should be backfilled with finished water, which may come from an adjacent operating filter or from finished water storage. An overflow weir and collection box should be placed with the crest located such that the water level will
be at the maximum level desired when the head for the weir is added to the weir crest elevation.

Flow measurement should be provided on the influent side by means of an orifice meter or a Venturi meter. Pressure difference should be measured by pressure gauges. A calibration curve should be placed on a wall in the vicinity of the meter and on the data recording sheets and should be included in the computer-processing software. A volumetric flow meter should be located on the effluent side of the filter. Flow control should be by means of a gate valve located downstream from the flow meter. The valve should be on the main influent pipe. The pattern may be duplicated for each filter, at the option of the engineer in consultation with the operator. Two piezometers should be provided for each filter, with taps in the headwater and tailwater. The piezometer tubes should have diameters of 2.5–5.0 cm (1–2 in.), with float balls and scale provided.

The hydraulic loading rate (HLR) for the peak daily flow may vary between 0.1 and 0.4 m/hr (meters per hour), or 1–10 mgad (million gallons per acre per day). The HLR may exceed 0.4 m/hr (10 mgad) only at the end of the design period, at the point when a filter is removed from operation for scraping. In addition, the foregoing HLR guideline may be exceeded only if the filter beds remaining in operation are biologically mature. Although the HLR may vary during the annual cycle and generally increase as the population grows, the flow should be steady over the daily cycle. The sand bed should be ≥1.0 m (3.3 ft) in depth at the start of operation. A depth of 1.3 m (4.3 ft) is favored, and a deeper bed may be used if desired by the engineer. The sand size recommended by tradition is $d_{10} = 0.2–0.3$ mm (millimeters), and the ratio of $d_{60}/d_{10}$ (UC) = 1.5–2.0. Media having some sand particles with $d_{10} ≥ 0.3$ mm and UC > 2 may be acceptable if a pilot study ascertains that acceptable removals are obtained.

Pilot Plant Studies

A pilot plant study is recommended to determine headloss increase with time, as each water is unique. If the sand contemplated for use in the filter deviates from recommended sizes, such pilot studies should be mandatory. The results of the pilot plant study should be plotted in terms of headloss versus time curves for each of the water quality seasons. The headloss versus time curves will indicate the cycle times between scrapings, based upon the maximum headloss to be imposed in the design.

Construction

The engineer should follow through with construction inspection and should develop the operating protocol. The latter should be done in consultation with the operator. The sand placed in the filter box should be washed sand. As a goal, the sand
should be washed to the extent needed to ensure that less than 10 percent of the effluent turbidity at start-up is due to erosion of fines from the sand bed.

**Operation**

Elevations of headwater and tailwater should be observed and recorded daily. Flow should be measured daily before and after any adjustment. Turbidity sampling should be done daily using clean containers. Measurements should be done in the plant laboratory using instruments that have been standardized using commercially available standards. Coliform sampling should be conducted with the frequency and handling required by state regulations and analyses should be performed by certified laboratories.

A self-explanatory record form should be designed for all data. Recommendations are that (1) processed data be shown on a form containing both original and processed data, organized into logical groupings; (2) plots be prepared from the forms and include cumulative flow volume versus time, headloss versus time, temperature versus time, turbidity versus time, and coliform level versus time; and (3) the plots be used to discern trends and to identify deviations and discrepancies and to show performance relative to applicable standards.

When the headwater level reaches the overflow level, or prior to that time, the headwater should be drained for scraping. The water level should be lowered in two stages: (1) to the level of the distribution manifold, and (2) to a level just below the surface of the sand bed, that is, 2.5–5.0 cm (1–2 in.). The first-stage lowering should be by means of the influent distribution orifices and manifold, and the second stage by means of the underdrain system, both drained to waste. Removal of the *schmutzdecke* should be done when headloss reaches, or is less than, the overflow level.

After scraping, the sand bed should be backfilled with finished water through the underdrain system. The backfilling should be at a slow rate, that is, <2 m/hr, in order to displace air until the water level reaches the sand surface, at which point the backfill rate may be increased.

The sand removed by scraping should be stored and then washed for recycling. The washing should be done when sufficient sand has accumulated to warrant such an operation. Washed sand should be stored until resanding is necessary. The washed sand storage should be covered to provide protection against contamination. For small installations—such as those with a single filter bed with a surface area small enough that two persons can scrape it within one shift of work—the operations are done by hand and only asphalt rakes, shovels, and buckets or wheelbarrows are needed.

Orifice meters should be installed in a way that allows for easy removal for cleaning. Any debris that accumulates behind the orifice plates must be removed. Taps
for pressure gauges should be flushed periodically. All instruments should be calibrated at regular intervals. Flow meters will retain calibration unless deposits occur; any such deposits should be removed.

Data may be processed using standard forms developed for commercial computer spreadsheet software. Calibration coefficients for flow meters and pressure gauges should be entered and identified on the spreadsheet and incorporated by reference in the processing functions. The same forms should be available in hard copy in the event that computer processing is not desired.

MODIFICATIONS

The slow sand system may be modified in some situations to mitigate short-term effects, such as high turbidity during one season of the year. Modification is contrary to the philosophy of slow sand filtration as a passive technology and should be used only after careful consideration of the trade-offs. In addition to the removal of suspended solids, removal of fulvic and humic acids (trihalomethane precursors) may be considered. Other modifications should be considered with caution. These are mentioned in this manual only to indicate that considerable experimentation has been in progress over recent years to explore the full potential of the slow sand technology.
Chapter 1

State of the Art

This manual is written for the general civil engineer who is considering slow sand filtration for a small community client. The manual explains the principles of slow sand filtration, provides design guidelines, outlines the purposes and procedures for pilot plant studies, describes the construction of a slow sand filter, and outlines operating protocol. The manual collates the knowledge, experience, and understanding of slow sand filtration (that is, the state of the art) and provides "how to" guidance in an easy-to-use form. The intent is to facilitate technology transfer for slow sand filtration. This chapter describes a slow sand filter, gives a historical perspective, outlines the theory of the process, and reviews practice. The orientation thus provided forms the basis for the guidelines subsequently presented.

1.1 DESCRIPTION

A slow sand filter is simple in design, construction, and operation. The filter is merely a bed of sand supported by a layer of gravel, all of which is confined within a box, with appurtenances to deliver and remove water. Figure 1.1 illustrates the concept. Within the gravel support, and on the floor of the box, are underdrains to remove the filtered water. The elevation of the tailwater is controlled by a weir plate on the effluent side of the filter. The flow of raw water to the filter is controlled by a valve, with a flow meter preceding it. The energy of the incoming raw water must be dissipated so that the sand bed is not eroded, which would cause "short-circuiting" through the sand bed. The energy can be dissipated by distributing some of the flow around the filter box and by providing a headwater depth of about 30 cm (centimeters), or 12 in. (inches), at the start of the filter run. The headloss at the start of a filter run is usually only a few centimeters, but it increases to whatever depth is permitted by the height of the filter box. Usually the greatest headloss permitted is about 2.0 m (meters), or 6 ft (feet). The headloss occurs because of a deposit, and perhaps microbial growth, that develops on the surface of the
filter. This deposit is called the *schmutzdecke*, a German word meaning "sludge blanket" or "dirty layer," both apt terms for the phenomenon observed.

To make the process work, design guidelines should be followed and appurtenances must be added. The goal is to provide a system that operates passively. In other words, the design engineer must endeavor to assure that proper plant operation will be self-implementing and self-evident to the operator. If the design engineer achieves this goal, then slow sand filtration can be an appropriate technology for small communities, where operators may have other responsibilities and where there is often chronic difficulty in operating rapid rate filters.

1.2 HISTORY

The first recorded use of sand filtration for a citywide water supply was in 1804 by John Gibb in Paisley, Scotland. The filter provided water for Gibb's bleaching business and for public purchase (Baker 1948 p. 77). The model for present practice, however, was a one-acre slow sand filter designed by James Simpson for the Chelsea Water Company in London and completed in 1829.

The London filter was the first of its kind and was a technological breakthrough in that the design laid the foundation for a widespread practice that continues today. Simpson based his filter design upon information he gained during a 2,000-mi (mile) study tour through Scotland, where he saw several installations, mostly for industrial use. None of these were like the forthcoming London filter, which he was to design. Simpson made the basic design a downflow filter and used scraping to remove

![Figure 1.1 Schematic Cross Section of a Slow Sand Filter](image-url)
accumulated material—that is, the schmutzdecke. The hydraulic loading rate, sand size, sand bed depth, water depth, and other design parameters that he delineated became the basis for the practice that followed. The London filter, which used the Thames River as its raw water source, was the first use of a treatment process for a piped public water supply (Baker 1948). The hydraulic loading rate, 0.15 m/hr (meters per hour), or 3.9 mgad (million gallons per acre per day), became a common design criterion and is one of the several criteria retained to current times.

The health benefits of filtered water soon became apparent in London, and in 1839 the city's five commercial suppliers began filtering their water. In 1852, the London city government required filtration prior to the sale of drinking water to the public, and it later established the Thames Conservancy Board to regulate potable water quality (Hazen 1913). By 1894, five successive increases in filter area had occurred for all London filters, and the total surface area was 470,000 m² (square meters), or 116 acres, producing 890,000 m³/d (cubic meters per day), or 234 mgd (million gallons per day).

Continental Europe began filtering public water supplies by the 1850s, with installations and dates as follows: Berlin, 1856; Altona, 1860; Zurich, 1884; Hamburg, 1893; and Budapest, 1894 (Hazen 1913). The health benefit of slow sand filtration was demonstrated dramatically by the 1892 cholera epidemic. In Hamburg, the epidemic resulted in 8,605 deaths. By contrast, the adjacent and downstream city of Altona had only a few deaths. Even though the intake for Altona's water supply was from the Elbe River downstream from Hamburg, Altona had slow sand filtration.

The first slow sand filter in the United States was installed in 1872 for the town of Poughkeepsie, NY, and was designed by James Kirkwood (Baker 1948). Installations followed at Hudson, NY, in 1874; St. Johnsbury, VT, in 1882; and Lawrence, MA, in 1894. By 1899, there were 20 slow sand filters in the United States, with an aggregate capacity of 200,000 m³/d (52.6 mgd), while rapid rate filters numbered 153 (Hazen 1913) and had an aggregate capacity of 900,000 m³/d (237 mgd). In 1940, the United States had about 100 slow sand filters and about 2,275 rapid rate filters (Logsdon and Fox 1988). Hazen (1913) provided a summary of countries and regions that had slow sand filtration installations in 1900; his list is reproduced in Table 1.1. The U.S. cities with slow sand filters in 1900 are listed in Table 1.2. Hazen qualified the listings in these tables as "partial" listings, probably to account for cities and countries that may have been overlooked by the survey.
Table 1.1
Summary of Countries and Regions Using Slow Sand Filtration in 1900

<table>
<thead>
<tr>
<th>Place</th>
<th>Population</th>
<th>Area of filters (acres)</th>
<th>Number of filters</th>
<th>Average daily consumption (mgd)</th>
</tr>
</thead>
<tbody>
<tr>
<td>United States</td>
<td>259,774</td>
<td>17.31</td>
<td>45</td>
<td>26.87</td>
</tr>
<tr>
<td>British Columbia</td>
<td>16,841</td>
<td>0.82</td>
<td>3</td>
<td>1.80</td>
</tr>
<tr>
<td>South America</td>
<td>500,000</td>
<td>4.15</td>
<td>3</td>
<td>—</td>
</tr>
<tr>
<td>Holland</td>
<td>1,414,021</td>
<td>22.75</td>
<td>47</td>
<td>31.48</td>
</tr>
<tr>
<td>Great Britain</td>
<td>10,100,738</td>
<td>161.80</td>
<td>161</td>
<td>382.73</td>
</tr>
<tr>
<td>Germany</td>
<td>4,639,080</td>
<td>106.22</td>
<td>185</td>
<td>117.13</td>
</tr>
<tr>
<td>Other European Countries</td>
<td>2,984,839</td>
<td>34.74</td>
<td>88</td>
<td>88.84</td>
</tr>
<tr>
<td>Asia</td>
<td>1,397,000</td>
<td>6.69</td>
<td>23</td>
<td>—</td>
</tr>
<tr>
<td><strong>Totals</strong></td>
<td>21,411,293</td>
<td>354.48</td>
<td>555</td>
<td>648.85</td>
</tr>
</tbody>
</table>

*No data.
Source: Adapted from Hazen (1913).

Table 1.2
Cities in the United States Using Slow Sand Filtration in 1900

<table>
<thead>
<tr>
<th>Place</th>
<th>When built</th>
<th>Population in 1890</th>
<th>Area of filters (acres)</th>
<th>Number of filters</th>
<th>Average daily use (mgd)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Poughkeepsie, NY</td>
<td>1872</td>
<td>24,000</td>
<td>1.36</td>
<td>3</td>
<td>1.67</td>
</tr>
<tr>
<td>Hudson, NY</td>
<td>1874</td>
<td>9,970</td>
<td>0.74</td>
<td>2</td>
<td>1.50</td>
</tr>
<tr>
<td>St. Johnsbury, VT</td>
<td>1882</td>
<td>3,857</td>
<td>0.14</td>
<td>3</td>
<td>0.70</td>
</tr>
<tr>
<td>Lawrence, MA</td>
<td>1893</td>
<td>3,268</td>
<td>0.11</td>
<td>1</td>
<td>0.09</td>
</tr>
<tr>
<td>Nantucket, MA</td>
<td>1893</td>
<td>44,654</td>
<td>2.50</td>
<td>1</td>
<td>3.00</td>
</tr>
<tr>
<td>Ilion, NY</td>
<td>1893</td>
<td>4,057</td>
<td>0.14</td>
<td>2</td>
<td>0.50</td>
</tr>
<tr>
<td>Mount Vernon, NY</td>
<td>1894</td>
<td>10,830</td>
<td>1.10</td>
<td>3</td>
<td>1.66</td>
</tr>
<tr>
<td>Grand Forks, ND</td>
<td>1894</td>
<td>4,979</td>
<td>0.42</td>
<td>1</td>
<td>—</td>
</tr>
<tr>
<td>Milford, MA</td>
<td>1895</td>
<td>9,956</td>
<td>0.25</td>
<td>1</td>
<td>0.70</td>
</tr>
<tr>
<td>Ashland, WI</td>
<td>1895</td>
<td>9,956</td>
<td>0.50</td>
<td>3</td>
<td>1.09</td>
</tr>
<tr>
<td>Hamilton, NY</td>
<td>1895</td>
<td>1,744</td>
<td>0.12</td>
<td>1</td>
<td>0.03</td>
</tr>
<tr>
<td>Lambertville, NJ</td>
<td>1896</td>
<td>4,142</td>
<td>0.28</td>
<td>2</td>
<td>0.25</td>
</tr>
<tr>
<td>Far Rockaway, NH</td>
<td>1896</td>
<td>2,288</td>
<td>0.92</td>
<td>2</td>
<td>0.93</td>
</tr>
<tr>
<td>Red Bank, NJ</td>
<td>1897</td>
<td>500</td>
<td>0.03</td>
<td>2</td>
<td>0.10</td>
</tr>
<tr>
<td>Somersworth, NH</td>
<td>1897</td>
<td>6,207</td>
<td>0.50</td>
<td>1</td>
<td>—</td>
</tr>
<tr>
<td>Little Falls, NY</td>
<td>1898</td>
<td>8,783</td>
<td>0.76</td>
<td>1</td>
<td>—</td>
</tr>
<tr>
<td>Berwyn, PA</td>
<td>1898</td>
<td>826</td>
<td>0.52</td>
<td>3</td>
<td>—</td>
</tr>
<tr>
<td>Harrisburg, PA</td>
<td>1899</td>
<td>1,200</td>
<td>0.12</td>
<td>2</td>
<td>0.15</td>
</tr>
<tr>
<td>Albany, NY</td>
<td>1899</td>
<td>94,923</td>
<td>5.60</td>
<td>8</td>
<td>11.00</td>
</tr>
<tr>
<td>Rock Island, IL</td>
<td>1899</td>
<td>13,634</td>
<td>1.20</td>
<td>3</td>
<td>3.50</td>
</tr>
<tr>
<td><strong>Totals</strong></td>
<td>259,774</td>
<td>17.31</td>
<td>45</td>
<td></td>
<td>26.87</td>
</tr>
</tbody>
</table>

*No data.
Source: Adapted from Hazen (1913).
1.3 THEORY

The purpose of filtration is to remove particles from a fluid. Table 1.3 lists many of the kinds of particles found in ambient raw waters. Zeta potentials are also given, to the extent that data are available. The zeta values are for pH 7. Zeta potential is a measure of the energy potential that characterizes particle interaction and is usually negative for particles in ambient waters. Of particular concern to the water supply industry are the pathogens, such as *Giardia lamblia* cysts, *Cryptosporidium* oocysts, and various bacteria and enteric viruses. Figure 1.2 is a photomicrograph showing cysts of *Giardia lamblia*, a pathogen that has been the focus of much attention since about 1978.

The tabular listing in Table 1.3 is simplified. For example, the category of parasitic eggs includes those from tapeworms, nematodes, and coccidia. Coccidia are protozoans.

<table>
<thead>
<tr>
<th>Category</th>
<th>Group/name</th>
<th>Size (micrometers)</th>
<th>Zeta potential (millivolts)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mineral</td>
<td>Clays (colloidal)*</td>
<td>0.001-1*</td>
<td>-15-20*</td>
</tr>
<tr>
<td></td>
<td>Silicatesd</td>
<td>no data</td>
<td>no data</td>
</tr>
<tr>
<td></td>
<td>Nonsilicatese</td>
<td>no data</td>
<td>no data</td>
</tr>
<tr>
<td>Biological</td>
<td>Viruses</td>
<td>0.01-1*</td>
<td>no data</td>
</tr>
<tr>
<td></td>
<td>Bacteria</td>
<td>0.3-10*</td>
<td>no data</td>
</tr>
<tr>
<td></td>
<td><em>Giardia lamblia</em> cysts</td>
<td>10^8</td>
<td>-33*</td>
</tr>
<tr>
<td></td>
<td>Algae, unicellular</td>
<td>30-50</td>
<td>no data</td>
</tr>
<tr>
<td></td>
<td>Parasite eggs</td>
<td>10-50^l</td>
<td>no data</td>
</tr>
<tr>
<td></td>
<td>Nematode eggs</td>
<td>10^l</td>
<td>no data</td>
</tr>
<tr>
<td></td>
<td><em>Cryptosporidium</em> oocysts</td>
<td>4-5^h,j</td>
<td>33^h</td>
</tr>
<tr>
<td></td>
<td>Biological concentrate</td>
<td>mixture</td>
<td>13.5^c</td>
</tr>
<tr>
<td></td>
<td>from 5-μm cartridge filter</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Other particles</td>
<td>Amorphous debris, small</td>
<td>1-5^h</td>
<td>no data</td>
</tr>
<tr>
<td></td>
<td>Amorphous debris, large</td>
<td>5-500^h</td>
<td>no data</td>
</tr>
<tr>
<td></td>
<td>Organic colloids^k</td>
<td>no data</td>
<td>no data</td>
</tr>
</tbody>
</table>

*Clays include montmorillonite, kaolinite, and illite, to name a few groups.*
*Tate and Trussell (1978).*
*Al-Jadhai and Hendricks (1989) measured -16.4 mv (millivolts) for raw water from Cache La Poudre samples having turbidity levels of 0.25 NTU (nephelometric turbidity units) and particle counts of 600,063 particles/10 mL (milliliters) using a 70-μm aperture.*
*The Safe Drinking Water Committee (1977) lists 28 minerals that belong to the silica group.*
*The Safe Drinking Water Committee (1977) includes compounds of iron, calcium, aluminum, magnesium, etc., in the nonsilicates group.*
*Beard and Tanaka (1977).*
*SJakubowski and Hoff (1979).*
*Ongerth (pers. com., July 27, 1990); data are for pH = 7.0.*
*Al-Jadhai and Hendricks (1989).*
*Hibler (pers. com., August 23, 1990).*
*The Safe Drinking Water Committee (1977) lists organic colloids as a type of particle found in ambient waters but gives no data for these particles.*
Cryptosporidium is a coccidia, but it is at the low end of the size range (Hibler 1990). Beaver and muskrat are loaded with coccidia, and the level of coccidia in their bodies can be an indicator of the level in the water. That, in turn, can be an indicator of filter plant performance. If coccidia are found in plant effluent, then Giardia cysts could be present also. Size ranges are not literal, as needle-shaped diatoms may be 100 μm (microns) in length, and some algae form chains several hundred microns in length.

How slow sand filtration removes these particles is not well understood, but research (Fox et al. 1984; Cleasby et al. 1984a, 1984b; Bellamy et al. 1985a, 1985b, 1985c; Pyper 1985; Bryck et al. 1987), pilot plant work, observations of full-scale plants, design experience, and operating experience all add up to a picture of what happens and provide a basis for rational slow sand filter design. Zeta potential is likely to be a factor in the removal of these particles, as it is in rapid rate filtration, but the mechanism is not understood and the zeta potential data are given only for reference. Ongerth (pers. com., July 27, 1990) noted that zeta potentials are affected by pH. He also noted that the zeta potentials of algae, which vary with species, are affected substantially by the presence of organic molecules, such as fulvic and humic acids, that adsorb on the particles.

Some of these particles can be removed from raw water at levels of 2-log to 4-log (99 to 99.99 percent) by biologically mature slow sand filters, as discussed in Section 1.3.1.
Although mature slow sand filters will remove most types of particles, including turbidity, Bellamy et al. (1985a) reported raw and finished water turbidities of about 4.5 and 3.5 NTU (nephelometric turbidity units), respectively, in their experiments with slow sand filters. Further investigation showed that the particles constituting the turbidity were montmorillonite and kaolinite having sizes of 2–6 μm, with many particles smaller than 0.5 μm.

1.3.1 Principles of Removal

This section reviews removal theory, as understood at this time, and hydraulic theory as described by Darcy's law. These theories provide a rationale for slow sand filtration that may be useful in the design of the filters.

Particle removal in slow sand filtration may occur by some of the same mechanisms as in rapid rate filtration. Iwasaki (1937) described the filter removal coefficient, \( \lambda \), in rapid rate filtration theory, and Ives (1961) verified it. This coefficient, as expanded by Yao et al. (1971), may be resolved into two components: \( \eta \), the collision probability coefficient, and \( \alpha \), the attachment coefficient. These two coefficients are expressed, respectively, in terms of the two steps of filtration removal: (1) transport, and (2) attachment. Once attachment has occurred, the biofilm on the sand grains making up the sand bed may metabolize organic contaminants, resulting in permanent removal of the contaminant particles. The *schmutzdecke* also provides some removal.

(1) Transport. Figure 1.3 illustrates the transport step, which is the mechanism by which collisions occur between particles and sand grains (called collectors in the literature). The transport mechanisms include interception, sedimentation, and diffusion. To understand these mechanisms, consider first the manner in which a fluid behaves around an obstruction, such as a sand grain. Figure 1.3 shows how the flow patterns of the fluid, which can be depicted in terms of streamlines, are altered by the presence of the sand grain, idealized in the figure as a sphere. If a particle, represented in the figure by a black dot, is carried with the flow (that is, if it is convected), it may collide with the sand grain, attach to it, and thus be removed.

(2) Interception. One way in which a particle may collide with a sand grain is through interception, depicted by Figure 1.3a. An interception can occur only if a particle is carried by one of the streamlines closest to the sand grain, such that a brushing effect occurs. The larger the particle, the more likely a brush will occur.

(3) Sedimentation. A second transport mechanism is sedimentation, depicted in Figure 1.3b. The force of gravity acts on all particles, giving a vertical velocity component as shown. When the vertical velocity component is added vectorially with the convection velocity, the resultant velocity of the particle may cause it to collide with the
sand grain. Sedimentation will have a perceptible role only for particles larger than 10 μm (Yao et al. 1971).

(4) Diffusion. A third transport mechanism is diffusion, shown in Figure 1.3c. The thermal energy of gases and liquids manifests itself in the form of random motion of molecules. When these molecules collide with a small particle, the particle will also move in a random fashion. The motion of the particle is then in a series of discrete steps, often called a random walk. If the particle is being convected by a flow, then the diffusion is superimposed on the convection flow, and the particle will move from one streamline to another. Eventually, the particle may collide with a sand grain surface. As may be inferred, the lower the convection velocity, the more steps a particle can take per unit time; hence, the probability of a collision increases as the interstitial velocity decreases. Also, as temperature increases, providing more thermal energy, the number of steps per unit time increases and so then does the probability of a collision. Diffusion is most important for particles smaller than 1 μm (Yao et al. 1971).

(5) Interstitial Flow. The streamlines shown in Figure 1.3 are, of course, idealized for a single particle. Within a packed bed with many sand grains, the streamlines have a more tortuous configuration, as depicted in Figure 1.4. By definition, the flows between any two streamlines are equal and the space within is called a streamtube. In the...
tortuous configuration of the streamlines, the streamtubes bifurcate (that is, branch), rejoin, and bifurcate again at random points.

This continuous bifurcation creates the opportunity for collisions between particles and sand grains by impingement. As indicated in Figure 1.4, any particle within the interstitial stream will most likely, at some point during its path, impinge upon a sand grain. The probability of an impingement within a given distance of travel depends upon the size of the sand grains, the velocity of the stream, and temperature. The smaller the sand grains, the higher the probability of an impingement; there are simply more bifurcations of an interstitial stream with smaller sand grains. Also, the lower the interstitial velocity, the higher the probability of impingement. As previously noted, lower velocity allows more "steps" of random motion by diffusion per unit distance. However, as interstitial velocity increases, there is a point above which higher velocity makes no further difference. Finally, higher temperature results in more random motion "steps" than lower temperature and hence in a higher probability of impingement.

(6) Collision Probability. The discussion thus far has related to the probability of a collision between a particle and a collector (sand grain), expressed by the coefficient $\eta$. The number of collisions per unit distance of travel determines the potential for removal by filtration. Whether removal actually occurs depends upon whether attachment occurs.
(7) Attachment. Unless attachment occurs, there is no removal. The fraction of particles that attach, relative to the number of collisions, is by definition the coefficient \( \alpha \). Research suggests that biofilm development on the sand grains provides an adsorptive surface for such attachment. Another idea is that extracellular enzymes coagulate the particles, thus permitting attachment. Whether \( \alpha \) is low or high is not known.

When a filter is first started, and before a biofilm develops, the coliform removal is about zero (Bryck et al. 1987), that is, \( \alpha = 0 \). After a biofilm develops, the removal rate is 2-log to 4-log, that is, \( \alpha \to 1.0 \), indicating the importance of the biofilm in slow sand filtration. (Because coliforms may die or be eaten by predators during the time it takes for them to reach an adsorptive surface, the attrition noted could be due to death or predation in addition to adsorption; certainly, after adsorption occurs, death and predation will occur.) When a biofilm develops to its maximum extent for the given conditions, the filter is called mature. The maximum extent of biofilm development has not been defined, and further research is needed to address this important question. However, research by Bellamy et al. (1985a) and Barrett (1989) and experiments with pilot filters at 100 Mile House, BC (Bryck et al. 1987), have shown that the maximum extent of biofilm development for nutrient-limited water is much less than for nutrient-rich water. Slow sand filters located in nutrient-limited situations may be expected to have 2-log coliform removals after biofilm maturity (Bellamy et al. 1985a); filters in nutrient-rich waters may be expected to have 3-log coliform removals (Bellamy et al. 1985a) and some may even have 4-log removals (Barrett 1989).

(8) Iwasaki's Equation. A mathematical description of rapid rate filtration particle removal in a packed bed of granular media was given by Iwasaki (1937) as,

\[
\frac{dC}{dz} = -\lambda C
\]  

(1.1)

in which \( C \) = concentration of particles (in number of particles/mL) 
\( z \) = distance from top of filter bed at which \( C \) is measured (in meters, m) 
\( \lambda \) = filter coefficient (cm\(^{-1}\))

Equation 1.1 when integrated has the form of an exponential decline of particle concentration with distance. The coefficient \( \lambda \) determines the efficiency of the filtration process. The higher the magnitude of \( \lambda \), the steeper the concentration profile. The coefficient \( \lambda \) is the product of the transport probability coefficient, \( \eta \), and the attachment coefficient, \( \alpha \) (for further discussion, see Yao et al. 1971).

(9) Metabolism. When biological particles attach to the biofilm, the microorganisms constituting the biofilm will most likely metabolize them. This hypothesis
explains the observations that while large spikes of Giardia cysts have been applied to pilot filters, only a few of the cysts have been observed to pass through the filters. According to this hypothesis, after attachment occurs, the cysts are attacked by the microfauna making up the biofilm. However, the removals may also be the result of straining (i.e., the physical retention of particles too large to pass through the interstitial pores). The idea of cysts being metabolized by a biofilm has not yet been fully established.

(10) Schmutzdecke. The term schmutzdecke is a German word that translates literally as "dirty skin." The word was adopted early in American practice and is still used. The schmutzdecke is defined here as a layer of material, both deposited and synthesized, on the top of the filter bed that causes headloss disproportionate to its thickness. After the schmutzdecke is removed, the headloss across the filter bed should recover to the "clean-bed" amount. The clean-bed headloss is normally 15 to 30 cm (6 to 12 in.); at the end of a filter run, the headloss will reach whatever depth is permitted by the height of the filter box.

As noted in the definition, the schmutzdecke is composed of both deposited and synthesized material. It is characterized usually as a gelatinous mat in which microorganisms thrive and, indeed, cause the major portion of the removals that occur. The following passage by Babbitt and Doland (1939, p. 536) may be considered representative of the lore:

The greatest portion of the action of a filter occurs at the surface of the sand in the layer of matter deposited thereon. This layer contains a zoogaeal jelly in which biological activities are at their highest. This layer of material at the surface is called the schmutzdecke (dirty skin). The successful operation of the filter is dependent on it. Until the schmutzdecke is built up, the bacterial removal may be low. The building up of the schmutzdecke, particularly in slow sand filtration, is known as the "ripening of the filter". It may require some time after a filter has been put into service before the desired efficiency of bacterial removal is secured. Frequently a satisfactory effluent can be obtained immediately. The character of the raw water, the rate of filtration, and other factors determine this. As the filter ages, the pressure necessary to force the water through the schmutzdecke and the sand becomes so great as to necessitate the cleaning of the filter.

Further description is given by Huisman and Wood (1974, p. 20):

On the surface of the sand there is a thin slimy matting of material, largely organic in origin, known as the schmutzdecke, or filter skin, through which the water must pass before reaching the filter medium itself. The schmutzdecke consists of threadlike algae and numerous other forms of life, including plankton, diatoms, protozoa, rotifers, and bacteria. It is intensely active, the various microorganisms entrapping, digesting, and breaking down organic matter contained in the water passing through. Dead algae from the water above and living bacteria in the raw water are alike consumed within this filter skin, and in the process simple inorganic salts are formed. At the same time nitrogenous compounds are broken down and the nitrogen is oxidized. Some colour is removed, and a considerable proportion of inert suspended particles is mechanically strained out.
Although the *schmutzdecke* has been described in the lore as a gelatinous zoogeal mass of living and dead microorganisms, its character, in fact, can vary widely. The *schmutzdecke* in the filter at Empire, CO, has been described as a light, easily suspended, inert, black carbonaceous deposit, about 1 mm in thickness. Figure 1.5 is a photograph of the Empire, CO, filter, showing the *schmutzdecke* on the right, composed of the carbonaceous detritus, and a section of scraped sand on the left. The raw water turbidity at Empire seldom exceeds 0.5 NTU. Schuler et al. (forthcoming) described a similar observation from the filtering of water in Pennsylvania having turbidities of 0.1–5.8 NTU. The *schmutzdecke* they described was "tightly packed and unattached to the sand." Filter scrapings they made on April 10, 1988, following 75 days of winter operation, showed almost no biological growth in the *schmutzdecke*. A diverse biological community was found, however, in the top layers of the sand bed. Bellamy et al. (1985a, 1985b) did not find a well-defined *schmutzdecke* in pilot filters at Colorado State University. Scraping the surface showed an effect, however, in that the clean-bed headloss was recovered. Collins (pers. com., August 16, 1990) reported that the *schmutzdeckes* at slow sand pilot filters in Portsmouth, NH, and Ashland, NH, were composed of gelatinous organic matter and resembled the classic description.

Whatever the character of the *schmutzdecke*, a deposit of some sort occurs in every slow sand filter and causes headloss to increase. Removing the *schmutzdecke* by scraping will cause the headloss to recover to the clean-bed level. While the *schmutzdecke* may have a primary role in removal when it is composed of a zoogeal mass, when it is merely a carbonaceous deposit, the sand bed maturity is most important.

(11) Relative Roles of the Schmutzdecke and Sand Bed in Particle Removal. In the literature on slow sand filtration, most particle removal is ascribed to the *schmutzdecke*. Data from Bellamy et al. (1985a), however, showed that at a hydraulic loading rate (HLR) of 0.12 m/hr, the total coliform removal was 3-log with a mature sand bed and *schmutzdecke*; when the *schmutzdecke* was scraped, the percent removal was still 2-log. Thus, the *mature* sand bed was felt to be responsible for most of the removals. In another test, Bellamy and co-workers (1985a) added nutrients to one of six filters and found that the removals of total coliform bacteria for that filter were 3-log as opposed to 2-log for the filter without nutrients added (the control). The nutrients added to the filter evidently caused an acceleration in the rate of development of the sand bed's biological maturity and perhaps in the overall level of its development.

The upper level of the filter bed, where bacteria concentrations are highest, is the most effective zone for particle removal. Hazen (1913) reported concentrations of 10^6 bacteria/gram of media at the surface of the filter bed and an exponential decline with
Figure 1.5 Scraped Sand Bed at Empire, CO, Contrasted With Schmutzdecke After 30 Days of Operation. The Schmutzdecke is composed of a carbonaceous deposit. (Photograph by D. W. Hendricks.)

depth to $10^5$ bacteria/gram of media at 2-cm depth. Collins et al. (1989) reported $10^9$ bacteria/gram of dry media at the surface, declining to $10^6$–$10^7$ bacteria/gram of media at 30–45 cm (12–18 in.). The conclusion may be that where ever the metabolizing bacteria are located, whether in the schmutzdecke or within the sand bed, there is capability to remove organic matter of various classes and nutrients. The question of the role of the schmutzdecke vis-à-vis the sand bed in removals cannot be answered definitively at this time. As noted, if the sand bed is biologically mature, then the schmutzdecke may have a less critical role; but if the sand bed is not mature, the schmutzdecke will have greater importance, assuming it has significant biological activity. The perspective of Huisman and Wood (1974, pp. 21–22) on this issue is useful. Their statements are consistent with the findings noted above.

When James Simpson installed his first slow sand filters nearly a century and a half ago, he had no idea of the complex processes of purification he was initiating. He looked upon his sand bed as a very effective strainer that would retain those particles that were larger than the interstices between the sand grains. This straining action does undoubtedly take place, though in view of the preliminary screening undergone by the water in passing through the schmutzdecke it is unlikely that mechanical straining within the bed constitutes more than a small part of the total
purification process. Only gradually, as the nature of colloids, bacteria, and viruses became known, did earlier concepts become obviously insufficient to explain the removal of these particles, the dimensions of which are much smaller than the pore sizes of the finest sand used in the filter bed. Nevertheless the fallacy of regarding filter media solely as straining mechanisms has persisted until comparatively recently, and unwarranted doubts about the efficacy of biological filtration have been raised by falsely equating the results of laboratory tests, in which pathogens and some parasites have been shown to pass through a column of clean sand, with the conditions that prevail in a working filter through which the same organisms undoubtedly could not pass.

A more significant property of the sand bed is adsorption, a phenomenon resulting from electrical forces, chemical bonding, and mass attraction interacting in a way that is not yet completely understood. Adsorption takes place at every surface at which water comes in contact with a sand grain. To appreciate the extent of this action it is necessary to visualize the interior of the sand bed as a series of grain surfaces over which the water must pass. The aggregate area of these surfaces is extremely high; in one cubic meter of filter sand there will be some 15,000 m²—one and a half acres—of surface. Over this the water passes in a laminar flow that is constantly changing in direction as it leaves one grain and meets the next. At each change of direction gravity and centrifugal forces act upon every particle carried by the water.

Between the grains are the pores, or open spaces, totalling some 40 percent of the total volume of the bed. Water passing over a grain surface is suddenly slowed down each time it enters one of these pores, and as a result millions of minute sedimentation basins are formed in which the smallest particles settle onto the nearest sand grain before the water continues on its downward path.

Hence during the passage of the water through the bed every particle, bacterium, and virus is brought into contact with the surfaces of the sand grains, to which they become attached by mass attraction or through the operation of electrical forces. The surfaces become coated with a sticky layer, similar in composition to the schmutzdecke, but without the larger particles and the algae, which have failed to penetrate. It sustains a teeming mass of microorganisms, bacteria, bacteriophages, rotifers, and protozoa, all feeding on the adsorbed impurities and on each other. The living coating continues through some 40 cm of the bed, different life forms predominating at different depths, with the greatest activity taking place near the surface, where food is most plentiful.

The food consists essentially of particles of organic origin carried by the water. The sticky coating holds the particles until they are broken down, consumed, and formed into cell material, which in turn is assimilated by other organisms and converted into inorganic matter such as water, carbon dioxide, nitrates, phosphates, and similar salts that are carried downward by the passing water. As the depth from the surface increases the quantity of organic food decreases and the struggle among the various organisms becomes fiercer. Other bacteria then predominate, utilizing the oxygen content of the water and extracting nutrients that would otherwise have passed unchanged in solution through the filter. As a consequence the raw water, which entered the bed laden with a variety of suspended solids, colloids, microorganisms, and complex salts in solution, has, in its passage through some 40-60 cm of filter medium, become virtually free of all such matter, containing only some simple (and relatively innocuous) inorganic salts in solution. Not only has practically every harmful organism been removed but also the dissolved nutrients that might encourage the subsequent growth of bacteria or slimes. It may be low in dissolved oxygen and may contain dissolved carbon dioxide but subsequent aeration caused by (the water) falling over the discharge weir will go far to remedying both these defects.

In tests on working filters it is not uncommon to find the total bacteria count reduced by a factor of between 1000 and 10,000, and the *Escherichia coli* count by a factor of between 100 and 1000. Starting with an average quality of raw water it is usual to find *E. coli* absent in a 100 mL sample of delivered water, thus satisfying normal drinking water quality standards.
1.3.2 Headloss

Headloss within a slow sand filter is caused by flow through the *schmutzdecke* and the sand bed. As the filter is operated, the *schmutzdecke* develops and its hydraulic resistance increases, causing most of the headloss. Removal of the *schmutzdecke*, usually by some means of scraping for small filters, will permit the headloss to recover to the clean-bed level. The "clean-bed headloss" is usually about 10 cm, but the level depends upon the hydraulic loading rate, the temperature, and the sand bed media characteristics. Darcy's law integrates these variables and is reviewed in the next section. Darcy's law can be applied to flow in the laminar range through any porous medium, including the *schmutzdecke*.

(1) *Darcy's Law.* Headloss within any porous medium is described by Darcy's law,

\[ v = -k \frac{dh_L}{dz} \quad (1.2) \]

in which \( v \) = superficial velocity, also called hydraulic loading rate, \( Q/A \) (m/hr)

\( h_L \) = headloss available across the filter bed from headwater to tailwater (m)

\( z \) = flow distance through porous media (m)

\( k \) = hydraulic conductivity of porous media (m/hr)

\( dh/dz \) = hydraulic gradient, loss of head/unit length of flow (m/m)

As noted in the definition for \( v \), superficial velocity is synonymous with the frequently used expression, hydraulic loading rate, HLR, which is defined,

\[ \text{HLR} = \frac{Q}{A} \quad (1.3) \]

in which \( \text{HLR} \) = hydraulic loading rate, defined as flow divided by plan area of sand bed (m³/m²/hr or mgad)

\( Q \) = flow of water (m³/s or ft³/s)

\( A \) = plan area of filter bed (m²)

For homogeneous porous media, Darcy's law usually is expressed in finite terms, that is,

\[ v = -k \frac{h_L}{\Delta z} \quad (1.4) \]

in which \( h_L \) = headloss across sand bed (m)

\( \Delta z \) = depth of filter bed (m)
Figure 1.6 illustrates the definitions of terms in Darcy's law. The sand bed is oriented horizontally to show more clearly how the hydraulic head, $h$, is associated with the flow distance, $z$, in the sand bed. The headloss, $h_{L}$, determines the superficial velocity, $v$, through the sand bed. In other words, for a given sand and a given headloss, $v$ is determined by Equation 1.2.

(2) Temperature Effect. Equation 1.5 expresses Darcy's law in terms of intrinsic hydraulic conductivity, $k'$:

$$v = \frac{k' \cdot h_{L}}{\mu \cdot \Delta z}$$

(1.5)

in which $k'$ = intrinsic hydraulic conductivity (N/m [newtons per meter] or lb/ft [pounds per foot]

$\mu$ = dynamic viscosity of water at given temperature (N-s/m² [newton seconds per square meter])

This equation shows that the hydraulic conductivity, $k$, of a sand is a function of both the dynamic viscosity, $\mu$, a fluid property, and the intrinsic hydraulic conductivity, $k'$, a porous media property.

![Figure 1.6 Sketch Showing the Relationships of the Variables Defined by Darcy's Law](image)
To illustrate the effect of temperature on superficial velocity, we can plug values for dynamic viscosities at various temperatures (obtained from Appendix Table A.1) into Equation 1.5. Take, for example, the values for 0°C and 25°C:

\[ \mu(0°C) = 1.8 \times 10^{-3} \text{ N-s/m}^2 \]

\[ \mu(25°C) = 0.9 \times 10^{-3} \text{ N-s/m}^2 \]

Because the intrinsic hydraulic conductivity, \( k' \), is constant for a given sand, the above data show that for a given headloss, the superficial velocity, \( v \), will increase by a factor of two when temperature changes from 0°C to 25°C. In other words, temperature is an important factor. Another way to express the effect of temperature on velocity is in terms of headloss. For example, again using Equation 1.5, if the superficial velocity, \( v \), is maintained constant, the headloss will be twice as much at 0°C than at 25°C.

3) Headloss. Figure 1.7 shows the headloss, as measured by piezometers, across a sand bed after the development of a schmutzdecke. Piezometers are installed in the walls of the filter box, approximately as shown. The largest headloss occurs across the schmutzdecke and is measured by piezometers A and B. The headloss across the sand bed is measured by piezometers B, C, and D. As illustrated, the headloss across the sand bed is small relative to that across the schmutzdecke.

The cycle of operation of a slow sand filter is as follows:

1. When the water level within the filter box is at its maximum height (elevation 1 in Figure 1.6), the schmutzdecke is scraped.
2. The filter is put back into operation with water at level 2, and overall headloss is the vertical distance from level 2 to level 5.
3. As the schmutzdecke develops again, the headloss across it increases, causing the water level to rise again to level 1.

Through this cycle, the velocity, and consequently the flow, \( Q \), to the filter box, are constant. The operator of a slow sand filter can follow the above cycle, using the piezometers to anticipate when scraping is needed. By maintaining records of headloss, flow, temperature, and bed depth, the operator can calculate the intrinsic hydraulic conductivity, \( k' \). If a filter bed clogging problem is suspected, the monitoring of \( k' \) permits diagnosis.

4) Intrinsic Hydraulic Conductivity. The intrinsic hydraulic conductivity, \( k' \), of a clean sand bed is a function of the sand size, the sand size distribution, and the aggregation of the sand (that is, the extent to which the sand packs grain to grain
vis-à-vis voids being formed by particle bridging). The hydraulic conductivity cannot be predicted; it can only be measured by tests. Table 1.4 gives hydraulic conductivity data from a laboratory test, from several pilot filters (30.54 cm in diameter), and from several full-scale slow sand filters. The terms $d_{10}$ and UC, used in the table, were developed by Allen Hazen around the turn of the century to characterize sands. They are defined in the table footnotes. The $k'$ values for the sands indicated are all within the same order of magnitude, ranging from $2.5 \times 10^{-7}$ to $2.5 \times 10^{-6}$ N/m. The highest $k'$ values are for sands having $d_{10}$ sizes of 0.62 and 0.92 mm (millimeters), respectively. These larger sands were used for experimental purposes—to examine the role of sand size in process efficiency. Their $k'$ values are included here to add perspective to the other data given; sand of this size is not intended to be considered for an installation.

When the intrinsic hydraulic conductivity, $k'$, is measured, the viscosity factor is removed from concern, permitting evaluation in terms of only a porous media characteristic. The $k'$ values may be used to monitor whether bed clogging is taking place and to determine whether a sand being considered for an installation has an intrinsic hydraulic conductivity that falls within an expected range. In addition, one may use $k'$ values to calculate the clean-bed headloss under different sand-bed-depth and temperature scenarios. As will be seen later, one can also use $k'$ values to determine the headloss penalty, such as by electing to design a sand bed that is deeper than
Table 1.4
Hydraulic Conductivities for Slow Sand Filters

<table>
<thead>
<tr>
<th>Installation</th>
<th>(d_{10}) (^a) (mm)</th>
<th>UC(^b)</th>
<th>Testing method</th>
<th>(k') ((N/m))</th>
<th>(k(25^\circ C)) ((m/hr))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Empire(^c)</td>
<td>0.21</td>
<td>2.67</td>
<td>Full-scale</td>
<td>(6.6 \times 10^{-7})</td>
<td>2.65</td>
</tr>
<tr>
<td>Empire(^c)</td>
<td></td>
<td></td>
<td>Lab column</td>
<td>(0.9 - 2.1)</td>
<td></td>
</tr>
<tr>
<td>100 Mile House(^d)</td>
<td>0.25</td>
<td>3.5</td>
<td>Full-scale</td>
<td>(5.05 \times 10^{-7})</td>
<td>2.04</td>
</tr>
<tr>
<td>CSU pilot plants(^e)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Phase I</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Filter #1</td>
<td>0.27</td>
<td>1.63</td>
<td>Pilot plants, 30.5-cm dia.</td>
<td>(4.6 \times 10^{-7})</td>
<td>1.46</td>
</tr>
<tr>
<td>Filter #2</td>
<td>0.27</td>
<td>1.63</td>
<td>Pilot plants, 30.5-cm dia.</td>
<td>(7.1 \times 10^{-7})</td>
<td>2.28</td>
</tr>
<tr>
<td>Filter #3</td>
<td>0.27</td>
<td>1.63</td>
<td>Pilot plants, 30.5-cm dia.</td>
<td>(10.3 \times 10^{-7})</td>
<td>3.31</td>
</tr>
<tr>
<td>Phase II</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Filters #2, 3, 4, 6</td>
<td>0.29</td>
<td>1.53</td>
<td>Pilot plants, 30.5-cm dia.</td>
<td>(8.9 \times 10^{-7})</td>
<td>3.56</td>
</tr>
<tr>
<td>Filter #5</td>
<td>0.62</td>
<td>1.59</td>
<td>Pilot plants, 30.5-cm dia.</td>
<td>(2.5 \times 10^{-6})</td>
<td>10.0</td>
</tr>
<tr>
<td>Phase III</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Filter #5</td>
<td>0.13</td>
<td>1.60</td>
<td>Pilot plants, 30.5-cm dia.</td>
<td>(2.5 \times 10^{-7})</td>
<td>1.01</td>
</tr>
<tr>
<td>CU pilot plants(^f)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sand #1</td>
<td>0.22</td>
<td>2.50</td>
<td>Pilot plants, 30.5-cm dia.</td>
<td>(6.78 \times 10^{-7})</td>
<td>2.74</td>
</tr>
<tr>
<td>Sand #2</td>
<td>0.92</td>
<td>2.28</td>
<td>Pilot plants, 30.5-cm dia.</td>
<td>(2.73 \times 10^{-6})</td>
<td>11.05</td>
</tr>
<tr>
<td>Sand #3</td>
<td>0.20</td>
<td>4.15</td>
<td>Pilot plants, 30.5-cm dia.</td>
<td>(6.26 \times 10^{-7})</td>
<td>2.54</td>
</tr>
</tbody>
</table>

\(^a\)The letter "d" followed by a subscript number is used to indicate particular sizes of sand. For example, "d\(_{10}\)" indicates that 10 percent of the grains in the sand bed are finer than the d\(_{10}\) particles.

\(^b\)UC is the uniformity coefficient and is defined as the ratio of the d\(_{50}\) size to the d\(_{10}\) size.

\(^c\)For the Empire full-scale filters, the measurement of 1.7 m/hr at 5\(^\circ C\) converts to 2.65 m/hr at 25\(^\circ C\) (Seelaus et al. 1988).

\(^d\)For 100 Mile House, \(k'\) is calculated for 25\(^\circ C\), that is, \(\mu = 0.90 \times 10^{-3} N\cdot s/m^2\) (Bryck et al. 1987).

\(^e\)For the CSU filters, calculations of \(k'\) and \(k\) were based upon graphic headloss data (Bellamy et al. 1985a) and interpolation was accurate + or - about 30 percent. The calculations for \(k\) and \(k\) reflect this uncertainty.

\(^f\)The CU pilot plant data were compiled from Barrett (1989).

---

recommended. The data in Table 1.4 may provide initial guidance on what \(k'\) to expect for sand of various size ranges. Example 1.1 illustrates the usefulness of \(k'\) data when estimating clean-bed headloss and shows how Darcy's law may be applied.

Example 1.1: Darcy's Law Calculation. Calculate the clean-bed headloss for the slow sand filter at Empire, CO. Bed depth = 1.30 m, HLR = 0.2 m/hr, and temperature = 15\(^\circ C\).

1. Apply Darcy's law, Equation 1.5: Use \(k'\) as given in Table 1.4 and \(\mu(15^\circ C)\) from Appendix Table A.1.

\[
\begin{align*}
0.2 \left( \frac{m}{hr} \right) &= \frac{6.6 \times 10^{-7} (N/m)}{1.14 \times 10^{-3} (N-s/m^2)} \cdot \frac{h_L (m)}{1.3 (m)} \cdot \frac{(3600 \text{ s})}{(hr)} \\
\end{align*}
\]

2. Solve for \(h_L\):

\[
h_L = 12.5 \text{ cm (15}^\circ \text{C)}
\]
3. At 0°C:

\[ h_L = 19.5 \text{ cm} \]

4. Comments: The clean-bed headloss increases from 12.5 cm to 19.5 cm as the temperature decreases from 15°C to 0°C. Even at the highest clean-bed headloss value, approximately 0.20 m, however, the sand bed headloss is only about 13 percent of the terminal headloss (as determined by the depth of the filter box).

The terminal headloss permitted for the Empire filter is about 1.5 m. Example 1.1 shows that the clean sand bed accounts for only 12.5-19.5 cm of this total. Thus, the schmutzdecke accounts for most of the headloss, that is, about 87 percent. Pilot plant modeling is most useful for determining the hydraulic character of the schmutzdecke and for observing how it changes over the annual cycle. Understanding how Darcy's law is applied, as outlined above, helps in interpreting pilot plant behavior and in anticipating the behavior of a full-scale filter.

1.4 PRACTICE

By 1900, slow sand filtration practice had been described thoroughly, mostly by European experience. American practice was based upon technology brought from Europe by James Kirkwood in 1867 and Alien Hazen around 1892. Present practice in slow sand filtration is largely unchanged from that described in Hazen's 1913 book. Although the technologies available for scraping large filter areas have expanded, the basic concept has not varied.

1.4.1 Europe

As noted in Section 1.2, James Simpson's one-acre filter, completed in 1829 for the Chelsea Water Company in London, became the model for practice. Simpson based this filter design upon a pilot plant study he conducted over an eight-month period. The design of the full-scale filter is not reported, but the pilot filter had a sand bed that was 0.61 m (2 ft) deep, supported by a 0.61-m (2-ft) layer of gravel, with 23-cm x 23-cm (9-in. x 9-in.) lateral drains and a 35.6-cm x 23-cm (14-in. x 9-in.) main drain. The depth of water on the filter bed was 38 cm (15 in.) and the hydraulic loading rate was 0.18 m/hr (4.7 mgad). For the full-scale filter, the hydraulic loading rate was 0.12–0.14 m/hr (2.7–3.6 mgad). The data given by Hazen are assumed to be in Imperial gallons; for the purposes of this manual, they have been converted to U.S. gallons.

Following the lead of the water companies in England, mainland European countries built slow sand filters, and the practice became well established in Europe during the latter half of the nineteenth century. In his 1913 book, Hazen stated (p. 3) that
many European cities with aggregate populations of 20 million persons had filtration (presumably slow sand, since rapid rate filtration was not established in Europe at that time). In his list were 5 cities in England and 15 cities on the continent, including Antwerp, Altona, Berlin, Budapest, Zurich, The Hague, Amsterdam, and Rotterdam. Hazen gave a thorough review of slow sand filtration practice at that time, the main points of which are summarized in Table 1.5.

