Water System Design Manual

Washington State Department of Health
Environmental Public Health
Office of Drinking Water

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Chapter 1: Introduction

1.0 Purpose and Scope

We developed the Water System Design Manual to establish uniform concepts for water system design and a framework for our own licensed engineers to perform consistent review of design documents. The manual is intended for Group A public water systems, which are regulated under the federal Safe Drinking Water Act and Chapter 246-290 WAC. We have separate design guidelines for Group B public water systems, which are so small that they are regulated only under Washington State law (Chapter 246-291 WAC).

This manual provides guidelines and criteria for design engineers to use in preparing portions of planning documents (WAC 246-290-100), project reports (WAC 246-290-110), construction documents (WAC 246-290-120), and source approval documents (WAC 246-290-135). This manual also clarifies engineering document submittal and review requirements. Most of the requirements in this manual are applicable to Group A water systems of all sizes. However, some of the design guidelines, such as the information on demand estimation and capacity analysis in Chapters 3 and 4 are focused more on water systems serving fewer than 1,000 connections.

1.1 Safety, Risk, and Reliability

The mission of the Department of Health Office of Drinking Water is to work with others to protect the health of the people of Washington State by ensuring safe and reliable drinking water. We believe water system owners, operators, and design engineers share this mission.

This manual identifies design requirements and design standards for ensuring safe and reliable drinking water sources and facilities. Where this manual doesn't simply restate a regulatory requirement it reflects our best thinking on what constitutes the basis for the design of a safe, reliable, and sustainable water system; one that does not result in exhausted sources, empty reservoirs, premature equipment breakdowns, contamination, low service pressures, or destructive pressure surges. In establishing these standards, we attempted to balance the reduction in risk against the added cost to provide that risk reduction and the capacity of water systems to maintain the associated physical and human infrastructure.

WAC 246-290-040 requires that all water system design work shall be prepared under the direction of a professional engineer licensed in the state of Washington and bear the seal, date, and signature of the professional engineer. Our licensed professional engineers review engineering documents with a focus on risk reduction and public health protection. In reviewing engineering documents we intend to ensure compliance with regulatory standards. We also strive to share our collective experience to promote construction and operation of appropriate, safe, reliable, and sustainable public water supply systems. Our ultimate goal is to help the design engineer and utility owner build a project that will be safe and reliable now and into the future. We do this by asking questions exploring risk versus available resources in the design phase, and helping water system owners and design engineers identify potential consequences of operational failure (e.g. contamination leading to illness, impacts of health advisories, permit restriction, or legal liability).

Design engineers need to know what we think are appropriate design standards, but they also need flexibility to approach the unique design circumstances they face. We encourage design engineers to consider various alternatives and options, provided the selected approach does not conflict with regulation. If the designer's selected approach differs from our standards, we expect the design engineer to provide us justification of his or her design decisions.

We're interested in hearing from users of this manual who believe we've misjudged the balance point between cost and public health protection. Please contact one of our regional offices with any comments or questions. We periodically review our design guidance and will update it as appropriate.

1.2 Applicability of Group A Rules

Our Group A rules apply to a water system that regularly serves 15 or more service connections or 25 or more people per day for 60 or more days per year. A water system, such as that serving people within a large building, is not subject to our rules if the building's water system meets all the following criteria:

1. It consists only of distribution or storage facilities without source or treatment facilities.
2. It obtains all its water from another regulated water system.
3. It is not an interstate passenger conveyance carrier.
4. It does not sell water directly to any person.

EPA has issued policies to clarify the applicability of the Safe Drinking Water Act in certain situations. Based on our review of these policies:

- Submetering individual dwelling units within a larger multi-family building does not trigger application of chapter 246-290 WAC. We do not consider apartment owners who install meters (sub-meters) and bill their tenants for actual water consumption as water systems subject to regulation.
- Installation of treatment within a building which serves 25 or more people per day for 60 or more days per year does trigger regulation under chapter 246-290 WAC. However, depending on the purpose and type of treatment, and the size and type of population served we may not require regulation of the building as a public water system. Please contact one of our regional offices for guidance.

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We have policies intended to clarify and interpret state drinking water regulations. At the time this manual was published we had a number of policies which may impact water system design and planning. We have attempted to reference our policies in applicable sections of this manual. As new policies may be added and others rescinded, we encourage you to review our policy web page to determine if any current policies apply to your design.

Many water systems were built before the current minimum design requirements were established in chapter 246-290 WAC. Design engineers **must** use the most recent standards and guidelines when designing new facilities or in planning expansion of an existing system (WAC 246-290 Part 3).

1.3 "Must" versus "Should"

Throughout this manual we use "**must**," "**will**," "**shall,**" or "**required**" when design practice is sufficiently standardized to permit specific delineation of requirements, or where safeguarding public health justifies definitive criteria or action (such as state statute or rule requirements). Design engineers have an obligation to satisfy the criteria in such instances.

"Should" or "recommend" indicate procedures, criteria, or methods that represent our view of best practices and can be approached with some degree of flexibility. Design engineers need to explain the basis of the altered approach or, in specific circumstances, why another approach may be more applicable.

1.4 Engineering Requirements

Water systems **must** be designed by professional engineers who are licensed in Washington State and who are qualified and experienced in designing drinking water systems and their various components (see chapter 18.43 RCW and WAC 246-290-040). There is a limited exception to these requirements for federal employees who practice engineering in the state of Washington for the federal government and possess a valid professional engineer's license from another state (RCW 18.43.130(6)).

Engineers are professionally bound to perform work only within their own fields of competence (WAC 196-27A-020(2)). For more complex projects, the involvement of structural, electrical, mechanical and other licensed professional engineers is usually necessary.

1.5 Minimum System Design Requirements

Good engineering practice (as determined by the Washington State Professional Licensing Board) **must** be used in all aspects of water system design (WAC 246-290-200). The design engineer **must** consider the water system operation under a full range of expected demands (minimum to maximum) and emergency conditions (WAC 246-290-200). "Emergency" means a natural or man-made event that causes damage or disrupts normal operations and requires prompt action to protect public health and safety. Examples include fires, power outages, water-main breaks, water system component or treatment process failures, floods, or recent evidence of contaminated drinking water.

1.6 Other Referenced Documents and Standards

We cite various waterworks-related laws, guides, standards and other documents in this manual to provide appropriate references. These references form a part of this manual, but it is not our intent to duplicate them. If references are not available, this manual defines the appropriate design procedures.

A number of waterworks industry standards and guidance exist, such as those from the American Water Works Association (AWWA), the American Society of Civil Engineers (ASCE), and *Recommended Standards for Water Works* (commonly called the Ten State Standards). If information in the manual conflicts with any referenced material, this manual should take precedence for purposes of designing water system facilities to meet our requirements. Otherwise, the design engineer is responsible for adequately justifying deviation from these guidelines when submitting the project design to us for review and approval.

Where applicable, all water system designs **must** also comply with locally adopted national model codes such as the *International Building Code* and *Uniform Plumbing Code* (WAC 246-290-200).

See Appendix C for a list of professional organizations and agencies with established standards and criteria referenced within this manual or the regulations.

Chapter 2: Project Reports and Construction Documents

2.0 General Engineering Project Submittal Requirements

Chapter 2 provides information to assist design engineers in preparing complete engineering documents submitted for our review and approval. Complete, concise, and accurate submittals enable us to work efficiently and to meet our target review timeframe. We strive to complete our initial review of project reports and construction documents within about 30 days. Incomplete submittals will delay the formal start of our review process.

Incomplete project submittals will result in delayed project review and may result in increased review fees (due to the need for multiple reviews). We will even return - un-reviewed - significantly incomplete or inaccurate submittals. This manual includes submittal checklists for many common projects. We suggest you reference them before submitting your final design to us for approval. Doing so will help ensure timely and efficient review of your submittal.

A complete submittal:

1. Includes a completed *Project Approval Application Form* (DOH 331-149-F)
2. Addresses all relevant elements identified in regulation and this manual
3. Articulates information clearly, concisely, and logically

2.1 Relationship of Project Reports and Construction Documents to Planning Requirements

Water systems satisfy the planning requirements in Chapter 246-290 WAC by preparing either a water system plan (WSP) or a small water system management program (SWSMP). WAC 246-290-100 identifies the conditions under which a water system **must** prepare a WSP or WSP amendment for our review and approval. Water systems not required to prepare a WSP **must** develop and implement a SWSMP (WAC 246-290-105).

These planning documents provide a structured process for water systems to:

1. Identify present and future needs.
2. Set forth means for addressing those needs.
3. Demonstrate the operational, technical, managerial, and financial capacity to achieve and maintain compliance with all relevant local, state, and federal plans and rules.
4. Demonstrate sufficient physical capacity and water rights for current and future needs. Together these comprise system capacity.

The water system planning requirement is closely linked with water system design. Water systems preparing a WSP must assess system capacity, identify deficiencies, and where needed establish an improvement plan necessary to maintain system capacity. Water systems preparing a

SWSMP must assess its infrastructure and list improvements associated with current and anticipated infrastructure deficiencies. Design engineers should reference information in a WSP or SWSMP when preparing a project report for new facilities or modifications to existing facilities.

If a water system contemplates preparing a project report or construction documents and the project is not included in a current, approved WSP, the water system and design engineer should contact one of our regional offices for guidance on specific planning requirements associated with the project.

WAC 246-290-100(3) allows a water system to combine project reports (but not construction documents) with a WSP. If all the information required for a project report under WAC 246-290-110 is in the water system's approved WSP, a separate project report is not required.

2.2 Submittal of Project Reports and Construction Documents

Unless a project is otherwise exempt from the project report or construction document submittal process (WAC 246-290-125), a water system proposing to construct any new water system, water system expansion, or improvement **must** first submit a project report (WAC 246-290-110) and construction document (WAC 246-290-120 and Policy J.21) to us for review and approval. Sections 2.8 and 2.9 describe exceptions to this requirement.

Design engineers should plan to submit engineering documents to us for our approval in paper form. At this time exceptions could be made, in advance, for small documents to be submitted electronically. Design engineers should consult with the appropriate DOH review engineer prior to considering an electronic submittal.

If the construction documents will be part of a bidding package we recommend design engineers submit construction documents in time to complete the review before bid solicitation starts. We expect to take about 30 calendar days to review construction documents, measured from the date we receive a complete submittal. If the design engineer fails to obtain approval prior to soliciting bids, the utility may have to solicit new bids or deal with significant change orders or contract amendments.

If the initial review results in a comment letter requiring an additional submittal and review cycle before approval, the time frame for approval will be significantly extended and may result in additional review fees. Comments requiring resubmittal are most often associated with:

- Incomplete submittals, where basic requirements of the submittal are not met
- Errors and/or omissions
- Lack of clarity or consistency in the design
- Insufficient evidence or justification that the project objective(s) will be met

Anyone constructing improvements to a water system that requires prior written approval from us and fails to do so is subject to administrative penalties. See WAC 246-290-050(7). Contact one of our regional offices if you are uncertain whether the planned project must be approved by us before construction.

2.3 Relationship between Project Approval and Operating Permit

Every Group A public water system **must** obtain an annual operating permit (see Chapter 246-294 WAC). Operating permit designations reflect compliance or approval status, or both. Project report and construction document approval does not change a water system's operating permit designation. However, the approval of a WSP or SWSMP, and the approval of construction documents and completion of the associated construction may satisfy requirements necessary to maintain or improve a water system's compliance or approval status, and therefore maintain or change the water system's operating permit designation.

2.4 Project Reports

A project report describes the basis for a project and includes calculations to show how the project will meet its objectives (WAC 246-290-110). Design engineers usually prepare the project report before preparing construction documents. See Section 2.5 and WAC 246-290-120 for construction document requirements. Project reports **must** reflect good engineering criteria and practices (WAC 246-290-200).

If you wish to receive a stamped and signed "Department of Health Approved" project report from us, you need to submit an additional project report and request that the additional report be stamped, signed, and returned.

The remainder of Section 2.4 provides a general outline of the items that, at a minimum, should be in all project reports. For specific project requirements, see the appropriate chapter of this manual or the applicable sections of Chapter 246-290 WAC. The level of detail we expect in the project report should reflect the complexity of the project. We've created checklists for a number of projects types (see Appendices B, F, and G) that provide detail on our expectations for project report and design submittal content.

2.4.1 Project Description

A complete and accurate project description provides us with valuable information about the project basis, orients us for an efficient design review, and serves as an important part of the overall design record once the project is approved. The project report **must** include the following general project-description information (WAC 246-290-110 (4)(a)) unless it is adequately described in a WSP:

1. A description of the problem or problems being addressed and why the project is proposed.

2. A summary of the recommended alternative (if applicable per Section 2.4.3), proposed construction schedule, estimated project cost, and financing method.
3. The relationship of the project to other water system components.
4. A statement of change in the physical capacity of the water system and its ability to serve customers, if applicable.
5. A copy of the environmental impact statement or determination of non-significance, or an explanation why the State Environmental Policy Act (SEPA) does not apply to the project. See SEPA Chapter 246-03 WAC. See Section 2.4.9 for more information on SEPA.
6. Source development information, if applicable.
7. The type of treatment, if applicable.

2.4.2 Planning

Section 2.1 provides general information on the relationship between planning and engineering document submittal and review. Planning is an important element in project design. If the following are not adequately addressed in an approved WSP or SWSMP, then the project report **must** address them (WAC 246-290-110 (4)(b)):

1. General project background, with population and water demand forecasts.
2. A service area map. Municipal water suppliers must identify their retail service area and their general service area on this map.
3. A description of the project's impact on neighboring water systems.
4. Local requirements, such as rates and duration of fire flow.
5. Additional management responsibilities, such as those in WAC 246-290-105, 415, and Chapter 246-292 WAC, Water Works Operator Certification. Also, see Section 2.4.8.
6. A project implementation and construction schedule, including project phasing, if applicable.
7. Estimated capital and operating costs, and financing method, if applicable.
8. A *Water Rights Self-Assessment Form*, if it is applicable to the type of project proposed.
9. Confirmation of local government consistency (WAC 246-290-108 and Policy B.07).

2.4.3 Analysis of Alternatives

A comparison of alternative solutions helps to ensure the completed project meets the project objectives (see WAC 246-290-110(4)(c)). These objectives may include minimum life-cycle costs, maximum efficiency and reliability, least life-time maintenance, shortest implementation schedule, or some combination of these outcomes. A poor or non-existent analysis of alternatives may result in the design failing to meet the project objectives, in expensive or unreliable operations, or non-compliance with operating requirements. To the extent possible, design engineers should match engineering solutions to not only the problem but also to the capacity of the water utility to maintain and sustain the infrastructure.

2.4.4 Water Quality

Water quality should be the most important consideration in every water system design. Every element of design should consider the impacts to water quality and public health. The project report **must** include a review of water quality as it relates to the purpose of the proposed project, including results of raw and finished water quality analyses conducted by a laboratory accredited to analyze drinking water compliance samples in Washington State (WAC 246-290-110(4)(d)). If the project involves water treatment or a filtration pilot study, see Chapter 12 and applicable sections of Chapter 246-290 WAC.

Design engineers should consider water system design holistically, so that the correction of one water quality problem (e.g., replacement source, pH adjustment, addition of a chemical disinfectant) does not lead to new or amplified water quality problems with the source or in the distribution system. Possible examples of such problems include:

- Installing reverse osmosis, gaseous chlorination, or pH adjustment that increases the corrosivity of the water, leading to increased levels of lead and copper
- Installing a new source that has a higher pH, leading to precipitation of iron and manganese
- Installing a new source that has a higher level of disinfectant by-products precursors, leading to higher levels of DBPs

2.4.5 Engineering Calculations

Submitting key calculations and making reference to appropriate data residing elsewhere is important to our ability to efficiently review the design. We want to be certain the design approach complies with design criteria (see Section 2.4.6). The project report **must** include relevant technical considerations necessary to support the project, such as a physical capacity analysis, hydraulic analysis, and sizing justification (WAC 246-290-110(4)(f)). For guidance on ways to analyze the physical capacity of a water system, see Chapter 4. For guidance on hydraulic analysis, see Chapter 6.

2.4.6 Design Criteria

Identifying the design criteria allows us to understand the overall project objective, project constraints to be considered, and minimum project requirements to be met. The project report **must** describe specific design criteria (WAC 246-290-110(g) and (h)) such as:

1. Design and construction standards, including performance standards, construction materials and methods, process control, and basis of sizing criteria, as applicable.
2. Locally adopted design standards relevant to the project, such as fire flow requirements.

Consult the appropriate chapters of this manual to determine whether any additional engineering and design information is required.

2.4.7 Water Rights and Other Legal Considerations

Some legal considerations, such as land ownership and water rights, should be addressed early in project design since they can affect the viability of a project. In preparing the project report, engineers **must** address water rights if the project involves a new, replacement, or modification to a source; increased withdrawal from a source; or an increase in the water system's physical capacity or service area (WAC 246-290-110(4)(e)). Design engineers should do so by completing a *Water Rights Self-Assessment Form* and submitting it with the project report. We encourage design engineers and water systems to review their water right self-assessment with the Department of Ecology before submitting documents to us for approval.

The Department of Ecology administers Washington's water rights permitting program. Our role is limited to ensuring water rights information is provided with the design engineer's submittal as required by our regulations, and sharing that information with Ecology. If the utility's water rights self-assessment indicates that completion of the project will exceed the utility's water right instantaneous or annual withdrawal limit, we may return the submittal and request the water system consult with Ecology before resubmitting.

Water rights also play a key role in adequacy determinations by Ecology and local governments. Our project-related correspondence (e.g., letter to acknowledge receipt of the submittal and our letter approving the submittal) will include a statement such as:

> *The department's approval of your water system plan does not confer or guarantee any right to a specific quantity of water. The approved number of service connections is based on your representation of available water quantity. If the Washington Department of Ecology, a local planning agency, or other authority responsible for determining water rights and water system adequacy determines that you have use of less water than you represented, the number of approved connections may be reduced commensurate with the actual amount of water and your legal right to use it.*

Such a statement will be included in our correspondence unless, on a case-by-case basis, our engineer determines that the project has no association with water resources, water availability, or the approved number of connections. Examples of a project that may have no association with any of these is the installation of a chemical injection treatment system to an existing source, replacement of an existing water main, or re-coating the interior of a reservoir.

Project reports **must** also identify other legal issues such as ownership, rights-of-way, sanitary control area, and restrictive covenants (such as water-related restrictions recorded on titles or deeds). Certain projects may also require coordination with the local boundary review board or Washington Utilities and Transportation Commission (WAC 246-290-110(4)(i)). Boundary review boards exist in most Washington counties. They guide and control growth of municipalities and special purpose districts.

2.4.8 Operations and Maintenance Considerations

If a project is expected to add considerably to the water system's operational and maintenance responsibilities (projects such as storage tanks, booster pump facilities, source of supply, and water treatment), a project report **must** include the following information in the project report (see WAC 246-290-110(4)):

1. Describe the routine operations tasks and frequencies.
2. Describe the preventive maintenance tasks and frequencies.
3. Identify the estimated annual operations and maintenance costs (energy, equipment, labor) and life-cycle costs. Include costs in an updated water system budget.
4. Explain whether the project triggers a requirement for a new or higher-level certified operator (Chapter 246-292 WAC) or - in the event of creating a new water system - a satellite management agency (Chapter 246-295 WAC and Policy B.05)).

2.4.9 State Environmental Policy Act Requirements

Before construction, SEPA requires certain types of projects to have an environmental impact statement, a SEPA determination of non-significance, or a document explaining why SEPA does not apply to the project (see WAC 246-03-030(3) and Policy A.03satellite)).

These requirements apply to:

- All surface water source development.
- All water system storage facilities greater than 0.5 million gallons.
- New transmission lines longer than 1,000 feet and more than 8 inches in diameter located in a new right of way.
- Major extensions to existing water distribution systems that will use pipes more than 8 inches in diameter and increase the existing service area by more than 1-square mile.

2.5 Construction Documents

Construction documents, such as detailed design drawings and specifications, **must** identify how a specific project will be constructed to satisfy the requirements and conditions established in the project report and/or the WSP (WAC 246-290-120). See Section 2.4 and WAC 246-290-110 for project report requirements.

If you wish to receive a stamped and signed "Department of Health Approved" set of construction documents from us, you need to submit an additional complete set of construction documents and request that the additional set be stamped, signed, and returned.

All construction documents should conform to the established standards of the engineering profession. Approval of construction documents shall be in effect for two years from the date of our approval, unless we determine the need to withdraw approval sooner. The design engineer should contact us if construction completion exceeds the two year approval window, to request an extension of time for the approval. Additional design conditions may be applied prior to approving such an extension.

2.5.1 Design Drawing Requirements

Design drawings submitted for our review and approval must be legible and should include the following:

- All information, complemented by the project specifications, necessary to construct the project including all its components in their proper location and orientation, and to demonstrate compliance with applicable regulations, follow standard practices, and satisfy the project's objectives and owner's needs.
- A location plan indicating the location of the water system.
- A service-area map showing the service-area boundary and the location of each project element.
- Name of project.
- Name of the legal owner of the water system.
- Scale.
- North arrow, where applicable.
- Date.
- Name, address, and phone number of the design engineer or consultant firm.
- Revision block with the initials of the design engineer and drafter.
- The stamp and signature of the design engineer. See the engineering registration requirements of Washington State (required by WAC 246-290-040).
- Location of all applicable easements, right of ways and property lines within the project area.
- Location of all existing above ground and underground utilities and structures within the project impact area.
- The 100-year flood elevation within the project area, where applicable. Pump stations, wells, reservoirs, and treatment plants funded by our state revolving loan fund must be protected from a flood two feet higher than the 100-year flood elevation.
- Seismic design standards for the location where the facility will be built. Additional seismic design requirements are in Section 11.5.

If the construction document submittal is for our review and comment only (not for approval), stamp the submission **Preliminary: For Review Only.** Be sure to communicate this to our regional staff at the time of submission. If we review and comment on preliminary documents, we will charge for the review by applying our fee regulation (WAC 246-290-990). Documents identified as "preliminary" or similar may be reviewed but will not be approved by us.

2.5.2 Project Specifications

Project specifications submitted for our review and approval should include the following:

- All information, complemented by the design drawings, necessary to describe the means, methods, and standards necessary to purchase, install, and test project components to satisfy the project's objectives and owner's needs.
- Name of the project.
- Name of the municipality, association, individual, or other entity that legally owns the water system.
- Date.
- Name, phone number, and address of the design engineer.
- The stamp and signature of the design engineer. See the engineering registration requirements of Washington State (required by WAC 246-290-040).
- A provision for the contractor to submit shop drawings for review by owners and design engineers.
- A detailed description of all equipment and water system start-up testing, disinfection and inspection (final acceptance) procedures (required by WAC 246-290-120(4)(c)). Cut-sheets of a product or material may not substitute for technical specifications.
- A summary of the means and methods for maintaining water service throughout the construction period, if necessary.
- Components in substantial contact with potable water must be certified under ANSI/NSF Standard 61 (required by WAC 246-290-220).

2.5.3 Change Orders

Changes orders considered after a project is approved could significantly alter the means, methods, objectives, components, and even outcome of the project. Water systems **must** submit each change order that significantly alters the scope of the project, drawings, or specifications to us for review and approval (WAC 246-290-120(4)(d)).

Examples of changes considered "significant" and, therefore, subject to our approval, are:

- Change in treatment process.
- Change in type of chlorination or disinfection process used.
- Change in elevations of tank or booster stations.
- Change of materials that are in direct contact with finished water.
- Change in control systems or control strategies.
- Change in size for a storage tank.
- Change in designated pumping capacity.

Examples of non-significant change orders include minor adjustments to valve and piping locations, piping configurations, security fencing materials, and a different pump model with the same pumping characteristics. Design engineers may note changes **not** considered "significant" on the record drawings ("as-built"). For guidance on whether a particular change order is significant, contact the appropriate regional office.

If significant changes to the approved project design are not approved it is not possible for an engineer to certify construction completion according to the approved design. See Section 2.13 for requirements to certify construction completed according to the construction documents approved by us.

2.5.4 Contractor-Supplied Design Components

Construction documents submitted for our review may call for contractor-supplied construction drawings and specifications to be submitted to the owner and design engineer after selection of the contractor. Such contractor-supplied drawings and specifications must be submitted to us for review and approval after our original design approval, regardless whether the contract has already been awarded. We will apply the professional engineering requirements of WAC 246-290-040 to contractor-supplied drawings and specifications.

2.6 Coordination with Local Approving Authorities

Construction projects may be subject to local permits or approvals. Compliance with our requirements does *not* guarantee compliance with local rules. Water systems are responsible for ensuring projects follow local approval processes. Design engineers can usually get information on the local approval process from county building departments and environmental health programs.

2.7 Design and Review Process

Figures 2.1, 2.2, and 2.3 reflect a typical process flow path for design and review of three general project types. Refer to general and specific project submittal checklists in Appendix A.3 for more detail.

2.8 Submittal Exceptions for Miscellaneous Components and Distribution Mains

2.8.1 Categorically Exempt

For the following types of projects, water systems are not required to submit project reports or construction documents to us for review and approval (WAC 246-290-125(1)):

1. Installing hydrants, valves, fittings, meters, and backflow prevention assemblies.
2. Repairing a water system component or replacing it with a component of similar capacity and materials described in the original approved design. For the purposes of replacing distribution mains, *similar capacity* includes up to one standard pipe size larger.
3. Maintaining or painting surfaces not contacting potable water.