With regard to hydraulic loading rates, Hazen stated that every water has its own special rate of filtration, which must be determined by local experiments, and that this rate may vary widely from case to case. A sentence from his 1913 book (p. 51) gives his thoughts on the matter most succinctly: "Thus it is possible that the rate of 1.6 mgad adopted at Hamburg for the turbid Elbe water, the rate of 2.57 used at Berlin, and the rate

<table>
<thead>
<tr>
<th>Design parameter</th>
<th>Metric</th>
<th>English</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hydraulic loading rate</td>
<td>0.06–0.3 m/hr</td>
<td>1.6–7.5 mgad</td>
</tr>
<tr>
<td>Filter bed area</td>
<td>Calculated as:</td>
<td>A = Q/HLR</td>
</tr>
<tr>
<td>Depth of filter bed</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Initial</td>
<td>0.61–1.22 m</td>
<td>2–4 ft</td>
</tr>
<tr>
<td>Final</td>
<td>0.30–0.61 m</td>
<td>1–2 ft</td>
</tr>
<tr>
<td>Sand specification</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Effective size, $d_{10}$</td>
<td>0.18–0.44 mm</td>
<td>0.0071–0.017 in.</td>
</tr>
<tr>
<td>Uniformity coefficient (UC)</td>
<td>1.5–4.7</td>
<td>1.5–4.7</td>
</tr>
<tr>
<td>Depth of gravel support</td>
<td>0.61 m</td>
<td>2.0 ft</td>
</tr>
<tr>
<td>Drains (tile)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Laterals (diameter/area)</td>
<td>10 cm/26.9 m²</td>
<td>4 in./290 ft²;</td>
</tr>
<tr>
<td>Main</td>
<td>15.2 cm/69.7 m²</td>
<td>6 in./750 ft²;</td>
</tr>
<tr>
<td></td>
<td>$A = 1/6,000 \times$ x area drained</td>
<td>$A = 1/6,000 \times$ x area drained</td>
</tr>
<tr>
<td>Depth of supernatant water</td>
<td>0.91–1.32 m</td>
<td>36–52 in.</td>
</tr>
<tr>
<td>Headloss permitted</td>
<td>0.45–2.0 m</td>
<td>18–80 in.</td>
</tr>
<tr>
<td>Cover recommended</td>
<td>Yes, for cold climates</td>
<td>Yes, for cold climates</td>
</tr>
</tbody>
</table>

*a Supernatant water depth should be sufficient to prevent hydraulic scour. If ice forms, the supernatant depth should be sufficient to prevent the ice block from touching the sand surface.

Source: Adapted from Hazen (1913), p. 25.
of 7.5 used at Zurich for the almost perfectly clear lake water are in each case suitable for
the respective waters." As a design hydraulic loading rate, Kirkwood (Baker 1948)
recommended 0.15 m/hr (3.9 mgad), and Hazen (1913, p. 51), despite the above remark,
suggested a rate of 0.10 m/hr (2.6 mgad). They did not state the flow basis for their
recommended HLRs (for example, peak hour, average daily, and so forth). On sand sizes,
Hazen (1913, p. 33) reviewed data from filtration systems in 19 European cities (with 56
sands). Most had \(d_{10}\) values of 0.30–0.36 mm and UC values of about 2. From his review,
Hazen suggested \(0.20 < d_{10} < 0.35\) mm.

Hazen also expressed opinions on a variety of other matters of slow sand filter
design and operation. Concerning acceptable headloss, he stated (1913, p. 65) that the
limit is economic rather than technical. He found no evidence that higher headlosses
would cause breakthroughs. With regard to the scraping and removal of the deposit of
material on the surface of the filters, he recommended that the sand that is removed be
taken to a sand washing apparatus to be washed and used again. He explained that a
filter should be filled from below to expel trapped air. He stated that the method for
rebuilding a sand bed should be to place the old sand above the washed sand used for
rebuilding the bed. He noted that the time interval between scrapings varied from 10 to
40 days for the German works, with the average being 25 days. Eleven German water
works filtered an average of 0.048 million L/m² (liters per square meter), or 51 million
gallons per acre between scrapings; Zurich, drawing clear water from L. Zurich, filtered
0.24 million L/m² (260 million gallons per acre). Hazen also recommended (p. 74)
operating the filter in the filter-to-waste mode after scraping, based upon the hypothesis
that an important part of the filtration occurs in the sediment layer deposited on top of
the sand.

Turbidity limits did not seem to be a major concern in European practice. Generally,
rivers with high sediment loads, such as the Mississippi River, are not prevalent in
Europe. Nevertheless, settling reservoirs were becoming a part of the filtration practice.

1.4.2 United States

The first American slow sand filter was constructed in 1872 for Poughkeepsie, NY,
which had a population of 20,000. The design was credited to James Kirkwood (Baker
1948, pp. 134, 150), although he resigned as the consulting engineer for the filtration
plant on December 31, 1872.

In 1869, just prior to the Poughkeepsie project, Kirkwood wrote the first work on
slow sand filtration for American practice (Kirkwood 1869). He had been retained by the
city of St. Louis to recommend improvements to the city's water supply system. He
advised the city to filter its supply and was sent abroad to obtain information on
European practices. His survey of European practice included tours of filters and filter galleries in 19 cities. In his resulting report, Kirkwood emphasized that due to the muddy character of the Mississippi River, settling should be a part of the filtration system for St. Louis. St. Louis decided not to build filters, but Kirkwood's report was published as a bound volume and undoubtedly had a role in the design of the Poughkeepsie filter.

In 1895, Allen Hazen wrote the first American treatise on filtration. Like Kirkwood, he preceded his writing with a tour of installations in Europe. His book was revised in 1913 and was the definitive work on the subject.

By 1900 only about 20 slow sand filters had been constructed in the United States, and five have been built in Canada (Baker 1948, p. 147). In 1940, the slow sand plants in the United States numbered about 100 in contrast to the 2,275 rapid rate filters in service. The same year, Canada had about 12 slow sand filters and 120 rapid filters. A 1983 survey of slow sand plants in the United States (Slezak and Sims 1984), which was updated in 1989 (Sims and Slezak 1991), identified 62 plants and provided information on 36 of them. Most of the plants were located in northeastern states, with about one third in New York. Although most served small communities, a third were for communities with populations of 10,000 to 100,000, and half of the plants were over 50 years old. Figures 1.8 and 1.9 show the raw and finished water turbidities and the raw and finished coliform concentrations, respectively, for the plants that provided data. Influent turbidities ranged from 0.4 to 10 NTU. Effluent turbidities ranged from 0.07 to 2 NTU with 99 percent of the plants producing water with turbidities lower than 1 NTU. Influent coliforms ranged from 2 to 700 per 100 mL; effluent coliforms ranged from 1 to 2 per 100 mL. These data and others have established slow sand filtration as an efficient process.

In a study to determine the extent of slow sand activity during recent years, Logsdon and Fox (1988) identified 39 slow sand plants built from the late 1960s through 1988, with 11 proposed plants. Of the 39 plants, 23 had design capacities of less than 1,000 m³/day. They noted the new attention focused on slow sand technology due to publications and presentations during the 1980s. They presented a paper on their study at a November 1988 London symposium convened to provide a forum for papers of persons working in the field. The proceedings of the symposium, edited by Graham (1988), included papers from Europe, Africa, and the Americas and covered filter design, operation, and management; pretreatment; biological aspects; process performance; process developments; and case studies from developing countries.

Cost, design, and operating practices of slow sand facilities in the United States were reported by Sims and Slezak (1991), adding to their 1984 survey of the extent of
slow sand filtration practice in the United States. They gave costs of sand ranging from $4 to $63/ton, and stated that cost was mostly dependent on transportation distance. Most facilities recycled their sand using on-site washing. The mean water depth was 1.73 m for the 25 plants responding (48 percent coefficient of variation, CV); the mean sand depth was 0.84 m (CV = 31 percent), and the mean depth of the support media was 0.55 m (CV = 71 percent). Most plants had sand sizes of $d_{10} = 0.30$ mm, UC = 1.7–3. Hydraulic loading rates were $<0.25$ m/hr (6.4 mgad) for 90 percent of the plants. Labor requirements for scraping were about 5.4 person-hours/100 m² (5 person-hours/1,000 ft²) to remove generally about 1–2 cm of sand. Logsdon et al. (1989) reported on performance of slow sand filters in the Pacific Northwest, looking at the role of design factors.

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### Table 1.8

<table>
<thead>
<tr>
<th>Percent Less Than</th>
<th>Turbidity (NTU)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.01</td>
<td>0.01</td>
</tr>
<tr>
<td>0.1</td>
<td>0.1</td>
</tr>
<tr>
<td>0.2</td>
<td>0.2</td>
</tr>
<tr>
<td>0.5</td>
<td>0.5</td>
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<td>1.0</td>
</tr>
<tr>
<td>5.0</td>
<td>5.0</td>
</tr>
<tr>
<td>10.0</td>
<td>10.0</td>
</tr>
<tr>
<td>20.0</td>
<td>20.0</td>
</tr>
<tr>
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<tr>
<td>99.9</td>
<td>99.9</td>
</tr>
<tr>
<td>99.95</td>
<td>99.95</td>
</tr>
</tbody>
</table>

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### Figure 1.8

Turbidity of Influent and Effluent Water for Slow Sand Filter Facilities Responding to Survey of Sims and Slezak. (Adapted from Sims and Slezak [1991].)
Figure 1.9 Coliforms in Influent and Effluent Water for Slow Sand Filter Facilities Responding to Survey of Sims and Slezak. (Adapted from Sims and Slezak [1991].)

1.4.3 Worldwide

Slow sand filtration has been advocated by the World Health Organization, especially for use in less developed countries. The 1974 book by Huisman and Wood has been the main published work facilitating technology transfer and has been the standard contemporary reference for persons interested in the subject. That work was followed by design and construction manuals by van Dijk and Oomen (1978) and by Visscher et al. (1987), which were also written with the objective of providing technology transfer to developing countries. Table 1.6 summarizes the basic design criteria recommended by Visscher (1988).

The merits of slow sand filtration for rural communities in developing countries are the same as for small communities anywhere. The technology is passive in nature and therefore does not depend upon active process control. But in addition, the construction uses mostly local materials and can utilize local labor, thus
### Table 1.6

<table>
<thead>
<tr>
<th>Design criteria</th>
<th>Recommended level</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hydraulic loading rate</td>
<td>0.1–0.2 m/hr</td>
</tr>
<tr>
<td>Filter bed area</td>
<td>5–200 m² per filter, minimum of 2 filter cells</td>
</tr>
<tr>
<td>Depth of filter bed:</td>
<td></td>
</tr>
<tr>
<td>Initial</td>
<td>0.8–0.9 m</td>
</tr>
<tr>
<td>Minimum</td>
<td>0.5–0.6 m</td>
</tr>
<tr>
<td>Sand size:</td>
<td></td>
</tr>
<tr>
<td>Effective size, d₁₀</td>
<td>0.15–0.30 mm</td>
</tr>
<tr>
<td>Uniformity coefficient, UC</td>
<td>&lt;5 (preferably &lt;3)</td>
</tr>
<tr>
<td>Depth of gravel support</td>
<td>0.3–0.5 m</td>
</tr>
<tr>
<td>Depth of headwater above sand</td>
<td>1 m</td>
</tr>
</tbody>
</table>

*Source: Adapted from Visscher (1988).*

providing economic benefits as well. Also, chemicals are not needed. Thus, if the plant is designed properly, there should be no dependence on outside sources for material goods or services, except for the purchase of disinfection chemicals, such as chlorine.

### 1.5 RESEARCH

Research has frequently had a pivotal role in water treatment practice. In the high-tech industries, research has always led the way by the development of new products. But in water treatment, research usually follows empirical criteria and practices. Practice is improved as research explains, corrects, recommends, and provides knowledge-based rationale. This section reviews some of the research that, together with the established empirical knowledge, provides the basis for the criteria and guidelines of this manual.

#### 1.5.1 The 1980s' Research Platform

The decade of the 1980s saw a resurgence in slow sand filtration in the United States and Canada. The progress was the result of the teamwork between funding agencies, universities, regulatory agencies, and consulting firms and culminated in slow sand again being considered a viable treatment technology.

**(1) United States and Canada.** Slow sand filtration practice was well established in the United States and Canada by 1900. Current practice is founded largely upon this
knowledge base, which was articulated by Hazen (1913). In about 1980, however, the USEPA (U.S. Environmental Protection Agency) Drinking Water Research Division initiated a research program on slow sand filtration that has established a new knowledge platform. After initial pilot research, the program supported several extramural projects, influenced the dissemination of knowledge on the subject, and stimulated interest in other slow sand research and in the construction of several slow sand plants. The EPA-sponsored research during the period 1981–1984 included work at Iowa State University (Cleasby et al. 1984a, 1984b); at Colorado State University on Giardia cyst removal and the role of process variables (Bellamy et al. 1985a, 1985b, 1985c); at Syracuse University on plant operation (Letterman and Cullen 1985); and at McIndoe Falls, VT, on full-scale operation (Pyper 1985). In parallel with the EPA-sponsored research, work was done by Sims at Utah State University, sponsored by the Utah Health Department, to examine the performance of construction-grade sand with uniformity coefficients as high as 4.4 (Slezak and Sims 1984), to study the removal of viruses by slow sand filters, and to examine the utility of amended slow sand (see Section 7.4), such as the use of ion-exchange minerals (McNair et al. 1987).

In 1984–1985, following the EPA research, the American Water Works Association Research Foundation (AWWARF), sponsored a project at Colorado State University to examine the performance of a full-scale slow sand filter. The project focused on the new Empire, CO, slow sand filtration plant (Seelaus et al. 1986; Hendricks 1988). At the same time, Health and Welfare Canada sponsored parallel research to evaluate the new plant at 100 Mile House, BC (Bryck et al. 1987). The AWWARF later sponsored research at the University of New Hampshire to look at the capacity of the slow sand filtration process to remove trihalomethanes (Collins and Eighmy 1988).

In addition to the above, Tanner (1987), working for the Idaho Health Department, assessed the performance of operating slow sand filters in northern Idaho. Performance of these filters was related to design by Logsdon et al. (1989). Also, Douglas Fogel, working with Ongerth at the University of Washington, was continuing to assess the performance of the slow sand filters at 100 Mile House, BC (Ongerth 1990). Finally, Barrett (1989), in experiments at the University of Colorado, observed total coliform removals of 99.99 percent using sand with $d_{10} = 0.92$ mm and UC = 2.0. The sand bed was mature, enhanced by nutrient-rich waters and a temperature of 25°C. The coarse sand gave run times of two months, compared with runs of six days for sands with $d_{10} = 0.21$ mm under the same conditions.

(2) Europe. Whereas there has been a resurgence of interest in slow sand filtration in North America, in Europe there has been continuous activity with this treatment technology, as indicated by the studies of Poynter and Slade in London (1977). In 1988,
the London Symposium convened by Graham included many presentations by European authors. The wide range of topics they covered indicates a viable research community and an extensive practice. Thus, slow sand filtration technology is well established in Europe. In London, for example, 85 percent of the city's water sources are slow-sand filtered, which has been the case since the mid-nineteenth century (Toms and Bayley 1988). Figure 1.10 shows a "package" slow sand filtration plant, a type of plant developed in the United Kingdom and indicative of the interest there in extending slow sand filtration technology to rural areas. A package plant has small capacity and is pre-engineered, with all components provided in the plant purchase from a manufacturer.

(3) Present Status. All of this recent research has not replaced the lore from the last century but has built upon its foundation, expanding, explaining, and correcting it in a few instances. The practice was and is empirical and has been successful. However, the recent research has provided a rationale for the design criteria and has paved the way for the present practice. The research has highlighted the fact that although a basis for practice exists, the theory of the slow sand filtration process needs additional work.

1.5.2 Findings

The research of the 1980s paved the way for current practice. Key findings that may bear upon practice include the following:

- *Giardia* cysts will be removed by the slow sand filtration process, with mature filters capable of 3-log to 4-log removal. This finding is based upon research by Bellamy et al. (1985a), in which pilot plants were spiked with millions of cysts.
- *Giardia* cysts and other particles of about the same size will be removed from water by full-scale filters when the particles occur at ambient concentration levels, such as 10–50 cysts per 1,000 L (Hendricks 1988).
- Process variables influence the removals of turbidity, coliform bacteria, and *Giardia* cysts. But, by far, the most important variable affecting removal rate is the biological maturity of the filter bed (Bellamy et al. 1985a, 1985b).
- The *schmutzdecke* has many forms, ranging from an inert carbonaceous deposit to a primarily microbiological layer. The raw water quality determines its character.
- The *schmutzdecke* accounted for about 1-log of the 3-log to 4-log total removal of coliforms in the pilot filters operated by Bellamy et al. (1985a).
- Pilot filters operated at Colorado State University removed only about 20 percent of the raw water turbidity, which was made up of particles <1 μm in size and had turbidities of about 3.4–4.5 NTU. Despite the low removals of turbidity, removals
Figure 1.10  Two Views of "Package" Slow Sand Filter in the United Kingdom. (Top photo courtesy Jean Darby, Department of Civil Engineering, University of California, Davis; bottom photo courtesy Tom Hall, Water Resource Center.)
were 3-log to 4-log for *Giardia* cysts and coliform bacteria, establishing the fact that turbidity removal is not indicative of the removals of biological particles.

- Pilot filters at 100 Mile House, BC, when spiked with coliforms during start-up, showed zero removals, indicating that the sand beds, at start-up, had little, if any, biological activity (Bryck et al. 1987).

- The length of the filter run, defined as the time period before scraping is required, is not necessarily long when turbidity is low. For example, the run time for the filter at Empire, CO, with raw water turbidities <0.5 NTU, is only about 30 days, whereas the run times for pilot filters at Colorado State University, with raw water turbidities in the range of 3 to 6 NTU, were several months. The run time at 100 Mile House, using raw water ≤1 NTU, was also several months during initial start-up.

- Typically, an abundant count of biological particles, such as parasite eggs, nematode eggs, coccidia, amorphous debris, and perhaps *Giardia* cysts, are found in the cartridge filter concentrate of ambient raw waters after 1,000–2,000 L of sampling. Yet, very low counts are found in cartridge filter concentrates after slow sand filtration. This finding was reported for the Empire filter, even during the start-up month of January 1985, as well as during succeeding months (Seelaus et al. 1988). The colors of cartridge filters after 1,000–2,000 L of sampling of raw water and filtered water, respectively, are indicative of the filter performance in removals of biological particles ≥10 μm in size. (Bacteria, as noted for the testing of pilot filters at 100 Mile House, BC, in November 1986, are not removed well during start-up.) Figure 1.11 illustrates the difference in color of cartridge filters after influent and effluent sampling, typical of the cartridge filter results obtained at Empire during the period 1985–1990. The colors were indicative of results of microscopic counts of the cartridge filter concentrates (work by Hibler, reported in Seelaus et al. 1988). Similar results were obtained during cartridge filter sampling at 100 Mile House, BC (Bryck et al. 1987).

- Hydraulic loading rate, though having an influence on removal efficiency, is not as important as indicated by the lore. The biofilm development overshadows in importance all other variables.

- Coarse sand (d_{10} = 0.92, UC = 2.0) may significantly increase filter run times when the raw water is warm and nutrient-rich (Barrett 1989).
The foregoing research findings have paved the way for current practice. They have provided confidence that the technology is applicable to contemporary problems and that the current design criteria are practical. The chapters that follow incorporate these findings and build upon the literature on slow sand filtration to define guidelines for contemporary practice.
Many factors contribute to the decision to select slow sand filtration. The slow sand process could be a mistake if misapplied. This chapter provides guidelines to determine whether the slow sand filtration technology is appropriate for the conditions at hand. In other words, does slow sand filtration fit the context of the water system? The context is the configuration of physical and social conditions that interact with the water system. Context includes water source characteristics, such as flows and quality, community characteristics, such as those that affect demand flows and usage patterns, and any external and internal factors that make a given system unique.

Numerous other factors common to most engineering projects must also be considered in planning a filtration system, but most are outside the scope of this manual. They include facility planning, developing a financing plan, advising on regulatory requirements, comparison of costs of slow sand with alternative processes, producing engineering plans and specifications, developing bid documents, advising on contractor selection, inspecting construction, writing an operating manual, and starting operation. (Those factors related to construction are presented in Chapter 5.) Thus, although this manual has a broad perspective and identifies many of the facets surrounding the design process, the primary focus is on the unique aspects of slow sand filtration.

2.1 SELECTION

The sections in this chapter consider alternative filtration technologies, costs, community context factors, and ambient water quality. Each of these factors must be considered when determining whether slow sand filtration is appropriate for a particular water treatment case.
2.1.1 Comparison With Other Technologies

The conventional filtration technologies for water treatment are slow sand, rapid rate, and diatomaceous earth. Each has different particle removal mechanisms and different design, operation, and range of water quality suitable for treatment.

(1) Slow Sand. The major component of a slow sand filter is the bed of sand through which the water flows. The particle removal process is passive. The filter's effectiveness is dependent mostly upon the development of a biofilm attached to the sand grains, which provides an adsorptive surface for the attachment of particles from the water. Slow sand filtration is attractive for small communities because it is passive in operation and is effective when matched with suitable water quality. Slow sand filtration needs no outside supplies, and operator intervention is minimal, requiring only the adjustment of flow to the plant, the monitoring of headloss and turbidity, and the scraping of the filter schmutzdecke. Pilot testing is needed to anticipate run length between scrapings and to determine whether pretreatment is necessary. A possible problem in the operation of the process is the slow rate of biofilm development in cold, low-nutrient water.

(2) Rapid Rate. The major component of a rapid rate filter is also a bed of granular media, such as anthracite and sand. However, the filter's effectiveness depends upon the administration of the "proper" dosage of coagulant chemicals to neutralize the negative surface charges on the particles to be removed so that the particles can be attached on the filter media. It is common to have problems achieving the proper coagulant dosage to match the water quality at hand. With improper chemicals or dosage, effectiveness is reduced. Without chemicals, the effectiveness of rapid rate filtration may approach zero because straining becomes the only means of particle removal.

Rapid rate filtration is applicable to a wide range of water quality conditions, and package (or modular) plants have been designed to make the process appropriate for small communities. Rapid rate is an active process in the sense that an operator has control over the resulting effluent water quality, that is, the operator makes the filtration process effective by matching chemicals and dosages to the particular raw water conditions. Coagulant chemicals are the supplies necessary for rapid rate filtration.

Because of the numerous variables operative in the rapid rate process, pilot testing should be done for every installation, but usually it is not. Pilot testing is increasing, however, especially for larger installations in which management understands that such testing is cost-effective.

(3) Diatomaceous Earth. A main element of a diatomaceous earth filter is the septum, which is a support fabric that holds a diatomaceous earth filter cake. Septa may be made of a variety of materials including stainless steel mesh and other kinds of fabric.
Process effectiveness depends mostly upon the grade of diatomaceous earth used. Operation requires that a supply of diatomaceous earth be fed into the raw water stream as a body feed. Diatomaceous earth is a supply item. One possible problem in this filtration process is the loss of the filter cake due to a disruption of the pressure gradient across the cake caused by a power failure. With such a loss of pressure gradient, the cake may fall off and filtration will not occur. Controls are available to shut off the system should a power failure occur. Pilot testing is imperative in order to match the correct grade of diatomaceous earth with the raw water quality and to anticipate run length for different grades.

Example 2.1: Case Study—Compare Results of Pilot Plant Studies for Rapid Rate, Slow Sand, Diatomaceous Earth, and Pressurized Rapid Rate Filtration. Pilot plant comparisons of four filtration technologies were conducted by the consulting engineer for the Village of 100 Mile House, BC, during the period from July to October 1983 (Dayton & Knight, Ltd., 1983; Bryck and Walker 1984). The engineer had the task of selecting one of the filtration technologies, and the pilot plant studies were the basis for decisionmaking. The results of the technical part of the study are summarized here. A cost comparison is given in Example 2.2.

1. Context: In the fall of 1981, 60 cases of giardiasis were confirmed in the Village of 100 Mile House, BC, which had a population of 2,000 persons and a projected population of 4,200 for the year 2002. The water supply was from the nearby Bridge Creek, a high-quality supply with turbidities usually <1 NTU. Chlorination was the only water treatment used. The Village engaged the services of their consulting engineer to recommend a filtration system.

2. Data:

<table>
<thead>
<tr>
<th>Year</th>
<th>Population</th>
<th>Average day</th>
<th>Peak day</th>
<th>Projected Flow Data for Village of 100 Mile House, BC</th>
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<tr>
<td></td>
<td></td>
<td>mil L/d</td>
<td>mgd</td>
<td>mil L/d</td>
</tr>
<tr>
<td>1983</td>
<td>2,075</td>
<td>1.55</td>
<td>0.342</td>
<td>3.30</td>
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<tr>
<td>1987</td>
<td>2,300</td>
<td>1.75</td>
<td>0.385</td>
<td>3.66</td>
</tr>
<tr>
<td>1992</td>
<td>2,800</td>
<td>2.11</td>
<td>0.465</td>
<td>4.45</td>
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<tr>
<td>1997</td>
<td>3,400</td>
<td>2.55</td>
<td>0.560</td>
<td>5.41</td>
</tr>
<tr>
<td>2002</td>
<td>4,200</td>
<td>3.11</td>
<td>0.685</td>
<td>6.68</td>
</tr>
</tbody>
</table>

*a A well field will supply 0.91-mil L/d (0.20-mgd) flow for peak day demands, which should be subtracted from peak day demands to obtain flow to the filters.

Source: Adapted from Bryck and Walker (1984).

3. Pilot plant studies: Four filtration technologies—gravity rapid rate, slow sand, diatomaceous earth, and pressurized rapid rate—were evaluated by the consulting engineer for consideration by the Village council. Pilot plant studies, run simultaneously during the period from July to October 1983, were the basis for the comparisons (Table 2.2). The goal of the slow sand pilot filter study was to determine the run length at HLRs of 0.20 m/hr (5.0 mgad) and 0.4 m/hr (10 mgad) and to ascertain removal rates.

35
Table 2.2
Pilot Plant Comparison Study at 100 Mile House, BC, July–October 1983

<table>
<thead>
<tr>
<th>Parameter</th>
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<tr>
<td></td>
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<td>Gravity rapid rate</td>
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<tr>
<td>Flow (peak 1992)</td>
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<td></td>
<td>(m/hr)</td>
<td>12.5</td>
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<tr>
<td></td>
<td>(mgad)</td>
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<td>5.0</td>
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<tr>
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<td>(ft&lt;sup&gt;2&lt;/sup&gt;)</td>
<td>160</td>
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<tr>
<td>Run length</td>
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<tr>
<td>Flow (peak 2002)</td>
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<td>6.68</td>
</tr>
<tr>
<td></td>
<td>(m/hr)</td>
<td>12.5</td>
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<td></td>
<td>(mgad)</td>
<td>4.75</td>
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<tr>
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<td>(gpm/ft&lt;sup&gt;2&lt;/sup&gt;)&lt;sup&gt;d&lt;/sup&gt;</td>
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<tr>
<td>finish</td>
<td>(NTU)</td>
<td>0.15–0.26</td>
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<td>Alum:</td>
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<td>d&lt;sub&gt;10&lt;/sub&gt; = 0.15 mm</td>
</tr>
<tr>
<td>Polymer:</td>
<td>0.35 mg/L</td>
<td>UC = 7.3</td>
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<td></td>
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<td>depth = 1.05 m</td>
</tr>
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<td></td>
<td></td>
<td>h&lt;sub&gt;L&lt;/sub&gt;(init.) = 0.43 m</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Precoat:</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.98 kg&lt;sup&gt;e&lt;/sup&gt; DE/m&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Body feed:</td>
</tr>
<tr>
<td></td>
<td></td>
<td>20 mg/L</td>
</tr>
<tr>
<td></td>
<td></td>
<td>h&lt;sub&gt;L&lt;/sub&gt; = 138 kPa&lt;sup&gt;f&lt;/sup&gt;</td>
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<tr>
<td></td>
<td></td>
<td>Run without chemical pretreatment</td>
</tr>
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<td>Run without chemical pretreatment</td>
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<td></td>
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</tr>
</tbody>
</table>
|<sup>a</sup>Grade of diatomaceous earth was Hyflo Super-Cel<sup>®</sup>, manufactured by Manville Corp., Denver, CO.
|<sup>b</sup>Flows are taken from Table 2.1, with 0.91 mil L/d subtracted to account for well field flow used for peak day.
|<sup>c</sup>Hydraulic loading rates were based upon results of pilot plant testing, which used recommended ranges.
|<sup>d</sup>gpm/ft<sup>2</sup> = gallons per minute per square foot.
|<sup>e</sup>The plants considered for 1984 construction were sized for 1992 peak day flow capacities of 4.45 mil L/d (0.94 mgd) using Table 2.1 data. The plants would be expanded when the plant flow demand reached 6.68 mil L/d (1.52 mgd), projected to occur in 2002. The illustration of area calculation for slow sand is:
|<sup>f</sup>Kilograms of diatomaceous earth per square meter of septum area.
|<sup>g</sup>Pressure conversion: 138 kPa = 20 lb/in.<sup>2</sup>.

Source: Adapted from Bryck and Walker (1984).

4. Discussion: An obvious difference between slow sand and the other filtration technologies is the size of the filter bed needed. With the filter bed area being a function of the HLR, and with the HLRS used in this study, the bed area for slow sand is about 65 times larger than that for the gravity rapid rate filter. For the 1992 peak plant flow, this ratio translates to an area of 12 m<sup>2</sup> for the rapid rate filter bed and 760 m<sup>2</sup> for the slow sand filter bed. Another basic difference is that the particle removal mechanisms are different for each filtration technology. For example, in the gravity rapid rate filter system, the particles must be charge neutralized by chemical coagulation before they can be adsorbed on the filter media. With slow sand, the particles are not chemically pretreated and the removal mechanisms are not known, although adsorption on the sand grain biofilm seems likely when the filter is mature (that is, when such a biofilm is present). With diatomaceous earth, the removal mechanism probably is straining, since removals are higher as the grade size is reduced. The rapid rate pressure filter was operated without coagulation.
Adsorption cannot be a mechanism under such circumstances; only sedimentation or straining could be operative. Both are likely to be low and so this form of filtration (that is, without chemical pretreatment) will not be effective.

5. **Pressure filters:** The question of pressure filters deserves special discussion. Sometimes the term pressure filter is synonymous with ineffective filtration, but there is no reason for such a reputation. If proper chemical pretreatment is practiced, the filtration process will be effective. It does not matter if the pressure gradient is due only to a head of water or is due to the pressure exerted on a closed vessel. The important factor is whether proper chemical pretreatment is practiced, not the cause of the pressure gradient. The pressure filter in the study was operated without chemical pretreatment only because the manufacturer recommended such a mode of operation, and the purpose of the study was to explore a range of possible options.

2.1.2 Cost Comparisons

Comparison of the costs of the three conventional technologies is difficult because of the many empirical factors involved. Such factors are unique to the water being treated and the social context of the community. Thus, the only way to accurately compare the costs is to evaluate cost and performance data for the three technologies generated under the same conditions. As noted in Example 2.1, the pilot plant studies of four technologies at the Village of 100 Mile House, BC, provided opportunity to compare technical performance data. This comparison was extended in Example 2.2 to a cost analysis for the three filtration technologies (plus the pressurized rapid rate), with sizing scaled up to full size for 1992 flows and with plant expansion planned for when the 2002 projected flow is reached.

**Example 2.2: Case Study—Compare Costs of Rapid Rate, Slow Sand, Diatomaceous Earth, and Pressurized Rapid Rate Filtration.** The pilot plant studies at the Village of 100 Mile House, BC, provided data on the four filtration technologies. The data included scaled-up sizings, as noted below, for estimating capital costs and operation and maintenance costs. Also provided were data on flows, the location of intakes, pumping requirements, and other contextual factors. From this information cost estimates were prepared and are shown in Table 2.3. The 1984 cost estimates are for a plant to be constructed in 1984 with a projected 1992 plant flow of 4.45 mil L/d (1.18 mgd). A modular expansion is planned when the plant flow is greater than 4.45 mil L/d (1.18 mgd), which is also expected to occur in 1992. Therefore, the 1992 cost estimates are for a projected 2002 plant flow of 6.68 mil L/d (1.76 mgd).

**Discussion:** The four technologies were evaluated using pilot plant test results, construction costs, and operation and maintenance costs. Diatomaceous earth was ruled out because its annual costs were markedly higher than those of the other technologies. Pressurized rapid rate also was ruled out because filtration without chemical pretreatment was not accepted by other jurisdictions and because the removal efficiency was low. Of the remaining two technologies, gravity rapid rate filtration had a lower annual cost than did slow sand, but slow sand was selected. The reasons for choosing slow sand were that it is reliable and easy to maintain; that the process is passive, requiring little operator attention and causing fewer staffing problems; and that the operating and maintenance costs were lower than those for gravity rapid rate filtration. The ease of operation and low operation and maintenance costs were key factors in the selection. Thus, although the cost analysis is important in selection, other factors must also be assimilated. The pilot plant testing and the cost analysis served as a basis for judgment, but the final decision incorporated additional factors, some of which could not be quantified.
Some discussion is warranted concerning generic characteristics of the filtration technologies that affect how much money is expended, where the expenditures occur, and the cost effectiveness of the installation. The following paragraphs further compare the three basic filtration technologies with respect to their respective generic factors.

(1) **Slow Sand.** Slow sand filtration is likely to have a higher capital cost than rapid rate filtration simply because the filter box is larger in area, as noted in Example 2.1. Consequently, more concrete is needed for slow sand. Yet, because a backwash is not involved in slow sand filtration, the plumbing costs are lower. Also, the capital costs for slow sand are mostly local expenditures, mainly for local materials and labor. For rapid
rate filtration, the capital expenditures may be, to a large extent, for equipment manufactured elsewhere. Balancing the higher capital cost for slow sand are its lower operating costs. The process is passive in nature and the requirements for operation are less than for rapid rate in terms of operator training and attention needed.

(2) **Rapid Rate.** For a small community, the capital cost of rapid rate filtration is likely to be lower than for slow sand if a package plant is purchased, but the capital cost may be about the same if the plant is built in place with concrete and steel. Operating costs will always be higher for rapid rate simply because of the tasks required of the operator, and the maintenance will be more extensive because of the greater complexity of the plant. In addition, the use of alum creates a sludge that requires disposal, a major problem in some situations.

(3) **Diatomaceous Earth.** The capital cost for diatomaceous earth filtration depends upon the bid cost of the equipment (tank septum, precoat tanks, mixers, body-feed tank mixers, controls, pumps) to the contractor. With this technology, the functional parts of the filter are purchased by the contractor instead of being built in place by the contractor, which is true also when proprietary package plants are purchased from a manufacturer. As shown in Table 2.3, of the four technologies, diatomaceous earth had the highest capital costs for the 1984 construction but was less expensive than slow sand for the 2002 construction. Of the $891,055 capital cost for 1984 shown in Table 2.3, $341,000 was for the purchase and installation of the filter units (U.S. dollars). The housing for the equipment may be a simple enclosure on a slab. Operation requires backwash and disposal of the spent diatomaceous earth. The cost of diatomaceous earth depends upon whether it is purchased bulk or in bags; it also depends on the shipping cost and on the grade of earth used. Such costs are variable, depending largely on transportation. At the Village of 100 Mile House, the free on board (FOB) 1984 cost was $0.45/kg ($0.20/lb) for 22.68-kg (50-lb) bags (Bryck and Walker 1984). As noted in Table 2.3, operating costs were the highest for diatomaceous earth.

2.1.3 Cost of Slow Sand

The major cost of a slow sand filter installation is the concrete work, with piping being the next largest cost. The distribution of these costs is illustrated in Example 2.3, which returns to the Village of 100 Mile House case study.

**Example 2.3: Case Study—Cost of Slow Sand Filtration at the Village of 100 Mile House, BC.** Bryck and Walker (1984) described the design for the filters at 100 Mile House and provided a breakdown of the costs. Table 2.4 shows the cost distribution for the categories of the construction.
1. Description of Design: The slow sand filter described in Table 2.3 for the 1984 construction had an area of approximately 774 m$^2$ (8,330 ft$^2$), which accommodated three filter cells, each 43.0 m $\times$ 6 m (141 ft $\times$ 19.7 ft) and each having a design flow of 1.2 mil L/d (0.32 mgd). The construction costs in Table 2.4 include the following items:

Pump Stations. The raw water pump station, which pumped water from Bridge Creek, comprised an infiltration gallery, an intake screen, a wet well, and three 3,730-watt (5-hp) pumps. The treated water pump station, which used the clear well, had three 7,460-watt (10-hp) pumps connected by vertical shafts to the floor above.

Control Building. The control building was heated and housed the chlorination equipment, the motor control center, the treated water pumps, the office, the pipe gallery, and a provision for standby power. The control building had a wall in common with the filter cells, located across the ends of cells.

Operation. The raw water pumps were controlled by the water level in the filters, whereas the treated water pumps were controlled by the water level in the Village's storage reservoir. The operator must manually adjust the butterfly valve of each filter to supply enough water to keep the clear well full. An overflow is provided from the clear well to the raw water pumping station.

2. Discussion: The largest cost for the slow sand filter, as seen in Table 2.4, is for concrete work, which was 43 percent of the total construction cost. The piping cost was 12 percent. The other cost categories were all within the same order of magnitude. The media cost is of particular concern because it is affected to a large extent by the transportation cost. Thus the media specifications need to be balanced against the cost of both sieving and transportation. The specific design determines the costs of each of the categories in Table 2.4. As seen in Table 2.4, the total cost for the 100 Mile House installation was U.S. $591,000.

<table>
<thead>
<tr>
<th>Category of work</th>
<th>Cost (U.S. $)</th>
<th>Percent of total cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete work (form work, reinforcing steel, concrete)</td>
<td>253,830</td>
<td>43</td>
</tr>
<tr>
<td>Piping (material cost for pipe, fittings, valves, supports, and labor)</td>
<td>68,190</td>
<td>12</td>
</tr>
<tr>
<td>Mechanical equipment (materials, installation, testing)</td>
<td>53,040</td>
<td>9</td>
</tr>
<tr>
<td>Earthwork</td>
<td>49,250</td>
<td>8</td>
</tr>
<tr>
<td>Architectural</td>
<td>45,462</td>
<td>8</td>
</tr>
<tr>
<td>Electrical</td>
<td>41,674</td>
<td>7</td>
</tr>
<tr>
<td>Miscellaneous metal</td>
<td>25,000</td>
<td>4</td>
</tr>
<tr>
<td>Corrosion protection and painting</td>
<td>24,246</td>
<td>4</td>
</tr>
<tr>
<td>Media:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Filter sand, 1,000 m$^3$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gravel 1,120 m$^3$; Gravel 2,120 m$^3$; Gravel 3,120 m$^3$</td>
<td>26,520</td>
<td>4</td>
</tr>
<tr>
<td>Placement of media</td>
<td>3,788</td>
<td>1</td>
</tr>
<tr>
<td><strong>Total construction cost</strong></td>
<td><strong>591,000</strong></td>
<td><strong>100%</strong></td>
</tr>
</tbody>
</table>

Conversion was made from Canadian dollars using the December 1984 exchange rate of U.S. $0.7577 = Can. $1.00. Significant figures shown are due to applying conversion from Canadian dollars to U.S. dollars and do not connote precision.

Source: Adapted from Bryck and Walker (1984).

Cost data for the Moricetown, BC, slow sand filter, built in 1989, showed approximately the same distribution of costs as for 100 Mile House. The total amount of the tender for Moricetown was U.S. $480,582 (Can. $634,265 was given in the bid.
The plant was designed to supply a peak flow of 0.922 mil L/d for a population of 900, with provision to expand to accommodate a population of 1,240 (Dayton & Knight, Ltd., 1989). The plant has two filter beds, each 2.0 m x 4.5 m x 4.0 m deep (6.56 ft x 14.8 ft x 13.1 ft) to give a total area of 18 m² (194 ft²).

2.1.4 Requisite Conditions for Slow Sand

Source water quality and community size are the key factors that determine whether slow sand should be selected. The following guidelines concerning water quality and community size may aid in the decisionmaking.

(1) Water Quality. The raw water quality determines the length of time between scraping operations. Slow sand filtration is limited to raw waters that will permit long filter runs, for example, 30 days to several months before terminal headloss is reached. There is no minimum acceptable run length, but runs greater than 30 days may be considered satisfactory for most situations. If run lengths of several months occur, the situation should be considered fortunate. Also, run length will decrease with increasing hydraulic loading rate if the solids loading rate is proportional to the hydraulic loading rate. Various kinds of particles cause headloss, including mineral sediments, organic detritus, and bacteria and larger microorganisms. The mix of water quality characteristics causing headloss is unique to the situation at hand, and a prediction of headloss is not possible. As a rule of thumb, however, when turbidity is too high, short filter runs are likely. In treating Ohio River water with a pilot slow sand filter in Cincinnati, Fox et al. (1984) observed that the turbidity-causing particulates (clay) gradually plugged the filter bed over a series of runs. Ultimately, the effluent turbidity was the same as the influent turbidity and the filter runs were reduced to less than one week in duration. Slow sand would not be acceptable with such short filter runs.

Table 2.5, which was adapted from Cleasby (1991) and extended to include additional entries, summarizes cycle lengths for a variety of slow sand filter installations having different raw water turbidities. The data show that a range of filter cycles may be expected and that raw water turbidity is not an indicator of cycle length. Cycle lengths cannot be predicted without pilot plant testing.

As noted by Cleasby (1991), the run lengths reported for high-quality surface waters emphasize the deficiency of raw water turbidity as an indicator of run length. For example, comparing the 6-month run length for the Horsetooth Reservoir water in Colorado, having 6–10 NTU raw water turbidity, with the 30-day run length for Mad Creek at Empire, CO, having generally <0.5 NTU raw water turbidity, shows that lower turbidity does not necessarily mean longer run lengths and that higher turbidity does
Table 2.5
Length of Filter Cycle for Slow Sand Filter Installations Having Different Raw Water Turbidities

<table>
<thead>
<tr>
<th>Plant or raw water source</th>
<th>Turbidity of raw water (NTU)</th>
<th>Filter cycle</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lake water (4 plants)</td>
<td>1–3</td>
<td>3–6 months</td>
<td>Letterman and Cullen (1985)</td>
</tr>
<tr>
<td>Reservoir water</td>
<td>7–9; peaks of 20–40</td>
<td>1.2–2 days</td>
<td>Letterman and Cullen (1985)</td>
</tr>
<tr>
<td>Idaho water samples</td>
<td>&lt;0.5</td>
<td>1.5–2 months</td>
<td>Tanner (1987)</td>
</tr>
<tr>
<td></td>
<td>0.5–1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pilot plant, gravel lake, Ames, IA</td>
<td>19 in summer w/algae 2.2 in winter</td>
<td>0.3 months 4 months</td>
<td>Cleasby et al. (1984a, 1984b)</td>
</tr>
<tr>
<td>Pilot plant, Horsetooth Reservoir, CO</td>
<td>6–10</td>
<td>6 months</td>
<td>Bellamy et al. (1985a, 1985b)</td>
</tr>
<tr>
<td>Mad Creek, Empire, CO</td>
<td>&lt;0.5 (seldom exceeds 1)</td>
<td>1.0 month</td>
<td>Seelaus et al. (1986)</td>
</tr>
<tr>
<td></td>
<td>(consistent over 5 years)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bridge Creek, 100 Mile House, BC</td>
<td>0.5–2.0</td>
<td>In months, cycle 1 = 7.2; 2 = 1.8; 3 = 2.0; 4 = 2.2. HLR = 0.15 m/hr</td>
<td>Bryck (1987)</td>
</tr>
<tr>
<td>Glacier Creek, Moricetown, BC</td>
<td>15–1,000; 1,000 reduced to 50 after sed.</td>
<td>No data</td>
<td>Bryck (pers. com., September 20, 1990)</td>
</tr>
<tr>
<td>Kassler Plant, Denver, S. Platte River, CO</td>
<td>0.5–1.5</td>
<td>1.0 month at max. rate (0.15 m/hr or 3.75 mgad)</td>
<td>Beer (pers. com., April 1990)</td>
</tr>
<tr>
<td>20 ha (hectares), or 50 acres, of wetlands and 4-ha (10-acre) impoundment, fed by springs, McIndoe Falls, VT</td>
<td>674 observations: &lt;1 = 48 percent of obs.; &gt;3 = 8 percent of obs.</td>
<td>4–8 months</td>
<td>Pyper (1985)</td>
</tr>
</tbody>
</table>

not necessarily mean shorter run lengths. The rationales for the observed run lengths for the two cases were: (1) the turbidity of the reservoir water was composed of fine particles, mostly $< 1 \mu m$, that passed through the filter bed, and (2) the water from Mad Creek deposited an inert organic detritus of small flaky particles on the surface of the sand bed. A similar finding was noted at the Moricetown plant, where 0.45-\mu m particles passed through the filter (Slezak, pers. com., 1990).

The data in Table 2.5 emphasize the importance of pilot testing over the annual cycle to predict run length. If the run length is greater than 30 days, then slow sand may be considered a candidate filtration technology. If the run length is less than 30 days, slow sand should not be automatically ruled out, but its use should be considered carefully. The only way to anticipate run length is to conduct pilot testing, which should be an expectation of the predesign process. Chapter 4 outlines pilot plant testing procedures for slow sand.

Another concern, common to almost every stream, has to do with occurrences of high turbidity levels over part of the year, for days or perhaps for weeks, caused by rainfall or seasonal surface runoff. Such elevated turbidity levels may be only up to 30 NTU, but they could be greater than 300 NTU or even 1,000 NTU. As a consequence, the filter is likely to be "blinded off" with a layer of sediments, and more frequent scrapings will be necessary. Such frequent scraping may be tolerable if the occurrence is infrequent. If slow sand is used, the high turbidities may be attenuated by the use of sedimentation basins, as at the Moricetown installation (raw water turbidities are stated in Table 2.5). In any case, guidelines cannot cover all situations and judgment is required.

Table 2.6 summarizes the major water quality guidelines for the selection of slow sand filtration. The guidelines outline general experience and are neither absolute nor comprehensive. Usually slow sand is most suitable for low-turbidity raw waters, which most often are associated with low pollution levels. Some of the constituents listed in Table 2.6 are either not removed by slow sand filtration (such as color), or research is under way to ascertain their degree of removals (such as total organic carbon [TOC]), or they cause nuisance problems in operation (such as algae and iron). Although there are listed and pending drinking water regulations for organic and inorganic chemicals, Table 2.6 lists traditional contaminants only.

(2) Community Size. Slow sand, as a resurrected technology, is considered appropriate for use by "small" communities. The motivation to use slow sand is the premise of low annual cost plus effective, reliable operation. For the most part, the size of the community determines whether slow sand filtration is the most appropriate option. At some point in population size, slow sand will become more expensive than rapid rate filtration. Also, at some point as population increases, communities will have
Table 2.6
Water Quality Guidelines for Selection of Slow Sand (SS) Filtration

<table>
<thead>
<tr>
<th>Constituent</th>
<th>Removal experiences</th>
<th>Constituent guidelines</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>True color</td>
<td>25 percent</td>
<td>5–10 Pt-Co&lt;sup&gt;a&lt;/sup&gt;</td>
<td>Removal depends on biofilm in filter</td>
</tr>
<tr>
<td>TOC&lt;sup&gt;b&lt;/sup&gt;</td>
<td>25 percent</td>
<td>None</td>
<td></td>
</tr>
<tr>
<td>Turbidity</td>
<td>27–39 percent,</td>
<td>&lt;10 NTU</td>
<td>SS appropriate</td>
</tr>
<tr>
<td></td>
<td>Horsetooth</td>
<td>&gt;25 NTU</td>
<td>Pretreatment recommended</td>
</tr>
<tr>
<td></td>
<td>Reservoir&lt;sup&gt;c&lt;/sup&gt;</td>
<td>10–50 NTU</td>
<td>Pretreatment recommended</td>
</tr>
<tr>
<td></td>
<td></td>
<td>50–120 NTU</td>
<td>Limited tolerance</td>
</tr>
<tr>
<td>Coliform bacteria</td>
<td>2-log to 4-log</td>
<td>None</td>
<td>Pilot plants&lt;sup&gt;c&lt;/sup&gt;</td>
</tr>
<tr>
<td>Giardia cysts</td>
<td>3-log to 4-log</td>
<td>2-log</td>
<td>Pilot plants&lt;sup&gt;c&lt;/sup&gt;</td>
</tr>
<tr>
<td>Algae</td>
<td></td>
<td>&lt;5 mg/m&lt;sup&gt;3&lt;/sup&gt;</td>
<td>Surface mat</td>
</tr>
<tr>
<td>Iron</td>
<td>Waverly, NY, has</td>
<td>None</td>
<td>Clogging potential</td>
</tr>
<tr>
<td></td>
<td>clogging due to</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>iron</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<sup>a</sup>Pt-Co = platinum-cobalt units.
<sup>b</sup>TOC = total organic carbon.
<sup>c</sup>Bellamy et al. (1985a, 1985b).

Source: Adapted from Cleasby (1991).

the requisite water management infrastructure and funds for full-time operators such that effective rapid rate filter operation will be likely. The engineer must determine the point at which rapid rate would be most economical for effective, reliable operation. The point of crossover on both curves depends upon the community context; therefore, an absolute criterion is not definable.

At the low end of the size scale for economical use of slow sand filtration is a water supply having more than 15 service connections, or serving more than 25 persons, the definition of a public water supply, which is regulated by EPA drinking water regulations (PL 99-339, 1986). At an HLR of 0.16 m/hr (4 mgad) and assuming a per capita use of 600 L/person/d (158 gpcd [gallons per capita per day]), the slow sand filter would be about 3.9 m<sup>2</sup> (42 ft<sup>2</sup>) in area or 2 m x 2 m (6 ft x 7 ft). A structure of this size could be built by local labor, subject to supervision by a professional engineer. The scraping operation could easily be done by one person (with a second person on hand for safety).

The upper population limit for the economical use of slow sand filtration will depend upon the situation. The probable cutoff point is a population greater than 1,000.
persons, but the limit could go to 10,000 persons. Local factors will have much to do with whether slow sand is used. The town of Salem, OR, a city of 100,000 people, for example, has slow sand filters that were built in 1958 and expanded in 1970 (Boydston, pers. com., January 24, 1990). Stayton, OR, population 5,000, has slow sand filters built in 1975 and expanded in 1987. Westfir, OR, population 400, is served by a slow sand filter built in 1986, and the Wickiup Water District in Oregon, which serves a population of 1,400, has had a slow sand filter in place since 1987. Thus, the size of communities in Oregon having slow sand filtration varies greatly; the examples would refute any specific guidelines.

Table 2.7 presents data from several sources that give an idea of the size of the labor force required and the unit cost of water for slow sand and rapid rate facilities. The 152-mil L/d (40-mgd) Kassler Plant of Denver, which required a work force of 18 persons, probably would not be built today. The nearby rapid rate Denver Foothills Plant, with an initial capacity of 473 mil L/d (125 mgd) and which requires a work force of only 20 persons, would be a more economical choice.

Table 2.7

<table>
<thead>
<tr>
<th>Facility</th>
<th>Persons employed</th>
<th>Annual costa</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Employment</td>
<td>$1,000 L</td>
</tr>
<tr>
<td>Slow Sand:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>100 Mile House, BCb</td>
<td>1 (1 hr/day + scrape)</td>
<td>0.16c</td>
</tr>
<tr>
<td>Denver Kassler Plant, 152 mil L/d</td>
<td>18</td>
<td>0.043</td>
</tr>
<tr>
<td>(40 mgd)d</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Empire, CO, serving 450 personsede</td>
<td>1</td>
<td>0.005</td>
</tr>
<tr>
<td>Moricetown, BC, serving 905 personsf</td>
<td>1 (1 hr/day + scrape)</td>
<td>0.18c</td>
</tr>
</tbody>
</table>

Rapid Rate:
Denver Foothills Plant, 473 mil L/d
(125 mgd)d

<table>
<thead>
<tr>
<th>Facility</th>
<th>Persons employed</th>
<th>Annual costa</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Employment</td>
<td>$1,000 L</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Denver Foothills Plant, 473 mil L/d</td>
<td>20</td>
<td>0.013</td>
</tr>
</tbody>
</table>

aCost is given in 1989 U.S. dollars unless stated otherwise.

bTotal construction cost was U.S. $59,100 (Can. $780,000); annual debt retirement cost = U.S. $121,200 (Can. $160,000); and annual operating cost = U.S. $15,700 (Can. $20,700). Bryck et al. (1987).

cConversion was made from Canadian dollars using the December 1984 exchange rate of U.S. $0.7577 = Can. $1.00. Significant figures shown are due to applying conversion from Canadian dollars to U.S. dollars and do not connote precision.


eThe facility was designed to serve 1,000 persons. Seelaus et al. (1988).

fPersonal communication, Don Kerven, Operator, facility in Empire, CO, 1990. He gave a time estimate of 40 person-hours per month, which includes one hour per day to visit the site, take samples, and record data; about two hours of scraping time per month using two persons; and six hours per month for draining the filter and refilling it with water. The water production calculation is based upon an assumed population of 500 persons and the per capita use of 750 L/person/d (200 gal/person/d), averaged over the year.

The cost per 1,000 L is based on a total debt of U.S. $30,338 (village) + U.S. $91,000 (province) amortized over 20 years, plus U.S. $15,684 operation and maintenance costs, divided by the volume of water produced per year at rated capacity. Jack Bryck (pers. com., September 20, 1990).
persons, would be favored. However, the Empire, CO, slow sand plant requires the time of an operator only once each day to obtain water samples and to measure headloss and turbidity and the time of two persons working about four hours each once each month to scrape the sand; these time requirements for labor add up to about 40 person-hours/month. Although a rapid rate package plant may be cheaper in capital cost than a slow sand plant, the package plant would require more attention to operation—for example, in the application of chemicals.

Because the rapid rate facility described in Table 2.7 has a capacity of 473 mil L/d (125 mgd), its unit cost for water is not comparable with those for the slow sand plants. Also, the reader should note that the cost of water for the Kassler Plant is for a 152 mil L/d (40 mgd) capacity, with debt already amortized. Nevertheless, the data in the table will facilitate initial estimates of person-hours needed for operation and unit costs.