For the following components installed under the “valve” submittal exception, at a minimum the following standards must be met:

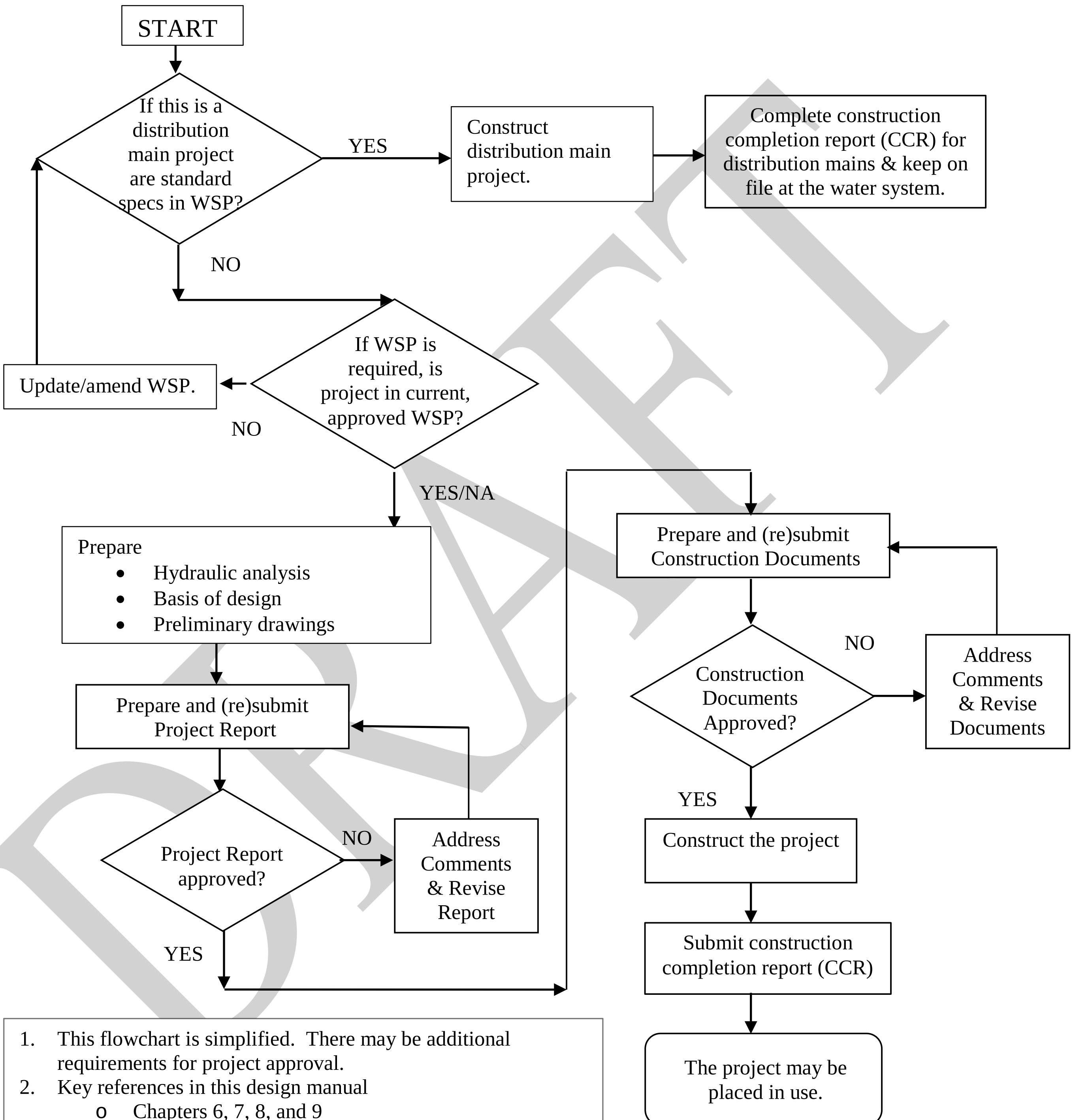
1. Automatic air-vacuum relief values installed in the distribution system – must meet WAC 246-290-200(1)(c). See Chapter 6 for further guidance on installation requirements.
2. Backflow prevention assemblies – must meet WAC 246-290-200(1)(g).

2.8.2 Exempt Distribution Main Projects

Water systems may elect not to submit project reports or construction documents for new distribution mains or larger-capacity replacement mains if they meet the following conditions (WAC 246-290-125(2)):

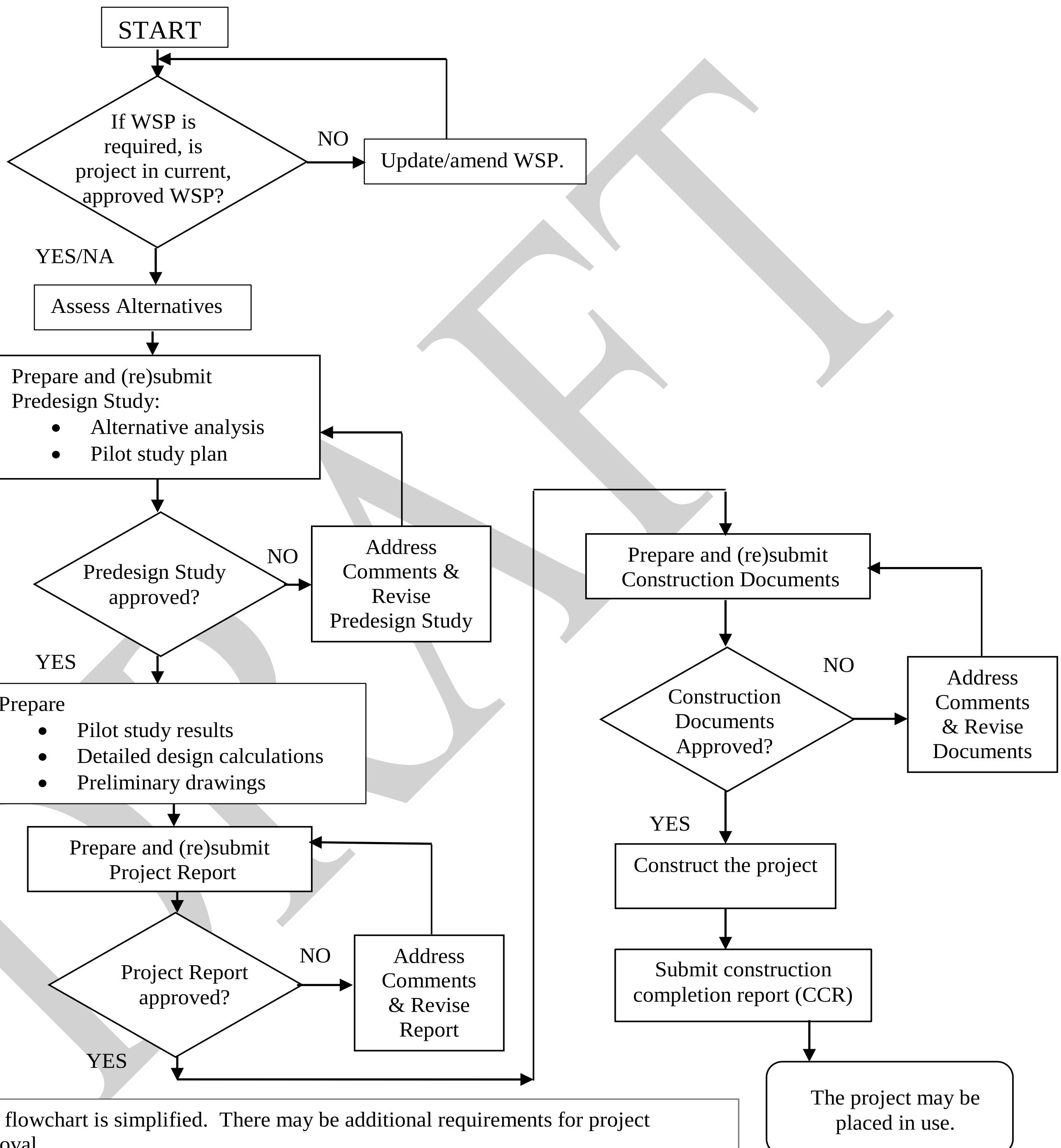
1. The water system has a currently approved WSP that includes standard construction specifications for distribution mains and an analysis of the hydraulic capacity of the basic transmission and distribution main configuration for the water system.
2. The water system maintains a completed *Construction Completion Report for Distribution Main Projects* (DOH 331-147) on file for each such project.

Figure 2.2
Distribution System – Project Development Flowchart



1. This flowchart is simplified. There may be additional requirements for project approval.
2. Key references in this design manual
   - Chapters 6, 7, 8, and 9
3. Key sections of chapter 246-290 WAC
   - WAC 246-290-110, -120, -230, -235
4. Design engineers may submit the project report and construction documents at the same time, and request simultaneous approval.

Figure 2.3
Water Treatment – Project Development Flowchart



1. This flowchart is simplified. There may be additional requirements for project approval.
2. Key references in this design manual
   - Chapters 10 and 11
   - Appendix F, H, and I
3. Key sections of chapter 246-290 WAC
   - WAC 246-290-110, -120, -250
4. Design engineers may submit the project report and construction documents at the same time, and request simultaneous approval.

2.9 Submittal Exception for Distribution-Related Projects (other than Distribution Mains)

For distribution related projects that are larger and more complex than a distribution main project, you may use the more extensive Submittal Exception Process under WAC 246-290-125(3) as long as all the conditions are met. Eligible projects are limited to:

- Storage tanks
- Booster pump facilities
- Transmission mains
- Pipe linings
- Tank coatings

Water systems that meet the eligibility criteria in WAC 246-290-125(3) and intend to follow the submittal exception process **must** make an initial written request to us on the *Water System Plan Submittal Form* (DOH 331-397). If you intend to apply for this submittal exemption, you should discuss the desired scope of exemption and planning document requirements during the pre-plan conference with the regional planner and engineer.

There is no submittal exception for source of supply (such as new or redeveloped wells or springs, refurbished wells, surface water intakes, interties) and water quality treatment projects (such as chlorination, corrosion control, filtration, iron and manganese removal, UV, and ozonation).

2.9.1 Design and Construction Standards for Reservoirs and Booster Pump Stations

To qualify for the Submittal Exception, the water system **must** include design and construction standards for distribution-related projects in an approved WSP (WAC 246-290-100(5)(b)).

We expect the following items to be part of the WSP narrative:

1. Reservoirs:
   - General location of tank sites.
   - Overflow and base elevations.
   - Map of service area indicating elevations of service connections.
   - Basis for sizing the storage volumes needed.
   - Hydraulic analysis of the water system or individual pressure zones evaluating the storage improvements.
   - Level control and alarms.
2. Booster Pump Stations (BPS):
   - General location of BPS site(s).
   - Sizing basis for BPS capacity (flow and head) needed.

- Hydraulic analysis of the water system or pressure zones evaluating the effect of BPS operation.
- Flow, pressure, and process control.

The following items should be part of the WSP standard specifications:

1. Reservoirs
    - Standard tank details, including level controls, high and low level alarm, external level indicator, access hatch, vent, drain, overflow (include sizing) drain and outfall, screens, and access ladder.
    - Material specifications to be used for tank construction together with construction specifications (concrete, steel, other). ANSI/NSF Standard 61 certified materials for all surfaces in substantial contact with the water.
    - Specifications for all coatings, including application, curing, and ANSI/NSF compliance. Water quality testing needed before activating tanks, such as volatile organic chemicals, if applicable (see Appendix G).
    - Leakage testing and disinfection procedures per AWWA C652 (include chlorinated water disposal specifications).
    - Site piping plans (generic). Also include isolation valving, sample taps (type and location), provision to improve circulation in tanks (reduce stagnation), and piping material specifications for pipes under the foundation slab, in the tank or in the yard.
    - Geotechnical considerations to be addressed, such as bearing strength and seismic considerations.
    - Water system-specific water quality concerns affecting treatment, such as coliform testing, chlorine residuals, pH, disinfection byproducts, and contact time requirements.
    - Security elements.
2. Booster Pump Stations:
    - Performance specifications for booster pumps, overload capacity, and minimum shutoff heads.
    - Electrical specifications, control strategies, and mechanisms.
    - Pipe material, construction standards, and specifications for internal BPS piping.
    - Specifications or standards for meters, control valves, and other appurtenances.
    - General structural and construction specifications and standards for BPS housing.

2.9.2 Rescinding Submittal Exception Authority

We will rescind a water system's eligibility for submittal exceptions under WAC 246-290-125(2) and (3) if the water system fails to maintain compliance with the eligibility criteria or conditions. At that point, the water system must submit all engineering documents (project reports and construction documents) to us for approval until it re-establishes compliance with the eligibility criteria.