2.1.5 Performance Required

The filtration requirement for slow sand, as promulgated by the Surface Water Treatment Rule (SWTR) (Federal Register 1989) of the U.S. Environmental Protection Agency, states a turbidity standard of ≤1 NTU 95 percent of the time, with no reading greater than 5 NTU. If a state agency determines, however, that higher turbidity levels will not interfere with disinfection, then a higher level may be permitted, but at no time may the level exceed 5 NTU. The SWTR requires further that a disinfectant residual of ≥0.2 mg/L be maintained. The combination of filtration and disinfection must achieve removal or inactivation of ≥3-log for *Giardia lamblia* cysts and ≥4-log for viruses. That slow sand can achieve >3-log removals for *Giardia lamblia* cysts was shown by Bellamy et al. (1985a, 1985b) in pilot plant studies, provided the filter bed is mature. Although bed maturity is likely after several months of operation, depending on temperature and nutrients, a simple measure is not available.

The number of EPA-regulated contaminants was 83 in 1989, and the list included trihalomethanes (THMs), which are disinfectant by-products; volatile synthetic organic chemicals (VOCs); synthetic organic chemicals (SOCs); inorganic chemicals (IOCs); microbiological contaminants; and radionuclide contaminants. Whereas slow sand may have the capacity to remove some of these contaminants—that is, those that are degradable—comprehensive research has not been conducted to establish which contaminants are removable and to what extent. Research has been conducted, however, by Sims at Utah State University (Foreman and Sims 1984; McNair et al. 1987) to investigate removals using a natural zeolite, e.g., clinoptilolite, filter medium. Clinoptilolite has shown selectivity for ammonium ions and heavy metals. The presence of clinoptilolite resulted in enhanced biological activity within the
*schmutzdecke* and some heavy metals removal until the bed capacity was exhausted. Collins et al. (1989) reported trihalomethane formation potential (THMFP) removals of 9–27 percent for filters in New England. They related the higher removals to enhanced biological activity at the surface of the sand bed due to harrowing the *schmutzdecke* into the filter media.

The above studies notwithstanding, the most prudent course of action for slow sand filtration is to use source waters of high quality. When slow sand is selected for marginal situations, pilot plant testing is needed to establish its potential for removing contaminants of interest. Although general guidelines do not exist to predict removals for every contaminant, one can predict the likelihood of removal for many. For example, significant and consistent removals of metal ions are not likely, and removals of organics are not certain.

### 2.1.6 Algae Control

If the surface deposit on the sand bed has much algae, a "mat" may be formed that will cause a rapid increase in headloss. For pilot filters in Iowa, Cleasby et al. (1984a, 1984b) reported run times of 9 days during algae blooms as compared to run times of up to 123 days during winter. Chlorophyll-a concentrations in the raw water ranged from 0.2 mg/m³ in midwinter to 132.7 mg/m³ in July (Cleasby et al. 1984b). Based upon his work with the Iowa water, Cleasby (1991) suggested that slow sand filtration be used only for waters with chlorophyll-a levels under 5 mg/m³.

If algae comes to the filter in the raw water, its concentration cannot be controlled unless the source water is from a reservoir. (Algae control is sometimes undertaken in reservoirs, for example, by treating the reservoir with copper sulfate or by practicing selective withdrawals.) The decay of the algae on the *schmutzdecke* may cause taste and odor problems in the effluent water. It is recommended that slow sand not be used to treat raw waters routinely subject to algae blooms or waters subject to major algae blooms. Slow sand may, however, be used if the bloom season is short enough that only minimal interruption of normal operation is likely.

Algae growth may occur also in the headwater above the filter bed or on the surface deposit of the sand bed. For a headwater depth of 2 m (6.6 ft), the detention time in the headwater ranges from 5 to 50 hours for filtration rates of 0.4 m/hr (10 mgd) to 0.04 m/hr (1.0 mgd), respectively. Because the regeneration time for typical algae species is about 2 hours nominally, but with a light-dark cycle needed to complete the cycle (Kugrens, pers. com., August 2, 1990), growth within the headwater is possible. The deposit on the surface of the sand bed may be in place for days, and algae growth can occur within it, although not so much as with full light intensity. A roof over the filter...
will reduce algae growth within the filter headwater and the surface deposit, but it will have no effect on algae brought in by the raw water. In any case, pilot plant testing can resolve questions.

2.1.7 Pretreatment Processes

As previously mentioned, raw water sources quite often have episodes of high turbidity that can reduce markedly the run time for a slow sand filter. Such episodes usually follow rainstorms or occur during spring runoff from snow melt. The latter situation occurs from about May to early July in streams draining high mountain watersheds in the western United States; turbidities in the streams may reach 30–50 NTU, with peaks as high as 200 NTU. During the remainder of the year the turbidity is usually less than 1 NTU. Turbidity patterns are site-specific, of course, and streams within a given region have their own unique behavior.

Sedimentation basins or roughing filters may be used to attenuate turbidity peaks and extend the run time of the filter. Two plants that have used sedimentation are the 152-mil L/d (40-mgd) Kassler Plant on the South Platte River in Denver (built around 1906) and the Moricetown Plant in British Columbia (started on June 15, 1989). Roughing filters, described in Section 7.6.1, are an alternative method used increasingly since about 1980, for reducing the sediment load to slow sand filters (Wegelin 1988). Roughing filters have several configurations, such as upflow and horizontal flow, and are composed of perhaps three cells of coarse gravel, with sizes in the first cell ranging from 12 to 18 mm, in the second cell ranging from 8 to 12 mm, and in the third cell ranging from 2 to 8 mm. The HLR is about 0.3–1.5 m/hr. Installations in Peru, Colombia, and the Sudan have reduced raw water turbidities from levels of 50–200 NTU to levels of 15–40 NTU; from levels of 10–150 to 5–15 NTU; and from levels of 40–500 to 5–50 NTU, respectively. The removal mechanism must be sedimentation, as with parallel plate settlers, common to rapid rate filtration.

1) Case: Kassler Plant. The South Platte River is a typical mountain stream with respect to turbidity cycles. To handle higher turbidities, the Kassler Plant had two ponds, or "settlers," which preceded the filters. Figure 2.1 is an aerial view of the Kassler Plant showing the ponds. Turbidity in the ponds during storms and spring runoff was about 10–50 NTU and was reduced by settling to about 1–2 NTU. As a note, the Kassler settling ponds were not used during the 20 years preceding closure of the plant in 1985.

2) Case: Moricetown Plant. The Moricetown filter, which was built to serve a population of 905 persons, was designed for an average flow of 0.32 mil L/d (0.08 mgd) and a peak flow of 0.92 mil L/d (0.24 mgd). The sedimentation basin was 415 m$^3$ (14,654 ft$^3$) in volume and held water for 12 hours at peak flow. Turbidity levels in the basin
were 15–1,000 NTU during a pilot plant test period from June 3 to October 9, 1987, with the high levels due to snowmelt and rain. During the months of November to May, turbidity was estimated to be near 1 NTU. Figure 2.2 is a flow schematic showing the layout of the plant, including the sedimentation basin. Not shown are the basins for the optional coagulation/flocculation process, although the coagulant chemicals are indicated. Figure 2.3 is a photograph showing the sedimentation basin. The basin is a simple pond, 2-m (6.6-ft) deep. The pond has a ramp to accommodate the equipment used to remove the accumulated sediment, which must be done about once every five years. The coagulation/flocculation processes were added to provide the coagulation option because of the uncertainty of whether the sedimentation basin alone could function adequately during episodes of high turbidity. The plant has two slow sand filters, each 90 m² (968 ft²), with widths of 4.50 m (14.8 ft) and lengths of 20.0 m (65.6 ft). The filter sand size had d_{10} = 0.25–0.30, with UC = 1.8–2.2. Chlorine contact time was two hours in a basin having a volume of 77 m³ (2,719 ft³). The clear well volume was 371 m³ (13,100 ft³).
From pilot testing at the Moricetown Plant, the operating cost was estimated to be U.S. $15,000/year, giving a water cost of $0.24/1,000 L ($0.91/1,000 gal). The annual cost and capital cost were estimated to be U.S. $45,883 and U.S. $450,000, respectively, assuming 8 percent annual interest and a 20-year repayment period. Despite some uncertainty over the episodes of high turbidity and the possible need to add
coagulation/flocculation processes, slow sand filtration was still considered the most appropriate technology for the situation, considering operating simplicity and cost. The engineers were fully aware that adding unit processes such as sedimentation and providing for coagulation/flocculation were deviating from the philosophy of slow sand filtration. Based upon their engineering judgment, for a case that was not clear-cut for any alternative, they recommended slow sand.

The Moricetown filter was started June 15, 1989. Raw water turbidities of 100 NTU have been measured in the intake water from Corya Creek (Glacier Creek), with 20–30 NTU reported for the water leaving the sedimentation basin and 3–4 NTU reported for the effluent water. A portion of the turbidity in the water leaving the filter was believed to be fines being washed from the new filter media. The filter performance is being monitored to determine whether the system of sedimentation/slow sand filtration is sufficient to reduce turbidities to the standard of 1 NTU. Although facilities for coagulation/flocculation are installed, the engineers prefer not to use these unit processes if possible.

(3) Conclusions. The above cases illustrate that the decision to use slow sand is not always clear-cut. Further, engineering judgment is needed to determine the extent to which a slow sand facility should incorporate auxiliary unit processes (discussed in Chapter 7). Complexity is contrary to the concept of slow sand filtration. If pretreatment is needed, the appeal of slow sand is diminished. Yet adding sedimentation or other pretreatment processes extends the range of situations in which slow sand can be applied. Adding coagulation/flocculation extends the range even further. Where to draw the line is not clear. Absolute answers are not possible for situations such as the Moricetown Plant (because of seasonal high turbidities) or the Denver Kassler Plant (because of size and operating costs), whereas the Empire, CO, and 100 Mile House plants were clearly situations in which slow sand could be selected with little doubt (because of low-turbidity raw waters all year and minimal operating costs).

If sedimentation is selected as a pretreatment process, several technologies are available, in addition to plain sedimentation, for removing particles. Two of them are plate settling and horizontal-flow roughing filters (Cleasby 1991). Roughing filters were reviewed by Wegelin (1988) and by Ives and Rajapaksi (1988) and are discussed in Chapter 7. Cleasby (1991) emphasized that plain sedimentation is not designed to remove colloidal particles and that many early filter systems incorporating plain sedimentation, such as those at Cincinnati and Pittsburgh, were considered unsuccessful due to the short run lengths, the penetration of colloidal material into the filter bed, and lower-than-desired turbidity removals. These experiences were one reason rapid rate
filtration had such widespread appeal after the path for its use was paved by Fuller's work at Louisville, KY, in 1896 (Baker 1948).

The practice in the United Kingdom is to precede the slow sand filters with pumped storage reservoirs, where detention times are 10–50 days and turbidity is reduced by settling from 17–31 NTU down to 1–10 NTU depending upon the season (Cleasby 1991). The algae blooms that occur in the reservoirs have stimulated the use of microstrainers ahead of the slow sand filters, which is another form of pretreatment. The filter runs for these conditions are about 6 weeks.

2.2 CONTEXT AND FORM

The theme of any design is context-form-fit (Alexander 1964). For a water treatment system, context refers to the array of community conditions, such as customs, values, wealth, labor skills, population, employment needs, materials at hand, regulatory pressures, technology support services, and water quality. Into this milieu, the engineer must attain a fit by choosing the most appropriate form of technology for the context at hand.

2.2.1 Community Context

The extent to which slow sand is an appropriate treatment technology for a given community is a matter of judgment. There are no rules, but knowing the issues that will affect the performance and acceptance of the technology will improve the fit achieved. For example, being sensitive to the financial situation, the political interest in high-quality water, the rate of population growth, and the regulatory mandates of a community will help to achieve appropriate fits between the technology and the community. Engineers consider passively many of the social factors in the design process. They should, however, consider them actively. Not to be aware of these factors is to invite problems.

(1) Population. Slow sand is considered most appropriate for small communities. A small community may include summer camps, trailer parks, associations of rural homes, resorts, work camps, and both unincorporated and incorporated towns. The upper limit is not clear; 10,000 persons has been mentioned, but there is no documentation to support this claim. This number is mentioned here only to indicate an upper population limit for slow sand. The specific population level at which the problems of slow sand exceed the benefits depends upon contextual factors and could be from less than 1,000 up to much higher levels, as noted for the cases in Oregon (Section 2.1.3[1]).
(2) **Community Character.** Each community has its own character, which is reflected in its customs, traditions, and values. A suburban community is different from a mining town, which is different from an agricultural center, a resort area, or a camp with only a seasonal population. Figure 2.4 is a photograph of the main street of the town of Empire, CO, a mining town that had a 1984 population of 450 persons. The reductions in the labor force of a nearby mine during the 1980s, due to a lowering in the price of metals, affected the financial health of the community and its consequent ability to finance water supply improvements. Thus, having a means to finance a needed capital improvement with a declining tax base was a key concern for the community when choosing a water filtration system. The Empire slow sand facility has been judged successful due not only to the work of the community's able consulting engineer but also to the positive attitude of the community. Figure 2.5 is a photograph of the Village of 100 Mile House, BC, which had a 1986 population of 1,925 persons. The economic driving forces for the Village are forestry and cattle ranching. The Village's slow sand installation is considered cost-effective and successful. The success of these installations was due in large part to the client-engineer relations, which in each case were on a professional level. A path to an unsuccessful outcome, with wasted money, is for the client to request bids for engineering services. The most important step for the community is to engage the services of a qualified consulting engineer who is capable of understanding community issues.

(3) **Political Climate.** The quality and style of political leadership in the community will be reflected in the mandates given to the engineer either directly or through the town staff. At the same time, the engineer is the professional in whom the community is placing confidence and is accountable to the community for achieving an appropriate fit between the design of the filtration system and the community context. The engineer must seek to understand the needs of the community in order to develop a design that provides the *most appropriate fit* within the community context. The task involves the use of judgment, rather than analytical capability.

(4) **Regulations.** Regulations and guidelines promulgated by regulatory agencies are usually the impetus that stirs communities to reexamine the quality of their drinking water. Compliance with regulations can engender attitudes ranging from interest in discovering and solving problems to hostility. The former creates a fertile climate for progress, whereas the latter needs to be overcome through discussion and education. In some instances regulations have been reassessed because of community resistance, but in general the regulations reflect the norms of the society as a whole.

(5) **Economics.** When comparing treatment alternatives, an economic analysis is performed to examine the uniform annual costs of alternative types of installations.
Figure 2.4 The Main Street of Empire, CO, 1989. The town is a mining locale, located at an elevation of 2,706 m (8,878 ft), about 64 km (kilometers), or 40 mi, from Denver. (Photograph by D. W. Hendricks.)

Figure 2.5 The Main Street of 100 Mile House, BC, 1989. The Village is a ranching and logging community, located about 500 km (310 mi) from Vancouver, BC. (Courtesy Village of 100 Mile House, BC.)
Such costs usually consist of amortized capital costs and take into account salvage value and operating and maintenance costs. Cost, however, is only one consideration. Reliability, ease of operation, maintenance, land costs, aesthetics, and so forth, should also be considered.

6) Financing. Whether a plant can be built depends upon financing. Factors affecting financing include bonding capacity, interest rates, eligibility for grants, tax base, income levels, and general capacity of the community to retire revenue bonds. The source of financing, the amount that can be financed, and the terms of financing are critical aspects of any public works project. Financing in British Columbia is done through the provincial Municipal Finance Authority, which aggregates borrowing needs from all municipalities and issues bonds as required. Each community has a borrowing ceiling. This system contrasts with the system of financing in the United States, where a community may sell bonds or seek grants or loans from a state or federal agency. Financing is discussed further in Section 6.6.

7) Engineer-Client Relations. The client places full confidence in the judgment and capabilities of the engineer. How the engineer is selected has been a major issue in the United States since 1972, when the American Society of Civil Engineers (ASCE) signed a consent decree with the Department of Justice to delete a Code of Ethics statement that read: "For ASCE members to participate in competitive bidding for professional engineering services is unethical conduct." As a result of the consent decree, some clients have come to expect cost bidding between engineers, a sure path to the possible selection of a less qualified consultant and to outcomes that will have wasted public funds. Selection of an engineer should be based upon qualifications and reputation. The traditional approach is for the client to seek a qualified engineer through a proposal selection process. Once the engineer is selected, the price is negotiated. If the negotiations are successful, the engineer is selected for the job. If the negotiations are not successful, the client seeks another engineer. In Canada, the traditional engineer-client relationship has been maintained. Firms in Canada do not advertise, another index of the level of professional practice in Canada.

2.2.2 Technology

The judgments made in designing a water treatment facility will be influenced by community factors. Will the design be modular, to allow for future expansions, or "standard," to take care of the problem at hand? Modular design requires a slightly greater investment than standard design in order to facilitate future expansion. Communities just on the brink of financing capability may wish not to spend even a modest amount in the present to save future costs. Those with more latitude may wish
to have such an option. The choices in technologies extend to appurtenances and disinfection. Flow meters, for example, should not be high-tech when a simple device will serve well. The philosophy of keeping everything simple and easy to maintain and operate should extend throughout the design.

2.3 FLOW VARIATION

Before a water filtration system can be designed, the demand for treated water must be ascertained for both the annual cycle and the daily cycle. The projected demand should also be determined. In addition, any unique daily or seasonal characteristics of flow cycles must be taken into account. For example, the treated water demands of a resort area will peak during the seasonal influx of tourists. Residents of communities that have below-freezing temperatures may be in the habit of letting their faucets run to prevent freezing. For such communities, the peak day for water demand may occur in the winter. Often, flow data are difficult to obtain for small systems either because flow meters have not been installed or are not maintained or because records are lacking. In such cases, design decisions should be deferred until such data are obtained. Flow data "drive" the rest of the design. The sizing of the filter bed, for example, will be proportional to the flow projections.

2.3.1 Flow Analysis

Any waterworks must be designed to accommodate varying flows, as dictated by the water demand and over the annual cycle, the daily cycle, and with increasing population. These categories of flow variation are addressed in the following section, along with a recommended approach to achieve steady flow operation.

(1) Annual Cycle. The flow demand varies over the annual cycle according to the characteristics of the community. Lowland communities in temperate climates will have their highest water demands in the summer. In such communities, summertime demands could be as much as 4 to 5 times higher than winter demands. Other towns, such as Empire, CO, have reported very high winter demands because of the practice of "bleeding" the water lines with open faucets to prevent freezing. Empire's winter demand is 3,000 L/person/d (800 gpcd). Figure 2.6 shows the annual cycle of per capita demand for Empire. In contrast to the winter demand, the summer demand ranges from 400–600 L/person/d (107–160 gpcd). The lower level is probably the domestic demand, while the continuous increase during the summer to 600 L/person/d (160 gpcd) probably reflects the outside watering practice. The changes over the annual cycle require periodic adjustments (daily or weekly) to the influent flow to a slow sand filter.
The shape of the graph in Figure 2.6 is opposite to what would be expected for residential single-family households in temperate climates, where the per capita demands are highest in the summer when lawn watering is prevalent. Each community will have a unique annual demand curve, reflecting its unique social character. The range may include winter resorts, summer resorts, trailer parks, Indian villages, suburban communities, and farm and ranch communities.

(2) Hourly Demand. Figure 2.7 shows the estimated hourly demand curves for water for Empire, CO, if the winter demand were to be reduced to 946 L/person/d (250 gpcd)—a goal of the water system. The curves were estimated for illustration purposes using the peak average daily flow for the 1984 population of 450 and for a projected Year 2000 population of 1,000 persons. The curves show that the flow of treated water varies over a range of almost 3 to 1 over the daily demand cycle and that the projected peak day demand is about 2.2 times the present peak day flow. The slow sand filter would have to be designed, therefore, to accommodate the average peak day demand of 0.946 mil L/d (0.25 mgd), based upon a projected population of 1,000 persons. The corresponding peak hour demand flow, from Figure 2.7, is 1.5 mil L/d (0.4 mgd). The filter bed area should
be sized under the assumption that scraping will be done on the peak day and that the remaining filter beds will be able to handle adequately the peak day flow.

(3) Steady Flow Operation Over the Daily Cycle. Figure 2.8 uses data from Figure 2.7, but is plotted to show the cumulative flow demand. The curves show that 150,000 L of storage are needed. The salient point is that adequate storage permits the slow sand filter to be operated under steady flow production. In other words, the filter can operate at a steady flow of 0.946 mil L/d (0.4 mgd) over the daily cycle in which the hourly flow has a 3:1 range.

(4) Design Flow. The design flow is based upon the projected maximum daily flow, which, in the case of Empire, CO, with a projected ultimate population of 1,000 persons, was 946,000 L/d. The hydraulic loading rate on the filter at the end of the design period should approach the HLR criterion used for design.
2.3.2 Treated Water Storage

As noted above, treated water storage is an intrinsic part of any filtration system. The purpose of having adequate treated water storage is to permit steady flow to be maintained through the filter. With steady flow, a possible reduction of filtration efficiency is averted and the ideal of passive operation is approached.

(1) Equalization Storage. The volume of treated water storage needed for flow equalization is determined by a mass flow diagram, illustrated in Figure 2.8, and is the largest vertical difference between the steady flow straight line and the cumulative demand curve. For the Empire filter, the volume required, as determined from the estimated hourly demand curve for the peak day, is about 150,000 L (40,000 gal), or 5.3 m x 5.3 m x 5.3 m (17.4 ft x 17.4 ft x 17.4 ft), about the size of a two-car garage with a high ceiling. The cost is moderate relative to the benefits of having the filter function as intended.

(2) Total Storage. The total storage is the sum of the storage determined for peak day steady flow operation (the equalization storage) plus the fire flow storage plus the
minimum storage needed for chlorine contact time plus the storage required for emergencies, as given in Equation 2.1:

\[ T = A + B + C \]  
\[ (2.1) \]

in which \( T \) = total storage required (m\(^3\) or ft\(^3\))
\( A(\text{peak}) \) = peak day storage volume required (m\(^3\) or ft\(^3\))
\( B \) = fire protection storage (m\(^3\) or ft\(^3\))
\( C \) (emer. storage) = emergency storage (m\(^3\) or ft\(^3\))

The peak day storage, \( A \), is determined as indicated above. Fire protection storage should be determined by local code or by the Fire Underwriter's code. Emergency storage is the volume deemed advisable to account for power outages and other events that could disrupt water production. Both \( B \) and \( C \) volumes are addressed in state and local codes.
Chapter 3

Design

Slow sand filters are simple in concept and design. But still, to make the process work, the principles must be understood and the practices known. This chapter reviews the basic knowledge needed for design.

3.1 PRINCIPLES

The effect of independent variables, such as the size of the filter bed, hydraulics, sand recovery, filter box design, freeze protection, and disinfection, on removal efficiency, hydraulics, and filter functioning are key concerns in design. These topics are discussed in the following sections.

3.1.1 Sizing the Filter Bed

The first step in the design is to size the bed. The bed area and depth are the basic dimensions that drive the rest of the design. The bed area is determined by the hydraulic loading rate (HLR) selected. Because flow is variable, as discussed in Section 2.3, and area is fixed, the HLR should be construed as both varying over the daily cycle, except as mitigated by storage, and increasing each year to the point at which the performance capacity of the filter is met. The term performance capacity is defined here as the point at which some measure of performance, such as effluent turbidity or the amount of other particles or the rate of headloss development, exceeds either regulatory requirements or community norms. Ideally, pilot plant studies can help to determine performance characteristics as affected by HLR. Either the rate of headloss development or turbidity may exceed the expected norms at some point in the future if HLR increases year to year.

(1) Area. The bed area is calculated by Equation 1.3, which is restated here as Equation 3.1.

\[
\text{HLR} = \frac{Q}{A} \quad (3.1)
\]
in which HLR = hydraulic loading rate, defined as flow divided by plan area of sand bed (m³/m²/hr or mgd)

Q = flow (mil L/d or mgd)
A = bed area (m² or acres)

To determine the bed area, one must first determine the design flow, Q, and the acceptable range for the HLR. These issues are discussed in Sections 2.3 and 3.2.1, respectively. Example 3.1 illustrates some of the considerations relevant to the HLR criterion.

**Example 3.1: Judgment in Use of HLR Criterion.** Calculate the HLR for the Empire slow sand filter. Two filter cells will be used; when one cell is out of operation, all flow will pass through the cell in service. The design per capita water use is taken to be 946 L/c/d (liters per capita per day), or 250 gpcd, which is the peak day demand for the summer. The peak day demand for winter, when residents of the community bleed the water lines to prevent them from freezing, is about 3,000 L/c/d (800 gpcd).

1. **Determine the maximum bed area, A(max):** The Empire site is located in the canyon formed by Mad Creek and had a limited area. Building on the site required rock excavation and fill. The largest slab area for the site was determined to be 9.75 m x 17.6 m (32 ft x 58 ft). From this slab area, two filter boxes were fitted, with internal dimensions of 9.14 m x 8.38 m (30 ft x 27.5 ft). Their total area was 153 m² (1,648 ft²). Thus, A(max) = 153 m² (1,648 ft²).

2. **Illustrate the HLR calculation for HLR(946 L/c/d, 1,000 persons):** The per capita water use of 946 L/d (250 gpcd) for Q(designed, 1,000 persons) = 0.946 mil L/d. Applying Equation 3.1 gives:

\[
\text{HLR} = \frac{Q(946 \text{ L/c/d}, 1,000 \text{ persons})}{A(\text{max})} = \frac{946 \text{ m}^3/\text{d} \times \text{d/24 hr}}{153 \text{ m}^2} = 0.26 \text{ m/hr (6.5 mgad)}
\]

3. **Check HLR for four combinations of HLR and population as outlined in Table 3.1.** The calculation is as illustrated in Step 2 of this example.

<table>
<thead>
<tr>
<th>Condition</th>
<th>Per capita use</th>
<th>Flow</th>
<th>HLR (m/hr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Population</td>
<td>L/c/d</td>
<td>gpcd</td>
<td>m³/d</td>
</tr>
<tr>
<td>500</td>
<td>946</td>
<td>250</td>
<td>473</td>
</tr>
<tr>
<td></td>
<td>3,000</td>
<td>800</td>
<td>1,500</td>
</tr>
<tr>
<td>1,000</td>
<td>946</td>
<td>250</td>
<td>946</td>
</tr>
<tr>
<td></td>
<td>3,000</td>
<td>800</td>
<td>3,000</td>
</tr>
</tbody>
</table>

*a A(max) = 153 m² (1,648 ft²).
*b A(max) = 76.5 m² (824 ft²).
4. Discussion: The HLR(946 L/c/d, 1,000 persons) is within the accepted criterion range, that is, 0.04–0.40 m/hr (1.0–10 mgd). When one of the two filter cells is taken out of service for scraping, the town's treated water storage of 757,000 L (200,000 gal) is sufficient to maintain steady flow through the filter remaining in operation, as illustrated in Figure 2.8. When the high winter per capita demand occurs, however, which is about 3,000 L/c/d (800 gpcd), the HLR will increase proportionately to give $\text{HLR}(\text{max}, 1,000 \text{ persons}) = 0.82 \text{ m/hr (20 mgad)}$, or $1.64 \text{ m/hr (40 mgad)}$ when only one filter is in operation. This high per capita demand in winter is due to the practice of households opening taps to prevent their water lines from freezing. Even with the 1986 population of 500 persons, the HLR was at the upper limit of 0.4 m/hr (10 mgad) during winter months and exceeded the HLR criterion when one filter was taken out of operation. Alternatives to reduce the $\text{HLR(\text{max}, 1,000 persons)}$ are (1) to find ways to reduce the high winter per capita demand, (2) to decrease the flow to operating filter during scraping and to use storage to satisfy the remaining flow requirement, and (3) to perform pilot testing to ascertain removal efficiencies at the high HLRs. For the winter flow at population 500, the present storage of 150,000 L is sufficient to handle 5 hours flow at half $Q(1,500 \text{ m}^3/\text{d}, 500 \text{ persons})$, which is almost sufficient time to accomplish the scraping task. As the population increases, a solution will be needed. The problem of having to deal with high per capita demand in the winter is not unique to slow sand. This dilemma must be resolved by judgment rather than by applying absolute criteria.

(2) Treated Water Storage. Determination of the treated water storage volume needed is reviewed in Section 2.3.2. Example 3.1 shows that in addition to providing equalization, fire, and emergency storage, another purpose of the storage volume may be to permit taking one of the filters out of service for scraping.

(3) Depth of Sand. The depth of the sand bed is determined by the number of years of operation desired before resanding is needed and by any constraints on the filter box depth. The years of operation are calculated as follows:

$$Y = \frac{D_t - D_i}{R \cdot f(\text{scraping})}$$  \hspace{1cm} (3.2)

in which $Y =$ years of operation before sand bed rebuilding is necessary

- $D_i =$ initial sand bed depth (cm or ft)
- $D_f =$ final sand bed depth before rebuilding (cm or ft)
- $R =$ sand depth removal per scraping (cm/scraping or ft/scraping)
- $f(\text{scraping}) =$ frequency of scraping (scrapings/year)

(4) Effect of Bed Depth on Removal Efficiency. The removal efficiency of the sand bed depends more upon its biological maturity than upon its depth. Removals of coliforms using pilot filters with a 1.0-m sand bed depth were 97 percent; with a 0.5-m sand bed, removals were 95 percent (Bellamy et al. 1985a, 1985b [p. 79]). Other studies have shown that most of the removal occurs within the top few centimeters of the sand bed, where the biological activity seems greatest. Investigations have been conducted to determine the minimum depth, $D_f$, of the sand bed before it must be rebuilt. Huisman and Wood (1974) recommended a depth of 0.5–0.8 m (1.6–2.6 ft). Visscher et al. (1987)
recommended a depth of 0.5 m (1.6 ft). Based upon the findings that most of the biological activity occurs within the top few centimeters of the sand bed and the findings of Bellamy et al. (1985a, 1985b) that removal efficiency was reduced from 97 percent for a 1.0-m sand bed to 95 percent for a 0.5-m sand bed, a 0.3-m (1.0-ft) minimum depth could be feasible. The extra usable bed depth could add 20–40 scrapings to the filter bed, which translates to 2–3 years in the case of the Empire filter. Thus, from the foregoing discussion the range in guidelines for minimum depth seems to be 30–80 cm (1.0–2.6 ft). The question is important enough that pilot plant testing is warranted.

(5) Effect of Bed Depth on Bed Life, Headloss, and Run Time. Examples 3.2 and 3.3 show the benefits and costs of using a deeper bed depth. Example 3.2 shows that several years of additional bed life are possible when a 1.3-m (4-ft) bed depth as opposed to a 1.0-m (3.3-ft) bed is used. Example 3.3 shows that the clean-bed headloss is only 26 cm (10 in.) for the deeper bed versus 20 cm (8 in.) for the 1.0-m (3.3-ft) bed. The examples also illustrate the utility of Darcy's law in answering some critical questions in design. Though a simple expression, Darcy's law may be used to address many kinds of questions.

Example 3.2: Effect of Bed Depth on Bed Life. Compare the bed life of a 1.3-m (4-ft) bed to that of a 1.0-m (3.3-ft) bed at Empire, CO.

1. Given data for Empire:
   (1) \( R = 0.5 \text{ cm/scraping} \)
   (2) \( f(\text{scraping}) = 12 \text{ scrapings/year} \)
   (3) \( D_f = 30 \text{ cm} \)

2. Calculation:
   (a) For \( D = 130 \text{ cm} \),
      \[
      Y = \frac{130 - 30}{0.5(\text{cm/scraping})\times12(\text{scrapings/year})} = 17 \text{ years}
      \]
   (b) For \( D = 100 \text{ cm} \),
      \[
      Y = \frac{100 - 30}{0.5(\text{cm/scraping})\times12(\text{scrapings/year})} = 12 \text{ years}
      \]

3. Discussion: A bed depth of 1.3 m (4 ft) gives a bed life of 17 years, whereas a 1.0-m (3.3-ft) bed depth gives a bed life of only 12 years. The bed life increases by 5 years merely by adding an additional 30 cm (1.0 ft) of bed depth.

The trade-off for the 1.3-m (4-ft) bed depth vis-à-vis a 1.0-m (3.3-ft) depth is the additional box depth and a higher clean-bed headloss for the first few years. The
additional box depth needed is the incremental bed depth of 30 cm (1.0 ft). This additional box depth should accommodate the additional clean-bed headloss required with the deeper bed.

The effect of increased sand bed depth on headloss is calculated in Example 3.3, which shows that a 1.3-m (4.0-ft) bed depth at 0°C has a clean-bed headloss of only 26 cm (10 in.) compared with 20 cm (8 in.) for a 1.0-m (3.0-ft) bed. The example also shows that the run time decreases by only two days with the deeper bed. Figure 3.1 illustrates the foregoing relationship for the Empire, CO, filter. The rate of headloss increase is shown to be linear for the purpose of illustration. Whether the increase is linear with time or exponential, the principle that illustrates the effect of a deeper bed depth is the same. The depositing material composing the *schmutzdecke* will have the same effect on headloss irrespective of bed depth. Filters in operation, such as at Empire and 100 Mile House, have shown a linear rate of headloss increase for weeks. The terminal headloss

---

**Figure 3.1** Comparison of Headloss Versus Run Time for Two Sand Bed Depths (D = 1.0 m and D = 1.3 m), Showing the Effect of Additional Sand Bed Depth on Run Time (see Example 3.3)
of these filters has then occurred quickly after sharp increases in the rate of headloss increase.

**Example 3.3: Effect of Using a Deeper Bed on Clean-Bed Headloss and Run Time.**

Compare the clean-bed headlosses and the filter run times for a 1.3-m (4-ft) bed depth and a 1.0-m (3-ft) bed depth at the Empire, CO, plant.

1. **Given data for Empire:**
   (1) HLR = 0.2 m/hr (typical HLR for Empire)
   (2) \( k' = 6.6 \times 10^{-7} \text{ N/m} \)
   (3) \( T = 0^\circ\text{C} \) (assume winter conditions); from Appendix Table A.1, \( \mu(0^\circ\text{C}) = 1.8 \times 10^{-3} \text{ (N-s/m}^2) \)
   (4) \( t = 30\text{-days run time} \)
   (5) \( h_L(\text{available}) = 1.5 \text{ m (5.0 ft)} \)

2. **Calculate clean-bed headloss:**
   (a) For \( D = 1.3 \text{ m} \):
      
      \[
      \text{HLR} = \frac{k'}{\mu} \frac{h_L}{\Delta z} \tag{1.5}
      \]
      
      \[
      0.2 \text{ (m/hr)} = \frac{6.6 \times 10^{-7} \text{ (N/m)}}{1.8 \times 10^{-3} \text{ (N-s/m}^2)} \frac{h_L}{0.013 \text{ (m)}} \frac{3600 \text{ (s)}}{30 \text{ (hr)}}
      \]
      
      Solve for \( h_L \).
      
      \( h_L(D = 1.3 \text{ m}) = 0.20 \text{ m (0}^\circ\text{C)} \)

   (b) For \( D = 1.0 \text{ m} \):
      
      (1) Solve Equation 1.5 for \( h_L \).
      
      \( h_L(D = 1.0 \text{ m}) = 0.26 \text{ m (0}^\circ\text{C)} \)

3. **Discussion:** A bed depth of 1.3 m (4 ft) gives a clean-bed headloss of 26 cm (10 in.), whereas a 1.0-m (3-ft) bed depth gives a headloss of 20 cm (8 in.). The clean-bed headloss increases by only 6 cm (2.4 in.) as a result of an additional 36 cm (1.0 ft) of bed depth.

4. **Comparison of run times:** Figure 3.1 illustrates the headloss-time curves for the sand bed depths of 1.0 m (3 ft) and 1.3 m (4 ft). The curves show that the 1.0-m (3-ft) sand bed has a 20-cm (8-in.) initial headloss while the 1.3-m (4-ft) sand bed has an initial headloss of 26 cm (10 in.). The Empire filter, which has a sand bed depth of 1.3 m (4 ft), has had filtration runs of about 30 days and a terminal headloss of 1.5 m (4.5 ft). The headloss increases after the initial headloss due to the accumulating surface deposit. As noted in Figure 3.1, the rate of headloss increase is: \( (1.50 - 0.26) \text{ m/30 d = 0.041 m/d} \). Thus, the run time for a filter with a sand bed depth of \( D = 1.0 \text{ m} \) would be: \( (1.50 - 0.20) \text{ m/0.041 m/d = 31.7 d = 32 d} \). This means the "penalty" for the 1.3-m (4-ft) sand bed, vis-à-vis a 1.0-m (3-ft) bed, is only 2 days. Weighed against this is the additional 5-year bed life before rebuilding for the deeper bed (Example 3.2). Also to be included in the bed-depth decision are the cost of concrete for the additional 36 cm of sidewall and whether the foundation soil can accommodate the additional soil pressure.
Although Hazen (1913) suggested bed depths of 0.67–1.3 m, or 2–4 ft (Table 1.3), a bed depth of about 1 m (3 ft) has become traditional. There is no reason, however, to limit the bed to this depth. The Empire filter, for example, broke this psychological barrier of 1 m (3 ft) with a bed depth of 1.3 m (4 ft). The result was an increase in bed life by about 5 years (Seelaus et al. 1988, p. 24) when compared with the bed life of a 1.0-m (3-ft) sand bed.

The deeper the sand bed, the longer the filter can operate before the bed must be rebuilt. The trade-offs are that a deeper sand bed requires a deeper box, with walls designed to handle the additional hydraulic pressure, and that the initial headloss is proportionally greater. As illustrated by the calculations in Examples 3.2 and 3.3, the trade-offs are small compared with the benefits of longer operation. For example, adding 30 cm to the bed depth lengthens the life of the filter bed by about 30 months, assuming that 1 cm of sand is removed by each scraping and that the scraping occurs once per month. (Note: At Empire the amount scraped has averaged only 0.5 cm per scraping; the 1 cm just cited is used to illustrate a maximum scraping rate.)

3.1.2 Hydraulics

A number of design decisions are driven by the results of hydraulic analysis. Major hydraulic functions are (1) to distribute the raw water without erosion of the sand bed; (2) to collect water uniformly from the filter; (3) to drain the headwater for sand bed scraping; (4) to provide for overflow of the filter box; (5) to measure the flow to or from the filter; (6) to control the flow through the filter; (7) to measure headloss through the filter bed; (8) to provide for a variety of plumbing needs, such as filter-to-waste, drains, directing flows, filling the dry bed from the bottom, and so forth; and (9) to avoid negative pressures within the sand bed.

In aggregate, the plumbing system that performs the above functions is the most complex system of the slow sand filter plant. But when viewed separately, the subsystems are simple. Figure 3.2, which shows the configuration of pipelines for slow sand filters at Moricetown, BC, illustrates the complexity of the plumbing system. The drawing shows only the basic subsystems (the complete drawing is reproduced as Figure 5.2). Figure 3.2 shows three plumbing subsystems: influent raw water, filtered water, and drainage. Within each subsystem further disaggregation is possible. For example, the filtered water plumbing has provision to backfill the sand bed with treated water. Also, the drainage system provides for overflows from the filters and from the chlorine contact basin and reservoir, for drawdown of headwater, for dewatering the sand bed, and for draining the chlorine contact basin. The sections that follow describe the plumbing subsystems.
Figure 3.2 Flow Configuration for Slow Sand Filters at Moricetown, BC. (Adapted from drawings by Dayton & Knight, Ltd. [1988a].)

1) Distribution. Figure 3.3 shows sand bed erosion in the slow sand filter at Empire, CO. The depression shown in the figure is about 20 cm (8 in.) deep and has a diameter of nearly 60 cm (2 ft). It formed because the entire influent flow was delivered to one point in the filter bed. The consequence of such a condition is short-circuiting of flow through the sand bed. To control sand bed erosion, such as shown in Figure 3.3, the kinetic energy of the flow must be distributed, dissipated, or both. Figure 3.4 illustrates the distribution approach, showing how the raw water influent flow may be distributed around the filter box. The lateral pipes must be large enough that the exit velocity is sufficiently low. Because criteria for flow distribution do not exist, selection of exit velocity and lateral pipe size is a matter of judgment. Example 3.4 illustrates an approach for determining exit velocity and lateral pipe size.
Example 3.4: Flow Distribution to Control Erosion of Sand Bed. Select a lateral pipe size for distributing the flow around the periphery of the sand bed as shown in Figure 3.4.

1. Terms. The terms used in the equations that follow are defined as follows:

- \( Q_{\text{orifice}} \): flow through orifice (L/s or ft\(^3\)/s)
- \( A_{\text{orifice}} \): cross-section area of orifice opening (m\(^2\) or ft\(^2\))
- \( d \): diameter of orifice (m or ft)
- \( C_d \): orifice discharge coefficient, given in Appendix Table B.1 (dimensionless)
- \( g \): acceleration of gravity (9.81 m/s\(^2\) or 32.2 ft/s\(^2\))
- \( \Delta h \): head difference between inside and outside of orifice (m or ft)
- \( h_L \): headloss in pipe flow (m or ft)
- \( f \): Darcy-Weisbach friction factor for pipe material (dimensionless)
- \( L \): length of pipe for headloss measurement (m or ft)
- \( D \): diameter of pipe (m or ft)
- \( v_{\text{pipe}} \): velocity of water flow within pipe (m/s or ft/s)

2. Determine the flow:

   (1) Assume HLR = 0.4 m/hr (10 mgad)—the approximate upper limit for HLR.
   (2) \( A_{\text{sand bed}} = 10.0 \text{ m} \cdot 30.0 \text{ m} = 300 \text{ m}^2 (3,228 \text{ ft}^2) \)
   (3) \( Q = A \cdot \text{HLR} = 300 \text{ m}^2 \cdot 0.4 \text{ m/hr} = 120 \text{ m}^3/\text{hr} = 0.033 \text{ m}^3/\text{s} (1.16 \text{ ft}^3/\text{s}) \)
3. Calculate, by the continuity equation (i.e., flow = area times velocity), the exit velocity from the header pipe:

(a) Assume a header pipe 30.5 cm (1 ft) in diameter, with a single outlet into the sand bed (not illustrated):

\[ v = \frac{Q}{A} \]  
\[ = \frac{0.033 \text{ m}^3/\text{s}}{0.073 \text{ m}^2} \]
\[ = 0.45 \text{ m/s} (1.47 \text{ ft/s}) \]

(b) Next, assume that 10 orifices distribute the flow from the 30.5-cm-diameter (1.0-ft) header pipe around the periphery of the filter box, as shown in Figure 3.4:
Q(orifice) = Q/10

= 0.033 m³/s/10

= 0.0033 m³/s (0.116 ft³/s)

Now assume the header pipe is fitted with 10 orifice plates that are each 25.4 cm (10 in.) in diameter and calculate the exit velocity from one of the 10 orifice plates:

\[ v = \frac{Q(orifice)}{A(orifice)} \]

\[ = \frac{0.0033 \text{ m}^3/\text{s}}{0.0507 \text{ m}^2} \]

\[ = 0.065 \text{ m/s (0.213 ft/s)} \]

Determine the required pressure within the header pipe to deliver 0.033 m³ through each of the 10 orifices. Appendix Table B.1 gives the orifice discharge coefficient, \( C_d = 0.77 \) for \( d/D = 0.83 \). Substituting data in the orifice equation gives:

\[ Q = C_d A(2gAh)^{1/2} \]

\[ 0.0033 = 0.77 \times 0.0507 \times (2 \times 9.8 \times h)^{1/2} \]

\[ h = 0.00036 \text{ m} \]

In other words, the differential head required across the orifice plate is only \( \approx 0.036 \text{ cm} \).

Now calculate the friction head loss along the header pipe. Assume the pipe is 17 m long, that is, \( L = (300 \text{ m}^2)^{1/2} \), and that the pipe is flowing full its entire length. Also assume the Darcy-Weisbach friction factor \( f \) is 0.012. Substituting numerical data in the Darcy-Weisbach equation for pipe flow gives:

\[ h_L = f \frac{L (v(pipe))^2}{D 2g} \]

\[ = 0.012 \times \frac{17 (0.45)^2}{0.305 \times 2 \times 9.81} \]

\[ = 0.007 \text{ m} \]

4. Discussion: If there is only a single discharge point, as in Step 2a, this exit velocity will certainly cause erosion of the sand bed unless the velocity is dissipated. Such a filter should be backfilled with perhaps a foot of water above the sand bed before each start-up after scraping. A more suitable design is shown in Figure 3.4, with calculations as in Step 2b of this example, with distribution by 10 orifice plates fitted on the header pipe around the inside periphery of the box. Even with the lower velocities from each orifice plate, some erosion can occur. Thus, even with this design, the sand bed should be protected by dissipating the kinetic energy of the velocity by means of starting the filter with 0.3 m (1 ft) of water above the sand bed. The 0.3-m (1-ft) water cushion will minimize bed erosion for the multiple outlet design, but for the single outlet design the water cushion will merely alleviate deep erosion of the bed.

Another consideration is that the flow from each of the lateral pipes should be approximately equal. Such flow equalization is accomplished by having a large header pipe relative to the orifice plate sizes. The principle is to have negligible headloss in the header pipe relative to the orifices. As noted above, the headloss across the orifices is only \( \approx 0.036 \text{ cm} \), whereas the header pipe will
have a friction loss of ≈ 0.7 cm. This calls for a more exact design as a "manifold" problem. Thus the pipe would be divided into segments with headloss calculated for each segment after accounting for the loss in flow through each orifice. The orifice size at the beginning of the header pipe may be adjusted to a smaller size so that the flows through each orifice are approximately equal. An alternate approach is merely to use a larger header pipe. With a larger header pipe, the pressures behind each orifice plate would be closer to equality.

As noted for the filter bed described in Example 3.4, the exit velocity with a single 0.3-m-diameter (1-ft) pipe is high enough to cause erosion. Also, even with the 10 distribution points around the filter bed, sand bed erosion is still a potential problem, albeit minimized. Therefore, in addition to distributing the flow uniformly around the bed, the bed should be backfilled with about 0.3 m (1 ft) of water to dissipate the kinetic energy of the exit flow. To accomplish the backfilling, the filters that are being filled should have a hydraulic connection with the operating filters. If the raw water inlet ports are located about 0.3 m (1.0 ft) above the highest level of the sand bed, the kinetic energy should dissipate as a "submerged jet." The weir plate controlling the tailwater elevation should be raised for this start-up period. When the headloss across the sand bed becomes about 0.3 m (1.0 ft), the weir plate should be lowered so that the crest is at the top of the sand bed.

Figure 3.5 Underdrain Pipes Placed on the Floor of the Slow Sand Filter at 100 Mile House, BC. (Courtesy Dayton & Knight, Ltd., Vancouver, BC.)
Figure 3.6 Type of Slotted Underdrain Pipe Used at 100 Mile House, BC. The pipe is SDR 26 PVC, 15 cm (6 in.) in diameter, with 131 slots/m/row and 3 rows around the diameter of the pipe. Each slot is 1 mm (0.039 in.) wide and 2.5 cm (1 in.) long. (Courtesy Dayton & Knight, Ltd., Vancouver, BC.)

(2) Collection. Figure 3.5 shows the underdrain pipe layout for one of the three slow sand filters at 100 Mile House, BC, which has dimensions of 43 m x 6 m (141 x 19.7 ft). The photograph was taken before the gravel was installed. Underdrain pipes may be purchased as perforated pipe or as slotted pipe. Figure 3.6 shows the slotted pipe used at 100 Mile House. The slotted pipes were 15-cm-diameter (6-in.) SDR 26 PVC with 131 slots/m/row and 3 rows around the diameter of the pipe. Each slot was 1 mm (0.039 in.) wide and 2.5 cm (1 in.) long.

The spacing of the underdrains is usually decided based upon practice. The underdrain pipes for the slow sand filters at the Village of 100 Mile House were spaced at 2 m (6.6 ft). The 2-m (6.6-ft) spacing should be satisfactory, since the headloss through the gravel support is negligible compared to the headloss through the sand bed. However, a spacing of 1 m (3.3 ft) is preferred; the closer spacing is an inexpensive means of providing an added certainty of uniform HLR as well as a safety factor with respect to clogging.

Additional certainty of having uniform HLR over the filter bed may be attained by applying hydraulic principles when sizing the underdrain pipes and the perforated
holes. The underdrain system is a manifold, and as stated in Example 3.4, the system's exact design requires an involved calculation process. One can, however, get an idea of the design, without carrying out such calculations, by applying a basic principle of manifold design, that is, that the headloss across the system points at which flow is distributed should be large compared to the headloss within the manifold header pipe. The idea is that the pressure within the manifold pipe, at each of the points of distribution, should be about equal. Figure 3.7 shows an underdrain layout, the header pipe, and the tailwater control for a hypothetical filter. Example 3.5 illustrates the principles in the design of underdrain laterals and orifices. An analysis is given also for underdrains consisting of the slotted pipe.

Figure 3.7 Underdrain Layout for Hypothetical Slow Sand Filter
Example 3.5: Determine the size of the underdrain system shown in Figure 3.7.

1. **Terms:** The terms used in the equations that follow that have not been defined previously are:

   q(lateral) = flow in underdrain lateral (m\(^3\)/s)  
   q(orifice) = flow through orifice in underdrain pipe (m\(^3\)/s)  
   N(orifices) = number of orifices in underdrain lateral  
   q(slot) = flow through orifice slot in underdrain pipe (m\(^3\)/s)  
   A(slot) = area of slot (m\(^2\))

2. **Determine the flow:**

   (1) Assume HLR = 0.4 m/hr (10 mgd)—the approximate upper limit for HLR.  
   (2) A(sand bed) = 10.0 m \(\times\) 30.0 m = 300 m\(^2\) (3,228 ft\(^2\))  
   (3) \(Q = A(sand\ \text{bed}) \times HLR = 300 \times 0.4 = 120 \text{ m}^3/\text{hr} = 0.033 \text{ m}^3/\text{s} = 1.16 \text{ ft}^3/\text{s}\)

3. **Size the laterals:**

   (a) Determine the flow for each lateral. The box has 10 laterals, spaced at 1.0 m (3.3 ft). The flow at the end of each lateral, q(lateral), is:

   \[
   q(lateral) = \frac{Q}{10} = \frac{0.033 \text{ m}^3/\text{s}}{10 \text{ laterals}} = 0.0033 \text{ m}^3/\text{s}/\text{lateral}
   \]

   (b) Determine the headloss for each lateral assuming a lateral flow of 0.0033 m\(^3\)/s for the full length:

   Trial 1, assume a lateral diameter of 0.20 m (8 in.).

   \[
   v = \frac{q(lateral)}{A} = \frac{0.0033 \text{ m}^3/\text{s}}{0.032 \text{ m}^2} = 0.10 \text{ m/s}
   \]

   \[
   h_L = \frac{fL(v\text{pipe})^2}{2g} = 0.012 \times \frac{20 \text{ m}}{0.202 \text{ m}} \times \frac{(0.10)^2 \text{ m}^2/\text{s}^2}{2 \times 9.81 \text{ m/s}^2} = 0.0006 \text{ m} = 0.06 \text{ cm}
   \]

   Trial 2, assume a lateral diameter of 0.15 m (6 in.). The same calculation using the Darcy-Weisbach equation gives \(v = 0.14 \text{ m/s}\) and \(h_L = 0.0016 \text{ m} = 0.16 \text{ cm}\).

   (c) Discussion: The 0.15-m-diameter (6-in.) lateral has a headloss of only 0.16 cm for its 20-m length for full flow, while the 0.20-m (8-in.) pipe has a headloss of only 0.06 cm. The smaller the headloss in the lateral pipe, the more even are the orifice flows between the two ends of the pipe. Before choosing a pipe, however, the headloss across orifices of different sizes should be examined.
4. **Size the orifices:**

(a) Assume that a headloss of 10 cm is desirable across the orifice for the 0.20-m lateral pipe. Also assume that the orifice size should be approximately 1.0 cm in diameter—small enough to restrict entrance of the gravel and yet large enough to avoid clogging. Using these assumptions, calculate the flow per orifice, \( q(\text{orifice}) \), using the standard orifice equation, with \( C_d = 0.62 \).

\[
q(\text{orifice}) = C_d A \sqrt{2g\Delta h}
\]  

\[
= 0.62 \frac{\pi}{4} (0.01)^2 \text{m}^2 \sqrt{2 \times 9.81 \frac{\text{m}}{\text{s}^2} \times 0.10 \text{ m}}
\]

\[
= 0.000068 \text{ m}^3/\text{s}
\]

\[
= 0.07 \text{ L/s (1.1 gpm)}
\]

(b) Calculate the number of orifices needed.

\[
N = \frac{q(\text{lateral})}{q(\text{orifice})} = \frac{0.0033 \text{ m}^3/\text{s}}{0.00007 \text{ m}^3/\text{s}}
\]

\[
= 47 \text{ orifices/lateral, or 2.4 orifices/m, which is sparse.}
\]

(c) Try another approach. Assume 20 orifices/m of lateral and an orifice diameter of 0.5 cm. Determine the corresponding headloss.

\[
q(\text{orifice}) = \frac{q(\text{lateral})}{N(\text{orifices})} = \frac{0.0033 \text{ m}^3/\text{s}}{20 \text{ orifices/m} \times 20 \text{ m}}
\]

\[
= 0.000008 \text{ m}^3/\text{s/orifice}
\]

\[
q(\text{orifice}) = C_d A \sqrt{2g\Delta h} \]  

\[
0.000008 = 0.62 \frac{0.005^2}{4} \sqrt{2 \times 9.81 \Delta h}
\]

\[
\Delta h = 0.022 \text{ m} = 2.2 \text{ cm}
\]

(d) Discussion: Since the orifice headloss is 2.2 cm, the headloss across the orifice located the the beginning of the lateral is 2.20 cm, while at the header end of the lateral the orifice headloss is 2.26 cm for the 20-cm (8-in.) lateral and 2.36 cm for the 15-cm (6-in.) lateral. The difference in the flows is proportional to the square root of the respective headlosses. Thus, if the flow is one unit at the beginning of the lateral, i.e. \((2.20)^{1/2}\), at the header end the flow is \((2.36)^{1/2}\) for the 15-cm (6-in.) lateral and \((2.26)^{1/2}\) for the 20-cm (8-in.) pipe, with ratios, 1: 1.04: 1.01, respectively. Thus the 15-cm (6-in.) pipe has a flow of 4 percent more at the header end of the pipe than at the beginning, while the 20-cm (8-in.) pipe has a 1 percent flow difference. The 4 percent difference should be acceptable and so the 15-cm (6-in.) lateral pipe should be satisfactory. If the orifice headloss is small, however, the effect of later pipe size becomes more.