2.10 Resolving Disputed Department of Health Review Decisions

When our review engineer and the water system or consultant cannot reconcile a difference, the water system or consultant may formally appeal our decision. We have established internal processes for these circumstances. Contact the appropriate regional office for a complete description of the Brief Adjudicative Proceeding (BAP) process (see Chapter 246-10 WAC).

2.11 Review Fees and Invoice

We charge fees for reviewing project documents. These fees may change periodically. The current fee can be found in WAC 246-290-990. After we complete a detailed review, we send an approval letter or review letter to the water system with an invoice for the review fee. A copy of the letter is sent to the design engineer. A fee estimator worksheet is available through our water system design web page.

Most of the planning and engineering document review fees listed in WAC 246-290-990 is fixed. These fixed fees cover the cost of reviewing the initial submittal and one re-submittal, in the event the initial submittal is not approved after the first review. If more than one re-submittal is necessary, we will charge an additional fee for each subsequent review.

Some of the fee-for-service activities listed in WAC 246-290-990 are assessed an hourly fee. We will charge each hour spent on the hourly fee-for-service activity, billed at the rate indicated in the rule. To keep the cost and review time to a minimum, design engineers should make sure each submittal is as complete and accurate as possible. Use the *Project Submittal Checklists* in the Appendices.

2.12 Project Approval Letter and Construction Completion

When all requirements for construction documents are met, we will send an approval letter to the system owner, with copies to the design engineer and others, as appropriate. A typical construction-document approval includes the following enclosures:

- An invoice for the review fee, if we did not already send it.
- A *Water Facilities Inventory Form*, if completion of the project will change any information on the form. We will request the water system update and return the inventory form upon completion of the project.
- A *Construction Completion Report Form* (DOH 331-121).

2.13 Construction Completion Report Forms

For clarification, there are three separate construction completion report forms. Each form is used in different circumstances, so it's important to know the difference.

1. *Construction Completion Report Form* (DOH 331-121). Use this form in the normal process of submitting documentation for a project that underwent our design review and approval and was constructed in accordance with the Department-approved design. It is the form referenced in WAC 246-290-120(5). We will send it with the construction approval document referenced in Section 2.12 above.
2. *Construction Completion Report Form for Distribution Main Projects* (DOH 331-147). Use this form only for distribution main projects not requiring prior written approval from us. The water system does not have to submit this form to us following construction completion. However, the water system **must** maintain a completed form on file and make it available to us upon request. This form is referenced in the submittal exception process (see Section 2.8 and WAC 246-290-125(2)).
3. *Construction Completion Report Form for Submittal Exception Process* (DOH 331-146). Use this form only for distribution-related projects not requiring prior written approval from us. Distribution-related projects include booster pump stations, storage tanks, internal tank coatings, and transmission mains. The water system **must** submit this report to us after constructing new storage tanks or booster pump stations, but only maintain a completed form on file for other distribution-related projects (WAC 246-290-125(3)(f)). This form is used in the submittal exception process (see Section 2.9 and WAC 246-290-125(3)).

If completion of the project changes any information on the water facilities inventory (WFI), the water system is responsible for submitting an updated WFI with the signed construction completion report.

2.14 Record Drawings

The engineer who manages construction or inspection typically provides record drawings to the purveyor when the project is complete. The purveyor **must** maintain a complete set of record drawings and provide them to DOH upon request (WAC 246-290-120(4)(e)).

2.15 Safety

Improperly designed facilities could put employees, contractors, and the public at risk. If someone gets hurt, the water system could face a lawsuit or citations and penalties from the Washington State Department of Labor and Industries (L&I). Design engineers should be aware of the full scope of state and federal regulation governing safe working environments (Washington Industrial Safety and Health Act, 49.17 RCW; Occupational Safety and Health Act).

This section on safety briefly summarizes issues design engineers must consider. More detailed safety information can be obtained by contacting L&I or accessing the L&I Web site at www.lni.wa.gov/safety/default.asp

Safety topics on the L&I Web site include:

- Asbestos
- Confined spaces
- Excavation and trenching
- Fall protection
- Guardrails
- Ladders
- Lead
- Lockout/Tagout

Contact information for L&I and Occupational Safety and Health Administration (OSHA) is in Appendix C.

References

WSDOH. 1997. *Water System Planning Handbook*, DOH 331-068, Washington State Department of Health, Olympia, WA.

Chapter 3: Estimating Water Demands

3.0 Applicability

Design engineers use water demand estimates to design new water systems or additions to existing water systems. To size any water system or its component parts, an engineer **must** estimate water system demand and consumers' consumption (WAC 246-290-221). This chapter provides basic, conservative water-demand design criteria engineers may use if they lack information that is more appropriate.

Design engineers using historical water use records to design future water system facilities should attempt to validate the information. Given the many variables that affect consumer demand (see Section 3.2), design criteria based on historical data should include a reasonable margin of safety. The more detailed the historical demand records are, the longer the period covered by that data, and the greater the designer's confidence in the validity of that data, the smaller the margin of safety needs to be. This basic concept applies to every recommendation in this chapter.

This chapter is divided into three parts:

1. Residential Demand Estimates – Focuses on water systems in which residential demands comprise a significant portion or all of the demand.
2. Nonresidential Demand Estimates – Focuses on water systems in which residential demands comprise an insignificant portion of total demand.
3. General Considerations - Covers issues applicable to both residential and non-residential demand estimates.

3.1 Demand versus Consumption

We expect design engineers to differentiate between the productive requirements of a water system and total consumptive demand. The difference in these two values includes the volumetric loss through distribution system leakage (DSL). DSL percentage is defined in WAC 246-290-820(2) and DSL volume is determined by Equation 3-3. For some existing water systems, DSL may be substantial enough that ignoring its contribution to productive requirements would create a meaningful deficit in design, and an inability to operate within approved design parameters. In the design of new water systems, we expect design engineers to consider the future state of the distribution system, and make an appropriate allowance for DSL.

Part 1 – Residential Demand Estimates

3.2 Consumer Demand

An equivalent residential unit (ERU) is central to the evaluation and design of water systems with significant residential demand. An ERU is defined as *a system-specific unit of measure used to express the amount of water consumed by a typical full-time single family residence* (WAC 246-290-010). This value is particular to the existing water system from which it was derived. In this manual, we refer to an ERU value reflecting various demand scenarios. These terms are defined below.

- **ERU_{MDD}** is the amount of water consumed by a typical full-time single family residence during high demand. It is intended to approximate the maximum daily demand of a typical full time single family home. It is the ERU value used in physical capacity assessment (see Chapter 4).

- **ERU_{ADD}** value is intended to approximate the average daily demand of a typical full time single family residence. The ERU_{ADD} value may be used in assessing factors bounded annually, such as a water supply safe annual yield and a water right annual volume (Qa).

For most water systems, consumption by consumers accounts for a significant majority of water that the system's supply must produce. In order of preference, the information sources for estimating consumer's consumption are:

1. Actual metered records, provided the design engineer considers such data complete and accurate (see WAC 246-290-221(1)).
    a. Information on water production should be available from every existing Group A public water system. The requirements to install and maintain source meters and to read source meters at least once per month (WAC 246-290-100, -105, and -496, and WAC 173-173-060) have been in place for many years.
    b. Most community water systems have service meters. As of January 2017, all community water suppliers with 15 or more connections **must** install service meters, and **must** calculate and report distribution system leakage (WAC 246-290-820). This primary consumptive data for an individual water system is considered most applicable for projecting future consumer consumptive use provided the data is complete and accurate, and the design engineer takes into account full-time and part-time consumers when evaluating this data.
2. Comparable metered data from an *analogous* water system, provided the system serves all or almost all full-time residential customers. See WAC 246-290-221(3)(a) and Section 3.2.3 for elements we consider important when considering consumptive data from an existing system in the design of a new one.
3. The consumer demand criteria presented in this chapter and in Appendix D.

The design engineer should assess the degree of confidence in the validity of available data. The smaller the degree of confidence the larger the design's margin of safety needs to be.

3.2.1 Evaluating Actual Water Demand

The analysis of historical water demand should include these considerations:

1. **Use actual water demand information.** Additional services should be based on actual water demands. Water systems cannot justify new services solely by committing to implement a water use efficiency program. See Section 3.8.
2. **Use multiple years of data.** The historical water production and consumption analysis should be based on meter readings covering at least two, but preferably more years. The meter readings should include daily production metered data for the peak usage period and weekly or monthly usage during the rest of the year. Most community water systems experience peak demand from June through September. Other water systems, such as ski resorts, may experience peak demand during the winter.

   For most water systems, the historical water use analysis **must** quantify distribution system leakage and total authorized consumption (WAC 246-290-820). See Section 3.8 for information on distribution system leakage and authorized consumption. Keep in mind system-wide ADD and MDD production data include consumption plus distribution system leakage as defined in WAC 246-290-820.
3. **Correlate data with occupancy.** Water demand data **must** be correlated with the number of full- and part-time residential service connections actually in use when the data was collected (WAC 246-290-221(1)). To quantify residential demands more clearly, the analysis should separate industrial, commercial, or other water demands from residential demands.
4. **Anticipate changes that might increase demand.** The analysis should address potential changes in demand (see Section 3.5).
5. **Normalize data based on climatic conditions.** Rainfall and temperature data should be reviewed to verify their effect on water system demand. Rainfall and cool weather usually decrease water demand, and hot, dry weather usually increase water demand (unless drought restrictions are imposed). Appendix C includes climatological organizations (NOAA, Office of State Climatologist, Western Regional Climate Center) whose data may assist with determining how current-year precipitation compares with the historic weather patterns.

   Design engineers should compare water demand data to historical climate information to determine if it is necessary to adjust historical demand data up or down. Summer temperature and precipitation data from the Office of the Washington State Climatologist will tell if the data period was an unusually wet/cool summer, average, or what was considered a hot/dry summer. Visit http://www.wrcc.dri.edu/summary/Climsmwa.html, and from the statewide map get precipitation and temperature data for the nearest gauging station.

   a. Water production and consumption data over wet/cool summer(s). In such cases, it would be appropriate to look at an expanded period of metered use data, or adjust the calculated ERU_{ADD} and ERU_{MDD} values higher, to account for the hotter, drier summers that will inevitably follow.

   b. Water production and consumption data over hot/dry summer(s). In such cases it would be inappropriate to dismiss the data as “worst case” unless the rainfall and temperature represented conditions approaching a two-standard deviation difference from the mean as measured over many years. A hot, dry summer that isn't a statistical anomaly is considered a normal operating condition, to which water system facilities must be designed to meet performance standards in WAC 246-290 Part 3.

3.2.2 Full-time and Part-time Single-Family Residential Users

Our preference is for design engineers to estimate ERU_{MDD} based on consumptive use. If consumptive use data is not available or considered invalid, then design engineers can use source meter data to estimate ERU_{MDD} provided that all customers are single family residential, all residences are occupied on a full-time basis, and the design engineer acknowledges that the ERU_{MDD} value includes consumption and DSL. When customers occupy homes intermittently, dividing total production by the total number of homes may significantly underestimate future demand as part-time customers become full-time customers.

Water demand design data **must** correlate to the number of full-time or part-time equivalent residential units in service at any time (WAC 246-290-221(1)). "Full-time" is a permanent place of residence. "Part-time" is a vacation home, used only seasonally, such as on holidays or weekends. The rule makes this distinction because water systems designed only for part-time residences may convert gradually over time to full-time residences (due to retirement, changing housing markets, and other factors).

We will not approve a water system for part-time residential use unless obligatory covenants or other binding agreements prohibit full-time occupancy. Water systems designed only for part-time residences cannot be expected to provide service levels adequate for full-time occupancy. Future demand assigned to each proposed residential dwelling unit **must** reflect full-time occupancy (WAC 246-290-221 (2)). The same consideration must be given to each existing part-time residence. This concept reduces concerns associated with part-time residents changing to "full-time" without sufficient water supply and delivery facilities. This concept also applies to part-time versus full-time multifamily residences.

Obtaining source meter records over any selected period is relatively straightforward. However, determining occupancy levels during that same period can be quite difficult. Water systems can use the following approaches to correlate source meter data with estimated occupancy levels. Each of these approaches has shortcomings, so we recommend using more than one to achieve an appropriate safety factor based on the degree of uncertainty.

- **Survey customers.** Very small water systems with about 50 or fewer connections may be able to use a survey to estimate daily or weekly occupancy for a short period of time when they can rigorously take meter readings. If the primary capacity limitation is associated with MDD, the survey could focus on the expected peak-demand period of summer.

- **Service meter records.** Many water systems have service meters, and all municipal water suppliers **must** install meters on their direct service connections (WAC 246-290-496(2)). Water systems usually read service meters monthly, bi-monthly, or quarterly. The frequency of meter reading limits the outcome of this method. In one comprehensive study, median indoor residential water use ranged from 54 to 64 gallons per capita per day for several communities throughout the United States (Mayer et al. 1999). When water use for a residence falls significantly below this range, residents probably occupy the dwelling intermittently. Reviewing service meter records may help you select the time to use an intensive meter-reading program to correlate demand with occupancy.

- **Assume full occupancy on holidays.** For some small recreational water systems, it may be reasonable to assume that all residences are occupied during certain times of the year, such as Memorial Day or Labor Day weekends. Other water systems may be able to assume full occupancy on other days. Meter readings on those days, especially if the water system assumes high demands will occur, could help to estimate peak day demand. You should supplement this approach with a customer survey on these target weekends.

- **Demand patterns.** Demands that vary significantly between billing periods could indicate an intermittently occupied residence.

- **Tax, voting, and other public records.** These may help to determine occupancy levels. However, there are several shortcomings to using public records to estimate occupancy. For example, people who live part-time in Washington and part-time in a warmer climate appear as full-time residents on assessor and voting records. Rental properties are another example. Similar to vacation homes, the assessor sends tax records for rental properties to owners at their primary residences. Renters may not be registered to vote where they reside.

Note: *When using service meter records to establish the ERU_{MDD} value, be sure to account for any part-time uses that occurred during the record-keeping period (such as the maximum month). Only residences that are occupied full time during the time of metered data collection should be used. Be sure to confirm the correlation between meter information and the various types of service (residential versus nonresidential) when determining the ERU_{MDD} value for the water system.*

3.2.3 Analogous Water Systems

Lacking metered water use records, engineers may use comparable water use data from an analogous water system (WAC 246-290-221(3)) to design a new water system. Because existing water systems must have and read source meters, there is generally no need to look elsewhere for appropriate production or demand information. The use of analogous system information is most appropriately applied to design of new water systems.

To be considered analogous, water systems **must** have similar characteristics (WAC 246-290-221(3)(a)). These characteristics include:

1. **Population and development pattern.** Demographics are the vital statistics of human populations such as size, growth, density, and distribution. Demographics change with the nature of the development. Population densities are different from single-family to multifamily residences, from housing provided for families to housing provided for single occupancy, and from individual lots to mobile home park developments.

2. **Lot size.** A major factor in water use related to larger lot sizes is in the irrigated area (lawns, gardens, and agricultural uses). However, it is possible to Xeriscape (use native flora, rockery, and pavement) multi-acre tracts with very little need for supplemental irrigation.

3. **Climactic zone.** Climate significantly effects water use. High temperatures and low precipitation usually lead to an increase in water use. To be considered analogous, water systems should have similar monthly and average annual temperature and precipitation. In areas where freezing temperatures are prevalent in winter, high demands may occur if users allow faucets to run to prevent freezing. You can also expect water demand to increase during the winter for water systems serving winter use activities, such as a ski resort.

4. **Cost of water service.** Water pricing structure relates to the use of "inclining block rates" versus "declining block rates." Both require the use of individual meters. For "flat rates," meters are most often not present and analogous water demands are more difficult to predict. To be considered analogous, the existing and proposed water systems should provide the same level of metering and have similar rate structures.

5. **Water conservation standards.** The analogous water system's conservation practices should be the same as the proposed water system. These practices include, but are not limited to, alternate day watering schedules, installing low-water-use fixtures, toilet-tank displacement devices, leak detection, and water demand reduction programs. Water use restrictions should be established that are the same for any voluntary or mandatory curtailment measures requested of analogous water system consumers. These may be in community covenants, bylaws, local ordinances, or on property deeds. It is very important to determine if the restrictions are enforceable. A legal opinion may be necessary to determine equivalent enforceability.

6. **Soil type and community landscape standards.** Soil types and landscaping can affect irrigation demands. Moisture retention and evaporation losses from sands and gravels differ from loams, silts, and clays. When designing a water system, engineers should check with the local Cooperative Extension office to determine and evaluate variables that may affect water demand. For example, water demands for landscaping vary largely between natural flora and more water-dependent plants.

7. **Maintenance practices.** The analogous water system's utility maintenance practices should be considered. These practices include the seasonality, frequency of, and volume of water used for line flushing, exercising hydrants and valves, and cleaning tanks.

Water-use patterns between water systems vary for more reasons than those presented above. Sociological factors also play a role. It is nearly impossible for a design engineer to predict the mind-set or water use ethic of consumers on a new water system. When basing a water system design on characteristics analogous to another water system, we recommend conservative water demand estimates. A safety factor is appropriate even if the proposed water system incorporates the same enforceable water use efficiency practices and use restrictions as the identified analogous water system. It may be wise to discuss this design approach with the regional engineer early in the design phase of a project.

3.2.4 DOH Default Water Demand Design Criteria (Appendix D)

Residential demand is the largest portion of total demand for most water systems. The design engineer with adequate historical service or source meter records can usually estimate residential demands with reasonable accuracy. Nonresidential demands related to industrial, commercial, and similar types of uses are also important and need to be estimated.

For new water systems or existing systems with inaccurate or insufficient records, and in the absence of analogous system information to draw from, the design engineer may use the information in Appendix D to estimate ERU_{ADD} and ERU_{MDD} for residential connections (WAC 246-290-221(3)). Limitations regarding use of water demand estimating criteria in Appendix D:

- ERU_{ADD} reflects consumptive use data (it does not include DSL), and therefore ERU_{MDD} generated under Appendix D excludes DSL.

- The information in Appendix D does not specifically address commercial and industrial demands. Design engineers should use information in Table 3-2 for non-residential consumer demand.

- Large-lot irrigation demands are not adequately represented by the data used in establishing demand estimating criteria in Appendix D. Design of new water systems intending to serve residential lots greater than ½ acre that will be irrigated by the public water system should undertake a detailed estimate of ERU_{MDD}. We recommend applying a value of 350 gallons per day to address in-house domestic demand (see Section 3.4.1) plus a detailed assessment of irrigation demand based on estimated irrigation demands in Table 3-2.

3.3 Water System Demand

Maximum daily demand (MDD) is defined in WAC 246-290-010 as *the highest actual or estimated quantity of water that is, or is expected to be, used over a twenty-four hour period, excluding unusual events or emergencies.* For the purposes of this manual, we take a broad view of MDD as it applies to the term *used*. We consider MDD to be the system-wide peak daily production requirement necessary to meet the consumptive demands of all types of connections; other intentional uses not associated with a connection; and the quantity of water lost through leakage or illicit uses. When we refer to MDD in this manual we mean the maximum daily source production/treatment required within a 24-hour period to meet all these withdrawals from the distribution system. Fire suppression is not a component of MDD. Engineers **must** design water system source and treatment so that together they can satisfy the maximum daily demand (WAC 246-290-222).

Average daily demand (ADD) is defined in WAC 246-290-010 as *the total quantity of water use from all sources of supply as measured or estimated over a calendar year divided by three hundred sixty five.* Likewise, we take a broad view of ADD as it applies to the term *used*. We consider ADD to be the system-wide average daily production requirement necessary to meet the consumptive demands of all types of connections; other intentional uses not associated with a connection; and the quantity of water lost through leakage or illicit uses. When we use the term "ADD" in this manual we mean the source production/treatment required to meet all these withdrawals from the distribution system during an entire year divided by 365.

Design engineers using advanced analysis of complex demand scenarios may need to separately analyze the component elements of MDD or ADD, such as identifying demand by customer class (such as $MDD_{residential}$, $MDD_{commercial}$).

3.4 Estimating Water System Demands

Engineers need to establish water demand estimates, with an appropriate factor of safety, to assess the adequacy of the water system's source and treatment capacity; to assess the adequacy of the water system's water rights; and to size pumping equipment, transmission lines, distribution mains, and water storage facilities properly.

Water systems **must** read source meters at least monthly (WAC 246-290-100(4)(b), 105(4)(h), and WAC 173-173-060). Design engineers **must** use metered production records to quantify MDD and ADD for most water systems (WAC 246-290-221). For new water systems without metered data, design engineers can use analogous water system data or the information in Appendix D to estimate the ERU_{MDD} and ERU_{ADD}.

Design engineers **must** assess the adequacy of the water system's water right, especially the attributes of annual volume (Qa) and instantaneous withdrawal (Qi) (WAC 246-290-110(4)). Qa is associated with ADD, and Qi is associated with MDD. See examples in Section 3.12.

3.4.1 Maximum Day Demand

Ideally the water system can provide the design engineer with daily production records from each source of supply. If a design engineer must rely on monthly source meter records, a peaking factor is needed to estimate the system-wide MDD from the maximum month's average day demand (MMADD).

Based on our analysis of 79 water systems in Washington State, we recommend the following MDD to MMADD ratio:

- 1.65 for systems serving fewer than 1,000 people
- 1.35 for systems serving 1,000 to 100,000 people

Appendix D.3 describes the results of this analysis in more detail. For new water residential-only systems without metered data, design engineers can use analogous water system data or the information in Appendix D.1 to estimate the ERU_{MDD}.

In general, the lower limit for ERU_{MDD} is 350 gallons/day/residential connection (WAC 246-290-221(4)). This demand estimate is consistent with the Department of Ecology on household water uses for developments that prohibit irrigation. There may be some projects with sufficient verified information (meter records, minimum of two years of data) to support an ERU_{MDD} value of less than 350 gallons per day. The data may only be used in support of expansion for that specific water system (WAC 246-290-221(4)).

Multifamily residences typically use less water per dwelling than separate single-family residences. Water uses for multifamily residences vary from water system to water system. They are usually specific to a given water system, but not always applicable to another water system. Engineers should view multifamily metered consumption data apart from single-family data when calculating ERU_{MDD}. Divide the total peak-day water use for the multifamily connection(s) by the water system-specific ERU_{MDD} to determine the number of ERUs contributed by multi-family connections.

Note: *In a few isolated cases in Western Washington, the ERU_{MDD} has been as high as 2,000 gpd/ connection. In Eastern Washington, the ERU_{MDD} for some water systems has been as high as 8,000 gpd/ connection. Design engineers should recognize that some water systems are outside the norm and will have much greater water demand, and that our assumptions about ERU_{MDD} = 2x ERU_{ADD} may result in a significant shortfall in supply during built-out peak day demand.*

3.4.2 Peak Hourly Demand

Engineers need PHD estimates to size equalizing storage, transmission lines, distribution mains, and some pumping facilities. The water system **must** be designed to provide PHD while maintaining a minimum pressure of 30 psi throughout the distribution system (WAC 246-290-230(5)). Water system specific diurnal demand curves can be developed and used to estimate PHD (AWWA 2012). Engineers usually need multiple diurnal demand curves because demand changes seasonally (AWWA 2012).

Design engineers may use Equation 3-1 to determine PHD for systems with predominantly residential demands. This equation is consistent with the maximum instantaneous demand values presented in previous editions of the state's design guidance manuals (WSDSHS 1973; WSDSHS 1983) known as the "Red Book" and "Blue Book" respectively. Equation 3-1 accounts for the ranges of PHD to MDD ratios reported as a function of water system size and by various water systems in Washington.

Key concepts associated with the use of Equation 3-1:

- Applicable to water systems with significant residential demand.
- N is the number of ERUs supplied by all sources. DSL has an associated number of ERUs (see examples in Section 3.12 and Worksheet 4-1). Therefore, N includes DSL. "N" is the number of connections only if there is no distribution system leakage and all connections are single family homes.
- Check to be sure that ERU_{MDD} times "N" equals total maximum daily source production.
- The ERU value is ERU_{MDD}. It is not appropriate to apply the ERU_{ADD} value to Equation 3-1.

Equation 3-1: Determine PHD

$$\mathbf{PHD = (ERU_{MDD} /1440) [(C)(N) + F] + 18}$$

Where	**PHD**	=	Peak Hourly Demand, total system (gallons per minute)
	C	=	Coefficient Associated with Ranges of ERUs
	N	=	Number of ERUs based on MDD
	F	=	Factor Associated with Ranges of ERUs
	ERU_{MDD}	=	Maximum Day Demand per ERU (gallons per day)

Table 3-1 identifies the appropriate coefficients and factors to substitute into Equation 3-1 for the ranges of single-family residential connections:

Table 3-1

Number of ERUs (N)	C	F
15 – 50	3.0	0
51 – 100	2.5	25
101 – 250	2.0	75
251 – 500	1.8	125
> 500	1.6	225

Note: *PHD relates to the hydraulic ability of a distribution system to accommodate a range of ERUs. A PHD evaluation determines the physical capacity of the whole water system, not each specific ERU. Engineers can use this equation to estimate the peak-hourly flow for the whole water system, or a specific pressure zone, after determining the number of ERUs.*

3.5 Anticipating Changes in Demand for Systems with Significant Residential Demand

Water demand estimates should address anticipated changes as a water system matures. An analysis should address how future water-use patterns may change. For example, vacation lots may become retirement homes, or be sold as permanent residences in a phased plan for development.