5. **Determine the size of the header pipe:**

(a) Assume the whole flow passes through the manifold pipe. Try a 46-cm (18-in.) pipe, for which \( v = 0.033 \text{ m}^3/\text{s} / 0.167 \text{ m}^2 = 0.20 \text{ m/s} \). For this condition, again apply the Darcy-Weisbach equation:
6. **Summary:** The final underdrain design will be sized as follows.
(a) Header pipe: $d = 0.46$ m (18 in.), which results in a headloss of only 0.05 cm at $Q_{\text{max}}$.
(b) Laterals: $d = 0.15$ m (8 in.), which results in a headloss of only 0.16 cm and a velocity of 0.14 m/s at $Q_{\text{max}}$.
(c) Orifices: $d = 0.5$ cm (1.3 in.), $N = 20$ holes/m, and $h_L = 2.2$ cm (which is large relative to the laterals and the header pipe).

7. **Discussion:** Filters should always operate with a uniform hydraulic loading rate (HLR) over the sand bed. Violation of this principle is a frequent cause of problems. Implementation of uniform HLR is dependent upon having a proper underdrain design. As a philosophy of underdrain design, the most prudent approach is to err on the side of over-design, albeit the cost will increase by the incremental price of larger pipe sizes. The investment is well placed, however, as the larger capital investment of the overall plant is thus protected. As noted above, the 15-cm (6-in.) pipe will be satisfactory for the conditions stated.

8. **Analysis for the commercial underdrain laterals used at 100 Mile House, BC:** The underdrains used at 100 Mile House by Dayton & Knight, Ltd., are 10-cm (6-in.) SDR 26 PVC pipe with 131 slots/row/m of pipe and 3 rows around the pipe circumference. The slots are 1 mm (0.039 in.) wide and 2.5 cm (1.0 in.) long.

Calculate the headloss across the orifices. The number of orifices for the layout of Figure 3.7 is:

$$N = \frac{131 \text{ slots}}{\text{m/row}} \times \frac{3 \text{ rows}}{\text{pipe}} \times \frac{10 \text{ pipes}}{\text{pipe}} \times \frac{20 \text{ m}}{} = 78,600 \text{ slots}$$

Next, the flow through the orifice slot is:

$$q(\text{slot}) = \frac{0.033 \text{ m}^3/\text{s}}{78,600 \text{ slots}} = 0.0000004 \text{ m}^3/\text{s/slot}$$

And the area of each slot is:

$$A(\text{slot}) = 0.001 \text{ m} \times 0.025 \text{ m} = 0.000025 \text{ m}^2$$

Thus, using Equation 3.4, the flow through the orifice is:

$$q(\text{orifice}) = C_d A \sqrt{2gA\Delta h}$$

$$0.0000004 \text{ m}^3/\text{s} = 0.62 \times (0.000025 \text{ m}^2) \sqrt{2 \times (9.81 \text{ m}) (\Delta h \text{ m})}$$

And the headloss per orifice is:

$$\Delta h(\text{orifice}) = 0.00003 \text{ m} = 0.003 \text{ cm}$$
Comments: With so little headloss across the orifices, the headloss within the lateral becomes more important. Because uniform flow is not certain, 0.20-m (8-in.) laterals with 0.6-cm headloss are advised. An alternative is to specify laterals having fewer orifices.

Further Analysis: At this point, principles of the hydraulic analysis should be reviewed. First, the outside pressure head (that is, within the gravel media and the pipe surface) at all points along any underdrain lateral should be approximately equal. The closer the spacing of the laterals, the more likely it is that this will be true, since the small headlosses within the gravel support will be less with shorter flow distance through the gravel. Second, with the pressure head the same across a given elevation outside the pipe, the difference in the pressure differential between one orifice and another is due to the headloss within the pipe. Therefore, examining orifice flows along the 15-cm (6-in.) lateral if the pressure differential across an orifice at the head of the lateral is 0.003 cm, the pressure differential will be 0.163 cm (= 0.003 cm + 0.160 cm) at the end of the lateral. The orifice flow near the collection header pipe is therefore:

\[
q_{\text{orifice}} = C_d A \sqrt{2gh}
\]

\[
= 0.62 (0.000025 \text{ m}^2) \sqrt{2 \times 9.81 \times 0.00163}
\]

\[
= 0.0000027 \text{ m}^3/\text{s}
\]

Note that the flow is 7 times higher at the header end of the pipe than at the beginning of the pipe (0.0027 L/s/orifice versus 0.0004 L/s/orifice). By comparison, for the 20-cm (8-in.) pipe, the orifice flow is calculated: \( q_{\text{orifice}} = 0.62 (0.000025 \text{ m}^2) \sqrt{2 \times 9.81 \times 0.00063 s} = 0.0000017 \text{ m}^3/\text{s} \) at the header end, which is still 4 times the flow at the beginning of the pipe. Actually, the problem is not as acute as these numbers make it seem because of the assumption that the lateral pipe is flowing full. If the flowing-full assumption gives a reasonable result, then the real conditions will be satisfactory. In the case discussed here, however, it is worthwhile to go to the trouble of a more precise analysis, that is, to take into account that the lateral pipe picks up flow with its length. But in any case, the HLR will be higher near the header pipe and lower at the beginning of the lateral. The importance of underdrain design is seen, in this example, as paramount in achieving uniform HLR.

(3) Drainage. To scrape the sand bed, the headwater must be drained to a level just below the sand bed surface. Figure 3.8 shows the pipe layout for draining the headwater. The profile view shows the drainage occurring in two drawdown stages: (1) the inflow distribution system removes the top portion of the headwater, (2) the underdrain system removes the remaining water. The plumbing here is simple, as only valves are needed to isolate the filter box. The positions of the valves are indicated in the profile view.

(4) Backfilling After Scraping. After scraping, the dewatered filter must be backfilled with finished water. Backfilling can be accomplished easily with the valve configuration shown in Figure 3.9. Part a shows normal operation, with the valves connecting the filters in a closed condition. Part b shows backfilling from Filter 2 to Filter 1, which assumes Filter 1 was the one scraped and now must be backfilled until the water depth reaches about 30 cm (1.0 ft) above the sand bed surface. Thus, the water level in Filter 2 must be higher than the intended water level in Filter 1. The valve connecting the filters must be open and the finished water line valves closed. Filter 3
Figure 3.8 Drainage System for Removing the Headwater From Filter Before Scraping

will continue in the filtration mode. The HLR for Filter 3 will increase during the backfilling unless the operator reduces the flow to it with the influent meter and valve. The treated water storage should be sufficient to satisfy demand during the time Filters 1 and 2 are not in operation.

(5) Overflow. The configuration for the overflow piping is simple, but it deserves special mention because of its importance in preventing over-topping of the filter box. Figure 3.10 shows the overflow weir box and the associated piping. The weir box should be installed above the sand bed so that the weir crest is just below the high water level. The weir crest should have sufficient length so that the head to carry the flow will not encroach on the freeboard (the vertical distance between the high water level and the top of the filter box). Example 3.6 illustrates the procedure for calculating weir length. The head permitted, 10 cm (0.33 ft), is reasonable and results in a weir length of only 60 cm (2.0 ft).
Example 3.6: Overflow Weir Sizing. Calculate the length of overflow weir for the weir box shown in Figure 3.10. Assume that H, the permissible depth of water above the weir crest is 10 cm (0.328 ft), and that P, the distance from the bottom of the channel to the weir crest, is 3.0 m (9.8 ft). (See section 3.3.1 for a discussion of weirs.)

1. Determine the flow:
   (1) Assume HLR = 0.4 m/hr (10 mgad)—the approximate upper limit for HLR.
   (2) A(sand bed) = 10.0 m x 30.0 m = 300 m² (3,228 ft²)
   (3) Q = A x HLR = 300 m² x 0.4 m/hr = 120 m³/hr = 0.033 m³/s (1.18 ft³/s)

2. Calculation: The weir coefficient is calculated using Equation 3.6.

\[
C_w = 0.40 + 0.05 \frac{H}{P}
\]  

(3.6)

Then substitute numerical data in the standard weir equation (equation 3.7), in which \( C_w \) is the weir coefficient and \( b \) is the length of the weir crest. See also Appendix Table B.2.

\[
Q = C_w \sqrt{2g} b H^{3/2}
\]  

(3.7)

\[
0.033 \text{ m}^3/\text{s} = \left[ 0.40 + 0.05 \frac{H}{P} \right] \sqrt{2 \times 9.81} b (0.10)^{3/2}
\]

\[
b = 30 \text{ cm (12 in.)}
\]
(6) Flow Measurements. The locations of flow measurement instruments are shown in Figure 3.11. Illustrated are: (1) the orifice meter on the influent side for the whole plant, (2) the orifice meters on the influent side for the individual filters, and (3) the volumetric flow meter on the exit side for the whole plant. The flow meters for the individual filters may be on either the influent side or the effluent side. The influent flow meter for the plant is used to determine the flow to the plant, which can be adjusted using the gate valve just downstream. The meters for the individual filters are used to ensure that each filter receives the same flow and to measure the volumes of water filtered between scrapings. The volumetric flow meter on the finished water side can provide data for records on water usage by the community. Metering devices for slow sand filters are discussed in greater detail in Section 3.3.
(7) Flow Control. Flow to the plant is controlled on the influent side by means of a gate valve located downstream from the metering device, as indicated in Figure 3.11. The flow into the plant should be steady over a 24-hr period. Treated water stored in the plant provides for varying hourly demands over the daily cycle. The treated water storage tank should have an overflow weir with drainage pipeline to waste, so that uncertainties in the daily demand can be met by having a treated water flow slightly in excess of demand flow.

(8) Tailwater Control. The effluent valve does not serve to control flow; its only function is to increase headloss so that the water level in the filter can be raised to about 0.3 m (1.0 ft) above the sand bed immediately after scraping in order to dissipate the kinetic energy of the jets of influent flow. The valve should be opened fully after the sand bed headloss increases to 0.3 m (1.0 ft). The water level is measured with piezometers.

A weir, and not the effluent control valve, is the recommended means to achieve tailwater elevation control. Figure 3.12 illustrates how a weir is used to control tailwater level, showing the weir plate on the effluent side of the filter and the crest elevation being set to the same elevation as the top of the sand bed. The weir plate should be designed so that it can be raised or lowered during operation. The initial position of the weir crest, at the start of a filter run, should be about 0.3 m (1 ft) above the top of the sand bed, but the weir plate should be adjustable enough that the weir crest can be lowered to the elevation of the surface of the sand bed once the headloss across the sand bed is greater than 0.3 m (1 ft). If flow measurement is desired, a hook gauge can be installed. The weir should be considered an auxilliary flow meter and not a replacement for other meters.

Figure 3.11 Flow Meters for Slow Sand Filters. The valve for flow adjustment is also shown, though the valves for other purposes are not shown.
(9) Headloss Measurement. Piezometers should always be installed in filters, particularly in slow sand filters, to measure headloss. The piezometers can be plastic tubes connected to points in the filter where pressure measurements are needed. One piezometer should be connected to the headwater above the sand bed, and a second one to the tailwater basin. Mounted side-by-side, with scales, these instruments permit easy measurement of water levels. Figure 3.13 shows the piezometers in one of the slow sand filters at the Village of 100 Mile House, BC. The tubes are large enough in diameter to be read easily and, in fact, have floats to facilitate reading. An additional tube inserted in the gravel support can be used to diagnose clogging problems, should they occur.

(10) Avoiding Negative Pressures. Negative pressures can cause the formation of gas bubbles, which may cause “air binding” and thereby disrupt the flow patterns of water movement through the sand bed. Design is the key to avoiding this problem.
Simply stated, if the weir crest for the tailwater elevation control is not permitted to be lower than the level of the sand bed during operation, negative pressure will not occur and, consequently, gas bubbles will be avoided. The only exception is when the influent raw water is supersaturated with an atmospheric gas.

Actually, the sand bed can be subject to a small amount of negative pressure without the occurrence of gas bubbles. But the safest rule is to design the filter so that the tailwater elevation is always at or above the level of the sand bed. An argument against such a design is that the filter box must be deeper than if this rule were not followed. But a more compelling argument is that the whole investment in building a slow sand filter system will be nullified if the sand bed is subject to frequent malfunctions and reduced effectiveness.

(11) **Gas Bubble Formation.** The source of gas bubbles in the filter is the dissolved gas in the influent water. Virtually all waters have dissolved gases. Most often, the gas forming the bubbles in the filter is oxygen. Oxygen is a product of the photosynthesis reaction. As oxygen is produced, the solution concentration of oxygen increases in the vicinity of the reaction. During daylight hours the oxygen production rate by photosynthesis may exceed the rate of oxygen consumption by microorganisms and
aquatic life and the oxygen mass transfer rate from the water to the atmosphere. When that occurs, the oxygen concentration increases accordingly. Such situations are common in ambient waters, such as streams, lakes, and ponds, and the oxygen concentration may exceed by a large margin the level that is in equilibrium with the atmosphere (in accordance with Henry’s law, which is discussed later in this section). Oxygen concentrations as high as 32 mg/L (milligrams per liter) have been measured (by the Winkler method) in slow-moving streams during the summer months in early afternoon, when photosynthesis is at its maximum (oxygen production by photosynthesis is proportional to the diurnal intensity of sunlight; see, for example, Kartchner et al. 1969). In turbulent mountain streams, oxygen concentrations of 15 mg/L have been measured by oxygen probe. Water with no biological activity (such as a laboratory beaker filled with distilled water) and with dissolved gas concentrations at equilibrium, in accordance with Henry’s law, between the water and the atmosphere, reaches oxygen saturation at 20°C at sea level with only 9.2 mg of oxygen per liter of water. When dissolved oxygen concentrations due to photosynthesis exceed the level as predicted by Henry’s law, gas bubbles may form. Virtually all natural waters have some level of biological activity; very seldom does water exchange gases only with the atmosphere.

The formation of gas bubbles, called "gas precipitation," is a phenomenon that occurs when the local pressure, \( P(\text{depth}=d) \), at a given depth, \( d \), in the water is equal to or less than the partial pressure, \( P(\text{gas } i) \), at which the particular gas concentration, \([i]\), is at saturation according to Henry’s law. That is, gas precipitation occurs when \( P(\text{depth}=d) \leq [i]/H_i \). Note that the concentration of a dissolved gas is indicated by brackets. Thus, for gas species \( i \), \([i]\) is the concentration of that gas. The Henry’s law coefficient for gas \( i \) is \( H_i \). In other words, gas precipitation is the spontaneous formation of gas bubbles by a given species \( i \) coming from the dissolved gas of species \( i \) in the water. Examples 3.7 and 3.8 explain further the conditions for gas precipitation.

**Example 3.7: Gas Precipitation.** Show how the occurrence of gas precipitation conditions may be determined.

1. **Conditions for Gas Precipitation:** Gas precipitation will occur when the solution concentration, \([i]\), of a given gas species, \( i \), in water exceeds the saturation concentration (with respect to the local pressure at the point of interest, i.e., \( H_iP(\text{local}) \)).

2. **Henry’s Law:** Henry’s law says that the concentration, \([i]\), of any gas, \( i \), in solution is proportional to its partial pressure \( P_{gas i} \) at the solution-gas interface. Henry’s law, in mathematical terms, is stated in Equation 3.8.

\[
[g\text{as of species } i] = H_{gas i} \cdot P_{gas i} \tag{3.8}
\]
in which \([\text{gas of species } i] = \text{concentration of gas species (square brackets symbolize concentration)} \ (\text{mg/L})\)

\(H_{\text{gas } i} = \text{Henry's law coefficient for gas of species } i \ (\text{mg species } i/\text{L/atmosphere} \ [\text{atm}] \text{ of species } i)\)

\(P_{\text{gas } i} = \text{partial pressure of gas species at gas-liquid interface (atm pressure of species } i)\)

\(= (\text{mole fraction of gas of species } i \text{ in the atmosphere}) \cdot (\text{total atmospheric pressure})\)

The gas may be any species, such as \(O_2, N_2, CO_2, NH_3\), and even \(H_2O\). Using oxygen as an example, Henry's law is stated: \([O_2] = H_{O_2} \cdot P_{O_2}\).

3. **Criterion for gas precipitation:** Gas precipitation will occur (Hendricks 1990) when:

\[\text{[gas of species } i\text{ measured}] > H_{\text{gas } i} \cdot P_{\text{local pressure in water}}\]  \[\text{(3.9)}\]

Thus, if the pressure at a given point \(A\) in the liquid is \(P_A\) and if the concentration of the gas in solution is greater than the product of \(H_{\text{gas } i} \cdot P_A\), then gas precipitation will occur spontaneously (and bubbles will be observed).

4. **Illustration:** Suppose that water carried into a filter bed has a dissolved oxygen concentration \([O_2] = 12 \text{ mg/L}\). The elevation is 1,372 m (4,500 ft), and therefore \(P(1,372 \text{ m}) = 630 \text{ mm Hg or } 0.83 \text{ atm}\), and temperature is \(25^\circ\text{C}\). The concentration of pure oxygen in water at sea level at \(25^\circ\text{C}\) is 40.0 mg/L, giving a Henry's law coefficient, \(H(O_2, 25^\circ)\), of 40.0 mg/L/atm. Will gas precipitation occur within the filter bed?

(a) Apply Henry's law to determine the pressure at which equilibrium will occur between the dissolved gas and the bubbles.

\[12 \text{ mg/L} = H(O_2, 25^\circ) \cdot P(\text{equilibrium})\]

\[12 \text{ mg/L} = 40.0 \text{ mg/L/atm} \cdot P(\text{equilibrium})\]

\[P(\text{equilibrium}) = 0.30 \text{ atm}\]

(b) If the absolute pressure at any point in the filter bed is less than 0.30 atm, gas precipitation will occur. The associated gauge pressure is:

\[P(\text{local gauge}) = P(\text{atmospheric}) - P(\text{local absolute})\]

\[P(\text{local gauge}) = 0.83 \text{ atm} - 0.30 \text{ atm}\]

\[= 0.53 \text{ atm (negative pressure)}\]

\[= 5.48 \text{ m (18.0 ft)}\]

(c) **Discussion:** From these calculations we see that gas precipitation will occur and bubbles will be observed if the hydraulic grade line (HGL) drops to 5.48 m (18.0 ft) below any given point in the sand bed. Figure 3.14 illustrates the foregoing discussion. The system in Figure 3.14a is a tall bell jar, 8.58 m high, filled with water having a dissolved oxygen concentration of 12.0 mg/L. The jar has been inverted and placed in a tray of water, which also has a dissolved oxygen concentration of 12.0 mg/L. Suppose that a membrane of plastic wrap seals the interface so that oxygen transfer cannot occur. With an increase in elevation, the pressure within the bell jar will decrease from the 0.83 atm measured at the level of the membrane. Thus, at 8.58-m (28.1-ft) elevation, the absolute
Figure 3.14 Illustration of Criterion for Gas Precipitation

pressure in the water is 0.0 atm. At the elevation at which the 12.0 mg/L oxygen concentration exceeds the product of

\[ H(O_2, 25^\circ C) \cdot P(\text{absolute pressure at given elevation within bell jar}) \]

gas precipitation will occur and gas bubbles will be noted. As noted above, the threshold absolute pressure for gas precipitation is 0.30 atm. At this pressure (and lower pressures), gas bubbles will be observed. In the system shown, this threshold pressure occurs at an elevation of 5.48 m above the surface.

Figure 3.14b shows graphically the Henry’s law criteria for gas precipitation. At the water surface elevation, the Henry’s law criterion for gas precipitation would require a dissolved oxygen concentration of 33.2 mg/L, that is, 40 mg dissolved oxygen/L/atm \cdot 0.83 atm total pressure. Because the relationship is linear with elevation, the threshold pressure-concentration relationship can be shown by a straight line. For all points under the curve, the oxygen will remain dissolved. For all points above the curve, gas precipitation will occur. If we enter the oxygen concentration, 12 mg/L, on the horizontal scale of Figure 3.14b, we see that the absolute pressure below which gas precipitation will occur is 0.30 atm. If we then project horizontally over to the bell jar, we see that the corresponding elevation above the water surface is 5.48 m. The practical consequence is that if the hydraulic grade line should drop to 5.48 m below any given point within the sand bed, gas bubbles will form, as noted above.

The distinction between the Henry’s law saturation with respect to the atmosphere and the “absolute” saturation as related to gas precipitation can be illustrated by removing the plastic membrane at the air-water interface. Immediately, a thin molecular film at the air-water interface will go to equilibrium with respect to the atmosphere and have a concentration of 7.9 mg/L dissolved oxygen (40 mg dissolved oxygen/L/atm \cdot 0.83 atm total pressure \cdot 0.209 mole fraction of oxygen in the atmosphere). A dissolved oxygen concentration gradient will be set up from the bulk.
5. _Another illustration:_ An everyday illustration of gas precipitation is seen in boiling water. For water, \([H_2O] = 1,000\ mg/L\). As temperature rises, the Henry’s law coefficient, which is the ratio of vapor pressure to the concentration of water (here 1,000 mg/L), also rises. At 100°C, the vapor pressure of water is 1.0 atm, and \(H[H_2O,100^\circC] = 1,000\ g/L/1.0\ atm\). The \(H\cdot P\) product is:

\[
H[H_2O,100^\circC] \cdot P(\text{local pressure} = 1\ \text{atm}) = 1,000\ g/L/\text{atm} \cdot 1.0\ \text{atm} \\
= 1,000\ g/L
\]

Thus, since \([H_2O]_{\text{actual}} = 1,000\ g/L\) at 100°C, the criterion for gas precipitation is satisfied and gas bubbles form. Although boiling water is explained by the fact that boiling occurs when the vapor pressure of the water increases to the local atmospheric pressure, the Henry’s law explanation shows the parallel with precipitation of any gas species.

**Example 3.8: Filter Pressure With Depth.** Show how pressures may be evaluated within the filter bed. Elevation is 1,372 m (4,500 ft), giving \(P(\text{local}) = 0.83\ atm\).

1. **Analysis of filter pressures:** Figure 3.15a shows a filter bed in vertical position, that is, as set up for operation. Figure 3.15b shows the filter bed inclined to facilitate hydraulic analysis. The line drawn from the headwater to the tailwater is the energy grade line/hydraulic grade line (EGL/HGL), which is the basis for the analysis. The EGL and HGL are approximately coincident because the velocity head \((v^2/2g)\) is negligible. The energy loss is through the filter bed, so the pressure at the top of the sand bed is that caused by the headwater. Starting at the headwater elevation at the top of the column, the energy loss is linear through the sand bed (assuming no surface deposit). The specific energy at the bottom of the sand bed is the elevation of the tailwater surface.

   In analysis, the sum of the elevation head, \(z\), and the pressure head, \(p/\gamma\), defines the HGL. By the same token, the distance from the HGL to the sand bed is the pressure head within the sand bed. When the HGL crosses the sand bed, as shown in Figure 3.15b, the pressure in the sand bed is zero. When the HGL drops below the sand bed, the pressure within the sand bed is negative. The maximum negative pressure head is at the exit from the sand bed. In the case shown, with the datum at 0.0 m, the tailwater elevation, point C, is at 0.20 m and the bottom of the sand bed is at 0.58 m. Thus, for point B, at the bottom of the sand bed, \(P_B/\gamma = -0.38\) m water. Also, at point C, the surface of the tailwater, \(P_C = 0.83\ atm\).

2. **Determine the pressure at the bottom of the filter:** Apply Bernoulli’s equation between points B and C, recalling that 1.0 atm = 10.33 m water:

\[
z_B + \frac{P_B}{\gamma} = z_C + \frac{P_C}{\gamma} \tag{3.10}
\]

Terms not defined previously are:

- \(z_B = \text{vertical distance from datum to point B in a closed hydraulic system (m or ft)}\)
- \(P_B = \text{pressure at point B in a closed hydraulic system (N/m}^2\text{ or lb/ft}^2\)
- \(\gamma = \text{specific weight of water (N/m}^3\text{ or lb/ft}^3\)

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Figure 3.15 Hydraulic Analysis of Filter Bed to Illustrate the Occurrence of Negative Pressures Caused by Low Tailwater Elevation. Variables are defined in the text.

Applying numerical data for absolute pressures gives:

\[ 0.58 \text{ m} + \frac{P_B}{\gamma} = 0.20 \text{ m} + 8.58 \text{ m} \]

\[ P_B = 8.20 \text{ m} \]

\[ P_B = 8.2 \text{ m} \cdot \frac{1.0 \text{ atm}}{10.33 \text{ m}} \]

\[ = 0.79 \text{ atm} \]

Note that \( P_B \) was determined as absolute pressure. Since \( P_C = P(\text{atmosphere, 1,372 m elev}) = 0.83 \text{ atm} \), a negative pressure of 0.04 atm exists at point B (38 cm of water), which is another way of expressing \( P_B \). Level C is the elevation of the tailwater.

3. Analysis of gas precipitation: The negative pressure in the filter bed reduces the magnitude of the product of \( H \cdot P(\text{absolute local pressure}) \). Thus, at the bottom of the filter bed, point B, the product of \( H(O_2, 25^\circ) \cdot P_B(0.79 \text{ atm}) = 40.0 \text{ mg/L/ atm} \cdot 0.79 \text{ atm} = 31.6 \text{ mg/L} \). From these calculations we see that a higher concentration of dissolved gas in the water than 31.6 mg/L will result in gas precipitation.
4. **Discussion**: By raising the tailwater elevation to point B by means of a weir, the HCL will be raised correspondingly and will be higher than the sand bed at all points. Thus negative pressures within the sand bed will not exist. For the case illustrated, the negative pressure level at point C is not likely to result in gas precipitation since dissolved oxygen concentrations in the raw water greater than 31.6 mg/L are not likely.

(12) **Plumbing Functions**. In addition to the above hydraulic functions related to making slow sand filtration work as a process, the plumbing system should provide mechanisms for a number of ancillary needs, such as the filter-to-waste operation, valves to direct flows, and drains of various sorts. Figure 3.2 shows a portion of the plumbing configuration needed for a slow sand filter, but as mentioned, the drawing is simplified. Filter-to-waste is sometimes recommended for use after sand bed scraping, but if the sand bed is biologically mature and if the disturbance during cleaning is minimal, the process efficiency will not be impaired significantly as a result of scraping. Nevertheless, it is prudent to provide for filter-to-waste. Even if it is not needed during operation, filter-to-waste is imperative at the start-up of a plant as well as when the sand bed is replaced. During start-up, the sand bed will shed its fines, causing short-term higher turbidity levels and unacceptable water quality. During this period, provision must be made for waste. This can be as simple as having valves to direct the effluent to a waste line (with an air gap to prevent cross-connection) rather than to the tailwater basin. Figure 3.8 shows the piping used to drain the filter. The same piping may be used for filter-to-waste.

Another type of valve arrangement should be utilized for hydraulically connecting multiple filter beds for the purpose of backfilling with filtered water after scraping. For adjacent filter beds, a continuous header pipe with a valve at the location separating the two beds is sufficient.

Without exception, all basins and floors in the filter should be provided with drains to allow for the disposal of drainage water. Compliance with National Pollution Discharge Elimination System (NPDES) permits may be a factor and should be ascertained. The main point, however, is that all basins will have to be drained at some time and drains must be incorporated into the design. Also, drains are needed to remove the wash water when floors are hosed. Gutters should be provided to collect the wash water and to permit the movement of wash waters toward the drains.

(13) **Pipe Gallery**. A pipe gallery is a necessary adjunct to a slow sand filter. Figure 3.16 shows the pipe gallery for the slow sand filter at 100 Mile House, BC. All pipelines are simple and uncluttered, with valves that are easy to operate and maintain. For small plants, the pipe gallery can be merely an additional "box" added to the end of the filter where raw water enters and finished water leaves. If the pipe gallery is the full depth of
the filter box, the piezometers and flow meters may be located in the pipe gallery. The operator should have easy access to all valves, meters, and piezometers within the pipe gallery and should not have to stoop or bend in difficult positions. The pipe systems include those for raw water (influent), finished water (effluent), headwater drainage, backfilling of finished water from an operating filter to a drained filter, and filter-to-waste. The pipes, valves, and meters should be color-coded and labeled so that the function of each is self-evident.

(14) Access to Filters. Operators must have easy access to the filter beds for inspection during operation, removing sand after scraping, and resanding the bed. The operators should be able to complete the tasks of scraping and resanding in a normal working fashion. Sand removal and placement should not require abnormal body positions that could result in injury. At 100 Mile House, BC, access to the filters is by means of ship doors, which are shown in the photograph in Figure 3.17.

(15) Hydraulic Profile. The starting point for any hydraulic analysis is the hydraulic profile. The hydraulic profile of the Moricetown filter is shown in Figure 5.2. Such a profile should be drawn based on headloss calculations in conjunction with trial-and-error selection of pipe sizes. From HGL determination, the filter box elevation can be set.
The latter is subject to the constraints of source water elevation and filtered water storage elevations.

(16) Headroom. Adequate headroom should be provided so that persons scraping the sand bed can assume normal posture. The distance between the top of the sand bed and the roof should be greater than 2.0 m. The amount of space that needs to be provided for headloss is about the same as the headroom required.

(17) Designing to Avoid Freezing. In northern latitudes, freezing temperatures must be considered. Two approaches are (1) to accept the presence of an ice block in the filter in winter and to design the walls to handle the thrust of the expansion, and (2) to prevent the occurrence of an ice block. The former is not good policy, however, as the nuisance and problems of ice are worth avoiding. Any layer of ice must be removed before sand can be scraped, so even a thin crust of ice will cause an inordinate increase in the labor requirement.

The Kassler slow sand filter, an outdoor filter operated from 1906 to 1985 by the city of Denver, was run in the winter months even though it routinely had a floating ice block, usually 0.3 to 0.6 m (1 to 2 ft) in thickness, on the water surface. The filter area, 2.47 ha (10.5 acres), was too extensive to cover with a roof. The important point was that the
ice block did not touch the surface of the sand. The horizontal force caused by the expansion of the ice block was taken by the sloped sidewalls of the filter. The sidewalls were 15-cm (6-in.) concrete slabs covering earth berms sloped 1:2, and had no structural difficulty with the ice. Scraping was timed for just before the winter season, so that the filter run could extend to spring when the ice block was expected to melt.

To prevent the occurrence of an ice block, a filter must be covered. The slow sand filters at Empire, CO, 100 Mile House, BC, and Moricetown, BC, are all covered. Figure 3.18 is a photograph of the slow sand filter at 100 Mile House, BC. The filter has a flatroof of precast concrete and earth sidewalls. With the insulation provided by the earth sidewalls, no auxiliary heat is needed, and the filter has had only a thin skin of ice on the water surface at any time during operation since November 1985. A small pump that maintains circulation at the headwater surface has also helped prevent ice buildup.

Figure 3.19 shows the earth insulation at the Moricetown, BC, filter. The earth is placed along the sidewalls and in a layer about 0.3 m (1 ft) deep on the slab roof. The filter box has a port to the operations room so that heat from the operations room can circulate above the headwater.

The slow sand filter at Empire, CO, is shown in Figure 3.20. The filter has a steep trussed roof, and two propane space heaters operate inside the filter building. The
propane is stored in a 3,800-L (1,000-gal) tank outside the filter and is sufficient to run the heaters during the winter season. A portion of the surface of the headwater is usually covered with a thin crust of ice. The Empire filter site is located at an elevation of 2,926 m (9,600 ft), and daytime winter temperatures may be \(-10^\circ\text{C}\), with much lower temperatures at night. Influent water temperatures are often \(0^\circ\text{C}\) in the winter; therefore, flows must not be stopped in any pipe or freezing will be likely.

The experiences of these filter plants show that covering the filter boxes will alleviate the problems of freezing at nominal cost. The roofs are relatively inexpensive for the size of the filters needed for small communities. The roofs also alleviate the problems of algae growth and protect the filter bed from debris and vandalism. Depending upon the situation, roofs may be merited based upon these factors alone.

### 3.1.3 Sand Recovery System

Sand recovery systems are relatively simple to construct and operate but require capital expenditure. If a storage area for dirty sand is provided, the initial capital expenditure for sand recovery may be deferred until the first resanding is required. Without sand recovery, new sand must be purchased for resanding.
The arguments for on-site sand recovery are that sand will be on hand for resanding; that the cost of resanding will be minimal; that the sand will have been washed and will be ready for use; that sand will not be indiscriminately discarded; and that the uncertainty that exists in any new sand acquisition concerning whether proper attention will be given to specifying the correct size of sand, and whether it can be obtained at a reasonable price, will be avoided. Factors in favor of not recovering the sand are that a capital cost can be deferred to the future, when new sand would have to be purchased and that the operation cost can be reduced by not having to handle sand. For the long term, however, having a sand recovery system is cost-effective and its provision will facilitate a sustainable operation and will provide an important measure of quality control. Facilities for sand storage and washing contribute to the "passive" theme of slow sand filtration and should be an integral part of any design.

Figure 3.21 illustrates schematically a sand handling system. First, the dirty sand must have a place to be stored until it is washed. The volume of the storage bin can be equal to the amount of space required to hold the sand from a single scraping, from a year of scrapings, or from the entire bed volume.
A simple flume is used to wash the sand. The water flow suspends the sand and debris. The sand settles into a box, and the debris, which remains in suspension, is removed with the flow. An NPDES permit may be required if the flow is to be discharged to a stream. Also, a settling basin may be needed to collect the solids that will settle out of the water before the flow is discharged. Since the solid material derives from erosion processes, a suitable land discharge site may alleviate the need for an NPDES permit.

After the washed sand settles, it is moved to another bin for storage. That bin should have drainage and a cover. Ideally, the bin should be accessible to a front-end loader, so that the sand can be removed easily during the resanding operation. The combined volumes of the storage bins for dirty sand and for washed sand should equal the total sand volume removed. The exact mix of clean and dirty sand stored will be determined by local factors and preferences. A suggested mix is to provide storage for one year of dirty sand, with the balance of storage for clean sand. This mix of storage implies annual washing, which should be convenient for most situations.

Example 3.9: Sand Recovery System for the Empire, CO, Filter. Size the storage bins for dirty and clean sand at Empire.

1. **Given data for Empire:** The sand bed at Empire is scraped about once each month. Approximately 0.5 cm of sand is removed during each scraping (see Example 3.2).

2. **Sand removal:** With two sand beds, each 9.60 m x 17.60 m, and with 0.5 cm removed at each scraping and 1 scraping/month, the volume of sand taken up with each monthly scraping is 0.85 m$^3$ (1.1 yd$^3$ [cubic yard]), requiring a total storage volume of 308 m$^3$ (402 yd$^3$) when 0.91 m
the 1.2-m sand bed is removed from the two filter boxes, i.e., 308 m$^3 = 9.60$ m x 9.60 m x 0.91 m x 2 boxes).

3. **Storage bins**: Assume that the bin for the storage of dirty sand has a volume to accommodate one year of scrapings, which is 10.2 m$^3$, or 13.3 yd$^3$ (10.2 m$^3$/yr = 9.60 m x 17.60 m x 0.005 m removed/scraping x 12 scrapings/yr). Thus, the volume needed for clean sand is 308 m$^3$ - 10.2 m$^3$ = 298 m$^3$. The clean sand storage bin should be as deep as possible because the site is excavated rock and is limited in area. An area about 10 m x 10 m could be developed, which would give a depth of 3.0 m (9.8 ft).

4. **Structure**: The storage bin should have a floor of reinforced concrete that is sloped to permit drainage. The drainage should be collected and removed from the site. The side pressure will not be great for a bin that is 3.0 m deep. A rough estimate is 1,700 kg/m$^2$ (360 lb/ft$^2$). Therefore a concrete block structure, or even a suitably designed wood building, may be considered. A roof should be added to protect the sand, and one end of the building should be open for equipment access. This should not be construed as a design recommendation but as an indication that the recovery system can be housed in an inexpensive structure.

5. **Comments**: At Empire, rock had to be excavated to provide a site for the slow sand filter and the budget was such that only the basic filter could be built. For economic reasons, a sand recovery system was not in the picture. In the future, Empire will have to face the problem of resanding and a retrofit to develop a sand recovery system. With only a little guidance from an engineer, the town could build such a system using its own staff.

### 3.1.4 Filter Box

Design issues for the filter box include the area, number of cells, layout, depth, structure, and water tightness. Table 3.2 provides an overview by summarizing data from three recently constructed slow sand filters. Given are the design population, total bed area, number of cells, bed area per cell, time out of operation for scraping, time required for scraping, and number of persons involved in scraping.

**Hydraulic Loading Rate and Area**. In principle, the hydraulic loading rate selected determines the filter area needed, per Section 3.1.1 and Equation 3.1. Example 3.1 reviewed the Empire case, exemplifying some of the exigencies of a real situation. The conditions incorporated in the HLR decision include peak day flow, peak hour, filtered water storage, and number of cells. The ideal protocol is (1) to select HLR(peak day); (2) to determine Q(peak day); (3) to determine filtered water storage for peak day, taking into account hourly variations and peak hour; (4) to determine the corresponding constant flow for peak day; and (5) to calculate the area required using Q(peak day, constant flow) and HLR(peak day).

The filter is a component of a system, and so the HLR and filter area should be considered in the systems context. For example, if the filtered water storage is adequate to make up for a filter being out of production for scraping, then the HLR to the filter cells still in operation can remain steady. The HLR should be permitted to be at the maximum during the peak day. If the peak day occurs only once per year, then
Table 3.2
Data on Slow Sand Filters

<table>
<thead>
<tr>
<th>Place/design population</th>
<th>Total area (m²)</th>
<th>Number of cells</th>
<th>Bed area per cell (m²)</th>
<th>Time required for scraping (hours)</th>
<th>Number of persons involved in scraping</th>
</tr>
</thead>
<tbody>
<tr>
<td>Empire/1,000 persons</td>
<td>153</td>
<td>2</td>
<td>76.6</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>100 Mile House/2,300 persons</td>
<td>774</td>
<td>3</td>
<td>258</td>
<td>4</td>
<td>3</td>
</tr>
<tr>
<td>Moricetown/900 persons</td>
<td>180</td>
<td>2</td>
<td>90</td>
<td>2</td>
<td>2</td>
</tr>
</tbody>
</table>

a The downtime for a filter includes the time to drain the headwater, dewater the sand bed to about 3 cm (1 in.) below the sand bed surface, scrape and remove the sand, backfill from the bottom, and refill the headwater. The downtime for the three installations in this table is about 24 hours each.
b The population of Empire, CO, was 450 in 1984. The plant was designed to filter 0.946 mil L/d (250,000 gpd) for a projected population of 1,000 persons, with an HLR of 0.26 m/hr (6.5 mgad) for a bed area of 153 m² (1,650 ft²). The flow demand in 1985 ranged from 0.189 to 2.000 mil L/d (50,000 to 530,000 gpd).
c The population of 100 Mile House, BC, was 1,925 in 1987. The plant was designed for a peak flow of 7.26 mil L/d (1.92 mgd) to serve a population of 2,300 (Bryck et al. 1987).
d The Moricetown, BC, plant was designed for a peak day flow of 0.922 mil L/d (0.24 mgd) to serve a projected 1996 population of 900. The plant can be expanded to serve 1,240 persons, the projected population for 2006 (Dayton & Knight, Ltd. 1989).

consideration should be given to allow the HLR to exceed the maximum. If the filter bed is biologically mature, it is not as sensitive to a nominal HLR increase. In other words, judgment may be required if the ideal protocol results in untenable positions (as noted for Empire in Example 3.1).

Because the filter area is determined by the HLR selection, it too clearly has a systems context. The selected area and associated HLR should be examined over the cycles of operation, taking into account the whole system. The critical periods are as determined locally but would include the time during which a filter bed is out of service for scraping, the peak day of per capita demand and its relationship to treated water storage, and the maximum demand, which would occur when the upper limit of population is reached.

Thus, the HLR is a "design link" to the rest of the system and to operation. The number of cells in the filter is also a factor in the design. The larger the number of cells, the less proportionate is the increase in HLR when a single cell is removed from service for scraping or resanding.

(2) Number of Cells. Every slow sand filter should have two or more cells so that when one is out of service for scraping or other reasons, another filter bed can continue producing sufficient amounts of water for the community. Whether more than two filter beds are used depends upon the time required for scraping and the capital costs. As
discussed above, the number of cells directly affects how the system operates and performs.

**Example 3.10: Number of Cells in a Filter Bed.** Using the data from the Empire, CO, filter, calculate an upper limit area for a single cell for a slow sand filter.

1. **Assumptions:** Assume that the filter bed can be out of operation for 16 hours and that three persons are available for scraping. Also assume that 4 hours are needed to drain the headwater (the time that should be required at Empire if headwater drainage plumbing was provided), and 4 hours are needed to put the filter back into operation, leaving 8 hours available for scraping. Also assume the scraping rate is 19 m²/person/hr (the rate at Empire).

2. **Calculate the maximum area of a cell:**

   \[ A_{cell} = \text{scraping rate in m}^2/\text{person/hr} \times \text{no. of persons} \times \text{hours allotted to scraping} \]

   \[ = 19 \text{ m}^2/\text{person/hr} \times 3 \text{ persons} \times 8 \text{ hours} \]

   \[ = 456 \text{ m}^2 \]

   in which \( A_{cell} \) = maximum area of a single cell based upon allotted downtime of a single cell, crew size, and rate of scraping per crew member (m²)

3. **Calculate the number of cells, using Empire data:**

   \[ \text{No. of cells} = \frac{A(\text{total filter area})}{A_{cell}} \]

   \[ = \frac{(169 \text{ m}^2/\text{cell} \times 2 \text{ cells})}{(456 \text{ m}^2/\text{cell})} \]

   \[ \leq 1 \text{ cell} \]

   From this calculation we see that Empire does not have enough total area to warrant concern about the maximum cell area based upon the criteria of labor available and time for scraping.

4. **Discussion:** The Empire filter has a total bed area of 338 m², comprised of two 169-m²/cells. The downtime for scraping should be only about 10 hours (but about 24 hours is actually required because the plumbing does not permit rapid headwater draw down). The allotted downtime, the crew size, and the scraping rate per crew member permit a cell area of 456 m². With one cell of the Empire filter being only 169 m², its area is well below the upper limit of 456 m² based upon downtime permitted and labor available. By comparison, the 100 Mile House plant has a total bed area of 774 m², with three cells, giving 258 m²/cell. The total downtime to scrape one of the three cells at 100 Mile House is about 24 hours.

   The calculation shows the factors relevant in determining the upper limit in area for a single cell. The magnitudes of the factors will depend upon local circumstances.

**(3) Layout.** An important part of the design is the layout of the filter area. Filter layout determines the plumbing configuration, the economy of the filter box construction, and whether future plant expansion will be feasible. Figure 3.22 is a schematic of the filter box and plumbing layout for the slow sand filter plant at 100 Mile House, BC. The plumbing layout is located in a pipe gallery along the front ends of the filter boxes, making access easy for both operation and maintenance. Further, avoidance
of a design involving a complex maze of pipes minimizes construction mistakes and allows for easier plant operation. At 100 Mile House, the pipe gallery is a part of the operations building, which has a laboratory, an office, a chlorine room, a chlorine contact tank, and a clear well for the treated water pumps.

The filter boxes at 100 Mile House have common walls, each having the capacity to withstand the hydraulic pressure when the adjacent filter box is drained. If additional boxes are added, they could be built alongside the existing boxes and the pipe gallery could be extended or they could be built on the other side of the pipe gallery, which would then serve both sets of filters. Although the configuration should be simple and allow easy maintenance, it is important to consider possible future expansion so that any new additions can be integrated easily into the operation of the plant.

To facilitate expansion, the design should be set up for a modular increase in the number of cells. This involves sizing the header pipes for a future peak day flow and designing the walls and building so that expansion is facilitated. The investment is nominal and will be repayed many fold when the time comes to consider new capacity.
(4) Depth of Box. The depth of the filter box can be stated as follows:

Depth of filter box = depth of gravel support (0.5–0.8 m)  
+ depth of filter media (1.0–1.5 m)  
+ maximum depth of water (2.0–3.0 m)  
+ freeboard depth (10–30 cm)

An important concern is that a person about 183 cm (6.0 ft) tall can stand comfortably when scraping the sand bed and not touch the underside of roof, or the roof truss, when the filter bed is at its maximum height.

The upper limits of filter bed depth and the headwater depth indicated above are slightly beyond the range of usual practice. As noted, the depths of the headwater and of the initial sand bed are limited by economic considerations rather than absolute criteria. Because the underdrain pipes are placed on the filter floor and the gravel is placed around the pipes, the pipe sizes are not measured in computing the depth of the filter box.

Figure 3.23 is a cross-section drawing of a slow sand filter, showing the underdrain, gravel support, sand, headwater, and freeboard, with typical ranges of depths indicated. At 100 Mile House, BC, the total distance from roof slab to concrete floor is 3.80 m (12.5 ft), with gravel taking up 0.83 m (2.7 ft); sand, 1.05 m (3.4 ft); maximum headwater depth, 1.80 m (5.9 ft); and freeboard, 0.12 m (0.40 ft). At Empire, CO, the depth of the filter box is 3.66 m (12.0 ft), with gravel taking up 0.61 m (2.0 ft); sand, 1.22 m (4.0 ft); maximum headwater depth, 1.52 m (5.0 ft); and freeboard, 0.31 m (1.0 ft). The depths of the headwater and the sand bed at 100 Mile House and Empire are representative of practice. To ensure that a 183-cm (6-ft) person will be able to stand comfortably on the sand bed, the following equation should be used when deciding the roof height:

\[
\text{distance above sand bed} = \text{headwater depth} + \text{freeboard} \geq 183 \text{ cm (6 ft)}
\]

(5) Structural Design. The structural design of the filter box depends upon the hydraulic pressure exerted on the inside of the filter box and the soil pressure exerted on the outside. The hydraulic pressure is calculated by applying the concept of the pressure prism. According to that concept, the pressure at any depth is:

\[
p = \gamma \cdot h
\]  

in which \(p\) = water pressure at depth \(h\) (pascals or lb/ft\(^2\))  
\(h\) = depth below water surface (m or ft)  
\(\gamma\) = specific weight of water (9,990 N/m\(^3\) or 62.4 lb/ft\(^3\))
The force on the wall is:

\[ F = (\gamma \cdot h/2) \cdot A(\text{wall}) \]  

(3.12)

in which \( F \) = force on wall (N or lb)

\( A(\text{wall}) \) = area of wall (m\textsuperscript{2} or ft\textsuperscript{2})

Example 3.11 illustrates the application of these equations.

**Example 3.11: Pressure and Force.** Calculate the pressure at the bottom of the filter at Empire and the force per unit width of wall.

1. **Data for Empire filter:** The maximum depth of water for the Empire slow sand filter is 3.65 m (12.0 ft).

2. **Calculate the pressure at a depth of 3.65 m (12.0 ft):**

\[ p(3.65 \text{ m}) = \gamma \cdot h(3.65 \text{ m}) \]

\[ = 9,990 \text{ N/m}^2 \cdot 3.65 \text{ m} \]

\[ = 36,463 \text{ N/m}^2 \]

\[ = 36.46 \text{ kPa (kilopascals)} \]

\[ = 750 \text{ lb/ft}^2 \]
3. Calculate the force exerted on a 1-m vertical section of wall:

\[ F = (\gamma \cdot h/2) \cdot A \]  

\[ = (9,990 \text{ N/m}^3 \cdot 3.65 \text{ m}/2) \cdot (3.65 \text{ m} \cdot 1.0 \text{ m}) \]

\[ = 66,546 \text{ N (for a 1-m vertical section)} \]

\[ = 4,492 \text{ lb (for a 1-ft vertical section)} \]

4. Discussion: The wall should be designed to withstand the force indicated above. In addition, the bending moment at the base of the wall should be calculated. Thus, the design of the filter boxes should be developed by an engineer knowledgeable in the field of structural design. The example is intended to give an idea of the horizontal force on a typical wall of a slow sand filter. Soil pressure should be determined for the local conditions by a person knowledgeable in the field of geotechnical engineering. The bottom floor and the roof must also be structurally designed.

3.1.5 Disinfection

Filtration and disinfection go together in accordance with the "double barrier" philosophy, which is the concept that two different removal mechanisms should be employed in any water treatment scheme to provide municipal drinking water. Usually chlorine will be used for disinfection, either in liquid form or as calcium hypochlorite. As a rule, chlorine gas is used only by communities with populations ≥200–500 persons. Figure 3.24 shows the chlorination building at Empire, CO. In Empire, chlorine gas is metered into the flow after slow sand filtration and prior to treated water storage.

Safety must be a key consideration for plants using chlorine disinfection. Facilities are required to meet state chlorine regulations and must have qualified operators and accountable management. The chlorine concentration used for disinfection will depend upon the contact time and the particular organisms of interest. Organisms most resistant to chlorination are the cysts of *Giardia lamblia* and *Cryptosporidium parvum*. The *Giardia lamblia* cysts have been found at most places where sampling has been done throughout the United States and Canada (Hibler, pers. com., August 23, 1990). *Cryptosporidium parvum* oocysts have been found extensively, but their occurrence has not been determined to the same extent as that of the *Giardia lamblia* cyst. The *Giardia lamblia* cyst is inactivated only at high C•T values, where C is the concentration of chlorine in mg/L and T is the contact time in minutes (min). The Surface Water Treatment Rule (SWTR, *Federal Register*, 1989, or 40 CFR, Parts 141 and 142) requires that T be for peak hourly flow and that C be for the disinfectant residual at the end of the contact time. (Note that "40 CFR" refers to Title 40, Code of Federal Regulations). Table 3.3 gives the C•T values specified by the *Federal Register* for achieving 1-log and 3-log inactivation of *Giardia lamblia* cysts with four disinfectants. The data in Table 3.3 show
clearly that inactivation of *Giardia* cysts requires very high C*T* values as compared to those for inactivating most bacteria (0.1–1) and viruses (1–10). The C*T* data for free chlorine in Table 3.3 were based in part upon data generated by Hibler et al. (1987); Appendix Figure D.1 is a plot showing the Hibler data.

Even more exceptional than the *Giardia lamblia* disinfection requirements are the C*T* values required to inactivate *Cryptosporidium parvum* oocysts, which are nominally 7,000 (Logsdon 1990). More comprehensive data on *Cryptosporidium* C*T* values are not available.

The extremely high C*T* values for *Cryptosporidium parvum* oocysts indicate that chlorine and less effective disinfectants will not work for removing this organism. Most systems will not be able to rid their water of *Cryptosporidium* by disinfection. The disinfection of *Giardia* cysts, however, is possible, but it requires the careful consideration of facilities to achieve the correct contact time and dosage.

At 100 Mile House, the chlorine contact time in the clear well at peak day flow is 20 min. With a chlorine dose of about 3.0 mg/L at the plant, resulting in about 0.5–1.0 mg/L in the distribution system, depending upon local circumstances, C*T* = 1.0*20 = 20. The peak day flow is about four times higher than the average daily flow at the 1986 population. For 1986 conditions, the C*T* = 1.0*80 = 80, which is sufficient for about 1.5-log inactivation of *Giardia* cysts at a temperature of 0.5°C. The residence time in the
transmission line and in the service reservoirs will add detention time, which should be calculated under peak flow conditions. Despite these nominal removals of Giardia and other organisms by disinfection, the main burden for protection of the community is on the filters. The filters provide the only effective protection from Cryptosporidium, for which chlorine C*T values may be as high as 7,000 for 2-log removal.

3.1.6 Filtration Removals

The slow sand filtration removal of Giardia cysts can be expected to be about 3-log for a biologically mature filter bed (Bellamy et al. 1985a, 1985b), rendering the Giardia risk very low to the population. The SWTR requires the combined filtration and disinfection to result in 3-log removal or inactivation of Giardia lamblia and 4-log overall removal or inactivation of viruses. The maturity of the filter bed has much to do with removals expected by filtration. A mature filter can consistently achieve 2-log to 4-log removals of Giardia cysts, but less removal may occur when the filter is not mature. The disinfection procedure should be designed to achieve ≥1-log inactivation of Giardia at all times. For Cryptosporidium parvum oocysts, Schuler et al. (forthcoming) reported filtration removals of ≥4-log at an HLR of 0.40 m/hr (10 mgad). Because Cryptosporidium cysts are so resistant to disinfection, the filters will have the entire burden of community protection.

The 1989 SWTR requires that the effluent water from slow sand filters must have turbidity levels under 5 NTU at all times and that turbidity levels must be <1 NTU in 95 percent of the samples taken. The 1 NTU may be increased by the state to 5 NTU if that level of turbidity will not interfere with disinfection. Turbidity must be sampled every
four hours or it must be continuously monitored, although for slow sand filtration the state may reduce the monitoring requirements to once per day.

3.2 DESIGN GUIDELINES

Design criteria for slow sand filters were first summarized by Hazen (1913). Indeed, his well-documented and comprehensive criteria on hydraulic loading rate, sand size, and gravel support, as well as his general guidelines, still are useful. The guidelines of Huisman and Wood (1974) were the next available and have been used widely by persons involved in slow sand filter design. The guidelines given here build upon these works but give precedence to the knowledge developed by the research, studies, and experience of the 1980s.

3.2.1 Hydraulic Loading Rates

Huisman and Wood (1974, pp. 43,44) noted that three Amsterdam filters operated at HLRs of 0.1, 0.25, and 0.45 m/hr for a year had no discernible differences in effluent water quality. They gave no criterion, as such, for HLR, but they discounted the role of HLR as an important factor. Shieh (pers. Com., September 19, 1990) reported that the slow sand filters at Salem, OR, were designed for an HLR of 0.40 m/hr (10 mgad) and were operated at about 0.35 m/hr (9 mgad) in 1990.