The analysis should consider if commercial activities associated with full build-out of the development or community is intended. MDD and PHD estimates for water systems serving general commercial and business needs should be based on the appropriate application of analogous systems, Table 3-2, and the UPC fixture method (see Appendix D.2).

Adjustments to any established design criteria should reflect actual or anticipated conditions. These adjustments should provide a realistic margin of safety for reasonably anticipated increases in demand. For some projects, future water system demand, or standby or fire suppression storage needs may exceed the engineer's initial estimate. This could occur when a water system experiences higher-than-expected growth, changing uses among existing customers, has historical supply reliability problems, or experiences higher or lower service demand due to changing economic and demographic influences.

3.5.1 Referencing Prior Years Data

Using several prior years' meter data will result in calculating different ERU_{MDD} from one year to the next. If the data is scattered, without any clear trend, then the highest ERU_{MDD} value within the study period should be applied to the design unless the design engineer can show that the data is unreliable or incomplete, or the highest ERU_{MDD} value is based on an unrepeatable event (for instance, a chronic failure of the reservoir level control, wide-spread installation of new landscaping requiring especially heavy irrigation, or use of the water supply to assist with dust control during an unusually active construction season). The intent of good water system design is to provide water systems with the capacity to supply the volume of safe drinking water demanded by its customers during all normal operating conditions. Applying an average of past normal operating conditions to future customers by definition excludes some normal operating conditions. Generally, water demand data spanning a period of several years should not be averaged to determine ERU_{ADD} and ERU_{MDD} for the built-out or planning year condition.

If production or consumption data reflect a clear trend toward higher or lower ERU_{MDD} with time, such trends should be factored into determining the selected design ERU_{MDD} value *while not extrapolating below the lower limit of the data set.*

In evaluating data, design engineers should exercise caution when water production data spans a period of imposed water use restrictions. A water system's decision to impose water use restrictions in response to drought conditions can artificially skew demand data lower. The National Drought Mitigation Center has historical drought information. Contact information is in Appendix C.

3.5.2 Commercial, Industrial, and Public Facilities

MDD and PHD estimates for industrial water systems can be based on customer contracted volumes (gpd or gpm), defined process needs, and/or analogous system data. Existing industrial and commercial users may have data logging capacity on their service meters, providing the design engineer with primary data on MDD and PHD that would be useful in designing for expansion of the system.

The analysis should address how future water-use may evolve without any change in the number of structures or spaces. For example, a concert venue may become more popular than anticipated, a second shift is added to a place of work, the irrigation or recreational water demands on the system increase as the clientele of a facility change, or a change in use of the facility occurs which changes water demand (e.g., a warehouse becomes a brewery and restaurant).

Part 2 – Nonresidential Demand Estimates

3.6 Estimating Nonresidential Water System Demand

Nonresidential water demand is the water consumed by users other than single or multifamily residential units. They can include:

- Commercial facilities (retail or wholesale businesses, restaurants, hotels, office buildings, and car washes).
- Industrial customers that require process water.
- Public facilities (schools, public hospitals, governmental offices, parks, landscaped roads, and cemeteries).
- Other large users, such as farms with irrigated crops.
- Recreational users (campgrounds, RV parks, ski resorts, and seasonal rental units).

Water systems that consist solely of these types of demand are usually classified as "transient noncommunity" or "nontransient noncommunity" water systems (see WAC 246-290-020). The ERU model <u>does not apply</u> to these types of water systems.

Design engineers should use different approaches to determine water demands for non-residential customers because these types of customers do not follow residential water use patterns. Applying the principles of Section 3.2, the following sources of information should be used to estimate ADD, MDD, and PHD for non-residential uses:

- Actual water use information correlated to the expected future uses (for an expanding non-residential system)
- Values from Table 3-2 or from an analogous nonresidential water system
- Fixture unit analysis based on Uniform Plumbing Code guidelines (see Appendix D.2 for guidance)

In evaluating data, design engineers should exercise caution when water production data spans a period of imposed water use restrictions. A water system owner's decision to impose water use restrictions in response to drought conditions can artificially skew demand data lower, and using artificially low production and demand data to design an expansion to the system would permanently lock in constrained water use conditions.

3.6.1 Procedures for Estimating Nonresidential Demands

Design engineers can base ADD and MDD estimates for **new** nonresidential water systems on similarly sized analogous facilities or water systems. Table 3-2 offers reasonable estimates of daily water demands for a variety of uses. Design engineers can create a reasonable estimate for MDD by multiplying the number of "units" (e.g., resident, RV, bed, patron, etc.) or maximum anticipated use (e.g., airport passengers, vehicle visits) served by the water system times the unit water demand value in Table 3-2. Table 3-2 does not account for outdoor watering needs and fire protection requirements associated with uses listed.

Design engineers may find other information sources more valuable than Table 3-2. The designer should review several information sources to ensure compliance with local codes and to provide for an adequate factor of safety in the design. Recommended resources include the:

- **Uniform Plumbing Code** (UPC). Under Appendix A of the UPC, engineers can total the number of water supply fixtures in a building and convert it to an estimated peak water system demand. Local jurisdictions may require a water system to use the UPC to estimate demand.

- **Department of Ecology**. Engineers should consult any specific water-demand estimates the Department of Ecology prepared to see if they reflect adjustments for the proposed water use efficiency practices.

- **American Water Works Association (AWWA)**. Design engineers should consult AWWA for information on recently developed or updated demand estimates. This source will be especially important if current data (based on published reports and research) was recently applied to estimates that reflect water use efficiency practices, regional demographic changes, or other adjustments to previous tabulations. If the design engineer cannot find pertinent information through other sources, refer to AWWA guidelines in Table 3-2 and the UPC.

- **National Forest Service.** FSH 7409.11 - SANITARY ENGINEERING AND PUBLIC HEALTH HANDBOOK, Chapter 40 – Drinking Water System Design and Construction, last revised October 1, 2004.

- **Regional Office**. If information in Table 3-2 does not appear to apply to the project, design engineers can contact us to determine appropriate criteria that may apply on a case-by-case basis.

Table 3-2: Guide for Maximum Daily Nonresidential Water Demand[1]

Type of Establishment	Water Used (*gpd*)
Airport (per passenger)	3 - 5
Bathhouse (per bather)	10
Boardinghouse (per boarder)	50
Additional kitchen requirements for nonresident boarders	10
Camp	
Construction, semi-permanent (per worker)	50
Day, no meals served (per camper)	15
Luxury (per camper)	100 - 150
Resort, day and night, limited plumbing (per camper)	50
Tourist, central bath and toilet facilities (per person)	35[2]
Cottage, seasonal occupancy (per resident)	50
Club	
Country (per resident member)	100
Country (per nonresident member present)	25
Factory (gallons per person per shift)	15 - 35
Highway rest area (per person)	5
Hotel (per person)	50
Institution other than hospital (per person)	75 - 125
Hospital (per bed)	250 - 400
Lawn and Garden (per 1,000 sq. ft., at 2-inches per week application rate)	180 gpd per 1000 sf[3]
Laundry, self-serviced (gallons per washing per customer)	50
Livestock Drinking (per animal)	
Beef, yearlings	20
Brood Sows, nursing	6
Cattle or Steers	12
Dairy	20
Dry Cows or Heifers	15
Goat or Sheep	2
Hogs/Swine	4
Horse or Mules	12
Livestock Facilities	
Dairy Sanitation (milk room)	500
Floor Flushing (per 100 sq. ft.)	10
Sanitary Hog Wallow	100

Type of Establishment	Water Used (*gpd*)
Motel	
Bath, toilet, and kitchen facilities (per bed space)	50
Bed and toilet (per bed space)	40
Park	
Overnight, flush toilets (per camper)	25[2]
Trailer/RV no sewer connection (per trailer)	25[2]
Trailer/RV connected to sewer (per trailer)	140[4]
Picnic	
Bathhouses, showers, and flush toilets (per picnicker)	20
Toilet facilities only (gallons per picnicker)	10
Poultry (per 100 birds)	
Chicken	5 - 10
Ducks	22
Turkeys	10 - 25
Restaurant	
Toilet facilities (per patron)	7 - 10
No toilet facilities (per patron)	2 ½ - 3
Bar and cocktail lounge (additional quantity per patron)	2
School	
Boarding (per pupil)	75 - 100
Day, cafeteria, gymnasiums, and showers (per pupil)	25
Day, cafeteria, no gymnasiums or showers (per pupil)	20
Day, no cafeteria, gymnasiums or showers (per pupil)	15
Service station (per vehicle)	10
Store (per toilet room)	400
Swimming pool (per swimmer) Maintenance (per 100 sq. ft.)	10
Theater	
Drive-in (per car space)	5
Movie (per auditorium seat)	5
Worker	
Construction (per person per shift)	50
Day (school or offices per person per shift)	15

Footnotes to Table 3-2

1. Table adapted from *Design and Construction of Small Water Systems* (AWWA, 1984) and *Planning for an Individual Water System (Assn for Vocational Instructional Materials*, 1982), unless otherwise noted.
2. Add the 25-35 gpd per camper value to the 25 gpd where trailer/RV is without a sewer connection.
3. U.S. Bureau of Reclamation, Argimet, 2015, for Eastern Washington locations.
4. WSDSHS. 1983. *Sizing Guidelines for Public Water Supplies*, Washington State Department of Social and Health Services, Olympia, WA.

3.6.2 Commercial, Industrial and Public Facility Demand

Water demands for commercial, industrial, and public facility categories range widely from less than, to significantly more than, a single-family residence. This is especially true for large farm irrigation needs or commercial and industrial processes

MDD and PHD estimates for industrial water systems can be based on customer contracted volumes (gpd or gpm), defined process needs, and/or analogous system data. Existing industrial and commercial users may have data logging capacity on their service meters, providing the design engineer with primary data on MDD and PHD that would be useful in designing for expansion of the system.

Design engineers can estimate MDD and PHD for water systems serving general commercial and business needs by appropriate application of analogous system data, Table 3-2, the UPC fixture method (see Appendix D.2), and/or other reference documents on non-residential water use. To estimate water demands, designers should use these planning guides together with documented water-use records for existing facilities within the water system, or comparable uses at other water systems.

The design engineer should identify the specific (existing, known, and planned) buildings and building sites that comprise the current and future customers of a commercial, industrial, or public facility water system. A table summarizing the MDD and PHD of each building will assist us in summarizing the scope of our design approval. The design engineer should also provide an estimate of the total maximum daily population served.

Documenting physical capacity for water systems serving primarily commercial, industrial, and/or public facilities is discussed in Chapter 4.

3.6.3 Farming and Crop Irrigation Demand

Engineers should consult with the local Cooperative Extension office when determining water-use estimates for farms. It may be possible to find water-use records for various farm practices in the area. Table 3-2 provides some water-use references by type and number of livestock. Irrigation needs can be extremely variable and may require additional investigation. An index of local extension offices can be viewed at http://extension.wsu.edu/locations.

Some variables that influence water demands for farming and crop irrigation are:

- Type of farm.
- Number and type of animals it produces.
- Type of crops it grows.
- Weather conditions.
- Geographic location.

3.6.4 Recreational Development Demand

"Recreational development" applies to facilities that individuals and families intend to use for vacations or holidays away from their normal place of residence. There is a full spectrum of recreational development types: Some are comprised of simple campsites suitable for tents or trailers in a manner similar to a state campground, while others may be an elaborate community of rustic housing equipped with most, if not all, the amenities of urban living. The design engineer should identify in the design submittal the specific buildings (e.g., camp lodge, bath/shower, and dining hall) and designated locations (e.g., camp sites, RV sites), and the total maximum daily population expected to be served.

Recreational developments may be eligible for reduced water system design criteria provided certain conditions are met. Reduced design criteria will only apply to sites intended solely for recreational occupancy. No permanent residential dwelling/structure, no matter how small, how simple, or how rustic, is permitted on a site designated for recreational uses. Note: Recreational development water systems that will serve residential dwellings not otherwise restricted from full-time occupancy **must** be designed consistent with demand values associated with permanent residences (WAC 246-290-221(2)).

We will consider reduced-design criteria if the project report and construction documents for a recreational development can demonstrate all of the following:

1. There are clearly defined sites for each occupant. Recreational developments can define sites by surveyed lot lines, permanent site markers, or surveyed-site centerlines drawn on a map that identifies the location of each site. We expect the number of sites or lots for the total tract to remain the same.
2. The acknowledged purpose of the recreational development is to provide space for short-term, transient, or seasonal use only.
3. Residential dwellings must be restricted from full-time occupancy.
4. We received satisfactory documentation of claims made with respect to items 1 -3 above. This may include a notation of the restrictions on the face of the plat, in covenants filed with the plat, or in individual deeds.

Ownership and operation of recreational developments vary along a wide spectrum. Some recreational developments operate on a membership basis while others sell facilities lot-by-lot, as in an ordinary residential plat. Recreational development water system owners that receive approval for reduced design criteria are obligated to operate their system within the approved design parameters. Ownership concentrated in a single decision-maker will have greater flexibility and capacity to ensure consumer demands do not exceed the design assumptions. Diffused ownership across many different owners (such as each lot owner in an association) may limit the ability to ensure design assumptions are not exceeded during operation. In responding to designs submitted for a recreational development water system under diffused ownership we may request additional documentation on the owner's authority to enforce water use consistent with design assumptions and on the owner's plans to respond to water shortages.

Design engineers can use Table 3-2 and other design references such as the UPC fixture method and the Department of Ecology wastewater flow tables (WSDOE 2008) to provide daily water use estimates for typical recreational and other nonresidential facilities.

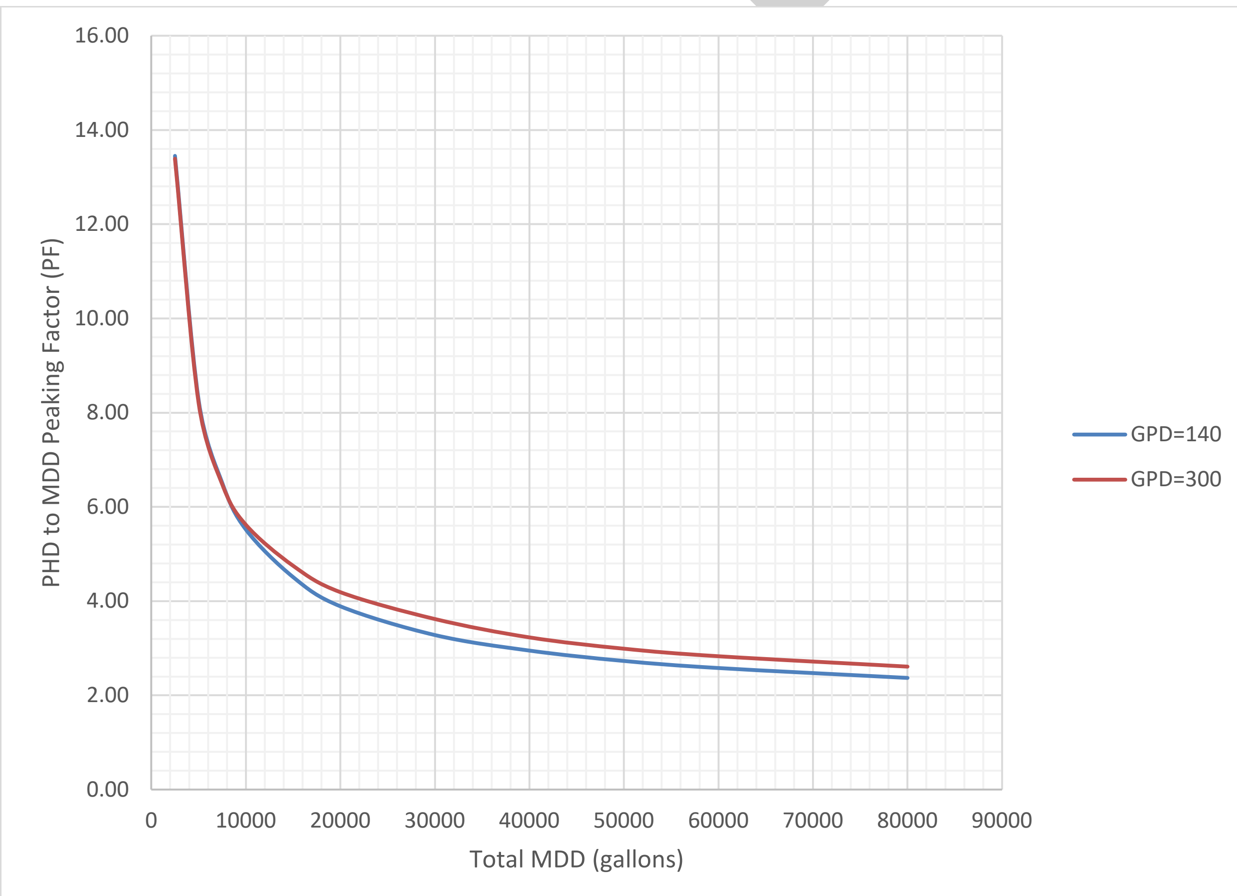
Regardless of the source of information, the MDD for recreational development water systems serving structures with internal plumbing suitable for short-term occupancy (such as overnight transient accommodation RVs and cabins) should be based on:

- No less than 140 gpd per site or lot.
- Full (site) occupancy.
- And include all other water uses including swimming pools, irrigation, water features, and commercial buildings.

The maximum daily demand to peak hourly demand peaking factor values in Figure 3.1 are applicable to water systems serving recreational demands. The two curves in Figure 3.1 were derived from Equation 3-1, applying 140 gpd and 300 gpd estimated maximum daily demand per recreational unit. The graph reveals differences in recreational unit MDD has little bearing on the peaking factor.

Once the unit MDD and the number of units are identified, the total estimated MDD can be determined. Using the graph and Eq. 3-2 below, designers can then estimate the system-wide PHD.

Figure 3.1
Recreational Water System
Maximum Daily Demand to Peak Hourly Demand Peaking Factor



Equation 3-2:

$$PHD_{recreational} = (MDD \div 1440) \times PF$$

Where	**PHD**	=	Peak hourly demand, total system (gallons per minute)
	MDD	=	Maximum daily demand, including DSL, total system (gallons per day)
	PF	=	MDD to PHD peaking factor, from Figure 3.1

3.6.5 Anticipating Changes in Demand for Systems with Significant Non-Residential Demand

Water demand estimates for non-community systems should address anticipated changes as a water system matures and business needs change. Changes in future water demands is likely a function of changes in the type and level of business done by the non-residential facilities served.

The analysis should address how future water-use may evolve without any change in the number of structures or spaces. For example, a concert venue may become more popular than anticipated, a second shift is added to a place of work, the irrigation or recreational water demands on the system increase as the clientele of a facility change, or a change in use of the facility occurs which changes water demand (e.g., a warehouse becomes a brewery and restaurant).

Adjustments to any established design criteria should reflect actual or anticipated design conditions. These adjustments should provide a realistic margin of safety for reasonably anticipated increases in demand. For some projects, water system demand, standby storage and/or fire suppression storage needs may exceed the engineer's initial estimate. This could occur when a water system experiences higher-than-expected growth, changing uses among existing customers, has historical supply reliability problems, or experiences higher or lower service demand due to changing economic and demographic influences.

3.7 Establishing Needed Fire Flow

The local fire protection authority or county fire marshal usually determines minimum fire flow requirements (WAC 246-290-221(5)). Design engineers should always confirm the fire suppression requirements associated with a given water system design with the local fire protection authority or county fire marshal.

3.8 Factoring Distribution System Leakage (DSL) in Design

Water use efficiency (WUE) requirements apply to municipal water suppliers. In general, municipal water suppliers are community water systems with 15 or more residential service connections. Some noncommunity water systems that serve water in a residential manner to 25 or more people at least 60 days per year (such as a second home community) are also considered municipal water suppliers. This determination is made on a case-by-case basis. For more information on WUE requirements, order the *Water Use Efficiency Guidebook* (DOH 331-375) or contact the appropriate regional planner.

Municipal water suppliers **must** meet certain leakage standards to minimize water lost through distribution system leaks. Most municipal water suppliers that lose more than 10 percent of the water they produce through DSL **must** take action to reduce their leakage (WAC 246-290-820).

Design engineers cannot use projections of water savings resulting from future leak detection and repair, or from future implementation of planned WUE measures, in establishing the design criteria for an expanding water system (WAC 246-290-221).

Design engineers **must** establish sizing criteria that account for water system demands during the highest demand periods, including DSL (WAC 246-290-222). The design information should be sufficient to estimate peak hourly demand (PHD) and MDD for the built-out condition. DSL is defined and calculated as:

Equation 3-3:

$$\mathbf{DSL = TP - AC}$$

Where:

DSL = Distribution system leakage (gallons per day)
TP = Total water produced and purchased over a full year, divided by 365 (gallons per day)
AC = Authorized consumption over a full year, divided by 365 (gallons per day).

Note: *Authorized consumption is the volume of metered and unmetered water used by consumers and others authorized to do so by the water system, including, but not limited to, fire- fighting and training, flushing of mains and sewers, street cleaning, and watering of parks and landscapes. These volumes may be billed or unbilled.*

Water system production “lost” through DSL is no longer available for customer service. As such, DSL reduces a water system’s ability to serve customers. A water system can increase its ability to serve more customers by reducing DSL. Some DSL will occur, even in very well maintained and managed water systems. For most water systems, it is impractical to eliminate all DSL (AWWA 2006). DSL is a demand component of every water system, and so should be included with the assessment of water system capacity. See Chapter 4.

For water systems, several factors influence the real water losses that are part of DSL including:

- Number of service connections.
- Length of water mains.
- Average operating pressure.
- Infrastructure condition (Thornton 2002; AWWA 2006).

Because these factors are independent of demand, DSL is more likely to be consistent on a volume basis than on a percentage basis throughout a given year. Engineers can use the most recent three-year average annual volume of DSL and divide by 365 to identify a daily volume of DSL.

3.9 Water Resource Issues

Competition over the state’s water resources by a growing population, natural resource interests, and a vibrant economy is steadily increasing. The Department of Ecology is the state agency responsible for managing the state’s water resources, implementing a regulatory program for allocation of those resources, and enforcing its provisions.

For designs involving new or expanding sources, or increases in water system capacity, engineers **must** address water rights as a part of a submittal to us (WAC 246-290-100(4)(f), 105(4)(e), 110(4)(e), 120(7), 130(3), and 132(3)(b)). In these submittals, the engineer must complete a *Water Rights Self-Assessment Form* as part of the water rights analysis unless otherwise noted. The project information on this form is used by Ecology to assess if the project and its associated water system demands are consistent with certain limits that may be specified on a utility’s water right permit, certificate, or claim. The relationship between water system design and water rights are discussed throughout this manual.

3.10 Source Adequacy and Reliability

The design of a water system frames the operational expectations and establishes the system’s adequacy and reliability to meet consumer demands. State rules require water systems to maintain a minimum level of service during normal (non-emergency) operating conditions (WAC 246-290-420). Consumers have a reasonable expectation to an adequate supply of water not just during average conditions but also during high demand periods. Design assumptions about source adequacy and reliability have a significant impact on the ability of the water system to meet future regulatory obligations and consumer expectations.

3.10.1 Design and Operating Requirements

WAC 246-290-230 establishes the requirement that water systems must be design to provide at least 30 psi throughout the distribution system during peak hourly demand conditions. WAC 246-290-420 requires water systems to be operated so that pressure throughout the distribution system is maintained at or above the approved design pressure. A source impacted by periodic drought or administrative restriction on withdrawal to the point that it cannot keep up with demand will require restrictions on demand to maintain pressure.

3.10.2 Surface Water Source Reliability

Surface water source reliability is dependent on environmental factors such as rainfall, snow pack, and runoff rates during drought conditions. Climate change may amplify these factors, making it all the more important design engineers consider the need for resiliency in the face of changing conditions over the life of the water system. Source reliability may also be dependent on legal restrictions to withdraw water, or on the design and maintenance of the source infrastructure such as a raw water impoundment. Source reliability can be expressed by how frequently a water system expects normal demand to go unmet, such as a one-in-50, or even -100 year drought.

We consider 98 percent source reliability an appropriate design standard for evaluating a watershed's capacity. This implies consumers should expect water system-imposed restrictions on water use to occur on average once every 50 years. The duration of the once in 50-year water supply restriction should be assessed at the time of the design and addressed in the utility's water shortage response plan. Additional information on assessing the reliability of surface water supplies is in Chapter 5.

If the design engineer adopts a lesser reliability standard consumers will be expected to accept a greater frequency of insufficient supply and more frequent mandatory demand curtailment. In this context, reliability becomes a balance between consumer expectations and the cost of meeting such expectations. The design engineer should document and provide engineering justification for the source reliability factor selected. See Section 4.4.2.3.

3.10.3 Ground Water Source Reliability

Ground water source reliability is dependent on environmental factors such as rainfall and hydrogeologic characteristics of the aquifer. Ground water source reliability may also be dependent on legal restrictions to withdraw water, such as a requirement from the Department of Ecology to interrupt well withdrawal during low flow periods in a nearby stream. Source reliability can be expressed by how frequently a water system expects normal demand to go unmet, such as a one-in-50, or even -100 year recurrence interval.

For wells that are subject to interruption due to low stream flow, we recommend a once-in-fifty year interval for interruption be used as the basis for establishing reliance on the source to meet normal water system demand.