The work of Bellamy et al. (1985a, 1985b), who ran three slow sand pilot filters, each 30 cm (11.5 in.) in diameter, for one year at HLRs of 0.04, 0.1, and 0.4 m/hr (1.0, 3.0, and 10.0 mgad), respectively, showed that HLR did have an influence on water quality. Table 3.4 shows the average percent removals calculated by Bellamy et al. for the data they obtained in their one-year study of biologically mature slow sand filters. The percent removals of *Giardia* cysts, total coliform bacteria, and turbidity all showed linear declines, generally, in percent removals with increasing HLR. The right-hand columns show the percent removal data processed in terms of calculated effluent concentrations for hypothetical influent concentrations indicated. These data show that the concentrations of *Giardia* cysts in the effluent water are very low for all three HLRs shown. For example, at HLR = 0.04 m/hr (1.0 mgad), removals of *Giardia* cysts averaged 99.992 percent (4-log), and at 0.40 m/hr (10.0 mgad), cyst removals averaged 99.982 percent (nearly 4-log). The corresponding effluent cyst concentrations were calculated to be 0.08 and 0.18 cysts/L, respectively. Removals of total coliform bacteria were 99.5 percent at 0.04 m/hr (1.0 mgad) and 95.6 percent at 0.4 m/hr (10.0 mgad); effluent concentrations were 5 total coliforms/100 mL and 44/100 mL, respectively, for the hypothetical influent concentration of 1,000 total coliforms/100 mL.
Several conclusions are evident from the results reported in Table 3.4. First, even though increasing HLR caused linear reductions in percent removals, the changes were continuous, which means that exceeding some arbitrary criterion or standard will not cause a sudden problem. The effect of increasing the hydraulic loading rate will be a continuous increase in effluent concentration, not a discontinuous, sudden effect. Second, the changes in effluent levels are not large within the HLR range of 0.04-0.40 m/hr (1.0–10.0 mgad). An increase in HLR from 0.04 to 0.4 m/hr (1.0 to 10.0 mgad) will cause the effluent cyst concentration to increase only from 0.08 to 0.18 cysts/L (for the hypothetical influent concentration of 1,000 cysts/L). Similar calculated results are seen for total coliform bacteria and turbidity. Such increases are small relative to the hypothetical influent concentrations.

Bellamy et al. (1985a, 1985b) chose the HLR range of 0.04–0.4 m/hr (1.0–10.0 mgad) for their pilot plant experiments based upon a survey they took of HLRs used in operating plants and on the discussion in Huisman and Wood (1974). As Huisman and Wood note, the length of run is reduced by higher HLR levels, and economic factors may weigh against higher HLRs even more strongly than the possible reduction in efficiency at the higher rates. Experimental work has not been reported on filter operation at HLR levels higher than 0.45 m/hr. The governing concern in exceeding the HLR of 0.45 m/hr would most likely be the reduction in run time rather than an unacceptable loss in filtration efficiency. The above discussion is qualified by the premise that the filter bed is biologically mature.

In conclusion, any HLR within the traditional range, that is, 0.04–0.4 m/hr (1.0–10.0 mgad), should be acceptable from the standpoint of filtration efficiency. Further, HLR is
not steady; it will change over the annual cycle and will show an increasing trend as population increases.

3.2.2 Underdrain Size and Spacing

The underdrain system should ensure uniform flow through the overlying sand bed. Uniform flow is achieved, first, by having a uniform distribution and sufficient number of collection orifices and, second, by designing the orifice area/conduit area ratio such that headloss within the underdrain conduit (the drainpipe) is negligible relative to the orifice. To illustrate, a hypothetical hydraulic grade line (HGL) through a filter system is shown in Figure 3.25a. The elements of the system causing hydraulic resistance include the schmutzdecke, the sand bed, the gravel support, the orifices, and the drainpipes. So that the headlosses can be associated with the resistance elements, the same system is shown in Figure 3.25b with the schmutzdecke-sand bed-gravel support rotated 90° counterclockwise to give a horizontal orientation. The headlosses are identified as \( h_{L}(\text{schmutzdecke}) \), \( h_{L}(\text{sand bed, gravel support}) \), \( h_{L}(\text{orifice}) \), and \( h_{L}(\text{drainpipe}) \), respectively, for each of the elements (the sand bed and gravel support headlosses are combined).

An important condition in the underdrain design is that the hydraulic head must at all points within the sand bed and the gravel support define horizontal planes of constant pressure. This means that the hydraulic head on the external side of the underdrain pipes must be the same along the length of each pipe, as illustrated in Figure 3.25c. The key point is that the pressure head inside the drainpipe decreases along its length due to friction, as Figure 3.25c illustrates (the vertical scale is larger in (c) than in (a) and (b) to emphasize this friction headloss inside the pipe). Therefore, the headloss across the orifices at the head of the pipe is less than for those at the end of the pipe by the amount of friction headloss due to flow within the pipe. That is,

\[
h_{L}(\text{pipe}) = f \frac{L \left[ v(\text{pipe}) \right]^2}{D \, 2g}
\]

(3.5)

in which \( h_{L}(\text{pipe}) \) = headloss in drainpipe in length, \( L \) (m or ft)
\( f = \) friction factor for pipe material (dimensionless)
\( v(\text{pipe}) = \) velocity of water flow within pipe (m/s or ft/s)
\( D = \) diameter of pipe (m or ft)
Figure 3.25  Headloss Across Underdrain Orifice Compared With Headloss in Underdrain Conduit. The entire hydraulic system is shown for reference.
Example 3.12: Analysis of Effect of Underdrain Pipe Size on Orifice Discharges Along Length of Pipe. Consider an underdrain pipe at the bottom of a gravel support. Let Point A be at the head of the pipe and Point B at the end of the pipe. Determine algebraically the difference in flows between the orifices at A and B.

1. Analysis: Figures 3-25a and 3-25b show a hydraulic analysis for the entire filter-underdrain system. Figure 3.25c shows the hydraulic analysis for the underdrain only. For the condition shown in Figure 3.25c, the flow through each orifice, q(orifice), can be stated by applying Equation 3.4:

\[
q(\text{orifice}) = C_d A(\text{orifice}) (2gh_L(\text{orifice}))^{1/2}
\]

The associated headlosses are:

\[
h_L(\text{orifice at A}) = p(\text{outside pipe}) - p(\text{inside pipe at A}) \quad (a)
\]

\[
h_L(\text{orifice at B}) = p(\text{outside pipe}) - p(\text{inside pipe at B}) \quad (b)
\]

\[
h_L(\text{drainpipe}) = p(\text{inside pipe at A}) - p(\text{inside pipe at B}) \quad (c)
\]

Substituting (c) in (b) gives:

\[
h_L(\text{orifice at B}) = p(\text{outside pipe}) - [p(\text{inside pipe at A}) - h_L(\text{drainpipe})] = h_L(\text{orifice at A}) + h_L(\text{drainpipe}) \quad (d)
\]

Now compare q(A) with q(B):

\[
q(A) = C_d A(\text{orifice}) (2gh_L(\text{orifice A}))^{1/2} \quad (f)
\]

\[
q(B) = C_d A(\text{orifice}) (2gh_L(\text{orifice B}))^{1/2} \quad (g)
\]

\[
= C_d A(\text{orifice}) (2g[h_L(\text{orifice at A}) + h_L(\text{drainpipe})])^{1/2} \quad (h)
\]

\[
q(B) - q(A) = C_d A(\text{orifice}) (2g[h_L(\text{orifice at A}) + h_L(\text{drainpipe})])^{1/2}
- C_d A(\text{orifice}) (2gh_L(\text{orifice A}))^{1/2} \quad (i)
\]

\[
= C_d A(\text{orifice}) \cdot (2g)^{1/2} [h_L(\text{orifice at A}) + h_L(\text{drainpipe})]^{1/2} - [h_L(\text{orifice A})]^{1/2} \quad (j)
\]

In other words, the difference between q(A) and q(B) is in the h_L(pipe) term. The important point is that:

\[
h_L(\text{pipe}) << h_L(\text{orifice at A}) \quad (l)
\]

Equation (l) states the basic criterion for underdrain design, which is that the headloss within the underdrain pipe should be negligible compared to the headlosses across the orifice.

(1) Design of Underdrains. The underdrains usually consist of pipes with orifices of holes, slots, or spaces. Design variables include the configuration of pipes, the spacing of laterals, the diameter of laterals, the number of orifices per lineal unit distance of lateral, and the diameter of the orifices. The diameter and spacing of the underdrain pipes and the diameter of the orifices may be determined theoretically by hydraulic calculations (see Section 3.1.2, Example 3.5, and Figure 3.26). The use of empirical
Figure 3.26 Underdrain Spacing Effect on Streamlines

Guidelines is, however, common. Such guidelines are given in the Ten States Standards (Great Lakes Upper Mississippi River Board 1987) and in design handbooks.

Figure 3.26 shows the effect of the spacing of underdrain pipes on streamlines. The flows between the streamlines are equal, by definition. The volume bounded by a set of streamlines is called a "streamtube." Figure 3.26a shows the streamlines for pipes spaced 0.5 m apart, and Figure 3.26b shows the streamlines for pipes 2.0 m apart. In Figure 3.26a, the streamlines are uniform through the sand bed and within the gravel support. In Figure 3.26b, the streamlines a, b, and c are not spaced equally. Streamlines a and b are
spaced closer than b and c. The reason is that streamtube b-c has a longer flow path within the gravel support, and hence the headloss is larger. The area needed for streamtube b-c must be larger than that for streamtube a-b. Thus, in principle, underdrain spacing affects the spatial distribution of the hydraulic loading rate. In practice, however, the effect of the larger spacing of drains in Figure 3.26b is likely to be negligible, since the hydraulic resistance in the gravel support is usually very little. Although a 2-m (6.5-ft) spacing may be satisfactory, a smaller spacing is prudent because underdrain pipes are inexpensive and uniform flow will be assured.

The pipe material for underdrains should be noncorrosive. Clay pipe is traditional, and in recent years plastic tubes have been used widely. The underdrain tubes at 100 Mile House, BC, are slotted SDR 26 PVC pipe; at Empire, the tubes are polyethylene. With respect to sizing, the tubes at 100 Mile House are 15.0 cm (6.0 in.) in diameter with 131 slots per row per meter (40 slots per row per foot) of pipe and extend the full length of the filter boxes, that is, 43 m (141 ft). The slots are in three rows and are 1.0 mm (0.039 in.) wide and 25 mm (1 in.) long, with 6-mm (0.25-in.) spacing between each slot. Three such tubes make up the underdrains, with one in the center and one 2.0 m (6.6 ft) on either side of the center tube. At Empire, CO, the underdrains are perforated polyethylene tubes, spaced at 1.52 m (5 ft).

Hazen (1913) suggested drainage area limits for different sizes of underdrain pipes as given in Table 3.5 (metric sizes supplied). He calculated the frictional resistance of the underdrain pipes for the Hamburg filters as 11 mm (0.036 ft). Lindley designed the Warsaw filters (Hazen 1913) to have underdrain headloss of only 5 mm (0.0164 ft) when filtering at a rate of 0.10 m/hr (2.57 mgd).

In addition to specifying the size of the orifices and the size and material for the conduits, the design must specify the conduit layout. For small filter areas, all underdrains can lead to a single header pipe at the end of the filter, just inside the filter box and parallel to the pipe gallery. If the filter bed area is large, a center header pipe with

<table>
<thead>
<tr>
<th>Diameter of drainpipe</th>
<th>Maximum area of sand to be drained</th>
</tr>
</thead>
<tbody>
<tr>
<td>cm</td>
<td>m²</td>
</tr>
<tr>
<td>10.2</td>
<td>26.9</td>
</tr>
<tr>
<td>15.2</td>
<td>69.7</td>
</tr>
<tr>
<td>20.3</td>
<td>142</td>
</tr>
<tr>
<td>25.4</td>
<td>258</td>
</tr>
<tr>
<td>30.5</td>
<td>409</td>
</tr>
</tbody>
</table>

*Source: Data from Hazen (1913).*
laterals may be desired. The main idea is to ensure small headlosses within the underdrain pipes.

3.2.3 Sand Size Specification

Comprehensive recommendations for sand size were given by Allen Hazen, one of the early pioneers in the field of slow sand filtration, who made careful and systematic studies as a basis for his recommendations. His 1913 treatise (Hazen 1913) on filtration is a useful point of departure for a discussion of current research and is reviewed here for historical perspective. In some cases, current research and practice have not supplanted Hazen’s work.

(1) Hazen’s Foundation. Hazen (1913) characterized a sand grain in terms of the mean diameter of an equivalent sphere, calculated as the cube root of the product of the dimensions as measured along the principal axes. For example, if the dimensions along the three axes are 1.38, 1.05, and 0.69 mm, the equivalent diameter is 1.0 mm. Also, Hazen reasoned that for a sand bed having a range of sand sizes, the flow is forced between the smaller particles occupying the spaces created by the larger grains and that the smaller sized grains determine the character of the sand for filtration. Thus, the idea of effective size, $d_{10}$, was born as a means to characterize a sand, along with the idea of the uniformity coefficient, UC. Hazen's definitions are:

\[
d_{10} = \text{the size of grain such that 10 percent by weight of the total sample is smaller, (mm), and}
\]

\[
UC = \text{the ratio of the size of grain that has 60 percent of the sample finer than itself to the size that has 10 percent finer than itself, that is, } d_{60}/d_{10}.
\]

(2) Hazen’s Recommendations for Sand Sizes. Hazen reported the sand sizes used for sand beds in 56 installations in Europe. A few of these are given in Table 3.6. Hazen noted that the entire $d_{10}$ range for 10 of the leading installations in England and Germany was between 0.31 and 0.40 mm. As noted in Table 3.6, the UC was generally <2.0. Hazen asserted that the quality of effluent depends upon the size of the sand grains. He reported the percent bacteria (B. prodigiosus) appearing in filter effluents for different $d_{10}$ sand sizes during 1892–1893 experiments at the Lawrence Experiment Station, Lawrence, MA. These data, listed in Table 3.7, show smaller amounts of bacteria for smaller sand sizes.

(3) Current Recommendations. The guidance of Huisman and Wood (1974) on $d_{10}$ and UC has been followed widely in practice and has been used as a starting point in most slow sand filtration research. On effective diameter, Huisman and Wood stated:
Table 3.6
Sand Sizes for Slow Sand Filters in Europe as Reported by Hazen (1913)

<table>
<thead>
<tr>
<th>Installation</th>
<th>( d_{10} ) (mm)</th>
<th>UC</th>
</tr>
</thead>
<tbody>
<tr>
<td>London, E. London</td>
<td>0.44</td>
<td>1.8</td>
</tr>
<tr>
<td>London, E. London</td>
<td>0.39</td>
<td>2.1</td>
</tr>
<tr>
<td>London, Chelsea</td>
<td>0.36</td>
<td>2.4</td>
</tr>
<tr>
<td>Birmingham</td>
<td>0.29</td>
<td>1.9</td>
</tr>
<tr>
<td>Antwerp</td>
<td>0.38</td>
<td>1.6</td>
</tr>
<tr>
<td>Hamburg</td>
<td>0.28</td>
<td>2.5</td>
</tr>
<tr>
<td>Altona</td>
<td>0.32</td>
<td>2.0</td>
</tr>
<tr>
<td>Berlin, Stralau</td>
<td>0.33</td>
<td>1.9</td>
</tr>
<tr>
<td>Berlin, Tegel</td>
<td>0.38</td>
<td>1.6</td>
</tr>
<tr>
<td>Budapest</td>
<td>0.20</td>
<td>2.0</td>
</tr>
<tr>
<td>Zurich</td>
<td>0.30</td>
<td>3.1</td>
</tr>
<tr>
<td>Hague</td>
<td>0.19</td>
<td>1.6</td>
</tr>
<tr>
<td>Amsterdam</td>
<td>0.17</td>
<td>1.6</td>
</tr>
</tbody>
</table>

*Source*: Data from Hazen (1913).

Table 3.7
Percent Bacteria in Filter Effluents for Different Sand Sizes in Lawrence Experiment Station Studies

<table>
<thead>
<tr>
<th>( d_{10} ) (mm)</th>
<th>Percent bacteria in effluent</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.38</td>
<td>0.16</td>
</tr>
<tr>
<td>0.29</td>
<td>0.16</td>
</tr>
<tr>
<td>0.26</td>
<td>0.10</td>
</tr>
<tr>
<td>0.20</td>
<td>0.13–0.01</td>
</tr>
<tr>
<td>0.14</td>
<td>0.04–0.03</td>
</tr>
<tr>
<td>0.09</td>
<td>0.02–0.02</td>
</tr>
</tbody>
</table>

*Source*: Data from Hazen (1913).

"Ideally the effective diameter of the sand, \( d_{10} \), should be just small enough to ensure a good quality effluent and to prevent penetration of clogging matter to such a depth that it cannot be removed by surface scraping. This effective diameter usually lies in the range 0.15–0.35 mm and is determined by experiment" (p. 53). They recommend a uniformity coefficient between 1.5 and 2.0. They restrict the UC so that the sand will have sufficient porosity. Their recommended upper limit is UC < 3.

(4) Research. Some research on slow sand filtration during the 1980s has focused on determining the most appropriate \( d_{10} \) and UC for new installations. The research has not, however, provided definitive answers. Experimental work by Bellamy et al. (1985b, 1985c) for biologically mature pilot filters 30.5 cm (12 in.) in diameter, with \( d_{10} \) sand sizes of 0.13, 0.29, and 0.62 mm and operated over a 10-month period, had average coliform
removals of 99.4, 98.6, and 96.0 percent, respectively. Although these data show that the 
d_{10}\text{ sand size has an influence on removal efficiency that decreases linearly with}
increasing d_{10}, the difference is only 0.8-log. In other experimental work, Barrett (1989)
showed that large sand size (d_{10} = 0.92 \text{ mm}) gave removals of 3-log to 4-log over a
2.5-month sampling period, after the sand bed was allowed to ripen for about two weeks
using nutrients. The key to the successful use of larger sand sizes is to have a biologically
mature sand bed. Even d_{10} sand sizes in the recommended range of 0.2–0.3 will not be
capable of their potential removals until the sand bed is biologically mature, as shown by
Bryck et al. (1987), who spiked pilot filters with coliforms at start-up and four months
later. Results of the spiking are given in Table 3.8. As shown in Table 3.8, coliform

<table>
<thead>
<tr>
<th>Elapsed time from start of spiking (hours)</th>
<th>Total coliform count (MPN/100 mL)(^{a})</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Raw water</td>
</tr>
<tr>
<td>------------------------------------------</td>
<td>---------------</td>
</tr>
<tr>
<td>Nov. 27, 1985(^{b})</td>
<td></td>
</tr>
<tr>
<td>0.00</td>
<td>49,000</td>
</tr>
<tr>
<td>0.67</td>
<td>—</td>
</tr>
<tr>
<td>1.67</td>
<td>540,000</td>
</tr>
<tr>
<td>3.00</td>
<td>13,000</td>
</tr>
<tr>
<td>6.00</td>
<td>3,500</td>
</tr>
<tr>
<td>8.00</td>
<td>13,000</td>
</tr>
<tr>
<td>11.00</td>
<td>1,600,000</td>
</tr>
<tr>
<td>13.00</td>
<td>23,000</td>
</tr>
<tr>
<td>15.00</td>
<td>110,000</td>
</tr>
<tr>
<td>18.00</td>
<td>23,000</td>
</tr>
<tr>
<td>19.50</td>
<td>3,300</td>
</tr>
<tr>
<td>30.00</td>
<td>—</td>
</tr>
<tr>
<td>30.50</td>
<td>79,000</td>
</tr>
<tr>
<td>March 20, 1986(^{c})</td>
<td></td>
</tr>
<tr>
<td>0.00</td>
<td>3,300</td>
</tr>
<tr>
<td>1.25</td>
<td>1,600,000</td>
</tr>
<tr>
<td>2.75</td>
<td>350,000</td>
</tr>
<tr>
<td>4.75</td>
<td>350,000</td>
</tr>
<tr>
<td>8.75</td>
<td>17,000</td>
</tr>
<tr>
<td>11.75</td>
<td>11,000</td>
</tr>
<tr>
<td>14.25</td>
<td>9,200</td>
</tr>
<tr>
<td>17.25</td>
<td>3,500</td>
</tr>
<tr>
<td>23.92</td>
<td>17,000</td>
</tr>
<tr>
<td>25.92</td>
<td>16,000</td>
</tr>
<tr>
<td>34.17</td>
<td>79,000</td>
</tr>
</tbody>
</table>

\(^{a}\)MPN means "most probable number" and refers to a standard test for the coliform group of bacteria. See, for example, Standard Methods (1989).
\(^{b}\)Water temperature was 1°C. Giardia cyst removal was 98.5 percent based upon a spike of 675,000 cysts.
\(^{c}\)Water temperature was 1°C–4°C. Giardia cyst spike was not successful due to attrition of cysts (4,400,000 cysts were supplied but only 6 were recovered from influent sampling and 0 from effluent sample).

Source: Adapted from Bryck et al. (1987).
removals were essentially zero at filter start-up on November 27, 1985. On March 20 the same pilot filter was spiked again and removals were seen to be nominally 1–2 log. Even with water temperatures of near 0°C to 4°C during this period, some maturing evidently did occur, as seen by the comparisons in coliform removals for the two dates.

This recent research with larger sand sizes indicates that a \( d_{10} > 0.4 \) mm can provide acceptable removals provided that the sand bed is biologically mature. However, sand sizes between 0.2 and 0.3 mm can be used with greater confidence, based upon past experience. This past experience includes studies with sand sizes between 0.3 and 0.4 mm, and results have shown that this larger sand can be used with nearly the same confidence. Pilot plant testing should be conducted when larger sand sizes are considered. As a recommendation, larger sand sizes should be used only in warmer climates where the filters will become biologically mature within a short period of time. Thus, although neither those filters with large sand particles nor those with smaller sand are fully effective until the sand beds are biologically mature, the margin of difference in effectiveness for using a smaller sand size may be important when biological maturity is lacking (see results of Bellamy et al. 1985b, 1985c, and Bryck et al. 1987).

For economic reasons, it may, in many cases, be necessary to consider a sand that has a \( UC > 3 \). The use of local sand (as was the case at Empire, CO) in lieu of a sand that meets specifications strictly may save much money and keep the funds local. Table 3.9 shows the cost for sand used at four U.S. and Canadian filters. The table indicates that the cost for the local sand used for the Empire filter was only $21/metric ton as opposed to the $128/metric ton for the Muscatine sand that would have met specifications.

(5) Determination of Sand Characteristics. The \( d_{10} \) and \( UC \) parameters must be determined by sieve analysis. From the data obtained, a plot must be drawn along the lines of the one shown in Figure 3.27. (Note: a probability scale, rather than the log scale shown in Figure 3.27, is recommended for the grain diameter.) The data plotted in the figure are for the Empire slow sand filter. The \( d_{10} \) and \( d_{60} \) values are read directly from the plot, that is, \( d_{10} = 0.24 \) and \( d_{60} = 0.64 \) mm. The calculation for UC gives \( UC = d_{60}/d_{10} = 0.64/0.24 = 2.67 \). Appendix Figure C.1 shows the conversion between grain diameter and U.S. Standard sieve sizes. Because plotting is often in terms of sieve results, the graph shown, with the conversion built in, may be useful and may be copied.

3.2.4 Gravel Support

The gravel support is aptly named because its function is to support the sand bed and to permit uniform drainage of the overlying sand. The cost of gravel was about $24/mT (metric ton) for the Moricetown filter, which was slightly less than the cost of
Table 3.9
Sand Sizes for Selected Slow Sand Filters Completed in the United States and Canada During the Period From 1985 to 1989

<table>
<thead>
<tr>
<th>Installation</th>
<th>(d_{10}) (mm)</th>
<th>UC</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Empire, CO</td>
<td>0.21</td>
<td>2.67</td>
<td>$21$/metric ton(^a) ($19$/U.S. short ton)</td>
</tr>
<tr>
<td>100 Mile House, BC</td>
<td>0.2–0.3</td>
<td>3.3</td>
<td>$11.60^{bc}/m^3</td>
</tr>
<tr>
<td>Moricetown, BC</td>
<td>0.15–0.35</td>
<td>2.0–2.5</td>
<td>$34^{bd}/m^3</td>
</tr>
<tr>
<td>CSU pilot plant</td>
<td>0.28</td>
<td>1.46</td>
<td>Muscatine: $128$/metric ton</td>
</tr>
</tbody>
</table>

\(^a\)1 metric ton (mT) = 1.1 U.S. tons (short, or 2,000 lb). Reported in Seelaus et al (1986).

\(^b\)U.S. dollars; conversion was made from Canadian dollars using the December 1984 exchange rate of U.S. $0.7577 = Can. $1.00.

\(^c\)Calculated from the following data: 1,000 m\(^3\) sand plus 560 m\(^3\) gravel for Can. $24,000 plus $10,000 for installation (Jack Bryck, pers. com., Sept. 20, 1990).

\(^d\)Calculated from the following data: 250 m\(^3\) sand for Can. $11,250 (Contract Bid Document 168.21.2, Dayton & Knight, Ltd., Vancouver, BC, 1988b).

---

Figure 3.27  Sieve Analysis Plot Showing How \(d_{10}\), \(d_{60}\), and UC Are Determined. (From Seelaus et al. [1988].)
the sand (shown in Table 3.9). By definition, uniform drainage requires minimal headloss. To accomplish both purposes, the gravel support must be graded, with finer material at the top and coarser material at the bottom. The size of gravel in each layer, the respective depths, and the headloss are discussed in the following sections.

(1) Size. The top layer of the gravel support should not permit migration of sand from the sand bed, nor should the gravel of any layer find its way to a lower level. The bottom layer should not permit entry of gravel to the underdrain orifices. Huisman and Wood (1974) gave the following rules for the design of the gravel support layers. The rules are applied in Example 3.13.

1. \( d_{90}^{(given\ layer)} / d_{10}^{(given\ layer)} \leq 1.4 \)
2. \( d_{10}^{(lower\ layer)} / d_{10}^{(upper\ layer)} \leq 4 \)
3. \( d_{10}^{(top\ layer)} / d_{15}^{(sand)} \geq 4 \)
4. \( d_{10}^{(top\ layer)} / d_{85}^{(sand)} \leq 4 \)
5. \( d_{10}^{(bottom\ layer)} \geq 2 \cdot d^{(drain\ orifice\ diameter)} \)

Example 3.13: Graded Gravel Support Design. The analysis of the sand bed of a slow sand filter gives sand sizes of \( d_{15} = 0.18\ mm \) and \( d_{85} = 0.3\ mm \). Determine the gravel support design following the rules of Huisman and Wood. (Problem adapted from Huisman and Wood [1974].)

1. **Top layer of gravel:**

   Apply rule 3:
   \[ d_{10}^{(top\ layer)} \geq 4 \cdot d_{15}^{(sand)} \]
   \[ \geq 4 \cdot 0.18 \]
   \[ \geq 0.7\ mm \]

   Apply rule 4:
   \[ d_{10}^{(top\ layer)} \leq 4 \cdot d_{85}^{(sand)} \]
   \[ \leq 4 \cdot 0.3 \]
   \[ \leq 1.2\ mm \]

   Therefore, the top layer of gravel should have \( d_{10} \) between 0.7 mm and 1.2 mm. Select \( d_{10} = 1.0\ mm \).

   Apply rule 1:
   \[ d_{90}^{(top\ layer)} \leq 1.4 \cdot d_{10}^{(top\ layer)} \]
   \[ d_{90} \leq 1.4 \cdot 1.0 \]
   \[ \leq 1.4\ mm \]

   The top layer of gravel should have the following specification: \( d_{10} = 1.0\ mm \), \( d_{90} \leq 1.4\ mm \).

2. **Second layer of gravel:**

   Apply rule 2:
   \[ d_{10}^{(second\ layer)} \leq 4 \cdot d_{10}^{(top\ layer)} \]
   \[ \leq 4 \cdot 1.0 \]
   \[ \leq 4.0\ mm \]
Apply rule 1:

\[ d_{90} \text{ (second layer)} \leq 1.4 \cdot d_{10} \text{ (second layer)} \]
\[ d_{90} \text{ (second layer)} \leq 1.4 \cdot 4.0 \]
\[ \leq 5.6 \text{ mm} \]

The second layer should have the following specification: \( d_{10} \leq 4.0 \text{ mm}, d_{90} \leq 5.6 \text{ mm} \).

3. **Third layer of gravel**:

Applying the same rules for the third layer gives \( d_{10} \leq 16 \text{ mm}, d_{90} \leq 23 \text{ mm} \).

4. **Apply rule 5 to test third layer as bottom layer**:

\[ d_{10} \text{ (bottom layer)} \geq 2 \cdot d \text{ (drain orifice diameter)} \]
\[ 16 \text{ mm} \geq 2 \cdot d \text{ (drain orifice diameter)} \]
\[ d \text{ (drain orifice diameter)} \leq 8 \text{ mm} \]

5. **Summary of gravel layers**:

<table>
<thead>
<tr>
<th>Layer</th>
<th>( d_{10} ) (mm)</th>
<th>( d_{90} ) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>First</td>
<td>1.0</td>
<td>1.4</td>
</tr>
<tr>
<td>Second</td>
<td>4.0</td>
<td>5.6</td>
</tr>
<tr>
<td>Third</td>
<td>16</td>
<td>23</td>
</tr>
</tbody>
</table>

6. **Comments**: The gravel support using three layers as specified will work if the orifices into the underdrain pipe are less than 8 mm in diameter. If the orifices are larger, more than three layers of gravel may be needed.

7. **Critique**: One of the authors of this work, G. Logsdon, Chair of the AWWA Filter Material and Standard Committee (1983–1986) suggests that the sieve range for a given layer of gravel should be about 2 to 1 as a practical matter. Having a range of 1.4, as suggested by Huisman and Wood, is too "tight" for reasons of cost. With such a tight specification, more sieving is required and more waste of media will result. Further reference on the topic is given in AWWA Standard B100-89 (American Water Works Association 1989). Huisman and Wood, in fact, state, "Where it is too difficult or expensive to grade the gravel within a layer to the recommended ratio of 1:1.4, the requirements may be relaxed to a factor of 1:2, but in this case the layers should have their \( d_{10} \) values restricted to three times that of the layer above" (p. 57). Applying this rule to Example 3.13 will then result in four layers, instead of three, with gradings of 0.7–1.4 mm, 2–4 mm, 6–12 mm, and 18–36 mm. Thus, the more relaxed version of the Huisman and Wood guidelines would concur with the above recommendation by Logsdon. Huisman and Wood were not dogmatic about the rules to be applied and suggested that some engineers may prefer that the \( d_{10} \) ratio between layers be ≤2, which would call for five layers in Example 3.13, that is, 0.6–1.2 mm, 1.2–2.4 mm, 2.4–4.8 mm, 4.8–10 mm, and 20–40 mm. Whatever the rules, Example 3.13 illustrates the procedure for determining the needed gradations.

(2) **Depth of Gravel Layers**. Another rule from Huisman and Wood (1974) is that the thickness of each gravel layer should be greater than three times the diameter of the largest stones. At the same time, they state that as a practical matter the minimum thickness of gravel layers should be 5–7 cm for finer material and 8–12 cm for coarser gravel. Hazen (1913) makes the case that only three layers of graded gravel, with a total thickness of 15 cm (6 in.), are needed and that such a design was successfully used in many installations. His recommendations are within the range recommended by Huisman and Wood, although they are on the low side.
The total depth of gravel is determined by the size of the orifices of the underdrain pipes, as implied by the Huisman and Wood rules and by Example 3.13. Thus, the smallest gravel depth will be for a porous plate type of underdrain. One example Huisman and Wood mention is a porous concrete underdrain made of aggregate 5–10 mm in size, which would require only one layer of 1.2–2.4 mm gravel to support a filter sand having \( d_{10} \leq 0.3 \) mm.

(3) **Headloss.** Basing their calculations upon the data in Table 3.10, using Darcy's law, and assuming an HLR of 0.5 m/hr (12.5 mgad), Huisman and Wood (1974) determined the total headloss across a gravel support to be 1.37 mm. Huisman and Wood stated the gravel size only as indicated in Table 3.10, that is, without reference to a specific size, such as \( d_{10} \). Nevertheless, the example illustrates the idea that the headloss through the gravel support is very low. Also, the hydraulic conductivity values in Table 3.10, or \( k \) values, for the different layers may be useful for estimating headlosses for other situations. As is evident, headloss in the gravel support is negligible.

(4) **Designs.** The designs of the gravel support media used at three installations—Empire, CO, 100 Mile House, BC, and Moricetown, BC—are summarized in Table 3.11. These designs provide a basis for judgment when applying the rules of Huisman and Wood (1974) given in Section 3.2.4.

(5) **Great Lakes Upper Mississippi River Board Recommendations.** The Great Lakes Upper Mississippi River Board (1987) recommended that hard, rounded particles be used for the gravel support media and that the underdrain system be composed of at least four layers. Table 3.12 gives sizes and depths of gravel fractions for perforated laterals as recommended by this board. The guidelines in Table 3.12 were followed for the Empire design, as can be seen by comparing the data in Table 3.11 for Empire with the guidelines in Table 3.12.

### Table 3.10

<table>
<thead>
<tr>
<th>Layer</th>
<th>Thickness (cm)</th>
<th>Gravel size (mm)</th>
<th>Hydraulic conductivity, ( k ) (m/hr)</th>
<th>Headloss (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top</td>
<td>6</td>
<td>0.7–1.4</td>
<td>25</td>
<td>1.20</td>
</tr>
<tr>
<td>Second</td>
<td>6</td>
<td>2–4</td>
<td>200</td>
<td>0.15</td>
</tr>
<tr>
<td>Third</td>
<td>6</td>
<td>6–12</td>
<td>1,800</td>
<td>0.02</td>
</tr>
<tr>
<td>Bottom</td>
<td>12</td>
<td>18–36</td>
<td>16,000</td>
<td>0.00</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td></td>
<td>1.37</td>
</tr>
</tbody>
</table>

Table 3.11
Gravel Support Designs at Three Slow Sand Installations

<table>
<thead>
<tr>
<th>Installation</th>
<th>Layer</th>
<th>Size Range</th>
<th>Depth</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>mm</td>
<td>cm (in.)</td>
</tr>
<tr>
<td>Empire, CO</td>
<td>Top</td>
<td>3–6</td>
<td>5 (2)</td>
</tr>
<tr>
<td></td>
<td>Second</td>
<td>6–13</td>
<td>10 (4)</td>
</tr>
<tr>
<td></td>
<td>Third</td>
<td>13–19</td>
<td>10 (4)</td>
</tr>
<tr>
<td></td>
<td>Fourth</td>
<td>19–38</td>
<td>13 (5)</td>
</tr>
<tr>
<td></td>
<td>Bottom</td>
<td>38–64</td>
<td>23 (9)</td>
</tr>
<tr>
<td>100 Mile House, BC</td>
<td>Top</td>
<td>3–6</td>
<td>15 (6)</td>
</tr>
<tr>
<td></td>
<td>Second</td>
<td>9–14</td>
<td>15 (6)</td>
</tr>
<tr>
<td></td>
<td>Bottom</td>
<td>20–63</td>
<td>30 (12)</td>
</tr>
<tr>
<td>Moricetown, BC</td>
<td>Top</td>
<td>2.5–3</td>
<td>15 (6)</td>
</tr>
<tr>
<td></td>
<td>Second</td>
<td>10–15</td>
<td>15 (6)</td>
</tr>
<tr>
<td></td>
<td>Bottom</td>
<td>40–60</td>
<td>30 (12)</td>
</tr>
</tbody>
</table>

Table 3.12
Gravel Support Design for Perforated Underdrain Laterals, as Recommended by the Great Lakes Upper Mississippi River Board

<table>
<thead>
<tr>
<th>Size</th>
<th>Depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>64–38 mm (2 1/2–1 1/2 in.)</td>
<td>12.7–20.3 cm (5–8 in.)</td>
</tr>
<tr>
<td>38–19 mm (1 1/2–3/4 in.)</td>
<td>7.6–12.7 cm (3–5 in.)</td>
</tr>
<tr>
<td>19–13 mm (3/4–1/2 in.)</td>
<td>7.6–12.7 cm (3–5 in.)</td>
</tr>
<tr>
<td>13–5 mm (1/2–3/16 in.)</td>
<td>5.1–7.6 cm (2–3 in.)</td>
</tr>
<tr>
<td>5–2 mm (3/16–3/32 in.)</td>
<td>5.1–7.6 cm (2–3 in.)</td>
</tr>
</tbody>
</table>

Source: Great Lakes Upper Mississippi River Board (1987).

3.3 APPURTEANCES

Slow sand filters require a number of appurtenances, for example, to measure flow, to read piezometric head, and to provide tailwater elevation control. These are described in this section. Other appurtenances, not described here, include devices for providing heat and for disinfecting the filter system.

3.3.1 Flow Measurement

Measurement of total flow is needed for water production records. Also a measure of instantaneous flow is required for operation. In addition, measurement of flow to and from individual filter beds is desirable to facilitate analysis of performance and to diagnose hydraulic problems.
Flow meters should be accurate, easy to use, reliable, low maintenance, and inexpensive. Such criteria are met by orifice plate meters for pipe installations. Weirs may be used if an overflow situation is a part of the design. Venturi meters are more accurate than orifice plates and have lower headloss, but they are more expensive than orifice plates, sometimes by an order of magnitude. Installation of flow meters on the influent side of the filter is most desirable. Influent-side installation permits adjustment of flow as desired by the operator. Effluent flow measurement cannot be used easily for upstream flow adjustment. The selection process described in this section for orifice plates and weirs is based upon the procedures outlined by Roberson and Crowe (1985). On the filter effluent side, a total flow meter is required so that measurements can be made of total flow volume delivered. Total flow meters range in type from diaphragm positive displacement to propeller. A review of propeller meters is given.

Installation of any meter in a pipeline should be downstream of bends or any other kind of disturbance. Although the rule of thumb is to position the meter several pipe diameters downstream from a disturbance, a better rule is: "The farther the meter is from the disturbance, the better." Fluctuations in a manometer may be observed even, for example, when the meter is located 50–100 pipe diameters downstream of a bend. Also, provision should be made for easy cleaning of the pressure taps, which may become clogged over time, and for inspection of the flow meter for deposits or erosion. Positioning the taps on the side of the pipe will minimize clogging due to sediment and will avoid the problem of gas entering a tap, which could occur for taps located at the top of a pipe. Flow meters can be considered accurate only if the conditions of their calibration are duplicated in the field.

1) Orifice Plate Meter. Orifice plate meters are recommended for flow measurement because they are simple, inexpensive, accurate, and easy to install. An orifice meter is a flat plate with a hole in the center that is placed between two flanges in a pipeline. The hole should be circular and should have a sharp edge so that standard coefficients can be used when determining flow. The flow is proportional to the square root of the pressure differential between upstream and downstream pressure taps, as indicated by Equation 3.4. Figure 3.28 shows the main features of an orifice meter. The variables d, D, and Δh are defined following Equations 3.4 and 3.13. Although a manometer is shown to illustrate pressure differential, pressure gauges are recommended over manometers. Mercury manometers should not be used because of the hazard of mercury being "blown" into the flow. As is well known, mercury is an acute health hazard. The taps should be easily cleaned. Figure 3.29 is a photograph of two orifice plates designed to fit into a flanged pipe.
The standard orifice equation is:

\[ Q = C_d A_{(orifice)} \cdot (2g \Delta h)^{1/2} \]  

(3.4)

in which \( C_d = f(d/D) \), per Appendix Table B.1

(3.13)

and \( Q = \) flow of water (m\(^3\)/s or ft\(^3\)/s)

\( C_d = \) orifice discharge coefficient, given in Appendix Table B.1 (dimensionless)

\( A_{(orifice)} = \) cross-section area of orifice opening (m\(^2\) or ft\(^2\))
\[ g = \text{acceleration of gravity (9.81 m/s}^2, \text{or 32.2 ft/s}^2) \]
\[ \Delta h = \text{pressure differential across the plate (m or ft)} \]
\[ d = \text{diameter of orifice in orifice plate (m or ft)} \]
\[ D = \text{diameter of pipe (m or ft)} \]

The coefficient, \( C_d \), is a function of the ratio \( d/D \) and is given in Appendix Table B.1. Over a period of time, as the average daily flow increases, a new orifice plate having a larger diameter hole may have to be installed. These plates should be on hand at plant start-up so the operator has easy access to such replacements. Instead of having the operator calculate the flow from these equations, the engineer should provide a graph showing the relationships of headloss to flow. With such a chart, the operator can easily determine the flow from whatever headloss is read on the pressure gauges or piezometers.

The orifice plate should be installed in a flanged section of the pipe so that removal is easy for cleaning of the pressure taps. The orifice plate should be located in a long, straight length of pipe to minimize disturbances caused by eddies. If continuous recording of flow is desired, pressure transducers may be installed. Example 3.14 illustrates how to size an orifice plate using Equation 3.4.

**Example 3.14: Orifice Plate Sizing.** Calculate the size of orifice plate for the Empire, CO, plant. Assume the pipe is 30.54 cm (12 in.) in diameter. Assume also that permissible headloss across the orifice plate is 60 cm.

1. **Given data for Empire:**

   \[
   Q(\text{max, 1,000 persons}) = 3,028 \text{ L/person/d} \times 1,000 \text{ persons} = 3,028,000 \text{ L/d} \\
   = 800 \text{ gpcd} \times 1,000 \text{ persons} = 800,000 \text{ gal/d}
   \]

2. **Calculation:**

   (a) First trial:

   (1) From Table B.1, select \( C_d = 0.62 \).
   (2) Calculate, using Equation 3.4:

   \[
   Q = 0.62 \times A(\text{orifice}) \times (2g\Delta h)^{1/2} \\
   3,028/24/3,600 = 0.62 \times \pi (d)^2/4 \times (2 \times 9.8 \times 0.60)^{1/2} \\
   d = 0.14 \text{ m (5.7 in.)}
   \]

   (b) Check \( C_d \):

   (1) \( d/D = 0.14/0.35 = 0.4 \)
   (2) From Table B.1, \( C_d = 0.615 \), which is close enough to the original assumption.

   (c) Determine the headloss at the low flow during the season of minimum demand and with 500 persons, that is, 278 L/capita/d (100 gpcd), giving \( Q = 189,250 \text{ L/d} \) (50,000 gpd).
3. Discussion: Because this amount of headloss for the low flow condition would be too small to measure accurately, Empire would have to use a smaller diameter orifice for the early years and substitute an orifice plate with a larger hole as the population increased. If, for example, 5 cm was selected as a measurable headloss for the low flow condition, the corresponding orifice hole diameter would be 7 cm (2.75 in.). That would be a practical size orifice plate for the early years.

For both orifice plates, headloss-flow graphs should be drawn, permitting a determination of the desired flow range for each, based upon the desired headloss range.

(2) Venturi Meter. The Venturi meter has lower headloss than an orifice plate and so may be more desirable when headloss is a major concern. Figure 3.30 is a sketch of a Venturi meter; the variables are defined in Equations 3.14 and 3.15. The configuration of a manometer installation is also shown. Note that although a manometer is used in the drawing to illustrate pressure differential, pressure gauges, rather than manometers, are recommended. As mentioned previously, manometer fluids are toxic and are subject to being "blown" into the flow.

The cost of purchasing a Venturi meter for a 30.5-cm (12-in.) pipeline may be several thousand dollars, whereas an orifice plate, by comparison, may cost only two hundred dollars. Equation 3.14 is the Venturi meter flow equation, which is similar to the orifice equation. The coefficients for the Venturi meter are given in Appendix Table B.1. As noted in the table, the coefficient is 1.0, contrasted with about 0.6 for an orifice plate. Headloss between the two types of meters may be compared using these coefficients.

\[ Q = C_v A_t \left(2gA_h\right)^{1/2} \]

in which \( C_v = f(d/D) \) per Table B.1

\[ Q = \text{flow of water (m}^3/\text{s or ft}^3/\text{s)} \]

\( C_v \) = Venturi meter discharge coefficient = 1.0 per Table B.1 (dimensionless)

\( A_t \) = cross-section area of Venturi throat (m\(^2\) or ft\(^2\))

\( g \) = acceleration of gravity (9.81 m/s\(^2\) or 32.2 ft/s\(^2\))

\( A_h \) = pressure between upstream pipe section and throat

\( d \) = diameter of orifice in orifice plate (m or ft)

\( D \) = diameter of pipe (m or ft)

(3) Rectangular Weir. A rectangular weir can be used to control water levels and measure flow. For the overflow within the filter box, a simple rectangular weir with contracted ends will control water level so that excess flow will not overtop the filter.
box. Flow measurement is, of course, not a requirement. To calculate the length of a weir, the standard equation given in Section 3.1.2 is applicable:

\[ Q = C_w \sqrt{2g \cdot b \cdot H^{3/2}} \]  \hspace{1cm} (3.7)

in which \( C_w = 0.40 + 0.05 \frac{H}{P} \) \hspace{1cm} (3.6)

and \( Q = \) flow (m\(^3\)/s or ft\(^3\)/s)

\( C_w = \) weir coefficient (dimensionless)
\( g = \) acceleration of gravity (9.81 m/s\(^2\) or 32.2 ft/s\(^2\))
\( b = \) length of weir crest (m or ft)
\( H = \) height of water level above weir crest upstream from effect of drawdown (m or ft)
\( P = \) distance from floor of channel to weir crest (m or ft)

Figure 3.31 presents two sketches of a rectangular weir with "end contractions," that is, the crest does not extend across the channel. The variables are those used in the flow equations presented above.

![Figure 3.30 Cross-Section Sketch of a Venturi Meter](image)

Figure 3.30 Cross-Section Sketch of a Venturi Meter

![Figure 3.31 Rectangular Weir With Contracted Ends, Showing Terms Used in Flow Equations 3.6 and 3.7](image)

Figure 3.31 Rectangular Weir With Contracted Ends, Showing Terms Used in Flow Equations 3.6 and 3.7
Example 3.15: Rectangular Weir Design for Measurement of Tailwater Overflow.
Calculate the length of a rectangular weir, b, for the tailwater overflow measurement, taking the flow, at the Empire, CO, plant. Assume the permissible depth of water above the weir crest, H, is 10 cm (0.328 ft), and that P = 2.0 m (6.6 ft). The calculation is for one filter.

1. **Given data for Empire:**

   \[ Q(\text{max, 1,000 persons}) = 3,028 \text{ L/person/d} \times 1,000 \text{ persons} = 3,028,000 \text{ L/d} = 800 \text{ gpcd} \times 1,000 \text{ persons} = 800,000 \text{ gal/d} \]

2. **Calculation:**

   Substitute numerical data into Equation 3.7:

   \[ Q(\text{max, one filter}) = C_w \sqrt{2g b H^{3/2}} \]

   \[ 3,028/24/3,600/2 \text{ filters} = \left[ 0.40 + 0.05 \frac{0.10}{2.0} \right] \sqrt{2 \times 9.81} b (0.10)^{3/2} \]

   \[ b = 0.31 \text{ m (1.0 ft)} \]

3. **Discussion:** With a maximum head of only 10 cm, which corresponds to a flow of 1.5 mil L/d per filter, the accuracy of flow measurement will be acceptable. At the same time, the rectangular weir will control the tailwater elevation with only a 10-cm change from zero flow to the maximum flow.

Example 3.16: Tailwater Weir Design for 100 Mile House, BC.
Show the required head, H, for the tailwater overflow weir used in the design at 100 Mile House, BC.

1. **Given data for 100 Mile House, BC:** The design peak flow for the 100 Mile House, BC, slow sand filter plant with three cells was 7.26 mil L/d (1.92 mgd).

2. **Calculate flow per cell:**

   \[ Q(\text{cell}) = Q(\text{plant})/\text{no. cells} \]

   \[ = 7.26 \text{ m}^3/\text{d}/3 \text{ cells} \]

   \[ = 0.028 \text{ m}^3/\text{s/cell} \]

3. **Assume that the weir is circular and has a diameter of 0.40 m and that \( P = 2.0 \text{ m} \). Calculate the head on the weir.**

   Substitute numerical data into Equation 3.7:

   \[ Q = C_w \sqrt{2g b H^{3/2}} \]

   \[ 0.028 \text{ m}^3/\text{s} = \left[ 0.40 + 0.05 \frac{H}{2.0} \right] \sqrt{2 \times 9.891 \times [\pi (0.40)] \times H^{3/2}} \]

   \[ H = 0.25 \text{ m (0.82 ft)} \]

4. **Discussion:**

   **Overflow Weir.** Figure 3.32 shows the tailwater overflow weir for one of the cells at the 100 Mile House plant. The overflow weir crest is the level of the maximum height of the sand bed. At the
maximum flow, the head on the weir will be about 0.25 m (0.82 ft). The flow from the weir is captured by a circular vessel and flows through a pipe to the chlorine contact basin. A flow meter is located after the exit from the filter underdrains.

Valve to Control Headwater Elevation. Shown also in Figure 3.32 is a valve on the downstream side of the flow meter. Such a valve may be used to raise the headwater elevation, after scraping, to alleviate sand bed erosion. When headloss reaches about 0.5 m, the valve may be opened fully. The valve may be used in lieu of a vertically movable tailwater weir but requires more operator attention for a few days after the bed is scraped and before the initial 0.5 m of headloss.

(4) Triangular Weir. A triangular weir, often called a V-notch weir, is accurate as a metering device and is inexpensive. Because the elevation difference over a given flow range is larger with a triangular weir than with a rectangular weir, the triangular weir is less satisfactory for water surface elevation control. The flow-head relationship is:

\[ Q = \frac{8}{15} C_{VN} \sqrt{2g \tan \left(\frac{\theta}{2}\right)} H^{5/2} \]  \hspace{1cm} (3.16)

in which 
- \( Q \) = flow (m\(^3\)/s or ft\(^3\)/s)
- \( C_{VN} \) = weir coefficient = 0.58 when \( \theta = 60^\circ \), given in Appendix Table B.2 (dimensionless)
- \( g \) = acceleration of gravity (9.81 m/s\(^2\) or 32.2 ft/s\(^2\))
- \( H \) = height of water level above weir crest upstream from effect of drawdown (m or ft)
- \( \theta \) = angle of notch in triangular weir (degrees)
For a weir having $\theta = 60^\circ$ and $C_{VN} = 0.58$, the discharge equation is as follows. For metric units,

$$Q = 0.79\, H^{5/2}$$

And for English units,

$$Q = 1.44\, H^{5/2}$$

Figure 3.33 presents two sketches of a triangular weir. The variables are defined in Equation 3.16. A weir with a $60^\circ$ crest is used most often because usually the coefficients are readily available from references.

**Example 3.17: Triangular Weir Head Calculation.** Calculate the head on a $60^\circ$ triangular weir at the Empire, CO, plant for maximum and minimum flows.

1. **For maximum flow:**

   $$Q(\text{max, 1,000 persons}) = 3,028\, \text{L/person/d} \times 1,000\, \text{persons} = 3,028,000\, \text{L/d}$$
   
   $$= 800\, \text{gpcd} \times 1,000\, \text{persons} = 800,000\, \text{gal/d} = 106,952\, \text{ft}^3/\text{d}$$

   **Metric** | **English**
   --- | ---
   $Q = 0.79\, H^{5/2}$ | $Q = 1.44\, H^{5/2}$
   $3,028/24/3,600 = 0.79\, H^{5/2}$ | $106,952/24/3,600 = 1.44\, H^{5/2}$
   $H = 0.29\, \text{m (29 cm)}$ | $H = 0.94\, \text{ft (11.3 in.)}$

2. **For minimum flow:**

   $$Q(\text{min, 500 persons}) = 278\, \text{L/person/d} \times 500\, \text{persons} = 189,250\, \text{L/d}$$
   
   $$= 100\, \text{gpcd} \times 500\, \text{persons} = 50,000\, \text{gal/d} = 6,685\, \text{ft}^3/\text{d}$$

   **Metric** | **English**
   --- | ---
   $Q = 0.79\, H^{5/2}$ | $Q = 1.44\, H^{5/2}$
   $189/24/3,600 = 0.79\, H^{5/2}$ | $6,685/24/3,600 = 1.44\, H^{5/2}$
   $H = 0.095\, \text{m (9.5 cm)}$ | $H = 0.31\, \text{ft (3.7 in.)}$
3. **Discussion:** The tailwater level range of 9.5 to 29 cm should be acceptable, and so the triangular weir would be the choice for flow measurement or for the dual role of tailwater elevation control and flow measurement. The rectangular weir would be the choice if tailwater control were the only function.

(5) **Propeller Meter.** A propeller meter consists of a propeller placed in a pipe section. The propeller diameter is matched to the pipe diameter. To measure flow, the shaft rotation is coupled to a flow volume indicator (or register). Figure 3.34 is a photograph of a propeller meter housing with a volume gauge. Propeller meters are constructed in sizes ranging from 50 mm (2 in.) to 3 m (120 in.) and are used widely in

![Figure 3.34 Photograph of a Propeller Meter, Showing Volume Gauge. (Courtesy Mike Herbst, Department of Water and Sewer Utilities, City of Fort Collins, CO.)](image-url)
many different kinds of situations (Huth 1990). Such meters are accurate to within ±2 percent throughout a 10:1 or 20:1 flow range.

The propeller has three or six blades and rotates in proportion to the flow velocity in the pipe section. The volume of flow that has passed through the propeller is proportional to the number of revolutions (rev). That is,

\[ V = k(\text{meter}) \times N \]  

(3.19)

in which \( V \) = volume of flow that has passed through the propeller (m\(^3\) or ft\(^3\))

\( k(\text{meter}) \) = coefficient of proportionality to calibrate flow meter (m\(^3\)/rev or ft\(^3\)/rev)

\( N \) = number of revolutions of propeller associated with volume, \( V \) (rev)

**Example 3.18: Volume-Flow Relation.** Show the derivation of Equation 3.19. The volume of flow that has passed through the pipe section can be calculated as follows:

\[
\begin{align*}
  \text{d}V &= Q \times \text{dt} \\
  &= v(\text{pipe}) \times A(\text{pipe}) \times \text{dt} \\
  &= [k(\text{meter}) \times \text{rpm}] \times A(\text{pipe}) \times \text{dt} \\
  &= [k(\text{meter}) \times \text{d}N/\text{dt}] \times A(\text{pipe}) \times \text{dt} \\
  V &= k(\text{meter}) \times N
\end{align*}
\]

in which \( V \) = volume of flow that has passed through the propeller (m\(^3\) or ft\(^3\))

\( Q \) = flow in pipe (m\(^3\)/s or ft\(^3\)/s)

\( t \) = time for flow of volume, \( V \) (s)

\( v(\text{pipe}) \) = velocity of water flowing within the pipe (m/s or ft/s)

\( A(\text{pipe}) \) = cross-sectional area of pipe (m\(^2\) or ft\(^2\))

\( k(\text{meter}) \) = coefficient of proportionality between velocity in pipe and rotational speed of propeller (m/s/rev or ft/s/rev) = coefficient of proportionality to calibrate flow meter (m\(^3\)/rev)

\( N \) = number of revolutions of propeller associated with volume, \( V \) (rev)

With Equation 3.19, the number of revolutions is converted to the flow volume reading on the register. Calibration of each flow meter must be done before installation to determine \( k' \). The flow volume is read from a digital register located at the top of the meter. Inside the meter a coupling, which may be direct-drive or magnetic-drive, as described by Huth (1990), connects the propeller to the internal mechanism. Magnetic drive eliminates the need for packing seals, and thus water is not likely to enter the register. The bearings of the propeller meter may be water-lubricated ceramic-sleeve bearings or stainless-steel ball bearings. The ceramic-sleeve type requires less maintenance and has a longer life than the stainless-steel bearings (Huth 1990).
According to Huth (1990), the meters can be equipped to measure flow as well as volume. Clogging due to debris is a possible problem; for that reason installing propeller meters on the upstream side of the sand bed is not advised.

3.3.2 Piezometers

Piezometers are tubes used to measure the pressure head at any point in a hydraulic system. The pressure tap should be at the point where a measurement is desired. For a slow sand filter, such taps should be in a wall retaining the headwater, just above the sand surface, and in the wall retaining the tailwater. The piezometers should be placed side by side and mounted on a metric scale. The tubes should be 2 to 4 cm in diameter and can contain colored plastic float balls to allow for easier reading. The operator can then use these piezometers to take the readings of water elevation in the filter. To help diagnose possible problems, for example media clogging, piezometers could also be tapped into the top gravel layer and the sand media, perhaps 30 cm above the top gravel support layer. Such taps should project into the media about 15–20 cm from the wall of the filter bed. The taps should be protected from media encroachment by means of a noncorrosive fabric covering the opening, with mesh size smaller than the media being penetrated.

3.3.3 Tailwater Elevation Control

Tailwater elevation should be controlled by means of a steel weir plate, preferably stainless steel. The essential point is that the weir plate should be movable so that the crest can be mounted above the sand surface elevation at the start of a run and can be lowered as the run progresses. At the start of a filter run, the filter bed should have about 30–40 cm of water cushion to minimize sand bed erosion from the influent water. As the run progresses, the weir can be lowered because the headloss across the schmutzdecke will develop to provide the needed water depth in the headwater. The weir plate may be bolted in place, but the operator will be able to change the level more easily if the plate can be fitted with a gear rack. Such racks can be fabricated and could be available commercially.
Chapter 4

Pilot Plant Studies

A pilot plant is a small version of a full-scale plant constructed for use in answering questions about full-scale operation. The questions relate most often to (1) the treatability of raw water, (2) design criteria, (3) operating costs, and (4) whether pretreatment is needed. Because every raw water is unique, the answers to these questions cannot be determined without pilot plant studies. The need for pilot studies is not unique to slow sand; rapid rate filtration plants should also be designed with the benefit of pilot plant studies, a fact recognized increasingly during the 1980s. The need is even more imperative for slow sand filters because, as a passive process, slow sand filtration must function as intended after the plant is built. The operator has few options for correcting malfunctions (Leland and Logsdon 1991). To go to full-scale design without a pilot study is economically unsound.

Usually, there is little question about the need for a pilot plant study. The only issue is whether the community can afford to build a pilot plant and conduct the study. Slow sand filtration plants are likely to be built by small communities with strictly limited budgets, and under such circumstances, the funds available may be just sufficient to build the facility. Although communities may hope to bypass the pilot study and design a filtration plant without resolving the critical questions mentioned above, that is not a prudent course of action. Even in communities with a limited budget, a pilot plant study still is a cost-effective means to ensure the successful design of a full-scale plant.

This chapter reviews the phases of a pilot plant study for slow sand filtration. These phases include the study plan, pilot plant construction, the study execution, data handling, data interpretation, and the application of study results to the design at hand. The questions to be addressed by a pilot plant study are reviewed first.
4.1 PURPOSE OF A PILOT PLANT STUDY

Three salient questions of a slow sand pilot plant study relate to the treatability of the raw water, the rate of headloss increase, and design criteria. The pilot study should also include an assessment of when filter maturity will occur and a determination of the removal potential of the filter. In addition, the turbidity-time relation should be analyzed so that the effect of sand washing can be evaluated. These questions are reviewed in the following sections.

4.1.1 Treatability of Raw Water

Treatability has to do with how well the filtration process removes specific contaminants from the raw water source and whether the rate of headloss increase is acceptable. If a filter is biologically mature, there is little question that microbiological organisms will be removed consistently at the 2-log to 4-log level (99 to 99.99 percent), as determined by Bellamy et al. (1985a, 1985b) for Giardia lamblia cysts and coliform bacteria, by Poynter and Slade (1977) for viruses and by Wheeler et al. (1988) for bacteria phages. As indicated by Bellamy (1987), if the filter bed is biologically mature, there is little reason to test the effectiveness of the filtration process for the removal of microbiological contaminants except for the benefit of interested parties. Thus, an objective of a slow sand pilot plant study is to ascertain the length of time needed for the filter bed to become biologically mature. Coliforms are often used as an indicator of filter maturity. These organisms should be removed at 2-log to 4-log when the filter bed is biologically mature; when the filter is started, removals may be less than 1-log and may even be zero.

Although microorganisms will be removed at the 2-log to 4-log level by biologically mature filters, such high removal levels are not assured for turbidity. Cases in which turbidities have not been reduced sufficiently have been those in which particles in the effluent stream were reported to be smaller than 0.5–1 μm (Bellamy et al. 1985a, 1985b; Slezak, pers. com., 1990). Letterman and Cullen (1985) reported on a full-scale slow sand filtration plant in New York that had a source water with turbidity not readily removed. Thus, pilot testing is recommended to verify treatability of the turbidity-causing material in the water.

As noted in Section 2.1.5, the turbidity standard for slow sand filtration plants in the United States is specified by the Surface Water Treatment Rule (SWTR, Federal Register 1989). According to the SWTR, filtered water from these plants should have turbidities under 1 NTU in 95 percent of samples collected, but in no instance may the turbidity exceed 5 NTU. If such a standard cannot be met, the state regulatory agency can permit a
turbidity standard of 5 NTU, providing interference with disinfection is not significant. The standard in Canada was set in 1989 to be ≤1 NTU; in Canada too, a maximum of 5 NTU is permitted if it can be demonstrated that disinfection is not compromised (Federal-Provincial Subcommittee 1989).

Turbidity will change with time in any surface water, and most streams have characteristic seasonal- and event- (e.g., rainfall) related turbidity levels. Pilot testing should continue long enough to cover these periods.

### 4.1.2 Rate of Headloss Increase

Another question concerning treatability is the rate of headloss increase. Every water is unique with respect to the characteristics of the *schmutzdecke* buildup and the consequent headloss. The rate of headloss increase is crucial to the determination of whether slow sand should be selected as the filtration technology and can be analyzed only by pilot plant testing. Figure 4.1 illustrates the two criteria with regard to headloss that must be determined in the pilot plant study: the length of run and the rate of headloss increase. The length of run is the amount of time that elapses from scraping until the headloss exceeds a specific criterion (2.2 m [7.2 ft] in the illustration). In Figure

![Figure 4.1: Headloss Increase With Time From Pilot Plant Testing to Determine Length of Run. Data points are hypothetical and are for the purpose of illustration.](image)
4.1, the rate of headloss increase was 0.034 m/d at time zero and 0.06 m/d at 64 days. As the run progresses beyond 64 days, as shown in Figure 4.1, the marginal benefits of continuing the filter run decrease rapidly. Thus, in the hypothetical case illustrated, a headloss of about 2.2 m would be used for design and a 64-day filter run should be expected.

4.1.3 Design Criteria

Hydraulic loading rate, sand size (d_{10} and UC), and minimum depth of the sand bed are the design parameters of major interest. Accepted criteria are given in Sections 3.1 and 3.2. Pilot testing to investigate these parameters is not needed unless deviation exceeding the upper limits is contemplated.

4.1.4 Filter Bed Maturity

The time period to attain filter bed maturity and the removal potential can be assessed by coliform spiking. The pilot filter should be spiked with coliforms at start-up and then at one-month intervals until removal rates stop increasing. The time required may be several months, depending upon the nutrient concentrations in the ambient raw water. Section 4.2.6 reviews the method for coliform spiking.

4.1.5 Effect of Sand Washing on Filtered Water Turbidity

Sand should be washed before installation in the filter box. To test the turbidity levels caused by residual fines, the turbidity of the effluent should be measured over time. After the sand is washed, the turbidity of the effluent should again be measured to assess the rate of removal of any residual fines still associated with the sand. Measurement of turbidity in the raw water and in the effluent is not expensive, and the testing will permit anticipation of both the role of sand washing and the turbidity removals from the ambient raw water.

4.2 STUDY PLAN

Every pilot plant study requires a work plan in order to ensure a productive testing program. The elements of a generic work plan include statements of purpose, goals, objectives, scope, significance, method, work plan, results expected, budget, and personnel. These elements may take on different names and forms depending on the nature and extent of the study and the established procedures of the organization. For example, some organizations use the term "scope of work" to refer to objectives and scope combined. Others do not refer to purpose (though that is a mistake, since one
should always be clear about why a study is being conducted). Whatever the names
given, and whatever the form given to the work plan, some kind of framework is
needed to address the questions of the study. The discussion here is in terms of generic
work plan elements.

The elements of a generic work plan are elaborated on in Table 4.1. The "question"
column in the table is intended to indicate the nature of each element. The "discussion"
column provides specific guidance on the content of the element.

4.2.1 Purpose

The statement of purpose answers the question, Why undertake a pilot study? The
basic reasons are to remove uncertainties about whether this type of filtration system
should be used, about design criteria, and about expectations during operation.

<table>
<thead>
<tr>
<th>Element</th>
<th>Question</th>
<th>Discussion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Purpose</td>
<td>Why?</td>
<td>State the motivation for the study. Why should the study be done?</td>
</tr>
<tr>
<td>Goals</td>
<td>Where?</td>
<td>Identify the achievements expected as a result of the study. Goals give direction.</td>
</tr>
<tr>
<td>Objectives</td>
<td>What?</td>
<td>State the specific accomplishments that will fulfill the goals.</td>
</tr>
<tr>
<td>Scope</td>
<td>How much?</td>
<td>Limits of the study should be stated so that time and cost constraints can be met.</td>
</tr>
<tr>
<td>Significance</td>
<td>Who cares?</td>
<td>Identify benefits of the study and who will receive those benefits.</td>
</tr>
<tr>
<td>Method</td>
<td>How?</td>
<td>Describe equipment, instruments, procedures, testing plan, data collection, data processing, and so forth, so that the way in which the project is to be accomplished is clear.</td>
</tr>
<tr>
<td>Work plan</td>
<td>When?</td>
<td>Identify tasks and subtasks associated with each category of the method and the expected time for completion. Also assign tasks to specific persons.</td>
</tr>
<tr>
<td>Results expected</td>
<td>Yield?</td>
<td>State the kinds of relationships expected between dependent and independent variables.</td>
</tr>
<tr>
<td>Budget</td>
<td>Cost?</td>
<td>Enumerate expected costs of each expenditure item, grouped into budget categories. Total expenditures must be less than or equal to budget allocated.</td>
</tr>
<tr>
<td>Personnel</td>
<td>Who?</td>
<td>List all personnel and state their roles in the project and their qualifications.</td>
</tr>
<tr>
<td>External review</td>
<td>Deficiency?</td>
<td>Review by others may indicate deficiencies.</td>
</tr>
</tbody>
</table>
These reasons are not, of course, unique to slow sand. Reasons for a pilot study specific to slow sand are to determine if the run length will be of acceptable duration and to determine whether turbidity criteria can be met readily. Once the purpose for a pilot study is clearly understood, goals may be stated.

4.2.2 Goals

Goals are statements of the overall achievements sought. The goals of a slow sand pilot plant study may be among the following: (1) to ascertain whether a given raw water is treatable by slow sand, (2) to develop design criteria, (3) to compare alternative designs, (4) to identify unanticipated seasonal conditions or problems, and (5) to answer questions of regulatory authorities (Bellamy 1987). Achieving the goals will satisfy the purpose.

4.2.3 Objectives

The objectives relate to the question, What is to be achieved? Objectives give resolution to goals and are specific, measurable, attainable accomplishments. For the goals stated above, some objectives of a pilot plant study could be (1) to determine the length of time needed for a filter bed to reach maturity; (2) to determine filter effluent turbidities over the annual cycle of influent turbidities as the filter bed is maturing; (3) to determine filter effluent turbidities over the annual cycle of influent turbidities after the filter bed is mature; (4) to determine the rate of increase in headloss across the filter bed with time due to *schmutzdecke* development; (5) to investigate the influence of design variables, such as hydraulic loading rate, sand size, size distribution, and bed depth, on effluent turbidities; and (6) to determine whether local sand can be used.

4.2.4 Scope

The scope relates to the extensiveness of the study. Among the questions of scope are, "Which objectives should be included? How many variables should be included? To what limits should each variable be tested? For how many months and seasons should testing be continued? Seldom is there enough money to conduct a study that will answer as many questions as one would like, and so the objectives and scope of a pilot plant study need to be delineated carefully. Objectives must be assigned priority, and their execution needs to be planned so that testing is minimal but is adequate to provide reliable answers to the design questions."
4.2.5 Method

The method of a pilot plant study delineates how the objectives are to be accomplished. The first step is to determine the dependent variables and the independent variables. Table 4.2 shows the array of dependent/independent variables that relate to slow sand filtration. Also shown in the table are plausible ranges for the independent variables. A pilot plant study program would consider only those variables that relate to the objectives and scope of the study.

Dependent variables include total coliform bacteria in the effluent, based upon influent spikes at one-month intervals; effluent turbidity, measured daily and based upon ambient influent levels; and headloss, measured daily and subject to ambient influences over the annual cycle. Independent variables include coliform bacteria during spikes; daily turbidity of raw water; and temperature, which is ambient. Other independent variables, such as hydraulic loading rate and sand bed depth, should be fixed at recommended levels, and the sand used should be a type that is locally available and that comes as close as possible to recommended specifications. The reason for the

<table>
<thead>
<tr>
<th>Dependent variable</th>
<th>Independent variable</th>
<th>Ranges for independent variables</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effluent concentrations</td>
<td>Influent concentrations</td>
<td>Not an expected test</td>
</tr>
<tr>
<td>Giardia cysts</td>
<td>Giardia cysts</td>
<td>Spike to (10^6) cfu/100 mL(^a)</td>
</tr>
<tr>
<td>Coliform bacteria</td>
<td>Coliform bacteria</td>
<td>Ambient</td>
</tr>
<tr>
<td>Turbidity</td>
<td>Turbidity</td>
<td>Ambient</td>
</tr>
<tr>
<td>Particles</td>
<td>Particles</td>
<td></td>
</tr>
<tr>
<td>Sand bed maturity</td>
<td></td>
<td>1-12 months</td>
</tr>
<tr>
<td>Age of sand bed</td>
<td></td>
<td>Ambient</td>
</tr>
<tr>
<td>Nutrient concentrations</td>
<td></td>
<td>Ambient</td>
</tr>
<tr>
<td>Temperature</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rate of schmutzdecke development</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hydraulic loading rate</td>
<td>0.04-0.4 m/hr (1-10 mgad)</td>
<td></td>
</tr>
<tr>
<td>Temperature</td>
<td>Ambient range</td>
<td></td>
</tr>
<tr>
<td>Season/month</td>
<td>12 months</td>
<td></td>
</tr>
<tr>
<td>Headloss increase rate</td>
<td>Local</td>
<td></td>
</tr>
<tr>
<td>Sand bed characteristics</td>
<td>Recommended</td>
<td></td>
</tr>
<tr>
<td>Sand size</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sand bed depth</td>
<td>Max. headloss permitted</td>
<td></td>
</tr>
<tr>
<td>Scraping frequency</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\(^a\)The term cfu means "colony forming units."
one-month interval in coliform spiking is to allow time for the sand bed maturity to develop.

(1) Sand Bed Maturity The reason for a coliform spike is to test a filter's removal efficiency as the sand bed progresses toward biological maturity. The removal efficiency at time zero is expected to be 0–25 percent and is expected to increase week by week until a removal rate of 2-log to 4-log is reached. As long as the coliform removal efficiency is increasing, the sand bed biological maturity is developing. When no further change occurs, the sand bed may be considered biologically mature. The length of time for the sand bed to reach maturity depends upon nutrient concentrations in the raw water and upon temperature.

For nutrient-rich warm water conditions, the time for maturity will be short. Bellamy et al. (1985a, 1985b) found that a nutrient-enriched pilot filter using water at 17°C gave 3-log removals after only 13 days and 4-log removals after 40 days. In the same study, a parallel filter, operated at ambient temperature, using a nutrient-poor natural water required 56 days to attain 2-log removals, with removals nominally 1-log for most of the 185-day study. Another parallel filter that was operated with ambient water at 5°C required 57 days to reach removals of about 1-log and showed no consistent improvement afterwards. Barrett (1989) found that about 14 days were required for biological maturity for a pilot filter using nutrient-enriched water at 25°C. During start-up, removals were about 1-log; after the filter became biologically mature, the removals were 3-log to 4-log.

Pilot filters were used at 100 Mile House, BC, to follow the progress of the full-scale plant, which started operation on November 27, 1985. On that same date, one of three pilot filters was spiked with coliform counts of 3,300/100 mL to 1,600,000/100 mL. (Data were presented in Table 3.8.) The filter showed removals of zero percent over the November 27, 1985, 30-hr spike. A second spike, on March 20, 1986, with raw water coliform counts that were about the same as for the previous spike showed effluent coliform counts of nominally 540/100 mL, or about 95 percent removals. Thus, the pilot study showed that for cold, low-nutrient conditions, biological maturity would require several months and when it occurred its performance capacity for high removal efficiency would be less than for the warm, nutrient-rich conditions.

Other means to assess sand bed maturity relate to indicators that significant respiration is occurring. Dissolved oxygen could serve as such an indicator. For a mature filter bed, the reduction in dissolved oxygen concentration between influent and effluent may be 2–4 mg/L. Nutrient concentrations, for example, of nitrogen and phosphorus, could also serve as indicators of sand bed maturity, but guidelines are not available. The bottom line is that definitive guidelines do not exist for assessing sand
bed maturity. However, judgments may be made based upon the experimental evidence noted above. Thus, for 15°C to 25°C water with sufficient nutrients, filter bed maturity will be established within two weeks and the filter bed will probably be fully mature within five weeks. For cold, nutrient-poor water, filter bed maturity requires several months and the level of maturity will be less in terms of log removals than that of the warm, nutrient-rich filter. If more certainty is needed than the foregoing guidelines, coliform spiking is the only sure means to assess maturity.

(2) Turbidity. Turbidity monitoring ascertains whether the slow sand turbidity standard (≤1 NTU or ≤5 NTU in the United States and ≤5 NTU in Canada) can be attained for the range of monthly ambient conditions. If the turbidity standard cannot be met, slow sand should not be used. Pilot testing will ascertain whether a problem is likely.

(3) Headloss. Daily headloss monitoring is necessary in a pilot plant study to predict when scraping will be necessary in a full-scale plant. A run time of 30 days should be acceptable for most plants. Whether a run time of less than 30 days will be acceptable will depend upon local factors. If a short run time occurs only a few times each year, slow sand may be acceptable. If the length of run is consistently too short, slow sand should not be used. Pilot plant testing over the annual cycle is advisable so that the durations of the filter cycles can be ascertained. If none is less than 30 days, slow sand can be selected with little hesitation. If a few are less than 30 days, judgments must be made to determine whether slow sand is a feasible choice.

(4) Other Variables. Whether other independent variables are included in a pilot plant study depends upon local circumstances. If any of the recommended design parameters are to be exceeded, for whatever reasons deemed appropriate, pilot testing can ascertain the effect. All variables except the variable being investigated should be maintained the same as in a control filter. The independent variable being investigated can then be changed systematically so that its effect on selected independent variables can be determined. If one or more of the other variables is allowed to change at the same time, the effect of the variable being investigated may be impossible to discern. If the interest is to examine a design variable such as bed depth or sand size, another pilot filter should be operated in parallel as a control.

4.2.6 Experimental Technique

A pilot plant study to investigate aspects of plant design has limited objectives, and the experimental techniques used should be just adequate to address the objectives of the study. The results sought are more focused in scope than for most research studies. The discussions that follow reflect this philosophy.
(1) Turbidity. Turbidity measurement is simple and easy, but it must be properly done. An important step is instrument calibration with prepared standards. Such prepared standards are available commercially. The correct use of instruments is also important, instruments should be used as specified by the manufacturer. For studies of low turbidity waters, care should be taken to ensure clean sampling bottles. All sampling bottles should be properly labeled with the sampling location, the sample number, the date, the hour, and the name of the person who did the sampling.

(2) Headloss. Headloss measurements should be taken from piezometers with taps located in the headwater and in the tailwater or, alternatively, at the bottom of the gravel support. The two piezometers should be aligned along a common scale. The difference in water surface elevations is the headloss across the surface deposit, the sand bed, and the gravel support. The gravel support headloss is usually negligible, so the initial reading will be the headloss across the sand bed only. With this measurement and the corresponding temperature, the intrinsic hydraulic conductivity of the sand bed, k', can be determined as described in Section 1.3.2. Headloss should be plotted with time. The increasing headloss over the filter cycle is due to the buildup of the schmutzdecke. As noted previously, the headloss rate is a function of raw water quality. From headloss studies, the duration of the filter cycle between scrapings can be determined.

(3) Temperature. Water temperature should be measured in the headwater above the sand bed. The thermometer should be able to be read from outside the filter column, with sensor located in-situ within the headwater. The thermometer should be calibrated with a laboratory-grade mercury thermometer.

(4) Coliform Spiking. Ideally, pure cultures should be used for coliform spiking. Sewage has coliform levels of about $10^9 \text{ cfu}/100 \text{ mL}$ and therefore is a usual source of coliforms for experimental use. Caution is warranted, however; in highly concentrated suspensions, the debris and fats present can interfere with the development process and result in a different type of schmutzdecke than that obtained when ambient water is used. The coliform spike should be mixed into the headwater above the sand bed without disturbing the sand bed surface. The spike can be added continuously over a 24-hour period or it can be imposed as a single spike of the headwater. For a continuous spike, a positive displacement pump must be used to feed the suspension. To maintain the spike over a 24-hour period, a plastic tank with a stirrer should be used. The volume of the tank should be equal to the flow times the desired elapsed time. The coliform concentrate suspension should be maintained at a cold temperature.

Sampling should be done from the headwater and from the effluent at about 30-minute intervals for the period of the spike and for a long enough time afterward for
the organisms making up the spike to have worked their way down the filter bed. The sampling can be done using sterile plastic bags that are commercially available. After sampling, the bags should be placed under ice. Ideally, the analysis should be done immediately. In any case, the time elapsed should not exceed 24 hours if Standard Methods (1989) is used for coliform enumeration. Because the interest is in coliforms associated with the volume of sample (that is, coliform densities in cfu/100 mL), the membrane filter technique is the method of choice.

4.3 PILOT PLANT CONSTRUCTION

Figure 4.2 shows a pilot plant set-up similar to the one used at Colorado State University and to those adapted for other studies (Bryck et al. 1987; Barrett 1989). The pilot plant is merely a cylinder (or pipe) that holds the gravel support, the sand, and the headwater. The main features are a cylinder to hold the media, a steady flow delivery, tailwater elevation control, valves and tubing, piezometers, and a means of measuring the flow. These components are described in the sections that follow.

4.3.1 Media Cylinder

The media cylinder should be about 4 m (13 ft) high, so that it is long enough to hold the gravel support, the sand bed, and the headwater. The diameter is not critical from a process standpoint, but a 30.5-cm (12-in.) diameter cylinder is easier to use than is a cylinder of smaller diameter. For cylinder material, an SC200 PVC pipe works well, as its walls are easy to tap. Other materials can be used, however, such as concrete culvert pipe and other noncorrosive materials. The cylinder may be pressurized if the pilot plant is to be located in a room with limited ceiling height.

Sidewall effects, i.e., the short-circuiting of flow along the walls of the sand bed, should not be a problem with a 30.5-cm (12-in.) diameter pipe. The published literature concerning sidewall effects was reviewed by Bellamy et al. (1985b), who found that although there had been no definitive study, those in the field seemed to be near consensus that any tube over 3–5 cm in diameter should not have sidewall effects (for most filter sands). From their conclusion, we can deduce that a 30.5-cm (12-in.) diameter pipe may be used with little concern about sidewall effects. (We are not, however, saying that a smaller diameter pipe is recommended.) Bellamy et al. (1985b) coated the inside walls of the pilot plant cylinders constructed for their experimental research with contact cement and then poured sand over them; the surface resulting could not have had wall effects.
A constant-head overflow cup should be used to collect the discharge from a pilot plant for safe disposal, and clear plastic piezometers should be positioned to allow for easy head measurements. In addition, a hole should be drilled in the side of the cylinder above the sand bed for the insertion of a thermometer (through a rubber stopper). The support plate for the gravel should be composed of stainless steel or plastic, and a coarse
mesh screen should be placed on the plate. The bottom of the cylinder should be a blind flange, with effluent lines collecting water by means of a threaded hole in the bottom of the flange. The cylinder should be raised 10–15 cm (4–6 in.) above the floor so that outlet tubes can extend from the bottom plate.

4.3.2 Delivery of Flow

The flow to a slow sand filter pilot plant must be "steady"—that is, it must not change with time—and it must be delivered at a specified rate. Two methods are available to achieve these conditions. The first method is to use a positive displacement metering pump. For the operation of a single pilot plant, this type of metering pump will be the easier method to use, will be a low-cost solution, and will provide the needed accuracy and reliability. Metering pumps can be ordered from most laboratory supply houses. The main concern is to purchase a pump that will handle the flow range needed for the pilot plant. For example, a pilot plant that has a media cylinder with an inside diameter of 29.2 cm (11.5 in.) and that is operated at an HLR of 0.4 m/hr (10 mgad) would require a flow delivery of 7.46 mL/s (0.002 gal/s) and should be adjustable to any lesser flow.

The second method of flow regulation is to use a constant-head orifice box (Cleasby et al. 1984b). Figure 4.3 is a photograph of an orifice box set up for experimental work at

Figure 4.3 Constant-Head Orifice Box for Metering Flow to a Slow Sand Filter. (Photograph by David Hendricks.)
Empire, CO, and used in studies by Barrett (1989). Figure 4.4 is a drawing showing details for construction of the orifice box shown in the photograph. The box has an orifice with a diameter of 2.2 mm (0.086 in.) and an overflow tube that serves as a weir. The flow should be set for a specified head and then adjusted to give the specified flow by screwing the brass overflow tube up or down as needed. The box has a screen to reduce the clogging frequency of the small orifice. The flow from the orifice is caught by an exterior box that is connected to the flow delivery to the pilot plant. The constant-head box can be used with a small centrifugal pump or a gravity feed, as the excess flow is discharged from the box by the adjustable overflow weir. The constant-head box is recommended for flow regulation when several pilot plants are to be operated at the same time (as in the studies by Bellamy et al. [1985b]), but this method will also work well with a single pilot plant.

Figure 4.4 Drawing of Constant-Head Orifice Box for Metering Flow to a Slow Sand Pilot Filter, Showing Dimensions for Construction

Note: 8.7 cm head on 0.22 cm orifice gives flow of 255 mL/min (HLR = 0.21 m/hr for 29.2 cm i.d. tube)
4.3.3 Tailwater Control

The tailwater elevation is controlled by an open break in the effluent line that should occur at the level of the top of the sand bed or higher. An overflow cup—a circular overflow weir with external collection—provides a measurable tailwater elevation. A common mistake is to let the effluent pipe run freely from the base of the filter; such a set-up will cause negative pressures within the filter bed, and air-binding will result (see Example 3.7). A less satisfactory alternative to an overflow cup is to use a large-diameter pipe with a U-bend at the elevation of the top of the sand bed. The U-bend should have a hole in the top to ensure that an open surface occurs.

4.3.4 Valves and Tubing

The valves and tubing should be made of PVC or other hard plastic. Valves can be the on/off ball type. Only two valves are needed in the pilot plant. They are located at the effluent T-connection to provide for drainage of the column. Flexible plastic tubing should not be used because its surface can develop a microbiological film and be a source of heterotrophic bacteria. This problem can also occur with hard plastic, but to a lesser extent. The bacteria problem can be controlled by injecting a 5 percent sodium hypochlorite (bleach) solution into the effluent tubing, being careful not to permit the chlorine to enter the gravel support.

4.3.5 Piezometers

Piezometers should be located in the headwater and within the gravel support layer just below the sand bed. The headloss through the gravel support layer and the effluent piping will be negligible compared with the headloss in the sand bed. The piezometers should be clear plastic and should have a diameter of ≥ 1–2 cm (0.5–0.8 in.) to minimize capillary effect. The tubes should be set up side by side, with an easily readable scale located behind them. An alternative to using piezometers is to make one careful measurement of the tailwater elevation and then measure the inside water surface as it rises day by day (see the case study, Section 4.7). Piezometers are, however, more convenient. The flow should be steady for a given test.

4.3.6 Energy Dissipation of Influent Flow

The delivery of flow to the pilot plant is designed to minimize surface erosion of the sand bed by submerging the effluent jet from the influent tubing with consequent dissipation of the turbulent energy from the jet occurring within the headwater. To control the headwater elevation, the tailwater overflow cup should be located so that the tailwater elevation is a few centimeters above the surface of the sand bed. A higher
tailwater elevation should be used initially during a test run, as in a full-scale filter, to provide a cushion for dissipating the turbulence of the flow delivery. The overflow cup should be vertically movable so that the water within the tube can be lowered to about 2–3 cm below the sand surface for scraping.

4.3.7 Flow Measurement

Flow should be measured volumetrically using a 1,000-mL cylinder and a stopwatch. Such measurements should be done periodically on the influent side to ensure that the metering system for the influent flow is operating as intended. A rotometer, calibrated volumetrically, may be used to monitor flow. The use of a rotometer does not replace volumetric measurements but may reduce the number of such measurements.

4.4 DATA HANDLING AND ANALYSIS

Data collection and analysis forms should be structured to facilitate subsequent analyses and interpretation. The person-hours spent on analyses of data often exceed the investment in generating data. Data forms that are designed with forethought will aid these analyses and will permit these tasks to be accomplished quickly relative to the alternative of no design. At the same time, the necessary documentation will be in place. This section reviews the principles for designing both data collection forms and data processing forms. A well-designed form can take advantage of computer software technology to store, process, analyze, and archive data.

4.4.1 Data Forms

Data collection forms must be self-explanatory and provide passive direction to those obtaining measurements. Figure 4.5 illustrates the type of form needed for a basic pilot plant study. The form should be flexible in design so that it can be easily modified to accommodate the situation at hand. For example, line 1.6 in the figure, "Person taking data," and line 1.7, "Notes," could be changed to column entries under "Variable Data" if different persons were to obtain data during the testing period and if comments were to be noted each day. The form may be modified also to include a column for a meter reading under "Flow Measurement." Additional columns should be added as needed for whatever measurements are of interest. The main point is to design a form that is simple and easy to understand. An ancillary diagram showing a flow schematic with the points of measurement indicated could be added to the form to minimize misunderstandings. Data collection forms should utilize computer spreadsheet software technology; however, if that technology is not available, the same principles would hold
Data Sheet
Pilot Plant Testing Program to Evaluate Feasibility of Slow Sand Filtration
Town of Beaver Falls, Colorado

1. Fixed Data:
1.1 Source water: Beaver Creek
1.2 Sand: d_{10} = 0.25 mm
1.3 Bed depth = 133 cm
1.4 Col. I.D. = 29.2 cm
1.5 HLR = 0.40 m/hr (set)
1.6 Person taking data: John Smith
1.7 Notes: Coliform spike on 01/15

2. Variable Data (page 1 of 2):

<table>
<thead>
<tr>
<th>Run No.</th>
<th>Date</th>
<th>Time (hours)</th>
<th>Temp. (°C)</th>
<th>Flow Measurement Volume (mL)</th>
<th>Piezometers (Tailwater Headwater (cm))</th>
<th>Turbidity Influent Effluent (NTU)</th>
<th>Coliform Samples (cfu/100 mL) Influent Effluent</th>
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</tr>
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<td>3.5, 0.45</td>
<td>10,000, 9,200</td>
</tr>
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</tr>
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<td>1,000</td>
<td>100.5, 154.0</td>
<td>4.5, 0.35</td>
<td>10,000, 9,200</td>
</tr>
</tbody>
</table>

*This section would have as many rows as needed to cover the time period of the pilot plant operation.

Figure 4.5 Sample Data Collection Form for Slow Sand Filter Pilot Study. The town, firm, and data entries in this figure are fictitious. Data (simulated in this example), usually entered by hand, are shown in italics.

for pencil-generated forms. Example 4.1 illustrates some of the considerations in developing a data collection form.

**Example 4.1: Design of Data Collection Form.** The hypothetical town of Beaver Falls, CO, has engaged a consulting firm to solve its drinking water problem, and slow sand filtration is being considered. The consultant has apprised the town of the need for pilot plant studies regardless of the choice of technology, and the town has agreed. Design a data collection sheet for a pilot plant study.

1. **Context:** The water source is Beaver Creek, a mountain stream that has turbidity levels of about 4-6 NTU most of the year, with occurrences of 20-50 during spring runoff in May and June. The water source is contaminated with *Giardia* cysts; VOCs (volatile organic chemicals) and SOCs (synthetic organic chemicals) are below the detection limit. The color is about 15 units during the summer months, and the associated humic acids cause about 50 µg/L trihalomethanes when the water is chlorinated. If slow sand is used, the sand will come from a local source and will be washed prior to placement. The sand bed depth will be set at 1.33 m (4 ft) for longer bed life.
The hydraulic loading rate of 0.4 m/hr (10 mgd) will be imposed. This HLR will be used during winter months when the town uses about 2,500 L/c/d (650 gpcd) to prevent pipes from freezing.

2. **Determine objectives:** The objectives of the pilot plant study will be to (1) determine whether the effluent turbidity will meet standards, (2) ascertain the rate of headloss increase, and (3) determine when the filter will be biologically mature. Also, the town wishes to ascertain whether the HLR of 0.4 m/hr, which is the high end of the recommended range, will pose any problem when the filter is biologically mature. Because THMs meet existing standards, they will not be investigated. If a question develops later, THM removal rates can be ascertained by studies in the full-scale plant.

3. **Method:** A single pilot plant will be set up, similar to the one shown in Figure 4.2. The dependent variables include effluent turbidity, effluent coliform concentration, and headloss. Independent variables include influent turbidity, influent coliform bacteria concentration, headloss across the sand bed, HLR, temperature, sand size, sand bed depth, time from start of operation, time from scraping event, and season.

4. **Selection of variables for study:** Effluent turbidity, effluent coliforms, and headloss are the dependent variables, with influent turbidity, influent coliforms, and elapsed time the respective independent variables. The influent turbidity will be ambient, and the influent coliforms will be imposed as spikes of about 10,000 cfu/100 mL maintained over 24-hour periods, with spikes at start-up and monthly thereafter. Temperature will be ambient, as will other water quality characteristics, which will not be measured. Design variables such as sand size and sand bed depth will be kept constant except when bed depth is diminished by scraping, another independent variable. The HLR will be maintained at a constant 0.4 m/hr.

5. **Data sheet:** The data sheet should be easy to use for the person conducting the testing program. It should be self-explanatory, not overwhelming, and not too large in size so that it is easy to handle in the field. The format of the sheet should be easily transcribable into a data processing format. Figure 4.5 is a sample data sheet that would meet most of the objectives of the study.

6. **Discussion:** The data sheet shown in Figure 4.5 should be modified to include a column for the flow meter reading. Although volumetric flow measurement is most accurate and is recommended, the hourly monitoring should utilize a flow meter previously calibrated. If the study includes the investigation of other variables, they should appear as additional columns under "Variable Data" on the data sheet.

4.4.2 Data Processing

A convenient way to process any amount of data is by means of a spreadsheet that holds transcribed original data and processed data all in one place (see Figure 4.6). One name for such a spreadsheet is a "master" data table. From such a table, specific data can be extracted as needed and graphical relationships can be constructed. Although in principle all of this can be done by hand, the use of readily available computer software facilitates the process. The first step is to design the format for the spreadsheet.
Data Processing Sheet
Pilot Plant Testing Program to Evaluate Feasibility of Slow Sand Filtration
Town of Beaver Falls, Colorado

1. Fixed Data:
   1.1 Source water: Beaver Creek
   1.2 Sand: $d_{10} = 0.25 \text{ mm}$
   1.3 Bed depth = 133 \text{ cm}
   1.4 Col. I.D. = 29.2 \text{ cm}
   1.5 HLR = 0.40 \text{ m/hr (set)}
   1.6 Person taking data: John Smith
   1.7 Notes: Coliform spike on 01/15

2. Variable Data (page 1 of 2):

<table>
<thead>
<tr>
<th>Run No.</th>
<th>Date</th>
<th>Time (hours)</th>
<th>Elapsed Time (hours)</th>
<th>Temp. (°C)</th>
<th>Flow Measurement Volume Time (mL/s)</th>
<th>Flow Q (mL/s)</th>
<th>HLR (m/hr)</th>
<th>Piezometers (cm)</th>
<th>Headloss (m)</th>
<th>k' (N/m)</th>
<th>Turbidity Influent (NTU)</th>
<th>Turbidity Effluent (NTU)</th>
<th>Coliform Samples (cfu/100 mL)</th>
</tr>
</thead>
</table>
   | 1       | 01/15/90  | 0800         | 0.0                  | 1.0        | 1,000 134                            | 7.46          | 100.5      | 151.2           | 50.7          | 5.05x10^-7 | 4.0                      | 0.40                     | 11,000 10
   |         |           | 0830         | 0.5                  |            |                                     |               |            |                 |               |          |                          |                          | 13,000 9,000
   |         |           | 0900         | 1.0                  |            |                                     |               |            |                 |               |          |                          |                          | 10,000 8,000
   |         |           | 0930         | 1.5                  |            |                                     |               |            |                 |               |          |                          |                          | 11,500 8,500
   |         |           | 1000         | 2.0                  |            |                                     |               |            |                 |               |          |                          |                          | 11,000 7,500
   | 01/16/90| 0800      | 24.0         | 0.5                  | 1,000      | 136                                 | 7.35          | 100.5      | 152.5           | 52.0          | 3.5       | 0.45                     |                          | 10,000 9,200
   | 01/17/90| 0800      | 48.0         | 1.0                  | 1,000      | 133                                 | 7.52          | 100.5      | 153.3           | 52.8          | 4.0       | 0.55                     |                          | 10,000 9,200
   | 01/18/90| 0800      | 72.0         | 0.5                  | 1,000      | 137                                 | 7.30          | 100.5      | 154.0           | 53.5          | 4.5       | 0.35                     |                          | 10,000 9,200

*This form is the same as that in Figure 4.5 but columns showing calculated data have been added. These columns include "Elapsed Time," "Flow," "HLR," "Headloss," and "k'." The calculations should be set up as formulas in the software program, for example, $Q = \text{Col 6/Col 7}$.  

Figure 4.6 Sample Data Processing Form for Slow Sand Filter Pilot Study. The town, firm, and data entries in this figure are fictitious. Data (simulated in this example), usually entered by hand, are shown in italics.
Example 4.2: Data Processing Spreadsheet Design. Design a data processing spreadsheet for a pilot plant study for the hypothetical town of Beaver Falls, CO.

1. **Delineate the form in which the data should appear when processed:** The task of data processing is to transform raw data to a usable form. For example, piezometer readings should appear as headloss. At the same time, the consolidation of all data, raw and processed, in a single spreadsheet facilitates data analysis. When data are in this tabular form, plots can be constructed easily by importing the appropriate columns into a compatible graphics package. (Alternatively, graphs can be constructed by hand.)

Figure 4.6 illustrates the next step, showing how the raw data from Figure 4.5 can be transcribed into a spreadsheet. Actually, the design should be in reverse (that is, the analysis spreadsheet should be designed before the data collection form). Based upon the objectives of the pilot plant study, one should first determine what plots are needed. Then, based upon such plots, what should be the kinds of processed data needed? And from the processed data, what raw data are required from the pilot plant operation? For example, to determine run length, a headloss versus time plot is required. The plot is constructed from the headloss and time columns of Figure 4.6. These columns are calculated from the data transcribed from Figure 4.5. Thus, the data collection form in Figure 4.5 should be the last in the process of designing a data handling protocol.

2. **Process the data into plots:** As noted, the next step is to construct plots of the processed data that will fulfill the objectives of the study. For example, a headloss-time curve permits determination of the run time. The influent/effluent turbidity-time curve permits determination of whether the effluent turbidity standards will be met. And the coliform-time plots permit ascertaining when the sand bed reaches biological maturity. The case study in Section 4.7 shows plots from pilot plant data at 100 Mile House, BC, and illustrates the end point of the pilot plant data analysis.

   The final step is to make judgments and decisions that fulfill the goals of the study.

### 4.5 DATA INTERPRETATION

Plots are the end point in data interpretation. As a general rule, all tables and graphics should be self-explanatory. The conclusions should be evident from the graphs. For example, in the headloss-time plot, maximum headloss should be shown as a horizontal line. The length of run will then be immediately evident as the abscissa distance to the intersection of the headloss curve with the maximum headloss line. Another aid to data interpretation is to include the turbidity standard on turbidity-time plots; with this information, the operator can tell immediately if the effluent turbidity exceeds the standard.

### 4.6 APPLICATION OF PILOT PLANT STUDY RESULTS

The first objective of pilot plant studies is to ascertain run length during normal and critical periods of water quality; the purpose is to provide information to aid in the decision on whether to use slow sand. The second objective is to determine whether effluent turbidity meets the standards of the regulatory agency. Again the purpose is to
provide data for an informed decision on whether to use slow sand. Only pilot plant testing will provide this knowledge.

After the basic questions have been addressed, or perhaps in conjunction, design questions may be of interest. For example, can a specific type of sand be used that deviates from established norms for $d_{10}$ and UC? Will the filtration system work effectively if a higher-than-recommended hydraulic loading rate is used? What will be the effect of using a deeper-than-usual filter box? If extended questions such as these are to be investigated, then it is useful to ascertain when biological maturity will occur. The filter bed will be effective in filtration only after the biofilm within the sand bed begins to develop. As discussed in Section 4.2.6, coliform testing will provide an indication of when the sand bed begins to develop its biological maturity.

When extended questions are of interest, the cost of pilot plant studies increases markedly, as the testing program becomes more sophisticated and tends toward an investigation. Also, the personnel require more training and the analyses become more costly. These kinds of studies may be too costly for a small community, albeit they may be desirable. As more fundamental knowledge is developed about mechanisms involved in the slow sand process, the need for some kinds of extended testing will be reduced.

4.7 CASE STUDY

A classic pilot plant study by Dayton & Knight, Ltd. (1983), compared the effectiveness and economics of rapid rate, diatomaceous earth, and slow sand for the Village of 100 Mile House, BC. Slow sand was selected for the full-scale plant because of anticipated lower operating costs, although the capital cost for slow sand would be slightly higher than that for rapid rate or diatomaceous earth. A review of their report (Dayton & Knight, Ltd., 1983) will reveal how a slow sand pilot plant study conducted at minimal cost can provide answers to the critical questions of effluent turbidity and length of run.

4.7.1 Context

In 1981, the Village of 100 Mile House had 60 confirmed cases of giardiasis attributed to the unfiltered, chlorinated water supply from Bridge Creek. After considering rapid rate filtration and conducting a cost estimate in 1982, Dayton & Knight, Ltd., examined also diatomaceous earth and slow sand. The firm subsequently recommended to the Village that a pilot plant study be conducted to compare the three basic filtration technologies. Study data would be used to select the technology while, at the same time,
the pilot plant would be effectively removing *Giardia* cysts. The study was conducted during the months of July to October 1983. The slow sand portion of the study is reviewed here.

### 4.7.2 Pilot Plant

Figure 4.7 includes a photograph and section drawing of the 1983 pilot plant set-up at 100 Mile House, BC. The photograph illustrates the simplicity of the set-up, which had only an influent pipeline of raw water, a filtration tube of precast concrete pipe 105 cm (42 in.) in diameter, and a 10-cm (4.0-in.) PVC drainpipe. The section drawing shows the details. As seen, the sand bed was supported by three layers of graded gravel (top layer, 10 cm with \(d_{10} = 0.6\) mm; middle layer, 10 cm with \(d_{10} = 5\) mm; bottom layer, 25 cm with \(d_{10} = 15\) mm). The 10-cm-diameter PVC drainpipe was perforated in the portion that drained the bottom gravel layer, and the bend in the pipe served as a weir. With such a set-up, the pipe should have a 2-3 cm (1 in.) hole drilled in the top of the bend to ensure (and verify) an air break for the tailwater control. The headwater level can be measured by a scale attached to the inside wall above the sand bed and to the top of the filtration tube, although the accuracy will not be as high as with piezometers. Also, a valve should

![Figure 4.7 Photograph and Section Drawing of Pilot Plant Set-up at 100 Mile House, BC. The top layer of the gravel support was 10 cm deep with \(d_{10} = 0.6\) mm; the middle layer was 10 cm deep with \(d_{10} = 5\) mm; the bottom layer was 25 cm deep with \(d_{10} = 15\) mm. The effluent was collected from a perforated 10.0-cm (4.0-in.) PVC drainpipe in the bottom layer of gravel. (Reprinted with permission from Dayton & Knight, Ltd., 1983.)](image-url)
be attached to the lower part of the effluent pipe to drain the column. This type of pilot plant set-up is easy to construct and is inexpensive because it uses materials at hand.

4.7.3 Operation

A pilot plant installation such as the one shown in Figure 4.7 can be operated by local personnel. The only requirements of the personnel at the 100 Mile House pilot plant were daily headwater measurement and daily sampling of the influent and effluent, followed by turbidity measurement and data recording. In obtaining the headloss data, only the water level inside the pipe was measured; the tailwater elevation was fixed at 150 cm. Because algae was considered the influence most likely to cause headloss problems with slow sand, the 100 Mile House study was conducted during the summer; the four-month period of operation was considered adequate. With properly designed data forms (as described in Section 4.4.1) and with a training period for personnel, pilot plant studies such as that at 100 Mile House should be successful.

4.7.4 Results

Figures 4.8 and 4.9 show headloss and influent/effluent turbidities, respectively, for the first cycle of operation before scraping at the 100 Mile House pilot plant. The maximum headloss possible, based upon the height of the column, was 200 cm. Figure 4.8 shows that the run time with 200-cm headloss would be about 33 days (extrapolating the curve), which was deemed acceptable. The study also sought to determine whether

Figure 4.8 Headloss-Time Plot for First Filtration Cycle, Pilot Plant at 100 Mile House, BC, July 18 to August 17, 1983. (Adapted from data from Dayton & Knight, Ltd. [1983].)
Figure 4.9  Influent and Effluent Turbidity Versus Time for First Filtration Cycle, Pilot Plant at 100 Mile House, BC, July 18 to August 17, 1983. (Adapted from data from Dayton & Knight, Ltd. [1983].)

An effluent turbidity of <0.5 NTU could be achieved. Figure 4.9 answers that question, showing effluent turbidities in the range of <0.1-0.5 for influent turbidities of 1-4 NTU.

4.7.5 Discussion

Several conditions of the 100 Mile House pilot plant study should lend insight to others planning pilot studies. First, the pilot plant set-up was improvised from materials on hand, keeping expenses low and avoiding the need to wait for supplies. Second, local persons were trained to make the needed measurements of headwater elevation and turbidity. Third, the pilot plant study was simple and its scope was limited, designed to answer only the critical questions of run time and effluent turbidity. Fourth, the duration of the study was only four months, yet the study covered the most adverse raw water quality problem—algae growth.

Based upon the success of the 100 Mile House study, a small community should not feel that a pilot plant study will be beyond their means. Nor should they be in a hurry to expedite construction of full-scale plants without adequate design data at hand. Financing authorities should not force the communities into untenable positions by placing time conditions on the use of funds; rather, the interest should be in the design of a facility that makes cost-effective use of funds. The consultant's job is to make the case for a pilot plant study. The alternative is to proceed with uncertainty.
Chapter 5

Construction

Slow sand filter construction can usually be accomplished by a local contractor using local materials and labor. Although the initial capital costs are generally higher than for "package" water treatment systems, with the construction of slow sand plants, more of the expenditure generally goes back to the local economy. Even though the capital costs may appear high, the annual operating costs are consistently low.

If a plant is to be constructed well, the contractor must have a clear and concise set of construction documents to bid from and follow. Further, the construction must be done in a quality manner in accordance with the documents.

5.1 ESTIMATING CONSTRUCTION COST

The engineer's final cost estimate is made when the design is complete, or nearly complete. The project is defined in great detail at this point, and the estimate is made on the basis of detailed quantities for the materials to be used and from supplier quotes. Due to having no control over the cost of labor, materials, or equipment, or over the contractor's methods of determining prices, or over competitive bidding or market conditions, the engineer's opinions of cost can only be made on the basis of experience and qualifications and represent the engineer's best judgment as a design professional familiar with the construction industry. The cost estimates for each slow sand filter must be based upon current local conditions.

The engineer's final estimate can be significantly different from estimates completed before the design was fully developed. If the estimate is higher than the owner is prepared to fund, the design may be revised to reduce costs or, preferably, construction bidding may be delayed until additional funding can be obtained. When the construction contract is bid, no further changes may be made in the design.
A systematic approach to estimating cost is very important. As a first step, material quantities must be enumerated in detail from the design drawings. Quantity takeoffs should clearly state the quantity and type of material being estimated.

The next step is to assign a cost to all of the items. A sample costing form is shown in Figure 5.1. The "Item" and "Page" columns refer to the engineer's quantity takeoff. The next column is used to describe the item being estimated, for example, PVC pipe. The remaining columns are used to calculate costs.

Unit material and labor costs can be found in published cost estimating manuals or they can be provided by local contractors. The manuals will provide regional costs; the contractors will provide costs specific to the site of the project. Two of the most commonly used manuals are the *Dodge Construction Cost Information System* manual (1988) and the *Means Mechanical Cost Data* manual (1988).

<table>
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<td>5</td>
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Figure 5.1 Sample Costing Form
After costs have been estimated for all items, the costs are totalled and a contingency is added to cover unknowns. This contingency usually is about 10 percent. The total is the amount that should be budgeted to pay the contractor.

Other costs that need to be budgeted are the costs of construction administration and resident engineering, responsibilities that are usually carried out by the engineer. Additional costs could include land acquisition, if the site is not already owned, and the costs for easements. The total project cost is the cost of construction plus all of the other costs.

5.2 FINANCING

The cost of construction is financed through bonds or loans and sometimes through grants. Water rates generate the revenues to pay the financing, operation, and maintenance costs. Table 5.1 lists publications from the American Water Works Association that are useful for developing water rates.

Though federal grants or loans generally are not available for the construction of community water treatment facilities, a number of state agencies can provide financial assistance in the form of loans and/or grants to eligible recipients. (The agencies vary from state to state. The appropriate state agency should be contacted regarding the availability of funds in that state.) In Colorado, for example, agencies providing funding for the construction of water treatment facilities include the following (Downs 1989):

1. The Energy Impact Assistance Fund, which provides grants and loans to a maximum amount of $300,000 and encourages local cash participation
2. The Community Development Block Grant, which provides grants to a maximum of $500,000, requires local cash participation, and benefits low- and moderate-income communities

Table 5.1
American Water Works Association Publications on Water Rates

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<td>The Rate Making Process: Going Beyond the Cost of Service</td>
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<td>Water Rates and Related Charges (Manual #26)</td>
</tr>
</tbody>
</table>
3. Colorado Water Resources and Power Development Authority, which provides loans of up to $10,000,000

These three agencies have either annual or ongoing funding cycles. The terms for loans are different at each agency, and the agencies stipulate eligibility. Although the specific sources of funds will vary from state to state and from country to country, the agencies listed for Colorado are indicative of the kinds of funding sources that should be explored.

The U.S. Farmers Home Administration provides grants and loans, based on need and on the availability of funds. Eligible recipients include public and nonprofit facilities, Indian tribes, and municipalities with populations under 10,000.

5.3 CONSTRUCTION SCHEDULE

The local climate will affect the construction schedule more than any other factor. In northern climates, surveying and geotechnical investigations should be done in the fall before the weather gets bad. Design and bidding can be done during the winter for a construction start in the spring. The following statements provide general guidelines for making schedule decisions.

1. The project can be bid at any time of the year. If the construction season is short, time can be saved by bidding during the off-season. A typical bidding period is 30 days.
2. Design drawing submittals can be reviewed during the off-season, prior to commencing construction.
3. Construction should commence during a period suitable for excavation and backfilling. Backfilling should not be done when the earth is frozen or when its moisture content is excessive. Costs will increase if concrete must be formed and poured in below-freezing conditions (as protection from freezing is needed). Because concrete work constitutes the largest cost, construction should commence when temperature conditions are proper for pouring.
4. Allow at least 45 days for plant start-up and testing after construction is completed.

5.4 CONTRACT DOCUMENTS

Contract documents are prepared in order to secure competitive bids. Prior to the execution of the contract between the owner (for example, the town council or the water
board) and the construction contractor, these documents are referred to as bidding documents. Standardized contract documents are generally used because (1) their language has stood the test of time; (2) legal decisions have been based on them; (3) engineers, contractors, and owners are familiar with their terms and content; (4) their standard wording helps to avoid omissions; and (5) time can be saved when they are used. The following sections describe contract documents and their content.

5.4.1 Bidding Information

Documents relating to bidding include: (1) Legal Notice and Invitation to Bid, (2) Instructions to Bidders, (3) Bidder's Qualification Statement, and (4) Bid Bond. These documents are described briefly in the following paragraphs.

(1) **Legal Notice and Invitation to Bid.** The Legal Notice and Invitation to Bid describes the project and provides basic information on how to bid. It is usually published in the legal notices section of local and regional newspapers and in trade journals. Items normally covered in the legal notice are:

1. The date, time, and place where bids will be received
2. A brief description of the project
3. The location at which bidding documents can be obtained
4. The cost of bidding documents
5. The period during which bids will remain valid and the required construction time limit
6. Bond information

(2) **Instructions to Bidders.** The Instructions to Bidders document gives the requirements for the preparation and submission of bids. Items normally included are:

1. A definition of terms
2. Qualifications required of bidders
3. Information about the site and access to it
4. The location at which subsurface information is available
5. Notice of a pre-bid conference
6. An interpretation of the bidding documents
7. Any bid security required
8. Contract time
9. Liquidated damages
10. Instructions for completing the bid form
11. Instructions for submitting bids
12. The procedures for modifying and withdrawing bids
13. Information about how bids will be opened
14. The basis for the award of the contract
15. Information about how contract documents will be processed

(3) **Bidder Qualification Statement.** The Bidder's Qualification Statement form, which is filled out by each bidder, contains information pertinent to the bidding company's qualifications to perform the work.

(4) **Bid Bond.** The Bid Bond, which is usually a percentage of the bid submitted by a company, serves as a guarantee that if the company's bid is accepted, that company will sign the contract and furnish the required performance and payment bonds.

5.4.2 **Bid Form**

The Bid Form is used by each bidder to submit its bid. The bidding company defines the bidding elements and alternatives and, where designated on the form, acknowledges the receipt of each addendum (if any were issued).

5.4.3 **Agreement Form**

The Agreement Form is signed by the contractor and the owner and is legally binding on both parties to do the work. Most often standard forms, published by various organizations, are used. The contract documents are attached to the agreement and made a part thereof. The contract documents usually consist of the following, as a minimum:

1. Performance Bond
2. Labor and Material Payment Bond
3. Notice of Award
4. Notice to Proceed
5. General Conditions
6. Supplementary Conditions
7. Specifications
8. Drawings
9. Addenda
10. Contractor's Bid

The bonds and notices are usually standard forms. The "general conditions" define the basic rights, responsibilities, and relationships of the parties involved in the
construction process. The most commonly used form is the "Standard General Conditions of the Construction Contract" that is prepared by the Engineers' Joint Contract Document Committee. This document has been approved and endorsed by the Associated General Contractors of America.

The "supplementary conditions" amend or supplement the general conditions and other provisions of the contract documents.

Addenda are instructions issued during the bidding process that modify the original bidding documents or previous addenda. The addenda must be sent to each bidder prior to the bid opening.

5.4.4 Specifications

Specifications are the part of the contract documents that contain the detailed technical provisions that relate to the construction or installation of the various parts of the work and to the materials used in the work. The specifications describe in words the construction of the project, which is also illustrated in the drawings submitted as part of the contract documents. The specifications describe in detail the nature of the materials and equipment to be used in the work and the quality, workmanship, and procedures to be used in constructing the project. The most common format for specifications is that of the Construction Specifications Institute (CSI). Information regarding CSI can be obtained by contacting The Construction Specifications Institute (601 Madison Street, Alexandria, VA 22314) or Construction Specifications Canada (1 St. Clair Avenue West, Suite 1206, Toronto, Ontario M4V 1K6). In general, the basic philosophy of the CSI concept is to establish a standard location for information and to state that information in only one location. This concept avoids confusion and repetition. The key elements and subdivisions of the CSI format are outlined below.

1. All work is classified under 1 of 16 divisions. These divisions are:

   1-General Requirements
   2-Sitework
   3-Concrete
   4-Masonry
   5-Metals
   6-Wood and Plastics
   7-Thermal and Moisture Protection
   8-Doors and Windows
   9-Finishes
10–Specialties  
11–Equipment  
12–Furnishings  
13–Special Construction  
14–Conveying Systems  
15–Mechanical  
16–Electrical

2. Each division is further divided into specification sections. For example:
   03100–Concrete Formwork  
   03200–Concrete Reinforcement

3. Each specification section is divided into three parts as follows:
   Part 1–General  
   Part 2–Products  
   Part 3–Execution

Example 5.1: Sample CSI Specifications Format. Delineate, following the CSI format, the specification sections for a slow sand filter similar to the one at Empire, CO.

1. Apply the CSI format to a slow sand filter: The following divisions and sections show how the specifications should appear for a slow sand filter such as that at Empire.

DIVISION 1–GENERAL REQUIREMENTS
   01010–Summary of Work  
   01040–Coordination  
   01060–Regulatory Requirements  
   01090–Abbreviations  
   01030–Alternates  
   01200–Project Meetings  
   01300–Submittals  
   01400–Quality Control  
   01500–Temporary Facilities and Controls  
   01600–Material and Equipment  
   01700–Contract Closeout

DIVISION 2–SITEWORK
   02050–Demolition  
   02200–Structural Earthwork  
   02220–Utility Trenching, Backfilling, and Compacting  
   02660–Water Transmission and Distribution  
   02505–Granular Paving

DIVISION 3–CONCRETE
   03100–Concrete Formwork  
   03200–Concrete Reinforcement  
   03300–Cast-In-Place Concrete  
   03600–Grout

DIVISION 4–NOT USED

DIVISION 5–METALS
   05500–Metal Fabrications

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2. Discussion: Division 16 was not used in the specifications for Empire because electrical power was not available at the slow sand filter site; all operations functioned by gravity. In cases in which electricity is available, Division 16 would normally specify grounding, lighting, controls, instrumentation, and motor control centers.

5.4.5 Drawings

The drawings define the geometry of the project, including dimensions, form, and details. Drawings are usually prepared using ink on mylar, and the scales used are those that will show the detail necessary for an accurate bid. Table 5.2 shows, as an example, the drawings included in the bid package for the Moricetown, BC, slow sand filter plant (Dayton & Knight, Ltd., 1988b).

Some of the drawings listed in Table 5.2 are reproduced in Figures 5.2 to 5.5. They illustrate working drawings done by the consulting engineer and provided to the contractor. Although the concept of a slow sand filter is simple, the contractor must be able to build the filter as the engineer intended; the plans are the major means of communication for a "meeting of the minds" between the engineer and the contractor.

Figure 5.2 provides an overall perspective of the slow sand process used at Moricetown. The raw water intake is located in Corya Creek. From the creek, raw water flows by gravity to the control building, where alum can be added for flocculation. From the control building the pretreated water flows to the sedimentation basin, where it settles for 24 hours at design conditions. From the sedimentation basin the water flows
onto the slow sand filters. After it is filtered, the water is chlorinated and stored in the clear well.

Figure 5.3 shows the overall design and layout of the filters. Note that each filter is covered to protect it from freezing. As shown in Detail A, the filter walls are grooved to prevent short-circuiting. Two access manholes are provided for each filter to allow for observation during filter operation.

Figure 5.4 shows the piping for the filters. Each filter has a 75-mm (3.0-in.) inlet consisting of an upturned elbow. The filter underdrains feed a 150-mm (5.9-in.) filtered water pipe. Each filter has a 150-mm (5.9-in.) overflow pipe. All wall penetrations are watertight. Section 1 shows the watertight doors at the end of each filter, which provide access for the operators for scraping and maintaining the filters and for replacing the media.

Figure 5.5 shows details used in the filter construction, including the watertight door and the pipe wall insert. Detail A shows the sight gauges (piezometers), which are

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**Table 5.2**

Drawings Developed as Part of the Specifications for the Construction of a Slow Sand Filter at Moricetown, BC

<table>
<thead>
<tr>
<th>Category</th>
<th>No.</th>
<th>Title</th>
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<tr>
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<td>Key Plan–Area Plan–Design Data–Index</td>
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<tr>
<td></td>
<td>G2</td>
<td>Process and Instrumentation Diagram–Hydraulic Profile</td>
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<tr>
<td></td>
<td>G3</td>
<td>Access Road–Power Plan</td>
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<tr>
<td></td>
<td>G4</td>
<td>Existing Site–Plan and Profile–Test Holes</td>
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<tr>
<td></td>
<td>G5</td>
<td>Miscellaneous Details</td>
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<tr>
<td></td>
<td>G6</td>
<td>Intake Site</td>
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<tr>
<td></td>
<td>G7</td>
<td>Intake Details–Drain Profile</td>
</tr>
<tr>
<td></td>
<td>G8</td>
<td>Site Plan–Sedimentation Basin–Outside Piping</td>
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<tr>
<td></td>
<td>G9</td>
<td>Sedimentation Basin Cross Sections–Typical Berm Section</td>
</tr>
<tr>
<td></td>
<td>G10</td>
<td>Sedimentation Basin Inlet and Overflow Manholes</td>
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<td>Operations Building</td>
</tr>
<tr>
<td></td>
<td>A2</td>
<td>Architectural Details</td>
</tr>
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</tr>
<tr>
<td>Electrical</td>
<td>E1</td>
<td>Electrical Layout</td>
</tr>
</tbody>
</table>

*Source: Adapted from Dayton & Knight, Ltd. (1988b).*
Figure 5.2 Process Diagram and Hydraulic Profile for Moricetown Plant (Drawing G2, from Dayton & Knight, Ltd. [1988b]).
Figure 5.4 Moricetown Filter Piping. (Drawing M1, from Dayton & Knight, Ltd. [1988b].)
Figure 5.5 Details From the Moricetown Filter Plans. (Drawing M3, from Dayton & Knight, Ltd. [1988b].)
used to monitor the headloss across the filters. All penetrations in the filter box are watertight and all concrete joints have a 150-mm (5.9-in.) waterstop.

5.5 INSPECTION

The role of construction inspection is to assure that the project is constructed in accordance with the contract documents. Thus, it is important that the construction inspector be thoroughly familiar with the drawings and specifications.

Several construction phases for which inspection is necessary in the building of a slow sand filter are described briefly in the following sections and are illustrated by photographs from the Empire, CO, slow sand filter. The photographs are located in sequence following the text discussions. That these phases were singled out for discussion does not mean that they are the only areas for which inspection is necessary; good inspection practice should be followed during all phases of construction. The photographs shown are from the engineer's file and are a part of the inspection record. Such photographs are advisable during the inspection.

5.5.1 Excavation

Figure 5.6 shows the site being excavated for the slow sand filter. Considerable rock was encountered, which made the excavation very costly.

5.5.2 Backfill and Compaction

After the site was excavated, it was backfilled with granular material to provide a base for the filter boxes. Figure 5.7 shows compaction of the backfill material. The backfill was tested using the ASTM D2922 (American Society for Testing and Materials 1981) procedure. The structural fill and backfill had to be at least 95 percent as dense as the maximum soil density.

5.5.3 Reinforcing Steel

After the subgrade was prepared and tested for compaction, reinforcing steel was placed for the filter box base slab. The reinforcing bars were checked for the correct spacing and size. All mud, oil, loose rust, and mill scale were removed from the bars. Figure 5.8 shows concrete being placed over the reinforced slab with a pumper truck. A minimum 28-day compressive strength of 3,000 psi was specified for the concrete. The concrete was tested for strength, air content, and slump in accordance with the requirements of ACI 301, Section 16.3 (American Concrete Institute 1984).

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5.5.4 Filter Walls With Reinforcing Steel

Figure 5.9 shows the walls of the filter box after the inside forms were placed and the reinforcing steel was erected. The steel was checked for proper spacing and size before the outside forms were placed. Note that the pipe penetrations have extra reinforcement to prevent cracking the walls. Each pipe penetration required seep rings to prevent leakage.

5.5.5 Waterstops

Waterstops are required in all joints, whether horizontal or vertical, that must be watertight. Waterstops are typically composed of PVC or rubber. Joints in the waterstop material should be welded. Figure 5.10 shows a PVC waterstop installed in a horizontal and vertical joint between the filter box and the clear well.

5.5.6 Gravel Support

After the forms were removed from the filter boxes, the boxes were cleaned and gravel was placed around the underdrains as shown in Figure 5.11. The sand was then placed on top of the gravel. Samples of the sand and gravel were checked for gradation and uniformity. Placement of sand and gravel was made in accordance with AWWA B100 (American Water Works Association 1989).

5.5.7 Roof Installation

Figure 5.12 shows wood framing and plywood sheeting being installed to support the metal roofing and siding that would cover the filter boxes. Roof installation generated a lot of debris, as shown in Figure 5.13.

5.5.8 Inside Filter Box

The filter sand was covered with plastic sheeting at all times during the erection of the cover to keep debris from contaminating it. After the cover was completed, the debris was removed and the filter boxes were cleaned. The filter boxes and clear well were then disinfected with chlorine solution in accordance with AWWA C652 (American Water Works Association 1986) and checked for leakage. The completed filter is shown in Figure 5.14.
Figure 5.6  Excavation for Slow Sand Filter Box. (Courtesy Versar Architects and Engineers, Inc., Greeley, CO.)

Figure 5.7  Compaction of Backfill Material. (Courtesy Versar Architects and Engineers, Inc., Greeley, CO.)
Figure 5.8 Reinforcing Filter Box Base Slab With Concrete. (Courtesy Versar Architects and Engineers, Inc., Greeley, CO.)

Figure 5.9 Steel Reinforcement and Formwork for Filter Box Walls. (Courtesy Versar Architects and Engineers, Inc., Greeley, CO.)
Figure 5.10 Typical Waterstop Installation. (Courtesy Versar Architects and Engineers, Inc., Greeley, CO.)

Figure 5.11 Gravel Placement Around Underdrains. (Courtesy Versar Architects and Engineers, Inc., Greeley, CO.)
Figure 5.12  Installation of Filter Box Cover. (Courtesy Versar Architects and Engineers, Inc., Greeley, CO.)

Figure 5.13  The Inside of the Filter Box During Installation of the Cover, Showing Debris Generated by This Process. (Courtesy Versar Architects and Engineers, Inc., Greeley, CO.)
Figure 5.14 The Completed Filter. (Courtesy Versar Architects and Engineers, Inc., Greeley, CO.)
A major appeal of slow sand filtration is the simplicity of operation. Relatively few tasks must be performed. The initial task is plant start-up. The routine tasks include scraping, sand handling, monitoring, and maintenance.

6.1 PLANT START-UP

Following the completion of construction, the plant requires a break-in period before the production of potable water can begin. The first task is to fill the sand bed from the bottom with raw water. The second task is to operate the filter in the filter-to-waste mode until the water produced is of acceptable quality. During this period of conditioning, the meters should be calibrated, the laboratory should be set up, and the monitoring routines and data forms should be developed.

6.1.1 Filling the Sand Bed

The new sand bed must first be saturated with water, which is done by backfilling slowly to displace air pockets. The rate of backfilling should be 0.1–0.2 m of bed depth per hour (0.3–0.6 ft/hr), and backfilling should be continued until the water level is sufficiently high to submerge the jets of incoming water. A flow meter is needed in the backfill line for measurement of the flow and to assure that the specified backfill rate is achieved. When backfilling is complete, the weir plate should be set with the crest at the level of the influent jets. At this point, the raw water filtering can begin. After the filtering operation starts, the water level in the filter box will rise slowly over a matter of days due to the *schmutzdecke* buildup. When the level reaches twice the distance between the sand bed and the influent jets, the weir plate should be lowered slowly so that the crest is at the level of the sand bed surface.
6.1.2 Filter-to-Waste Operation

Washed sand should be specified by the engineer for use as filter media. The use of unwashed sand adds an uncertainty concerning how long the fines may appear in the effluent.

At the Empire, CO, plant, which used unwashed sand, turbidity levels were >2 NTU immediately after start-up and declined to <1 NTU only after one month, at which time the filter was put on line to the town. Several months were required, however, before the effluent turbidity was less than the 0.4 NTU found in the ambient water.

Even when washed sand is used, the filter should be operated in a filter-to-waste mode for sufficient time to wash out any residual fines. During this period, the filter should be operated at high hydraulic loading rates and the effluent turbidity should be measured daily. The resulting turbidity-time plot will indicate when the fines have been purged and the effluent turbidity has reached an acceptable level.

6.1.3 Ripening or Maturation Period

Following the start-up of a new filter or a rebuilt filter bed, a "ripening" process will occur. This process, whereby the filter bed reaches maturity, is discussed in Sections 1.3.1 and 4.1.1. The ripening period will range from about one week to several months. Warm temperatures and high nutrients will decrease the ripening time. Unless an indicator organism such as coliforms occurs in the raw water, the determination of the state of filter bed maturity will not be possible. Pilot testing with coliform spiking, prior to construction, will give an indication of filter ripening time.

6.2 OPERATING TASKS

The tasks involved in operating a slow sand filter pertain largely to the filter media and filter bed. These tasks include scraping the sand surface, washing and storing the sand, and rebuilding the filter bed.

6.2.1 Scraping and Backfilling

Slow sand filters should be scraped when the headwater rises to the overflow level. The following process is recommended for scraping the filters:

1. Remove any floating material
2. Slowly drain the water level to several centimeters below the surface of the sand
3. Scrape the top 1–3 cm of sand
4. Remove the scraped sand from the filter box
5. Wash the filter walls if they are dirty

Dropping the water level to below the sand surface is most easily accomplished by using a drain installed in the filter wall just above the sand level. With this drain, the operator can rapidly draw off the headwater. Asphalt rakes have been found to be effective for scraping the sand at Empire, CO, and at the 100 Mile House facility in British Columbia. The filter should be put back in operation as soon as possible after scraping so that the biological development of the filter bed is disturbed only minimally.

The time required for scraping a slow sand filter depends largely on two factors: the depth of sand removed and the method used to transport the dirty sand from the filter. Letterman and Cullen (1985) reported a typical time requirement of 5 person-hours per 93 m² (1,000 ft²) of sand when 2.5 cm (1.0 in.) of sand was removed in shovel loads and a hydraulic conveyance was used to remove the dirty sand from the filter. At the Empire facility, where sand is removed in buckets, scraping is performed at a rate of 19 m²/person/hr (205 ft²/person/hr). Figure 6.1 is a photograph of the scraping operation at Empire, CO. The tool shown is an asphalt rake, which is used to scrape the black

![Figure 6.1 Scraping Operation at Empire, CO, Filter Plant. (Photograph by D. W. Hendricks.)](image)
schmutzdecke into windrows. The operators have been trained to scrape the top 0.5 cm of the sand bed, thereby removing only the thin deposit on the sand surface. After the sand is scraped into windrows, it is shoveled into 20-L (5-gal) buckets and carried from the filters.

The equipment used for work on the filter, and even the weight of the workers, could force the surface material into the sand bed. To prevent this from happening, operators can walk and transport sand-removal equipment on boards laid on the sand surface. Also, because operators' boots could be a source of contamination, each operator should dedicate a pair of boots to scraping; the boots should be cleaned before and after this procedure. Scraped sand does not need to be replaced until a specified minimum sand bed depth is reached. The minimum bed depth recommended by Visscher et al. (1987) is 0.5 m (20 in.). Bellamy et al. (1985c) reported excellent coliform removal rates with filter beds 0.5 m (20 in.) deep. Huisman and Wood (1974) suggest that the filter bed should be at least 0.7 m (28 in.). For simplicity, the minimum and maximum bed depths should be noted on the inside walls of the filter box, preferably around the perimeter.

The filter housing should provide adequate headroom, lighting, and ventilation for operators who perform the scraping. If local codes do not permit lighting inside the filter box, some provision must be made for light from another source. A mechanism for the removal of scraped sand (such as a pulley) and for the replacement of sand should be included in the plant design.

The timing of filter start-up should be planned so that no two filters at a facility will need to be scraped simultaneously. The treatment plant must be able to meet peak demand even when a filter bed is shut down for scraping.

After scraping, the filter box should be backfilled slowly with filtered water. If the effluent turbidity is too high, the filter should be operated in the filter-to-waste mode until turbidity levels are acceptable. Filtration can then proceed as described.

6.2.2 Flow Adjustment

Flow demand will vary daily and seasonally. The treated water storage should be adequate to accommodate the variation in demand over the daily cycle. The daily demand will, however, change seasonally and will also change due to random events, with the result that the operator must make valve adjustments to increase or decrease the raw water flow coming into the plant. The operator can use the volume of water in treated water storage as the guide for the degree to which the flow should be adjusted, since the tank level should return to the same level each day. The volume of water higher or lower than that of the previous day gives the required flow adjustment. For example, if the volume of treated water storage on a particular day at Empire is 75,000 L less than the
volume measured at the same time on the previous day, the flow should be increased by 75,000 L/day.

6.2.3 Washing the Sand

The slow sand filter plant site should include facilities for washing sand. The process suggested for washing scraped sand is depicted in Figure 3.21. Dirty sand is transferred in slurry form to a wash flume, where the turbulence suspends the fines and the sand. The sand is then allowed to settle and the wash water is discarded to waste through a trough. (Caution: finer sand particles can be lost during washing, resulting in an increase in the effective size of the filter media.) The washed sand should be transferred to a concrete slab for drainage and then to a storage bin for clean sand.

6.2.4 Rebuilding the Sand Bed

The sand surface can be scraped many times, over a period of several years, before the minimum sand bed depth is reached, at which time the sand bed should be rebuilt. Figure 6.2 summarizes the steps in rebuilding the sand bed as outlined by Huisman and Wood (1974). Figure 6.2a shows the sand bed after many layers have been removed by scraping and as it stands ready for resanding operations. First, in resanding, most of the residual sand is removed from a section of the filter box so that only a shallow layer of sand remains above the gravel support, as shown in Figure 6.2b. The lower depths of the sand bed should be rebuilt using "new" sand from storage (Figure 6.2c). Old sand—that is, unwashed sand previously removed from the lower depths of the filter bed—should then be placed on top of the new sand (Figure 6.2d and e). This method will assure that sand containing microorganisms will form the upper zone of the filter bed. The diverse population of organisms in this sand will facilitate the ripening of the newly built bed. Care must be taken to not contaminate the old sand during the rebuilding process. When the rebuilding is completed, the sand bed should be structured like that shown in Figure 6.2f.

Once the desired sand bed height is obtained, the sand surface should be leveled and the filter should be started up as described in Section 6.1. That is, the filter should be backfilled with water, purged of fines, and given time to ripen. Ripening of a rebuilt sand bed should require less time than for a new sand bed due to the organisms present in the upper regions of the filter bed.
6.3 MONITORING AND REPORTING

Certain operational and water quality parameters must be regularly monitored and reported. These parameters include headloss, turbidity, disinfection, and concentrations of certain biological and chemical contaminants.

6.3.1 Headloss Versus Time

A record of headloss as a function of filter operation time is useful for determining when a filter will need scraping. Therefore, daily measurements, beginning at plant start-up, should be made of the headwater and tailwater levels. A plot of headloss versus time is shown in Figure 3.1. From such a graph, the time to terminal headloss can be predicted. Similarly, headloss data should be obtained from any other piezometer tube measuring the pressure within the sand bed.

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Figure 6.2 Steps in Resanding Filter Bed. (Adapted from Huisman and Wood [1974].)
6.3.2 Intrinsic Hydraulic Conductivity Versus Time

Calculating the intrinsic hydraulic conductivity on a daily basis is not necessary. However, the examination of intrinsic hydraulic conductivity levels may be useful for determining the origin of hydraulic problems that could develop during the filter run. Therefore, temperature measurements should be made along with headloss measurements to permit this calculation. From records of headloss and temperature values, and using the bed depth and HLR, the intrinsic hydraulic conductivity can be calculated by Equation 1.5.

6.3.3 Contaminants

Community water facilities are required to monitor and report levels of all contaminants listed in the National Primary Drinking Water Regulations (NPDWR) developed by the Environmental Protection Agency (40 CFR, Parts 141 and 142). Individual states may also have additional regulatory requirements for drinking water contamination. The appropriate state regulatory agency should be contacted regarding such additional requirements. The national regulations set limits on the types and amounts of contamination allowed in drinking water, based on maximum contaminant levels (MCLs) for selected organic and inorganic chemicals, turbidity, and microbiological and radiological contaminants. They also define monitoring and reporting requirements and stipulate that systems that do not meet the NPDWR requirements must notify users publicly. A copy of the NPDWR can be obtained from regional offices of the EPA, from the U.S. Government Printing Office, or from state regulatory agencies.

(1) Turbidity. The national regulations require that, effective June 1993, the turbidity levels of slow sand filtered water be 1 NTU or less in 95 percent of the measurements taken each month and that turbidity at no time exceeds 5 NTU. According to the regulations, a state may allow the turbidity value to be greater than 1 NTU, but below 5 NTU, in 95 percent of the measurements if the state determines that the higher turbidity level does not significantly interfere with disinfection. Although the national regulations require that water facilities measure turbidity every four hours by grab sample or that they continuously monitor it, for slow sand filter systems the regulations allow states to reduce the frequency of turbidity sampling to once per day (40 CFR, Parts 141 and 142).

(2) Microbiological Contaminants. The microbiological quality of a water supply is determined by analyzing for the coliform group of organisms. Coliforms are usually present in waters contaminated with human and animal feces and are frequently
associated with outbreaks of waterborne diseases. They therefore serve as indicators of contamination by pathogenic organisms.

Effective December 31, 1990, compliance with the NPDWR maximum contaminant level (MCL) is based on the presence or absence of total coliforms in a sample rather than on an estimate of coliform density. For systems analyzing at least 40 samples per month, no more than 5.0 percent of the monthly samples may be total-coliform positive. For systems analyzing fewer than 40 samples per month, no more than one sample per month can be total-coliform positive (40 CFR, Parts 141 and 142). The required frequency of bacteriological sampling depends on the size of the population served by the water supply and is stipulated in the NPDWR. The regulations require that analyses for total and fecal coliforms/E. coli, as well as the heterotrophic plate count (HPC), be performed by laboratories certified by state agencies or by the EPA for such analyses. Methods approved for the analysis of coliform bacteria include the membrane filter method and the 10-tube multiple tube fermentation method. Both tests are described in Standard Methods (1989). The minimal media ONPG-MUG test (Colilert®) may also be approved for compliance monitoring (40 CFR, Parts 141 and 142). Consult with your state regulatory agency for bacterial monitoring methods.

According to the national regulations, the results of bacteriological tests must be retained for at least five years. A sample form for recording coliform analysis is included in Section 6.3.7. At the minimum, the records must indicate the following:

1. Date, place, and time of sampling
2. Name of the person who collected the samples
3. Reason for sampling (routine or check)
4. Whether the sampled water was raw or treated
5. Details on the analysis, including the date it was performed
6. Identification of the laboratory, the analysis method used, and the name of the analyst
7. Test results

6.3.4 Disinfection

The disinfectants most commonly used in domestic water supplies are chlorine and chlorine-containing compounds. Therefore, guidelines for monitoring and reporting disinfection are presented here for chlorine disinfection and complement the review of the topic in Section 3.1.5. Communities using ozone or ultraviolet radiation for disinfection should refer to appropriate EPA and state guidelines.
The chlorine flow should be measured daily to assure the proper dosage of chlorine. The chlorine dosage should be adequate to satisfy the chlorine demand and to provide a free residual concentration at the consumer's tap of at least 0.2 mg of chlorine per liter of water. The disinfectant dosage is typically calculated using C•T values. C•T is the product of the residual disinfectant concentration (C) in milligrams per liter and the corresponding disinfectant contact time (T) in minutes. The Federal Register (1989, 40 CFR, Parts 141 and 142) contains a table of C•T values, summarized as Table 3.3, for achieving 1-log and 3-log inactivation of Giardia lamblia for a variety of water temperatures and pH values. EPA guidance manuals, such as by Malcolm Pirnie, Inc. (1989), should be consulted for greater detail on how to use C•T values to achieve desired inactivations of Giardia and other contaminants. Information on how to obtain these manuals is available from EPA regional offices and from state regulatory agencies. The national regulations require that filtration plus disinfection achieve at least a 99.9 percent (3-log) removal or inactivation of Giardia cysts and a 99.99 percent (4-log) removal or inactivation of viruses. The level of disinfection required for achieving these removal/inactivation rates is set by the state. Generally, however, a 2-log (99 percent) reduction in Giardia cysts should be expected for slow sand filtration, especially after the filter has become mature, leaving only 1-log (90 percent) reduction for the disinfection.

The level of residual chlorine should be measured at least daily, both at the point where the water leaves the filtration plant and at the point in the distribution system that is at the greatest distance from the plant. Samples from the most distant point must meet water quality criteria for residual disinfectant and coliforms. Engineers and water utilities should refer to the Surface Water Treatment Rules promulgated by their regulatory agencies for current requirements.

Approved methods for measuring the chlorine level in water include the DPD colorimetric test, which uses the chemical N,N-diethyl-P-phenylene-diamine. This test is described in Standard Methods (1989). Test kits are available commercially for the field analysis of chlorine, but care must be exercised in selecting which kit to use. Some of the kits designed for measuring chlorine in swimming pools do not use approved methods. The use of chlorine-selective probes is also an approved method for measuring chlorine in water.

Because free residual chlorine is only relatively stable even in the absence of sunlight, agitation, and certain organic and inorganic chemicals, samples should be collected in clean, sterilized containers that are completely free of sodium thiosulfate and should be analyzed immediately.
6.3.5 Instrumentation

Two classes of instruments are required for a slow sand filtration facility: analytical instruments and measuring devices. Analytical instruments are needed to outfit the support laboratory at the filtration plant. Measuring devices include flow meters, thermometers, and gauges. In addition, safety instruments are required for the disinfection processes.

(1) Analytical Instruments. As noted, a laboratory should be available for instruments and as a place to conduct the analyses. The laboratory should have running water and sufficient bench space for the instruments and set samples and cabinet space for reagents and standards. Analyses that are performed daily, such as for turbidity, chlorine, and pH, should be done on site in such a laboratory. Therefore, each slow sand facility must have a turbidimeter, chlorine-analysis equipment, and a pH meter. Only 2–3 m (6–9 ft) of bench space are needed for this level of analytical capability. If coliform analyses are performed on site, an incubator and the appropriate laboratory ware and reagents are required. It is important to remember that many water quality tests have crucial time, transport, and sample-storage limitations, which are specified in the national regulations. For metals and organics, the use of an outside certified laboratory should be arranged, and sampling and analysis should be done at intervals specified by the regulations.

(2) Measuring Instruments. Only a few metering instruments are required for a slow sand filtration facility (see Section 3.3.1). A flow meter with a flow control valve is required on the inlet side of each filter cell. Volumetric flow meters are advised for the effluent side so that records can be kept of the total volume of water treated per day. Temperature should be measured also, using a laboratory-grade thermometer. The flow meters should be checked periodically for possible obstructions and should be removed periodically for cleaning. Material can be trapped, for example, on the upstream side of orifice plates; they should be positioned so the obstructions can be easily removed. Pressure taps and piezometers should be checked weekly for possible clogging of the openings. Total flow meters should be removed about once each year for inspection and possible calibration. When a meter is removed from the filter, a substitute meter should be placed in the line. Such "spares" should be purchased as a part of the initial capital investment of the plant so that the plant is fully operational at start-up.

Where continuous monitoring of turbidity is required, turbidimeters with associated pump(s) and sampling lines should be set up to continuously sample both influent and effluent water flows. For most slow sand installations, however, daily grab sampling is sufficient for monitoring turbidity.
The instrumentation required for disinfection will depend on which disinfectant is chosen. Solid and liquid disinfectants require meters for proper dosing. Regulators are needed for gas disinfectant systems. In addition, safety equipment and alarms are necessary if the disinfectant source is a gas (chlorine or chlorine dioxide).

6.3.6 Recording, Reporting, and Filing of Records

Records must be kept of all water quality parameters evaluated both in-house and at certified laboratories. Analytical results must be reported to the appropriate federal and state agencies in compliance with NPDWR and state regulations. The frequency of reporting is stipulated by the regulatory agencies and may depend on the population served, as discussed in Section 6.3.3 for coliforms.

The length of time that records must be retained is specified in the federal regulations. For example, the results of bacteriological tests must be kept for at least five years. State requirements may vary from federal requirements and should be consulted. An archives system should be set up, however, to retain all useful monitoring and reporting records within an easy-to-use filing system.

Additionally, records of other occurrences should be kept, including the following:

1. Any cleaning of the filter inlet, bed, or distribution network
2. Interruptions of the water supply to the filter or of any filter operations
3. Failure or malfunction of equipment, even if it does not result in the interruption of operation
4. Headloss through the filter bed and filtered water flow rate
5. Temperature
6. Precipitation events

A record should also be kept of the total volume of water filtered each day per filter unit. These data will allow an examination of the volume of water filtered per cycle, which is likely to vary seasonally.

6.3.7 Forms

The format for recording and reporting water quality test results is not stipulated in the regulations. However, the following items should be included in the reporting forms:

1. Title of the form, identifying the water quality parameter being analyzed
2. Date and time the sample was obtained
3. Location in the plant from which the sample was obtained
4. Name of the person who collected the sample
5. Date, time, and location of analysis
6. Name of the person performing the analysis
7. Any dilution or special sample preparation
8. Results of the analysis
9. Comments on weather events or other conditions (for example, equipment malfunction) that could have affected the parameter on the day the sample was taken

Trends will be more easily identifiable if the results of periodic sampling are recorded on the same page. Figure 6.3 is a sample form for reporting the results of coliform monitoring.

6.4 MAINTENANCE

Daily maintenance tasks related to the operation of the filters consist of checking that the inflow is not clogged, measuring the flow and level of the headwater, and clearing floating material and scum, if any, from the water surface. (Enclosed filters will be less likely to have litter on the surface of the headwater.) Pumps, chlorinators, and other equipment should be checked daily to verify that they are working properly. All flow-measuring devices should be monitored, including those metering the supply to the community. Piezometers should also be read daily.

Daily maintenance tasks peripheral to the operation of the filters include measuring chlorine residuals in the water at various points in the distribution system (as discussed in Section 6.3.4), checking the supply of disinfectant, and noting the level of water in the storage tank and clear well.

6.5 EQUIPMENT NEEDED FOR OPERATION

The number of pumps needed in a slow sand filtration facility will be determined largely by the hydraulic layout of the treatment scheme. For example, the slow sand filter plant in Empire, CO, operates without pumps or any other electrical equipment and is located nearly 0.5 km (0.3 mi) from the nearest power lines. Propane is used for auxiliary heat. The Empire plant can operate without pumps because it is located on a steep hillside between the water supply and the community served. At gravity-feed facilities such as Empire, control valves are needed instead of pumps.

At most slow sand filter sites, however, water must be pumped from the water supply, and other pumps may also be required. For these facilities, the pumps must be
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### Monthly Summary

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Figure 6.3 Sample Form for Slow Sand Operating and Evaluation Data. (Adapted from forms of Colorado Health Department.)
capable of supplying peak water needs. It is recommended that sufficient spare pumps and parts be kept on hand to remedy failures of such equipment.

Much of the large equipment needed at a slow sand filtration plant pertains to the filtration media. Specifically, equipment is required for the original placement of sand in the filter boxes and for subsequent resanding. Additionally, the operators for very large slow sand filters, such as those with surface area greater than 400 m$^2$ (4,300 ft$^2$) for one cell, may wish to use machines for scraping the filter beds, for transporting the scraped sand, or both. A study by Letterman and Cullen (1985) determined that the shortest scraping time per unit filter surface area for large filters was accomplished when a motorized buggy was used for transporting the sand. For small filter plants, such equipment is not warranted as the required tasks can be done more quickly and easily by manual methods than by the use of mechanical aids. At the Empire plant, for example, the surface sand is hand-scraped with asphalt rakes and hauled out of the filter boxes in buckets. This task can be done within 2–4 hours by two persons.
Chapter 7

Modified Slow Sand

This chapter reviews selected modifications to the slow sand filtration process developed in research reported during the 1980s. By definition, a modified slow sand system deviates from the slow sand concept of a simple, passive technology. Nevertheless, the range of slow sand applications may be extended by the use of selected modifications, and the engineer must evaluate the benefits of applying them vis-à-vis the alternative of using another technology. Thus, although knowledge of some of the tentative extensions of practice is useful, they are not for routine application. With this caveat in mind, the following advancements are addressed:

- Increased applicability of slow sand for raw waters of marginal quality
- Improved removals of organic precursors
- Improved schmutzdecke removal techniques that minimize filter downtimes and ripening periods

Modifications to slow sand filtration that enhance operational and treatment performance may be grouped into (1) pretreatment techniques, (2) filter media upgrades, and (3) alternative schmutzdecke removal techniques. Pretreatment techniques include the use of roughing filters and pre-ozonation. Filter media modifications include the utilization of filter mats, the enhancement of bacterial populations through filter harrowing, and surface amendments. Schmutzdecke removal techniques as alternatives to surface scraping include the use of filter mats and filter harrowing.

Table 7.1 shows which of these modifications address which slow sand filtration problems. As can be seen from the table, several modifications may address more than one filtration problem. For example, filter mats may be used to increase raw water applicability and may minimize downtimes caused by schmutzdecke removal. Filter harrowing may reduce filter downtime due to schmutzdecke removal, reduce ripening periods, and improve organic precursor removals by increasing the density of the filter.
Table 7.1
Slow Sand Filter Modifications

<table>
<thead>
<tr>
<th>Problem</th>
<th>Modification</th>
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<tbody>
<tr>
<td>Raw waters of marginal quality</td>
<td>Roughing filters</td>
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<td>Filter mats</td>
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<tr>
<td>Organic precursor removal</td>
<td>Pre-ozonation</td>
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<td>Surface amendments</td>
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<tr>
<td></td>
<td>Filter harrowing</td>
</tr>
<tr>
<td>Schmutzdecke removal</td>
<td>Filter mats</td>
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<tr>
<td></td>
<td>Filter harrowing</td>
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</table>

media bacterial population. This chapter summarizes the design of each of these slow sand filter modifications and evaluates their benefits.

7.1 ROUGHING FILTERS

A number of pretreatment techniques can be used to reduce high source-water turbidities to a level considered acceptable for slow sand filter operation. For small communities, coarse-media roughing filters are considered to be the most promising of these pretreatment techniques because of the simplicity of their use (Wegelin 1988).

The basic components of a roughing filter include:

- A filter box divided into two or preferably three compartments
- Gravel beds of decreasing size for each compartment
- Inlet, outlet, and intercompartmental control structures to ensure an even flow distribution across the filter cross section
- A filter flow control device

The flow direction in roughing filters can be either vertical or horizontal, as illustrated in Figure 7.1. Horizontal-flow roughing filters are considered to have greater silt storage capacity and lower hydraulic cleaning needs than upflow or downflow roughing filters.

The size of the gravel, the length of filter material, and the filtration rate are the principal parameters for roughing filter design (Wegelin et al. 1986; Wegelin 1988). Specifically, the size of the graded gravel medium varies from roughly 20 mm to 4 mm in the sequence of coarse, medium, and fine compartmental packs. The cumulative filtration length is in the order of 6 m (20 ft) to 12 m (39 ft). The filter height is limited to
Figure 7.1 Roughing Filters, Showing Three Flow Directions. (Adapted from Wegelin [1988].)

1.5 m (5 ft) to permit easy cleaning. The cross-sectional width depends on the capacity of the filter but generally ranges from 2 m (6.5 ft) to 5 m (16 ft). Filtration rates for roughing filters designed for high-turbidity raw water generally vary from 0.3 m/hr to 1.5 m/hr. Practical design experiences with roughing filters are summarized in Table 7.2.

As seen in Table 7.2, roughing filters can achieve significant reductions in raw water turbidity and in levels of fecal coliform bacteria. Also, they can remove apparent color and algae. Reports indicate that peak turbidity removals may range from 60 percent to 90 percent (Wegelin 1988; Wolters et al. 1989), with the higher reductions generally associated with the higher turbidity loadings, as noted in Table 7.2. The table also shows that coliform bacteria may be removed to a similar extent as turbidity. Peak turbidity and bacteria removals are generally obtained after a pronounced maturation period, which is
consistent with the findings of Bellamy et al. (1985a, 1985b) for the performance of slow sand filters; that is, the biological maturity of the filter is the most important parameter of performance. Removals are inversely related to filtration rate and gravel size, which is consistent with theoretical expectations (reviewed in Section 1.3.1).

Color removals have not been well documented but pilot downflow gravel-packed roughing filters were reported to have reduced apparent color by 45–80 percent. The particulate fraction of color is most likely removed by filtration.

The Thames Water Authority uses gravity roughing sand filters and microstrainers to reduce algal loadings to its slow sand filters (Rachwal et al. 1988). The roughing filter media consists of 0.5–0.7 m (20–28 in.) of sand with an effective size of 0.75 mm and a uniformity coefficient of 1.7 overlaying 0.5 m (20 in.) of graded gravel. At a flow rate of 5 m/hr, removals of algal and particulate organic material averaged 50 percent, with a range of 30 percent to 80 percent depending on particle size and algal species. Increasing the bed depth to 0.7–0.9 m (28–36 in.) increased average removals to 67 percent at flow rates of up to 10 m/hr.

The primary concern with regard to the efficiency of roughing filtration is the effectiveness of the hydraulic flushing techniques used to clean the filter media and restore the bed's removal and storage capacities. Fluidizing the bed as practiced for rapid rate sand filtration is difficult to accomplish in roughing filters because of the heavy, coarse media used. Cleaning is generally accomplished by hydraulic surges that are
initiated by rapid openings and closings of the inlet and outlet drainage valves (Wolters et al. 1989). The cleaning action results from the introduction of high velocities and the generation of pressure waves within the filter bed. The cleaning efficiency is influenced by the arrangement of the underdrain system, the characteristics of the filter media, and the characteristics of the deposited material to be removed. Experience to date with cleaning procedures is limited, necessitating the need for more laboratory and field evaluations. After roughing filters are in service for several years, periodic hydraulic cleaning may not be sufficient to reestablish filter efficiency. In such cases, the filter media must be manually removed and washed.

Another concern with roughing filters is the lack of information regarding the type and size of particulate matter that can be removed with different combinations of gravel size, filter length, and loading rate. Recent investigations by Wegelin et al. (1986) have provided some information toward understanding filter efficiency and toward the development of a filtration model for the horizontal roughing filter. However, those investigations were limited to kaolin stock solutions. The effectiveness of roughing filters with raw waters containing various combinations of clay particles, algae, and dissolved organic matter still needs to be evaluated.

7.2 FILTER MATS

The operational performance of slow sand filters can be considerably improved by the application of a layer of nonwoven synthetic fabric to the sand surface, a procedure that has been the subject of a joint project, many years in duration, of Imperial College in London and the Thames Water Authority (Rachwal et al. 1988; Mbwette and Graham 1988; Graham and Mbwette 1990). The principal objectives in applying a layer of nonwoven, synthetic fabric to the sand surface are to concentrate the macro-particle removals on the fabric layer, thereby avoiding the need to remove the sand for cleaning, and to extend the filter run time (Graham and Mbwette 1990).

The rationale behind applying a nonwoven fabric layer to the top of the slow sand filter is to concentrate the treatment processes within the fabric layer instead of within the top sand layers. Nonwoven fabric is considered to be a more efficient filtration medium than sand because of its greater porosity and specific area (Graham and Mbwette 1990). The benefits of using fabric mats in slow sand filters include:

- Longer filter run times because fabrics result in a lower rate of headloss development than sand
- A simpler filter cleaning arrangement, since cleaning involves only the removal and cleaning of the fabric
A number of commercial, nonwoven synthetic fabrics with various physical properties, as shown in Table 7.3, were recently evaluated for operational effectiveness (Mbwette 1989). The selection of a fabric for a slow sand filter should be based on the structural properties (porosity, specific surface area, and total fabric thickness) that will result in optimal performance for the filter conditions (Graham and Mbwette 1990). The fabric filterability, as defined by fabric surface area and total thickness, must be designed to prevent any significant penetration of influent material into the sand layer. In general, the total fabric thickness should not exceed 2–3 cm (1–1.5 in.) in order to facilitate fabric removal and cleaning operations.

A ratio of the mean run time of the fabric-protected filter unit to the mean run time of a reference, unprotected filter unit can be used to evaluate fabric effectiveness independently of raw water quality and filtration velocity (Graham and Mbwette 1990). Generally, an optimal run time ratio corresponds to a fabric surface area in the 13,000–14,000 m²/m³ (3,962–4,267 ft²/ft³) region. Fabrics with a high surface area (for example, >25,000 m²/m³ or 7,620 ft²/ft³) will reach terminal headloss more rapidly than conventional filter sand, regardless of the depth of the fabric. Conversely, fabrics with a low specific surface area (for example, <3,000 m²/m³ or 914 ft²/ft³) have been shown to extend filter runs times, with the improvement increasing with fabric depth. However, these low filterability fabrics resulted in significant particle penetration into the underlying sand layer.

The primary concerns with the use of filter mats are associated with fabric cleaning and the lack of large-scale applications to raw waters of varying quality. Manual cleaning of the fabric mat has not presented any difficulties in small-scale installations (that is, mats with areas smaller than 27.9 m² or 300 ft²), but a suitable method of cleaning has yet to be established for larger fabric areas. Further pilot plant data are required to determine the effectiveness of fabric mats for various raw water qualities.

<table>
<thead>
<tr>
<th>Property</th>
<th>Minimum</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit layer thickness (mm)</td>
<td>0.36</td>
<td>20</td>
</tr>
<tr>
<td>Bulk density (g/mL)</td>
<td>0.02</td>
<td>0.4</td>
</tr>
<tr>
<td>Mean fiber diameter (μm)</td>
<td>27</td>
<td>48</td>
</tr>
<tr>
<td>Porosity</td>
<td>0.56</td>
<td>0.99</td>
</tr>
<tr>
<td>Specific surface area (m²/m³)</td>
<td>1,100</td>
<td>36,300</td>
</tr>
</tbody>
</table>

7.3 PRE-OZONATION

Studies have shown that the ozonation of raw water prior to slow sand filtration controls the growth of algae in the headwater without decaying the biologically active *schmutzdecke* (Rachwal et al. 1988) and increases the biodegradability of organic compounds present in natural waters (Narkis and Schneider-Rotel 1980; Kuo et al. 1978). The control of algae growth is beneficial because it extends the filter run length; the increase in biodegradability is beneficial because it improves the removal of organic precursor material, such as aquatic humic substances, thereby minimizing the occurrence of disinfection by-products, such as trihalomethanes.

The increase in biodegradability after ozonation is mainly due to the production of lower-molecular-weight compounds from large organic molecules, such as humic and fulvic acids, which are more resistant to biodegradation (Gilbert 1979). Van der Kooij et al. (1989) found that the concentration of easily assimilable organic carbon (AOC) increased with increasing ozone dosages for a variety of natural waters. In their study, various ozone dosages caused only a limited reduction in dissolved organic carbon, but the concentration of ultraviolet-adsorbing (254-nm [nanometers]) compounds was clearly reduced. The decrease of ultraviolet (UV) adsorption can be linearly related to the increase in AOC concentration. Hence, biodegradable compounds are produced by the oxidation of UV-adsorbing organic compounds in natural waters, for example, by the oxidation of humic substances. Sontheimer and Hubele (1986) reported that ozonation is essential in promoting the biodegradation of aquatic humic material. They found that without ozonation, biodegradation removed 8 percent of the dissolved organic carbon (DOC) from their sample. After doses of 1.82 mg O₃/mg DOC, biodegradation removed 55 percent of the DOC. Biological growth rates of ozonated DOC were also four times higher than those of controls.

Aquatic humic material may also be degraded by bacterial populations that have been induced to co-metabolize, degrade, and utilize aromatic compounds commonly associated with fulvic and humic acids. Work by Collins and Eighmy (1988) has shown that benzoate ring cleavage and mineralization activity are proportional to aquatic organic matter removal, particularly for smaller-molecular-weight fractions, in *schmutzdecke* microbial populations.

Pilot and full-scale evaluations of pre-ozonation on slow sand filtration, conducted in England by Rachwal et al. (1988), showed that pre-ozonation doses of 3 mg/L increased run length. The main effect of pre-ozonation was related to reductions in UV absorbance at 254 nm and 400 nm of up to 70 and 90 percent, respectively, compared to reductions of 15 and 25 percent, respectively, for the control slow sand filters. Most of these reductions
were due to the ozone itself. Only 10 percent of the reduction was attributed to occur within the slow sand filter. Pre-ozonation also resulted in a 25 percent removal rate for total organic carbon compared with 15 percent for conventional filters.

In pilot studies at the University of New Hampshire, pre-ozonation has been used to increase the biodegradability of the source water organic matter, particularly organic precursors, and sodium benzoate has been added to the filters to augment the bacterial populations that catabolize substituted aromatics (Eighmy et al. 1991). The ozone contactors have contact times of 8-9 minutes resulting in virtually complete ozone transfer with minimal residuals in the contactor effluent lines. The researchers have reported that the raw water turbidity, the nonpurgeable dissolved organic carbon (NPDOC), the UV absorbance (254 nm), and the trihalomethane formation potential (THMFP) ranged from 1.5 to 2.0 NTU, 7.5 to 8.0 mg/L, 0.30 to 0.35, and 400 to 500 µg/L, respectively. All of the pilot filters were able to achieve significant reductions in turbidity, with effluent levels frequently below 0.5 NTU. Ozone-induced reductions of organic precursor material were significant, with removal levels approaching 50 percent.

In the New Hampshire studies, a major limitation with ozonation or benzoate-induced organic removals has been the rapid headloss development observed in the slow sand filters. The headloss curves for the pre-ozone and benzoate-induced filters were shown to be exponential, taking only 20 days each to reach 1.5-m headlosses. The control filter showed no significant headloss during the same period. The former curves were representative of cake filtration, suggesting that the organic and inorganic filter-blocking materials are not penetrating the filter sand. It may be that a larger sand size than is normally used is needed for long filter runs of highly colored waters in enhanced slow sand filters. Further investigations relating filter media specifications to increased organic precursor removals are needed.

7.4 SURFACE AMENDMENTS

The substitution of a layer of sand with granular media more noted for their ability to adsorb and remove organic precursor material was evaluated in a pilot study by Collins et al. (1990). Typically, an 8-cm (3-in.) layer of surface amendment (the replacement granular media) was supported by a 30-cm (12-in.) layer of sand, which in turn was supported by a 15-cm (6-in.) layer of graded gravel. Specifications for the support sand and surface amendments are summarized in Table 7.4.

The pilot filtration study was conducted on a highly colored, low-turbidity water source. The raw water organic parameters of NPDOC, UV absorbance (254 nm, pH 7), and THMFP averaged 11 ± 3 mg/L, 0.450 ± 0.100, and 750 ± 250 µg/L, respectively, with
the higher levels generally occurring near the end of the filter run. Raw water turbidities averaged \(2 \pm 1\) NTU, with the higher values again near the end of the run. Filtration rates averaged 0.1 m/hr (0.04 mgd).

Organic precursor removals by slow sand filters were influenced by the characteristics of the filter media. Average THMFP, NPDOC, and UV absorbance (UVA) percent removals for the various amended pilot filters are shown in Figure 7.2. Organic precursor removals for the filters amended with anionic resin and granular activated carbon (GAC) frequently exceeded 90 and 75 percent, respectively. These values were much higher than those reported for municipal, conventional slow sand facilities (Collins et al. 1990). Other studies have shown GAC-amended slow sand filters to achieve even higher organic precursor removals, frequently exceeding 90 percent (Fox et al. 1984). GAC removal in the Collins et al. study was limited by the relatively small depth—8 cm (3 in.)—of amendment used. Higher precursor removals would be expected from deeper beds of GAC. The effect of the anionic resin did not appear to be limited by bed depth. Most removals appeared to have occurred in the top layers. The kinetics of precursor removals by ion-exchange is apparently faster than the transport and attachment mechanisms inherent to GAC.

The treatment efficiencies of the aluminum oxide, anthracite, and clinoptilolite amended filters were similar to those of conventional slow sand filters. Typical organic precursor removals by the amended filters in the Collins et al. (1990) pilot study frequently averaged 5–25 percent, and those values are in agreement with the approximately 20 percent removals reported by other researchers (Fox et al. 1984; Rachwal et al. 1988). The slightly higher percent removals noted for the aluminum oxide amended filter might be attributable to the smaller effective size of the amendment media. The poor removals noted for the clinoptilolite amended filter, which was a covered filter,
were attributed to the lack of sunlight-induced algal growths typically observed on the *schmutzdecke* of uncovered slow clinoptilolite/sand filters. Other experiments with pilot filters having a 20-cm (8-in.) layer of clinoptilolite gave 50 percent longer filter runs (Foreman and Sims 1984). Studies have also shown that the biological activity within the *schmutzdecke* was increased by the role of the ion-exchanger in concentrating ammonium ions. The ammonium ions were released as ammonia, a nutrient that enhances bacterial growth (McNair and Sims 1986; McNair et al. 1987). Clinoptilolite as an ion-exchanger can remove a range of cations, such as iron, silver, copper, zinc, cadmium, and other toxic heavy metals and nuisance metals (Andrews 1990).

The principal disadvantages associated with anionic resin or GAC amended slow sand filters are rapid headloss development and the costs of amendment cleaning and regeneration. The excellent organic precursor removals demonstrated by anionic resin and GAC amended filters were tempered by their exponential headloss curves. Increases in organic removals will increase headloss development rates, thus requiring more frequent *schmutzdecke* removals, especially of the amendment surface. Handling of the GAC or anionic resin layer may be simplified by containing the amendment between
layers of a synthetic filter fabric. A permeable quilted blanket may be sectionally placed on and removed from the filter sand surface in the same way that filter mats are used. The blocked GAC layer may be washed and/or replaced; the exhausted anionic resin layer could be regenerated without a noticeable decrease in efficiency by rinsing it with water and soaking it with a sodium chloride solution. The removal of the *schmutzdecke* and the removal of the amendment layer for a slow sand filter with a large-scale surface amendment have yet to be demonstrated.

### 7.5 FILTER HARROWING

Operators at the West Hartford, CT, slow sand facility do not remove the *schmutzdecke* from their filters; they disrupt the layer by harrowing, and a portion of the debris is removed by headwater drainage (Collins et al. 1990). When the filter headloss approaches a terminal headloss of 1.8 m (6 ft), the operators drain the supernatant water to a height roughly 0.3 m (1.0 ft) above the sand filter and rake the sand media. They simultaneously keep the filter surface sumps open, causing a steady discharge of the overlaying water. As the rake is dragged over the sand, colloidal debris in the top 0.3 m (12 in.) is loosened and caught by the moving water stream and is eventually discharged. When the supernatant water drops to a level of less than 0.1 m (3 in.) above the sand, operations are suspended until the filter has refilled by reverse flow to a depth of 0.3 m (12 in.), at which time harrowing is resumed. The process is repeated until the entire filter surface has been harrowed. Filter run lengths generally last 4–8 weeks before harrowing is required. The entire filter sand bed is removed and thoroughly cleaned once every 8–10 years.

Only fine clay colloids and other small particulate debris are removed by filter harrowing, but there appear to be some major treatment advantages with this method of scraping the sand. For a large filter, harrowing typically requires significantly less time and manpower than the usual scraping method. One driver can usually harrow a 1,350 to 2,000 m² (1/3–1/2 acre) filter surface in less than 2 hours. Moreover, harrowed filters can be put back on line within hours instead of days or weeks. While a majority of the colloidal debris of the surface deposit is washed away with this method, the *schmutzdecke* bacterial population is apparently raked into the depths of the filter sand bed. The ability to maintain a high bacterial population after cleaning enables the harrowed filters to be quickly placed back on line without a deterioration in treatment performance.

Coring of three mature full-scale slow sand filters over two seasons revealed a significant relationship between the mass removal rates of trihalomethane formation.
potential (mg/m²/hr) and the filter media biomass (Collins et al. 1990). Filter biomass was quantified indirectly by acriflavine direct cell counts (AFDC) and directly by Folin reactive material (FRM) over the filter bed depth. Two of the full-scale filters, those in Springfield, MA, and New Haven, CT, utilized the surface scraping cleaning method; the third filter, in West Hartford, CT, used filter harrowing. Higher THM precursor material removals were observed for the West Hartford slow sand filters, which had greater bacterial biomass. The West Hartford filters consistently outperformed the more conventional slow sand filters.

The additional cost of harrow cleaning over conventional filter scraping is estimated to be less than 5 percent. At the West Hartford facility, filter harrowing reduced sand washing costs by 22 percent and filter downtimes by 18 percent when compared to manual surface scrapings (Minkus 1954).

7.6 SUMMARY

The use of conventional slow sand filters has been a consistently effective means of economically providing treated water of acceptable quality from low-turbidity raw waters. However, there is a considerable need to upgrade the ability of slow sand filters to treat waters of marginal quality with respect to turbidity and algal content. In addition, regulatory pressures to lower the THM maximum contaminant level may require slow sand filters to remove significantly more organic precursor material than is now removed. There also is strong interest in increasing filter loading rates to lower the initial construction costs to levels more conducive with the financial resources of small communities without compromising the low maintenance benefits of slow sand filtration. Cost-effective modifications to slow sand filter performance can be made to meet these needs.

Modifications to the slow sand filtration process to enhance operational and treatment performance, demonstrated in pilot studies or full-scale installations, include roughing filters, filter mats, pre-ozonation, surface amendments, and harrowing techniques that minimize filter cleaning downtimes and ripening periods. Evaluations of each of these modifications are described below.

7.6.1 Roughing Filters

Roughing filters have significantly reduced raw water turbidity, coliform bacteria, and algal content. The primary concern with this modification centers on the effectiveness of the hydraulic flushing techniques used to restore the filters' removal and storage capacities. Another concern is the lack of certainty in knowledge regarding
the type and size of natural particulate matter that can be removed with different combinations of gravel size, filter length, and loading rate.

7.6.2 Filter Mats

Filter mats placed on top of the sand surface provide longer filter run times and a simpler filter cleaning technique than with conventional filters. Filter cleaning requires only the removal and cleaning of the fabric. The primary concerns with this modification are associated with the lack of large-scale applications to raw waters of varying quality and the lack of a suitable fabric cleaning method for large-scale installations.

7.6.3 Pre-Ozonation

Pre-ozonation may increase organic precursor removals in slow sand filters by increasing the production of lower-molecular-weight and more biodegradable compounds from large organic molecules that are more resistant to biodegradation. A major limitation with pre-ozonation is that it increases headloss development in conventional slow sand filter media. Further investigations relating filter media specifications to increased organic precursor removal are needed.

7.6.4 Adsorption and Ion-Exchange

Slow sand filters amended to include layers of anionic resin or granular activated carbon have shown significant organic precursor removals. The principal disadvantages with such amended slow sand filters are that headloss develops rapidly and that cleaning and regeneration methods and costs have not been defined.

7.6.5 Harrowing

Filter harrowing requires less time and manpower for schmutzdecke removal than does the surface scraping method. Moreover, because this method maintains the bacterial population of the filter, harrowed filters can be quickly placed back on line after cleaning, without a deterioration in treatment performance. This is particularly advantageous if the water source is of low turbidity. Filter harrowing is the only modification to the slow sand filtration process that has been evaluated on a full-scale basis in the United States.
Chapter 8

Guidelines

This chapter provides a summary of the guidelines for the design and operation of slow sand filters that were discussed in detail in this manual. The guidelines are based upon the research knowledge generated since 1980, the practice since that date, and review of past knowledge and experience.

8.1 STATE OF THE ART

Slow sand filtration is a technology that was commenced in 1829 with the construction of filters for London by James Simpson, who set forth the basic design. Simpson's design for London became the standard for practice and is still the standard today. In the United States and Canada, slow sand filtration has not been a favored technology, but interest is being revived due to a resurgence of research that started about 1980. The research findings have both verified past practice and provided new findings useful for contemporary practice. The guidelines herein incorporate these research findings and the experience of recent plant designs. The guidelines also build upon past practice and incorporate it, except as superceded by the findings of the 1980s. Figure 8.1 is a photograph of a slow sand filter that was recently built for a small community in accordance with the guidelines for contemporary practice reviewed in this manual.

8.2 PREDESIGN

Before any design can proceed, a host of "inputs," performance specifications, and conditions must be determined. These factors "drive" the design. Some of the more salient ones are reviewed in the following sections.
8.2.1 Community Size

Slow sand filtration is especially suited for small communities because the technology is effective, passive, and inexpensive with regard to operating costs. Without question, communities having populations of 1,000 to 2,000 persons, or even as high as 5,000 persons, should be not too large for slow sand to be "appropriate." With larger plant sizes, however, the labor costs of processing sand will be greater than those for rapid rate filtration. The point of this crossover will depend upon circumstances. The city of Salem, OR, with a population of 107,000 (with the water system serving 135,000), uses slow sand filtration, as does West Hartford, CT, with 300,000 population served, and other cities (see Slezak and Sims 1984).

8.2.2 Raw Water Quality

Preferred turbidity levels for influent waters to slow sand filters are <10 NTU (Sims and Slezak 1990; their Figure 1.6 shows that 90 percent of the communities using slow sand have raw water turbidities less than 10 NTU). As an upper limit, 30–50 NTU has
been mentioned as a rule of thumb, but the limit is more a matter of engineering and judgment than an absolute level. Factors affecting allowable influent turbidity levels include effluent turbidity, length of run, and whether pretreatment is acceptable. Color and volatile organics are other considerations, and if the raw water concentrations are high, then pilot plant work should be done to determine removals.

8.2.3 Performance Required

The slow sand filtration process is expected to remove biological particles from influent water at 2-log to 4-log levels when the filter bed is biologically mature. These particles include cysts, oocysts, algae, bacteria, viruses, parasite eggs, nematode eggs, and amorphous organic debris. The foregoing applies only to biological particles present in the influent flow; it does not apply to species that find ecological niches and grow within the filter bed as a part of the biofilm around the sand grains (contributing to the maturity of the sand bed). The biofilm will be sloughed from the filter and will appear in the effluent flow. Cartridge filter sampling using 1 μm pore size filters may be used to assess the performance of the filter. Also, total coliform bacteria, if present in the influent water in sufficient densities, is a useful indicator of removal efficiencies. Because mineral particles may pass through a filter bed that is effective in removing biological particles, turbidity removal may not be a useful indicator of performance. Nevertheless, the effluent turbidity must be measured, since it has to comply with regulatory requirements. The EPA Surface Water Treatment Rule (Federal Register 1989) requires a turbidity standard of <1 NTU 95 percent of the time and stipulates that no reading may be greater than 5 NTU. If the state agency determines that interference with disinfection will not occur at higher turbidities than 1 NTU, a higher level will be permitted, but at no time may the level exceed 5 NTU.

8.2.4 Algae Control

Algae growth can be controlled by the installation of a roof over the filter beds. Algae growth will cause a more rapid increase in the rate of headloss and may cause taste and odor problems. Algae have no role in treatment if the sand bed is biologically mature, albeit nutrient uptake may occur due to their metabolism.

8.2.5 Pretreatment

As a general rule, waters that require pretreatment should be avoided. If, however, there are seasonal peaks of turbidity or if algae blooms occur for a short duration, and if slow sand filtration is still considered the most appropriate technology, then pretreatment may be considered. Pretreatment can be carried out through the use of a
sedimentation pond or a roughing filter. If organic precursors are present, the removal methods mentioned in Sections 7.3 and 7.4 may be considered.

8.2.6 Design Flow

The level of flow used to size the filter bed area (known as the design flow) should be the maximum daily flow projected to the end of the design period, taking into account the removal of one filter bed for scraping (unless storage is provided to compensate). The design period will depend upon local circumstances, but 20 years is common.

8.2.7 Treated Water Storage

Treated water storage should be sufficient to permit a steady flow of water through the slow sand filter over any daily cycle. The water storage volume should also be sufficient to allow for fire protection and emergencies.

8.3 DESIGN

Although a slow sand filter is simple in concept, its design has many considerations and includes layout, filter box sizing, plumbing, hydraulic factors, process design, appurtenances, and disinfection. These factors are reviewed in the following sections.

8.3.1 Overall

(1) Layout. The components of the design include intake, pretreatment (if any), the filter box, piping, disinfection, and treated water storage. Their layout is site specific.

(2) Filter Box. The filter box should include two or more cells of equal size that are independently operated. The filter bed area is calculated as the maximum expected flow divided by the maximum permissible hydraulic loading rate when all but one cell is in service. The depth of the filter box is the sum of the gravel support, sand bed, maximum water depth, and freeboard. The distance from the highest level of the sand bed to the nearest structural member of the filter box should be at least 2.0 m (6.6 ft) so that an operator will be able to stand comfortably erect while performing operations within the sand bed. The structure of the filter box should be designed by a civil engineer qualified in structural design, and the foundation should be designed by a qualified geotechnical civil engineer.

(3) Housing. Though not a requirement of the design, a slow sand filter housing is useful. By covering the filter beds, it prevents freezing, inhibits algae growth, and protects the sand bed from windblown debris. The housing should be equipped with
ventilation and should have access ports for entry, sand handling, and sand transfer. Portable electric lighting, which is in compliance with electric codes, should be provided.

(4) **Plumbing.** Provision must be made for running water directly from the filter to waste (a process known as filter-to-waste), for drainage of the headwater, for backfilling of the sand bed with filtered water, and for the adjustment of flow to each filter. An overflow should be installed at the maximum headwater level. Taps should be installed for easy sampling at appropriate sample points.

8.3.2 Hydraulics

(1) **Influent Water Distribution.** The water enters the filter through the headwater. The incoming flow should be distributed around the filter bed at low velocity, and the headwater should be ≥0.3 m (1.0 ft) deep to minimize sand bed erosion.

(2) **Tailwater Control.** A tailwater weir should be designed so that the weir crest can be raised to provide the ≥0.3 m (1.0 ft) water cushion for the influent and then lowered to the top of the filter bed during the filter run. To facilitate draining and scraping of the sand, the weir must be lowered further so that the headwater drops to a depth just below the sand surface.

(3) **Underdrains.** The underdrain system should be designed using the manifold hydraulic principle; that is, the headloss within the main pipe should be small compared with the headloss through the orifices into the main pipe. If the manifold principle is maintained, then the hydraulic loading rate across the filter bed should be uniform.

(4) **Gravel Support.** The purpose of the gravel layers is to support the sand bed and to minimize headloss between the sand bed and the underdrains so that the collection of water by the underdrains is not affected by the gravel support headloss. The gravel support should be graded, with the smallest gravel size on top and the largest on the bottom surrounding the underdrains. The gravel support should be designed such that gravel from one layer will not penetrate an underlying layer. The design is empirical and established rules should be followed, such as those given by the Ten States Standards (1987), by AWWA Standard B100, or by Huisman and Wood (1974), as described in Section 3.2.4.

(5) **Drainage.** Drainage of the headwater should be provided for by the influent distribution manifold system. Final lowering of the water surface to the desired level can be done through the underdrain system.

(6) **Backfilling After Scraping.** The water level should be lowered to about 2–5 cm (1–2 in.) below the sand surface before scraping. After scraping, the sand bed and the headwater should be backfilled with finished water, which may come from an adjacent
operating filter or from finished water storage. Finished water storage, if used, should be free of chlorine. The headwater should be backfilled to the level of the influent distribution orifices (to provide the ≥0.3 m [1.0 ft] water cushion). Valves and piping should be provided to accomplish the foregoing tasks.

(7) Overflow. The overflow weir and collection box should be positioned above the sand bed so that when the head for the weir is added to the weir crest elevation, the water level will be at the maximum level desired.

(8) Flow Measurements. Flow measurement on the influent side of the filter should be provided by means of an orifice meter or a Venturi meter. Pressure difference should be measured by pressure gauges. A calibration curve should be placed on a wall in the vicinity of the meter and should also be included on the data recording sheets and in the computer processing software. A volumetric flow meter should be located on the effluent side of the filter.

(9) Flow Control. Flow should be controlled by means of a gate valve located downstream from the flow meter. The valve should be on the main influent pipe. The pattern may be duplicated for each filter cell, but that is at the option of the engineer in consultation with the operator.

(10) Tailwater Elevation Control. The tailwater elevation should be controlled by means of a movable weir. Each filter should have such a means of tailwater control.

(11) Headloss Measurement. Two piezometers should be provided for each filter, with taps in the headwater and tailwater. Additional piezometers may be placed between the headwater and tailwater taps, at the option of the engineer in consultation with the operator. The piezometer tubes should have diameters of 2.5–5.0 cm (1–2 in.), with float balls and scales provided.

8.3.3 Process Design

(1) Hydraulic Loading Rate. The hydraulic loading rate (HLR) for the peak daily flow may vary between 0.1 and 0.4 m/hr (1–10 mgad). The HLR may exceed 0.4 m/hr (10 mgad) only at the end of the design period when one of the filters is removed from operation for scraping, and even then the HLR may exceed 0.4 m/hr only if the filter beds remaining in operation are biologically mature. While the HLR may vary during the annual cycle and generally increase as the population grows, the flow should be steady over the daily cycle.

(2) Sand Bed Depth. The sand bed should be ≥1.0 m (3.3 ft) in depth at the start of operation. A depth of 1.3 m (4.3 ft) is favored, but a deeper bed may be used if desired by the engineer. The minimum bed depth should be ≥0.5 m (1.5 ft), assuming the filter media is biologically mature. A staff gauge to measure sand bed elevations in metric
units should be placed on the wall of each filter box to denote minimum and maximum sand levels.

(3) Sand Specifications. The sand size recommended by tradition is $d_{10} = 0.2$–0.3 mm, with UC = 1.5–2.0. Media having a sand size with $d_{10} \geq 0.3$ mm and UC > 2 may be acceptable if a pilot study ascertains that acceptable removals are obtained. Sieve analyses should be performed on the sand being considered. The sand size distribution should be plotted as in Appendix Figure C.1, with $d_{10}$ and $d_{60}$ clearly marked.

(4) Freeze Protection. In regions in which freezing temperatures occur, ice blocks should be prevented from forming on the headwater. The combinations of a roof and earth backfill insulation and of a roof and auxiliary heat have proven effective. If the formation of an ice block cannot be, or is not, prevented the sidewalls should be designed to accommodate or withstand the associated horizontal force. All pipes and equipment shall be protected from freezing.

8.3.4 Appurtenances

(1) Pipe Gallery. Pipes should be grouped into a pipe gallery to facilitate operation and maintenance. All meters and valves should be accessible from within the pipe gallery so that the operator can easily perform all needed operations and maintenance and can read all instruments without having to assume abnormal postures. Adequate space should be provided so that the operator can remove meters and valves from their positions and from the gallery.

(2) Sampling Taps. Both influent and effluent lines should have sampling taps for both grab samples and cartridge filter sampling with hose connections.

(3) Laboratory. A room with benches and distilled water should be provided for use in holding and processing samples. The laboratory should have, as a minimum, a turbidimeter and should keep an inventory of turbidimeter standards, clean sample bottles, and sterile plastic containers (proprietary plastic bags are most convenient) for coliform sampling.

(4) Office. An office should be provided with a desk and filing cabinet for records and reports. A computer is recommended for processing data and producing plots: The computer may be used to store data, but hard copies should be maintained in the files.

(5) Shop. Operators at the plant should be able to remove flow meters, adjust weir plates, and perform minor maintenance. The plant should have an inventory of tools for such purposes.
8.3.5 Disinfection

Disinfection should be provided subsequent to filtration. The installation should comply with state regulations.

8.4 PILOT PLANT STUDIES

Each water is unique and prediction is not feasible for the rate of headloss increase, the suitability of a sand, and other characteristics of operation. For this reason a pilot plant study is recommended to evaluate the suitability of slow sand and to predict operating characteristics.

8.4.1 Purpose

A pilot plant study is recommended to determine the headloss increase with time, as each water is unique. If the sand contemplated for use deviates from recommended sizes, a pilot study should be mandatory. Treatment efficiency should be ascertained at the same time.

8.4.2 Study Plan

A study plan should be developed prior to conducting a pilot plant study. The study plan should state the purpose of the study, objectives, scope, methods, and task schedule.

8.4.3 Method

The measurements to be taken should be stated and the instruments and equipment to be used should be indicated. A flow diagram of the pilot plant set-up should be given.

8.4.4 Results

The headloss data obtained in the pilot plant study should be plotted on headloss versus time curves for each of the water quality seasons. The time required for the sand bed to reach biological maturity may be determined by coliform spiking tests done at two-week or monthly intervals, with the first spike occurring during start-up. Influent and effluent levels of coliforms should be plotted with time for each spiking test.

8.4.5 Data Interpretation

The headloss versus time curves will indicate the cycle times between scrapings based upon the maximum headloss to be imposed in the design. The HLR must be stated. The plot of effluent coliform levels and corresponding percent removals or log-
removals against time in months for each of the coliform spikes will indicate the time to sand bed maturity.

8.5 CONSTRUCTION

All plans, specifications, and bid documents should be prepared by the project engineer. The engineer should conduct construction inspections and should develop the operating protocol in consultation with the plant operator.

8.5.1 Plan Approval and Building Permit

Prior to construction, plan approval must be obtained from the appropriate regulatory agency. A building permit is needed after the plan is approved and before construction begins.

8.5.2 Inspection

As the owner's agent, the engineer should inspect the construction and photograph and videotape all components of the structure to ensure compliance with plans and specifications.

8.5.3 Washed Sand

The sand placed in the filter box should be washed sand. As a goal, washing should be to the extent needed to ensure that less than 10 percent of the effluent turbidity at start-up is due to the erosion of fines from the sand bed. Such a goal does not require quantitative evaluation, unless desired, but is stated to indicate that the washing should be thorough.

8.6 OPERATION

8.6.1 Measurements, Sampling, Recording

1) Headloss and Temperature. Elevations of headwater and tailwater should be observed and recorded daily. Headwater temperature should be recorded daily in degrees Celsius.

2) Flow. Influent flow should be measured daily before and after any adjustment. The volumetric flow on the effluent side should also be measured daily.

3) Turbidity. Water should be collected daily in clean containers for turbidity sampling. Measurements should be done in the plant laboratory with an instrument that has been standardized using commercially available standards.
(4) **Coliforms.** Coliform levels should be sampled frequently. Sampling should be handled as required by state regulations, with analyses made by certified laboratories.

(5) **Records and Reports.** Self-explanatory record forms should be designed for all data. Recommendations are that (1) original and processed data be shown on the same form, organized into logical groupings; (2) plots be prepared from the forms, including cumulative flow volume versus time, headloss versus time, temperature versus time, turbidity versus time, and coliform versus time; (3) the plots be used to identify trends as well as deviations and discrepancies; and (4) that the plots show performance relative to applicable standards. Such data processing will be facilitated by the use of a computer software package, preferably one recommended by the state regulatory agency.

8.6.2 **Operations**

(1) **Drainage of Headwater.** When the headwater reaches the overflow level, or prior to that level, the headwater should be drained and the sand scraped. The water level should be lowered in two stages: (1) to the level of the distribution manifold, and (2) to a level just below the surface of the sand bed, that is, 2.5-5.0 cm (1-2 in.) below the surface. The first stage of drainage should be by means of the influent distribution orifices and manifold, and the second stage by means of the underdrain system. In both stages, the drained water is channeled to waste.

(2) **Scraping.** The *schmutzdecke* should be removed when the headloss reaches, or is less than, the overflow level. The depth of sand removed should be as small as is possible in order to remove just the *schmutzdecke*. Asphalt rakes can be used, with the sand raked to form windrows and then shoveled into buckets. The sand should be carried to a sand storage bin. For larger plants, mechanical methods of moving sand may be used.

(3) **Backfilling After Scraping.** After the sand bed is scraped, it should be backfilled with finished water through the underdrain system. The rate of backfilling should be slow until the water level reaches the sand surface (<2 m/hr), so that air trapped in the sand can be displaced. When the water level reaches the sand surface, the backfill rate may be increased. Backfilling is stopped when the headwater level reaches the level of the influent distribution manifold and orifices. At that point, the influent water may enter through the influent distribution system and the filtration mode can begin.

(4) **Filter-to-Waste.** When a slow sand filter is started initially or after resanding, the operating mode should be filter-to-waste so that residual fines can be flushed from the filter. Also, the filter-to-waste mode may be used to permit sand bed maturity to develop. The time to maturity may range from two weeks for a warm water that is nutrient rich to several months for cold waters that are nutrient poor.
(5) **Sand Washing.** The sand removed by scraping should be stored and then washed for recycling. The washing should be done at intervals when sufficient sand has accumulated to warrant such an operation.

(6) **Sand Storage.** Washed sand should be stored until resanding is necessary. The washed sand storage should be covered to provide protection against contamination.

(7) **Equipment for Operation.** For small installations, such as those with only a single filter and with a bed surface area small enough that two persons can complete the scraping operation within one shift of work, all operations can be done by hand and only asphalt rakes, shovels, and buckets or wheelbarrows are needed.

(8) **Cleaning Flow Meters.** Orifice meters should be installed in a way that allows for easy removal for cleaning. Any debris that accumulates behind the plate must be removed. Because small debris will pass by a Venturi meter, this type of meter should be used for installations in which debris accumulation is excessive. Taps for pressure gauges should be flushed periodically for cleaning.

(9) **Instrument Calibration.** All instruments should be calibrated at regular intervals. Flow meters will retain calibration unless deposits occur; any such deposits should be removed. Pressure gauges should be calibrated by imposing a known static pressure on the gauges and noting the corresponding gauge readings. Volumetric flow meters should be calibrated at annual intervals. At least two meters should be on hand so that a substitute meter is available if needed. Turbidity meters should be calibrated weekly using commercially available standards.

(10) **Data Processing.** Data may be processed by hand or computer on standard forms. For computerized data processing, commercially available spreadsheet software should be used. Calibration coefficients for flow meters and pressure gauges should be entered and identified on the forms and incorporated by reference in the processing functions. The computer forms should be saved in hard copy.

(11) **Sand Bed Maturity.** Sand bed maturity cannot be measured unless removal rates can be determined by means of an indicator, such as coliform bacteria. If adequate coliform levels are present in the ambient water, such as >100 cfu/100 mL, or if a pilot filter is operated in parallel with the full-scale filter and is spiked with coliform, then filter maturity may be assessed. Removals of 2-log to 4-log should be expected for a mature filter bed. A rule of thumb is that filter bed maturity may be assumed if the filter bed has been in operation for six months without being drained and without flow being interrupted for more than 24 hours. The foregoing is based upon work by Bellamy et al. (1985a, 1985b) and by Bryck et al. (1987) in which coliform removals of ≥2-log were noted within 4 months for cold, oligotrophic influent waters. However, filter bed maturity should be determined experimentally whenever possible.

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8.7 MODIFICATIONS

Modifications to the slow sand filtration process may be made in some situations to mitigate the short-term effects of some variables, such as high turbidity during one season of the year. Modification of the process is contrary to the philosophy of slow sand filtration as a passive technology, and modifications should be made only after careful consideration of the trade-offs. In addition to the removal of suspended solids, the removal of fulvic and humic acids (trihalomethane precursors) may be considered. Other kinds of modifications should be considered with caution and are mentioned here only to indicate that considerable experimentation has been in progress over recent years to explore the full potential of slow sand technology.

8.7.1 Suspended Solids Removal

(1) Sedimentation. The least costly method of removing high levels of suspended solids is to use a settling pond with ≥24 hours of detention time at peak flows. Accumulated solids may be removed from the pond annually or when accumulation warrants it.

(2) Roughing Filters. A roughing filter is inexpensive and requires less space than a pond. The sediment may be removed from the roughing filter by backwashing.

8.7.2 Organic Precursor Removal

Pre-ozonation has been used to break down organic precursors such as fulvic and humic acids and has been shown to provide removals on the order of 50 percent. Although the use of ozone is not compatible with the slow sand philosophy, its use has been established and may be considered. Prechlorination, on the other hand, should not be used.

(1) Surface Amendments. Various surface amendments have been used experimentally. Granular activated carbon, for example, will remove in excess of 80 percent of organic precursors. Clinoptilolite has been used to concentrate ammonia, which may be released as a nutrient to enhance bacteria growth.

8.7.3 Other

(1) Nutrient Additions. Some mountain waters have low levels of nutrients in addition to low winter temperatures, conditions that result in slow maturity of the filter bed. The time to filter bed maturity may be decreased by the addition of nutrients, such as nitrates and phosphates, to the filter.

(2) Harrowing. Filter harrowing has been used in full-scale plants in the northeast United States. Harrowing is faster than scraping and may be considered for large plants.
Definitions of Terms

A = plan area of filter bed (m²)
A₀ = maximum area of a single cell
A(max) = maximum calculated plan area of filter bed (m² or ft²)
A(orifice) = cross-section area of orifice opening (m² or ft²)
A(peak) = peak day storage volume (m³ or ft³)
A(pipe) = cross-section area of pipe (m² or ft²)
A(slot) = area of slot (m²)
Aₜ = cross-section area of Venturi throat (m² or ft²)
A(wall) = area of wall (m² or ft²)

b = length of weir crest (m or ft)
B = fire protection storage (m³ or ft³)
C = concentration of particles (number of particles/mL)
C(emerg storage) = emergency storage (m³ or ft³)
Cₜ = orifice discharge coefficient, given in Appendix Table B.1 (dimensionless)
Cᵥ = Venturi meter discharge coefficient = 1.0 (dimensionless)
CᵥN = triangular weir discharge coefficient (dimensionless)
Cₜ = weir coefficient = 0.58 when θ = 60°, given in Appendix Table B.2 (dimensionless)

d = diameter of orifice in orifice plate (m or ft)
d₁₀ = sand grain size, 10 percent of which is finer than d₁₀ (mm)
d₆₀ = sand grain size, 60 percent of which is finer than d₆₀ (mm)
d₉₀ = sand grain size, 90 percent of which is finer than d₉₀ (mm)
D = diameter of pipe (m or ft)
Dᵢ = final sand bed depth before rebuilding (cm or ft)
dh/dz = hydraulic gradient, i.e., loss of head/unit length of flow (m/m)
Dᵢ = initial sand bed depth (cm or ft)

F = force on wall (N or lb)
f = Darcy-Weisbach friction factor for pipe material (dimensionless)
f(scraping) = frequency of scraping (scrapings/year)

\( g = \text{acceleration of gravity (9.81 m/s}^2 \text{ or 32.2 ft/s}^2 \) 
\([\text{gas}] = \text{concentration of gas species (square brackets symbolize concentration) (mg/L)} \)

\( h = \text{depth below water surface (m or ft)} \)
\( h_L = \text{headloss used generically for several kinds of hydraulic conditions, such as headloss available across the filter bed from headwater to tailwater, headloss in pipe flow, etc. (m or ft)} \)
\( h_L(\text{drainpipe}) = \text{headloss in drainpipe in length, L (m or ft)} \)
\( \Delta h = \text{pressure difference two points in which water is flowing, such as across an orifice plate, a Venturi meter, or within a pipe, or within a sand bed; may be the same as headloss (m or ft)} \)
\( H = \text{head on weir, i.e., height of water level with respect to weir crest, upstream from effect of drawdown (m or ft)} \)
\( H_{\text{gas}} = \text{Henry's law coefficient (mg/L/atm partial pressure)} \)
\( H_i = \text{Henry's law coefficient for gas i} \)
\( \text{HLR} = \text{hydraulic loading rate, defined as flow divided by plan area of sand bed (m}^3/\text{m}^2/\text{hr or mgad)} \)

\( i = \text{gas species} \)
\( [i] = \text{gas concentration} \)

\( k = \text{hydraulic conductivity of porous media (m/hr)} \)
\( k(\text{meter}) = \text{coefficient of proportionality to calibrate flow meter (m}^3/\text{rev)} \)
\( k' = \text{intrinsic hydraulic conductivity (N/m or lb/ft)} \)

\( L = \text{length of pipe for headloss measurement (m or ft)} \)

\( N = \text{number of revolutions of propeller associated with volume, V (rev)} \)
\( N(\text{orifices}) = \text{number of orifices in underdrain lateral} \)

\( P = \text{distance from bottom of channel to weir crest (m or ft)} \)
\( p = \text{pressure, used generically as hydraulic pressure such as below a water surface, within a pipe, within the sand bed, etc. (pascals or lb/ft}^2 \) at depth h \)
\( P_B = \text{pressure at point B in closed hydraulic system (N/m}^2 \text{ or lb/ft}^2 \) \)
\( P(\text{depth} = d) = \text{local pressure at given depth, } d \)

\( P_{\text{gas}} = \text{partial pressure of gas species at gas-liquid interface (N/m}^2 \text{ or lb/ft}^2 \) 

\( P(\text{gas } i) = \text{partial pressure} \)

\( Q = \text{flow of water; term is used generically to indicate flows in a variety of situations (m}^3/\text{s or ft}^3/\text{s)} \)

\( q(\text{lateral}) = \text{flow in underdrain lateral (m}^3/\text{s)} \)

\( Q(\text{orifice}) = \text{flow through orifice (L/s or ft}^3/\text{s)} \)

\( q(\text{orifice}) = \text{flow through orifice in underdrain pipe (m}^3/\text{s)} \)

\( q(\text{slot}) = \text{flow through orifice slot in underdrain pipe (m}^3/\text{s)} \)

\( R = \text{sand depth removal per scraping (cm/scraping or ft/scraping)} \)

\( T = \text{total storage volume required (m}^3 \text{ or ft}^3 \)

\( t = \text{time for flow of volume, } V \text{ (s)} \)

\( U_C = \text{ratio of } d_{60}/d_{10} \)

\( V = \text{volume of flow that has passed through the propeller (m}^3 \text{ or ft}^3 \)

\( v = \text{superficial velocity, also called hydraulic loading rate, defined as } Q/A \text{ (m/hr or ft/hr)} \)

\( v(\text{pipe}) = \text{velocity of water flowing within pipe (m/s or ft/s)} \)

\( Y = \text{years of operation before sand bed rebuilding is necessary (years)} \)

\( z = \text{flow distance through porous media, used generically for different situations, such as, in the case of a filter, the distance from top of filter bed at which C is measured (m or ft)} \)

\( z_B = \text{vertical distance from a datum to point B in closed hydraulic system (m or ft)} \)

\( \Delta z = \text{depth of filter bed (m or ft)} \)

\( \alpha = \text{attachment coefficient, defined as fraction of particle that attaches to a sand grain } \)

"collector" \( \text{relative to the number of collisions} \)

\( \gamma = \text{specific weight of water (9,990 N/m}^3 \text{ or 62.4 lb/ft}^3 \)

\( \eta = \text{collision probability coefficient, defined as fraction of particles that strike a sand grain } \)

\( \text{collector relative to the number approaching (particles striking collector per unit cross-section area per unit time divided by particles approaching per unit cross-section per unit time)} \)
\( \theta \) = angle of notch in triangular weir (degrees)
\( \lambda \) = filter coefficient, which is the slope of a particle concentration profile on a semi-log plot at any point, \( z \), measured from surface, at any given time, within a filter sand bed (cm\(^{-1}\))
\( \mu \) = dynamic viscosity of water at given temperature (N-s/m\(^2\))
\( \nu \) = kinematic viscosity (cm\(^2\)/s)
\( \pi = 3.1416 \)
\( \rho \) = density of water (g/cm\(^3\))
### Abbreviations

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>AFDC</td>
<td>acriflavine direct cell counts</td>
</tr>
<tr>
<td>AOC</td>
<td>assimilable organic carbon</td>
</tr>
<tr>
<td>ASCE</td>
<td>American Society of Civil Engineers</td>
</tr>
<tr>
<td>atm</td>
<td>atmosphere</td>
</tr>
<tr>
<td>AWWARF</td>
<td>American Water Works Association Research Foundation</td>
</tr>
<tr>
<td>c</td>
<td>capita</td>
</tr>
<tr>
<td>°C</td>
<td>degrees Celcius</td>
</tr>
<tr>
<td>cm</td>
<td>centimeter</td>
</tr>
<tr>
<td>cfu</td>
<td>colony forming unit</td>
</tr>
<tr>
<td>CSI</td>
<td>Construction Specifications Institute</td>
</tr>
<tr>
<td>C*T</td>
<td>product of residual disinfectant concentration, C (in mg/L), and disinfectant contact time, T (in min)</td>
</tr>
<tr>
<td>CV</td>
<td>coefficient of variation</td>
</tr>
<tr>
<td>d</td>
<td>day</td>
</tr>
<tr>
<td>DOC</td>
<td>dissolved organic carbon</td>
</tr>
<tr>
<td>EGL</td>
<td>energy grade line</td>
</tr>
<tr>
<td>EPA</td>
<td>Environmental Protection Agency</td>
</tr>
<tr>
<td>FOB</td>
<td>free on rail</td>
</tr>
<tr>
<td>FRM</td>
<td>Folin reactive material</td>
</tr>
<tr>
<td>ft</td>
<td>feet</td>
</tr>
<tr>
<td>g</td>
<td>gallon</td>
</tr>
<tr>
<td>GAC</td>
<td>granular activated carbon</td>
</tr>
<tr>
<td>gpcd</td>
<td>gallons per capita per day</td>
</tr>
<tr>
<td>gpd</td>
<td>gallons per day</td>
</tr>
</tbody>
</table>
gpm  gallons per minute

ha  hectare
HGL  hydraulic grade line
HLR  hydraulic loading rate
hp  horsepower
HPC  heterotrophic plate count
hr  hour

in.  inch
IOCs  inorganic chemicals

kg  kilogram
km  kilometer
kPa  kilopascal

L  liter
lb  pound
L/c/d  liters per capita per day

m  meter
MCL  maximum contaminant level
m³/d  cubic meters per day
mg/acre  milligrams per acre
mgad  million gallons per acre per day
mgd  million gallons per day
mg/L  milligrams per liter
m/hr  meters per hour
mi  mile
mil L/d  million liters per day
min  minute
mL  milliliter
mm  millimeter
MPN  most probable number
mT  metric ton
mv  millivolt
µ  micro
µm  micrometer
N  newton
nm  nanometers
N/m  newtons per meter
N-s/m²  newton seconds per square meter
NPDES  National Pollution Discharge Elimination System (an acronym introduced with Public Law 9250, 1972)
NPDOC  nonpurgeable dissolved organic carbon
NPDWR  National Primary Drinking Water Regulations
NTU  nephelometric turbidity units
Pt-Co  platinum-cobalt units
rev  revolution
s  second
SI  System International
SOCs  synthetic organic chemicals
SWTR  Surface Water Treatment Rule
THM  trihalomethane
THMFP  trihalomethane formation potential
TOC  total organic carbon
UC  uniformity coefficient
USEPA  U.S. Environmental Protection Agency
UV  ultraviolet
UVA  ultraviolet absorbance
VOCs  volatile organic chemicals
WHO  World Health Organization
yd³  cubic yard
Glossary

air-binding – the presence of gas bubbles in the sand bed, causing increased headloss and possible rupture of the schmutzdecke on their release

appurtenances – accessories, such as meters

atmosphere – unit of pressure; 1 atm = 14.7 lb/in.² = 101.325 kPa = 760 mm mercury

backfilling – saturation of the sand bed by filling it slowly from the bottom upwards to displace air in the bed

biologically mature – see "mature"

bleeding – allowing faucets to drip to prevent pipes from freezing

breakthrough – passage of contaminants through the sand bed

cfu, colony forming unit – a measure of bacteria concentration

chlorine demand – the chlorine fed into the water that reacts with oxidizable impurities and may therefore not be available for disinfection, reported in units of mg/L

chlorine residual – the concentration of chlorine, in mg/L, remaining in the water after the chlorine demand has been met

clear well – a facility for the storage of treated water

coliform removal, coliform removal efficiency – the extent of retention of coliform bacteria by a filter

constant-head box – a device that controls flow rate to the filter(s) by maintaining a constant head of liquid over fixed orifices; used in pilot plant studies

control building – the facility that houses meters

Cryptosporidium parvum – a pathogenic protozoan that causes enteritis and/or severe diarrhea; the oocyst is resistant to chlorine disinfection

C•T – the product of residual disinfectant concentration (C) in mg/L and the corresponding disinfectant contact time (T) in minutes

cumulative flow demand – the total volume of flow calculated over a given time period; the cumulative flow volume is calculated by summing the product of the flow rate and the duration of that flow rate over the time period of interest
$d_{10}$ – the size of the sieve opening through which 10 percent of the sand will just pass; also called the "effective size".

design flow – the maximum daily flow for a projected population

detritus – nonliving organic matter

dewatering – draining the headwater to below the sand bed surface

diatomaceous earth – high-silica skeletal remains of algae, used as a filtration material

drawdown – drainage of the headwater

effective size – see "$d_{10}$"

energy grade line, EGL – a graphic representation of the total energy head, which shows the rate at which energy decreases; the EGL always drops downward in the direction of flow unless there is an energy input from a pump

filter harrowing – disturbing the filter media surface with a large implement consisting of a series of teeth or disklike blades

filter mats – nonwoven synthetic fabrics placed on the surface of the filter media to facilitate cleaning

filter-to-waste – the process of disposing of tailwater (filter effluent) while the filter product water is of unacceptable quality

fines – the smallest particles in unwashed sand

gas precipitation – a phenomenon whereby dissolved gas comes out of solution due to supersaturation

giardiasis – disease caused by the protozoan *Giardia lamblia*

*Giardia* – name often used in conversation for the pathogenic organism *Giardia lamblia*

*Giardia lamblia* – a pathogenic protozoan that causes severe diarrhea

gravel support – coarse media on which the filter media is placed, and which surrounds and covers the underdrains

headloss – loss of media permeability; increased flow resistance

headwater – the raw water in the filter box directly above the filter bed; also called supernatant water

hydraulic conductivity – permeability, reported as length/time

hydraulic grade line, HGL – the piezometric head line, i.e., a graphic representation of what would be the free surface if one could exist, and the same conditions of flow were maintained; if the velocity head is constant, the drop in the hydraulic grade line between any two points is the value of the loss of head between those two points
hydraulic loading rate – volumetric flow rate divided by filter surface area, resulting in
units of length/time

log removal, logR – defined as logR = log N_in − log N_out, in which N_in is the concentra-
tion in the influent and N_out is the concentration in the effluent of whatever
constituent species is being measured. Conversion to %R is: %R = 1 − 10^-logR

mature – the state of a filter when coliform removal has reached its optimum level

newton – unit of force; 1 N = 0.2248 lb force

package plant – a commercially available prefabricated filter

Pascal – unit of pressure; 1 Pascal = 1 N/m^2 = 0.000145 lb/ft^2

peak flow – a community’s maximum daily water demand

percent removal, %R – defined as \( \frac{N_{in} - N_{out}}{N_{in}} \cdot 100 \); see also "log removal"

performance capacity – the flow rate above which the performance of the filter (effluent
turbidity, rate of headloss development, etc.) no longer satisfies regulatory or
community requirements

pilot plant – a small-scale replica of a proposed or existing full-scale facility, useful in
determining, at relatively low expense, the feasibility of the full-scale plant in
achieving the desired finished water quality given the raw water characteristics

poise – unit of dynamic viscosity; 1 poise = 0.10 N-s/m^2

pre-ozonation – oxidation of the raw water prior to filtration

reservoir – the filter box zone above the filter media; the location of the headwater or
supernatant water

ripening – the process whereby a diverse biological community develops within a filter
bed

roughing filter – a pretreatment consisting of a series of chambers of coarse media that
serves to reduce raw water turbidity

run length – the period of time between filter start-up and terminal headloss

sand bed – the filter media

schmutzdecke – a German word that translates literally as "dirty layer" and was adopted
early in American practice; the layer of material deposited on the top of the filter
bed that causes headloss disproportionate to its thickness

scour – disturbance of the filter media, usually caused by high-velocity discharge of
water into the filter

scraping – removing the top few centimeters (approximately 1 in.) of the filter media,
including the schmutzdecke

sedimentation – settling
Stoke – 1 cm²/s

supernatant water – headwater; the raw water in the reservoir


tailwater – the filter effluent

THMFP – trihalomethane formation potential

turbidity – cloudiness of the water

underdrain – a network of pipes that collects filtered water and channels it out of the filter box

uniformity coefficient, UC, \( d_{60} / d_{10} \) – the ratio of the sieve size through which 60 percent of the sand will pass to the size through which 10 percent will pass

UVA – ultraviolet absorbance; relates to instrumental method to measure dissolved organic carbon

weir – a plate or other device used to control the water level
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## Appendix A

# Viscosity of Water

<table>
<thead>
<tr>
<th>Temp. (°C)</th>
<th>Dynamic viscosity(^a) (poises)(^b)</th>
<th>Kinematic viscosity(^b,c) (Stokes)</th>
<th>Dynamic viscosity(^d) (N·s/m(^2))</th>
<th>Kinematic viscosity(^e) (m(^2)/s)</th>
<th>English</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1.787 × 10(^{-2})</td>
<td>1.787 × 10(^{-3})</td>
<td>1.787 × 10(^{-6})</td>
<td>3.73 × 10(^{-5})</td>
<td>1.92 × 10(^{-5})</td>
</tr>
<tr>
<td>1</td>
<td>1.728 × 10(^{-2})</td>
<td>1.728 × 10(^{-3})</td>
<td>1.728 × 10(^{-6})</td>
<td>3.61 × 10(^{-5})</td>
<td>1.86 × 10(^{-5})</td>
</tr>
<tr>
<td>2</td>
<td>1.671 × 10(^{-2})</td>
<td>1.671 × 10(^{-3})</td>
<td>1.671 × 10(^{-6})</td>
<td>3.49 × 10(^{-5})</td>
<td>1.80 × 10(^{-5})</td>
</tr>
<tr>
<td>3</td>
<td>1.613 × 10(^{-2})</td>
<td>1.613 × 10(^{-3})</td>
<td>1.613 × 10(^{-6})</td>
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(continues)
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<tr>
<th>Temp. (°C)</th>
<th>Dynamic viscosity a (poises) b</th>
<th>Kinematic viscosity b,c (Stokes)</th>
<th>Dynamic viscosity d (N-s/m²)</th>
<th>Kinematic viscosity e (m²/s)</th>
<th>Dynamic viscosity f (lb-s/ft²)</th>
<th>Kinematic viscosity g (ft²/s)</th>
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<td>0.8010×10⁻²</td>
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<td>1.66×10⁻⁵</td>
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<td>0.7844×10⁻²</td>
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<td>0.7531×10⁻²</td>
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<td>0.719×10⁻³</td>
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<td>0.705×10⁻³</td>
<td>0.710×10⁻⁶</td>
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<tr>
<td>37</td>
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<td>0.6961×10⁻²</td>
<td>0.692×10⁻³</td>
<td>0.696×10⁻⁶</td>
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<td>0.75×10⁻⁵</td>
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<td>38</td>
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<tr>
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<td>0.6703×10⁻²</td>
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<td>0.68×10⁻⁵</td>
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<td>43</td>
<td>0.6178×10⁻²</td>
<td>0.6234×10⁻²</td>
<td>0.618×10⁻³</td>
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<td>0.67×10⁻⁵</td>
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<td>44</td>
<td>0.6067×10⁻²</td>
<td>0.6124×10⁻²</td>
<td>0.607×10⁻³</td>
<td>0.612×10⁻⁶</td>
<td>1.27×10⁻⁵</td>
<td>0.66×10⁻⁵</td>
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<td>45</td>
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<td>0.6019×10⁻²</td>
<td>0.596×10⁻³</td>
<td>0.602×10⁻⁶</td>
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<td>0.65×10⁻⁵</td>
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<td>46</td>
<td>0.5856×10⁻²</td>
<td>0.5916×10⁻²</td>
<td>0.586×10⁻³</td>
<td>0.592×10⁻⁶</td>
<td>1.22×10⁻⁵</td>
<td>0.64×10⁻⁵</td>
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<td>47</td>
<td>0.5755×10⁻²</td>
<td>0.5817×10⁻²</td>
<td>0.576×10⁻³</td>
<td>0.582×10⁻⁶</td>
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<td>48</td>
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<td>0.5719×10⁻²</td>
<td>0.566×10⁻³</td>
<td>0.572×10⁻⁶</td>
<td>1.18×10⁻⁵</td>
<td>0.61×10⁻⁵</td>
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<td>49</td>
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<td>0.5626×10⁻²</td>
<td>0.556×10⁻³</td>
<td>0.563×10⁻⁶</td>
<td>1.16×10⁻⁵</td>
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<td>50</td>
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<td>0.547×10⁻³</td>
<td>0.553×10⁻⁶</td>
<td>1.14×10⁻⁵</td>
<td>0.59×10⁻⁵</td>
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</tbody>
</table>

b 1 poise = g·s/cm (gram-second/centimeter), 1 Stoke = 1 cm²/s.
c Kinematic viscosity, \( \nu \), is calculated as:

\[
\nu = \frac{\mu}{\rho}
\]

in which: \( \nu \) = kinematic viscosity (cm²/s)
\( \mu \) = dynamic viscosity (g·s/cm)
\( \rho \) = density of water (g/cm³)

d Dynamic viscosity, \( \mu \), is taken from column 1; density, \( \rho \), is taken from Weast (1978, p. F-11), with interpolation made between 5°C temperature increments.

e To convert dynamic viscosity in poises to System International (SI) units:

\[
\mu(\text{N·s/m}^2) = \left(\frac{\text{g}}{\text{cm}^2 \cdot \text{s}}\right) \cdot \left(\frac{1 \text{ kg}}{1000 \text{ g}}\right) \cdot \left(\frac{100 \text{ cm}}{1 \text{ m}}\right) \cdot \left(\frac{\text{N·s}}{\text{kg}}\right) = 0.1 \frac{\text{N·s}}{\text{m}^2}
\]

f To convert dynamic viscosity in poises to English units in lb-s/ft², multiply poises by 2.089×10⁻³ (Weast 1978, p. F-50).
g To convert kinematic viscosity in Stokes to English units in ft²/s, multiply Stokes by 1.0761×10⁻³ (Weast 1978, p. F-50).
# Appendix B

## Flow Meter Coefficients

### Table B.1

Meter Coefficients for Flow Measurement

<table>
<thead>
<tr>
<th>Meter type</th>
<th>Metering formula</th>
<th>d/D</th>
<th>C_d</th>
</tr>
</thead>
<tbody>
<tr>
<td>Orifice</td>
<td>( Q = C_d A (2gh)^{1/2} )</td>
<td>0.10</td>
<td>0.60</td>
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<tr>
<td></td>
<td></td>
<td>0.20</td>
<td>0.60</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.30</td>
<td>0.60</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.40</td>
<td>0.615</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.50</td>
<td>0.625</td>
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<td></td>
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<td>0.60</td>
<td>0.65</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.70</td>
<td>0.70</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.80</td>
<td>0.77</td>
</tr>
<tr>
<td>Venturi</td>
<td>( Q = C_v A_t (2g\Delta h)^{1/2} )</td>
<td>0.50</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.60</td>
<td>1.04</td>
</tr>
</tbody>
</table>

Notes: Coefficients are for \( R \geq 10^5 \). Reynold's number is defined as \( R = \frac{pvD}{\mu} \). For lower Reynold's numbers, the coefficient increases for orifices and decreases for Venturi meters. For most flows, \( R \geq 10^5 \) can be assumed. \( D \) is the diameter of the pipe, and \( d \) is the diameter of the orifice or Venturi throat.

Table B.2
Weir Coefficients for Flow Measurement

<table>
<thead>
<tr>
<th>Meter type</th>
<th>Metering formula</th>
<th>H/P</th>
<th>C&lt;sub&gt;w&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rectangular weir</td>
<td>( Q = C_w \sqrt{2g} \ b H^{3/2} )</td>
<td>0.00</td>
<td>0.40</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.10</td>
<td>0.405</td>
</tr>
<tr>
<td></td>
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<td>0.41</td>
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<tr>
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<td>0.415</td>
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<td>0.40</td>
<td>0.42</td>
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<tr>
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<td>0.50</td>
<td>0.425</td>
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<td></td>
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<td>0.60</td>
<td>0.43</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.80</td>
<td>0.44</td>
</tr>
<tr>
<td>V-notch weir</td>
<td>( Q = \frac{8}{15} C_{VN} \sqrt{2g} \ tan \left( \frac{\theta}{2} \right) H^{5/2} )</td>
<td>60°</td>
<td>0.58</td>
</tr>
<tr>
<td>Metric</td>
<td>( Q = 0.79 H^{5/2} )</td>
<td>60°</td>
<td>0.58</td>
</tr>
<tr>
<td>English</td>
<td>( Q = 1.44 H^{5/2} )</td>
<td>60°</td>
<td>0.58</td>
</tr>
</tbody>
</table>

Notes: For a rectangular weir, \( C_w = 0.40 + 0.05H/P \). For a V-notch weir, \( C_{VN} = 0.58 \) when \( \theta = 60° \). The terms of the equations used in this table are as follows: \( Q \) = flow in m\(^3\)/s or ft\(^3\)/s, \( C_w \) = weir coefficient; \( g \) = acceleration of gravity (9.81 m/s\(^2\) or 32.2 ft/s\(^2\)).

Figure B.1 Definition Sketches for Orifice Meter and Venturi Meter, Showing Pressure Taps

Figure B.2 Definition Sketch for Rectangular Weir With End Contractions, Showing Terms Used in Flow Equation and Weir Coefficient Equation

Figure B.3 Definition Sketch for a Triangular Weir
Appendix C

Plotting Paper for Filter Sand

<table>
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<th>SAND</th>
<th>COARSE TO MEDIUM</th>
<th>GRAVEL</th>
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U.S. STANDARD SIEVE SIZES

Figure C.1 Sieve Analysis Plotting Form With Grain Diameter to Sieve Size Conversion
Appendix D

C•T Data for *Giardia lamblia* Cysts

Figure D.1 Chlorine C•T Data for 3-Log (99.9 Percent) Inactivation of *Giardia lamblia* Cysts as Affected by Temperature in 0.5–5.0°C Range, With pH 6.0–8.0. Curves were obtained based upon interpolation between data points shown (Hibler et al. 1987). Percent inactivation was determined by infection rate of gerbils after ingestion of chlorinated water having indicated residual C•Ts, assuming 1–5 cysts needed to infect a gerbil, with 1–4 gerbils infected averaged with zero infected.