The pump test protocol (see Appendix E) selected should identify the aquifer safe yield. Based on the pump test protocol and quality of data, the design engineer should apply an appropriate factor of safety (e.g., multiply pump test results by 0.85) in calculating safe yield to account for unknown hydrogeologic conditions and future climatic conditions.

3.10.4 General Factor of Safety and Contingency Planning

We recommend against designs based on 24-hours per day pumping to meet future MDD. Designing or evaluating a system assuming some period less than constant pumping to satisfy projected MDD provides a factor of safety and an increased ability to meet unexpected demands. We recommend assessing source capacity based on an assumption of pumping a source no more than 20 hours per day.

Source adequacy and reliability are important long-range planning elements and must be addressed in water shortage response planning under WAC 246-290-100(4)(f) and -415(2)(d). Plans to ensure long-range water system adequacy and reliability should address:

- Water shortage response activities such as accessing alternative water supplies and notifying us.
- Long-term adequacy of water rights for meeting growth expectations of the water system.
- Conservation as a mitigating practice to reduce the frequency or degree of curtailment.
- Water resource trends (such as declining aquifer levels, declining dry period stream flows, establishing in-stream flow requirements, increasing salt water intrusion)

3.10.5 Summary of Water Supply Reliability Recommendations

Recommendations for source and water system reliability appear in various chapters of this manual. The following is a brief summary of our recommendations for water supply reliability.

1. Two or more supply sources are available
2. Permanent and seasonal sources are capable of replenishing depleted fire suppression storage within 72-hours (continuous, 24-hour source pumping may be assumed for replenishment), or sooner if required by the local fire authority, while concurrently supplying the MDD for the water system.
3. Permanent and seasonal source capacity is enough to supply MDD in a period of 20 hours or less of pumping.
4. With the largest source out of service, the remaining permanent and seasonal sources can provide a minimum of ADD for the water system.
5. Source of supply pump stations have power connections to two independent primary public power sources, have in-place auxiliary power available (auto transfer capable), and/or maintain adequate gravity standby storage (see Chapter 7).
6. The firm watershed yield (for surface water sources) provides 98 percent reliability to meet normal, anticipated system demands; and a factor of safety applied to a well pumping test safe yield determination.

3.11 Factor of Safety

We support the design of robust and resilient water systems that are based on the best available demand data. In the absence of reliable and applicable information, we expect design engineers to apply their professional judgment and to document their assumptions. Despite using even the best available information, uncertainty about future conditions and validity of assumptions will persist. That's why we recommend the use of a factor of safety (FS) in the design of water systems.

Using an appropriate FS is common in the engineering profession. Below we describe in general terms the range of FS given the degree of confidence the design engineer has in the data, and the scope of design assumptions.

- Mostly or entirely confident: FS = 1.10 to 1.15
- Somewhat confident: FS = 1.15 to 1.25
- Mostly uncertain: FS = 1.25 to 1.5

3.12 Example Exercises in Estimating Water System Demand

To illustrate the standards and concepts described in this chapter we offer the following examples. These examples are not intended as a recipe for design engineers. All reference to water rights assumes prior verification by the Department of Ecology.

3.12.1 New Community Water System

Known: Planned 100-lot subdivision with ½ to 1 acre lots. All lots will have single family residential homes expected to have at least 3,000 square feet of living space. The subdivision is located in Benton County. Average annual rainfall for the project location is 9 inches per year. The soil is sand.

Find: ADD, MDD, PHD, ERU_{ADD}, and ERU_{MDD}

Solution:

1. Reference information in Appendix D.
2. Mostly confident in Appendix D ADD information. Apply FS to ERU_{ADD} of 1.10
3. ERU_{ADD} = [8000 ÷ 9] + 200 = 1090 gpd x 1.1 = 1,200 gpd
4. Assume future DSL to be 10 percent of annual consumption. DSL = 1,200 x 100 connections x 0.10 = 12,000 gpd
5. Total system-wide ADD = annual consumption plus DSL. [1,200 gpd x 100] + 12,000 gpd = 132,000 gpd. Use 130,000 gpd
6. Qa on the water right must provide at least 130,000 gpd x 365 days per year = 47.5 MG per year.
7. Determine ERU_{MDD}. Select an ERU_{MDD} to ERU_{ADD} peaking factor of 2.0 as recommended in Appendix D.

8. Mostly uncertain in Appendix D MDD to ADD peaking factor for Eastern Washington. Apply FS to peaking factor of 1.3.
9. ERU_{MDD} = 1,200 x 2 x 1.3 = 3,120 gpd
10. Translate DSL into ERUs. 12,000 gpd ÷ 3,120 = 3.8 ERUs. Total number of ERUs, “N”, supplied by sources equals 100 + 3.8 = 104
11. Total system-wide MDD = [3,120 x 100 connections] + 12,000 = 324,000 gpd
12. Design source pumping capacity to meet MDD in 20 hours (Section 3.10.4): 324,000 gpd ÷1200 min per day = 270 gpm
13. Qi on the water right must provide at least 325,000 gpd ÷1440 = 225 gpm. Ideally, Qi is at least 270 gpm.
14. Use Equation 3-1 to determine PHD
    a. PHD = [(3,120 ÷1440) x (2.5 x 104 +25)] +18 = 630 gpm.

3.12.2 Expanding (Existing) Community Water System

Known: An existing 100-lot subdivision has ¼ acre lots. All services are single family residential dwellings. The existing subdivision was fully built-out by the mid 1990’s. Proposal is to add 200 additional single family residential ¼ acre lots. All homes are primary residences, and occupied on a full-time basis. The existing system is located in Klickitat County.

DSL is indeterminable due to incomplete service metering.

Monthly source production is generally recorded. The maximum monthly production for the past 10 years is as follows (from oldest to most recent): 4.1 MG, 3.4 MG, 3.2 MG, 3.0 MG, 2.5 MG, 2.2 MG, 2.3 MG, 2.2 MG, 2.0 MG, and 2.5 MG. Further investigation into oldest maximum monthly production data (4.1 MG) reveals two anomalies: Undetected failure of the reservoir control system, resulting in significant waste of water through a prolonged reservoir overflow; and a nearby wildfire which the local fire authority used the community water supply to suppress.

Annual production for the past 10 years is as follows (from oldest to most recent): 26 MG, 22 MG, 20 MG, 20 MG, 19 MG, 19 MG, 18 MG, 18 MG, 16 MG, 17 MG and 17 MG. Average climatic conditions prevailed during this period, and there were no restrictions imposed on water use.

Find: ADD, MDD, PHD, ERU_{ADD}, and ERU_{MDD} for the proposed expanding water system

Solution:

1. Discard first year data for monthly and annual source production volume.
2. Apply a factor of safety of 1.10 to the annual and maximum monthly production data, based on a high degree of confidence in the remaining data set.
3. Annual production data reflects a trend toward lower production with time. Use 17 MG per year to determine future ERU_{ADD}. 17 MG x 1.10 = 18.7 MG.
4. Assume DSL to be 10 percent of annual production. DSL = 1.9 MG/year
5. Annual consumption = Production – DSL = 18.7 MG/yr – 1.9 MG/yr = 16.8 MG/yr
6. ERU_{ADD} = [16.8 MG/yr] / 100 / 365 = 460 gpd
7. Maximum monthly production for the past three years does not indicate any trend. Use high value = 2.5 MG/month to determine ERU_{MDD}. 2.5 MG x 1.10 = 2.75 MG
8. Maximum monthly consumption = Max monthly production – DSL (monthly volume)

- 2.75 MG – [1.9MG ÷12] = 2.6 MG

9. ERU_{MDD} = [2.6 MG x 1.7 peaking factor (see Section 3.4.1)] ÷ 100 ÷ 32 days between measurements = 1,380 gpd
10. Translate existing DSL into ERUs. [1.9 MG/yr ÷ 365] ÷ 1380 = 3.8 ERUs. Use 4
11. Future DSL will be proportional to the number of lots served because the distribution system is expanding proportionately.
12. Future number of ERUs = 300 + 12 = 312
13. Total future system-wide ADD = [300 x 460 gpd] + [(1.9 MG x 3) ÷ 365] = 154,000 gpd. Use 155,000 gpd
14. Qa on the water right must provide at least 155,000 gpd x 365 days per year = 56.5 MG per year.
15. Total future system-wide MDD = [300 x 1,380 gpd] + [(1.9 MG x 3) ÷ 365] = 430,000 gpd.
16. Design source pumping capacity to meet MDD in 20 hours (Section 3.10.4): 430,000 gpd ÷1200 min per day = 360 gpm
17. Qi on the water right must provide at least 430,000 gpd ÷1440 = 300 gpm. Ideally, Qi is at least 360 gpm.
18. Use Equation 3-1 to determine PHD
    a. PHD = [(1380 ÷1440) x (1.8 x 312 +125)] +18 = 680 gpm

3.12.3 New Mixed-Use Community Water System

Known: Proposed new planned unit develop with 100 1-acre single family home lots, 2-acre community park, and a 100-unit RV park. The subdivision is located in Benton County. Average annual rainfall for the project location is 9 inches per year.

Find: ADD, MDD, PHD, ERU_{ADD}, and ERU_{MDD}

Solution:

1. Reference information in Appendix D.
2. Mostly confident in Appendix D ADD information. Apply FS to ERU_{ADD} of 1.10
3. ERU_{ADD} = [8000 ÷ 9] + 200 = 1,090 gpd x 1.1 = 1,200 gpd
4. Determine ERU_{MDD}. Select an ERU_{MDD} to ERU_{ADD} peaking factor of 2.0 as recommended in Appendix D.
5. Mostly uncertain in Appendix D MDD to ADD peaking factor for Eastern Washington. Apply FS to peaking factor of 1.3.
6. ERU_{MDD} = 1,200 x 2 x 1.3 = 3,120 gpd
7. Irrigation of the 2-acre community park: Use 180 gpd per 1,000 sf (Table 3-2) and assume 182 days of irrigation per year.
    a. 180 x (2 x 43,560) ÷ 1,000 = 15,681 gpd x 182 days = 2.85 MG (annual)
    b. MDD for park irrigation is 15,681. Use 16,000 gpd
8. 100-unit RV park consumptive demand estimated at 140 gpd/unit (see Table 3-2). Assume the RV park is 100% occupied during the spring, summer, and fall; closed during the winter months.
9. Annual consumption.
    a. Single family homes = 1,200 x 100 connections x 365 = 44 MG
    b. Community park irrigation = 2.85 MG

c. RV park = 100 units x 140 gpd/unit x 100% occupancy x (365 x 0.75) = 3.8 MG
d. Total annual consumption = 50.6. Use 51 MG

10. Average daily consumption = 51 MG ÷ 365 = 140,000 gpd
11. Assume DSL to be 10 percent of annual consumption. DSL = 140,000 x 0.10 = 14,000 gpd
12. Total system-wide ADD = 140,000 + 14,000 = 154,000 gpd.
13. Qa on the water right must provide at least 154,000 gpd x 365 days per year = 56 MG per year.
14. Translate DSL into ERUs. 14,000 gpd/3,120 = 4.4 ERUs.
15. Determine total number of ERUs, "N", supplied by sources
    a. 100 SFHs = 100
    b. DSL = 4.4
    c. Community park = 16,000 ÷ 3,120 = 6.4
    d. RV park = 100 units x 140 gpd/unit = 14,000 ÷ 3,120 = 4.4
    e. Total number of ERUs, "N" = 115. System is predominantly residential.
16. Total system-wide MDD = (100 x 3,120) + 16,000 + 14,000 + 14,000 (DSL) = 356,000 gpd.
17. Design source pumping capacity to meet MDD in 20 hours (Section 3.10.4): 356,000 gpd ÷1200 min per day = 297 gpm. Use 300 gpm
18. Qi on the water right must provide at least 356,000 gpd ÷ 1440 = 247 gpm. Ideally, Qi is at least 300 gpm.
19. Use Equation 3-1 to determine PHD
    a. PHD = [(3,120 ÷ 1440) x (2.0 x 115 +75)] +18 = 680 gpm.

3.12.4 Expanding Mixed-Use Community Water System

Known: Existing 100-lot subdivision built-out in the 1980's with 1 acre lots each. The existing system is located on Bainbridge Island. All services are single family residential.

The proposal is to add 200 additional single family residential lots on ¼ acre lots, a 2-acre community park, and a 100-unit RV park.

The three-year average DSL was calculated at 15 percent. Monthly source production is recorded. Service meters are read every two months. Through a survey the water system determined that 40 homes are currently used as seasonal residences (summer only), and 60 homes are occupied on a full-time basis.

Annual production for the past 5 years is as follows (from oldest to most recent): 16 MG, 13 MG, 17 MG, 18 MG, and 16 MG. Annual consumption of the 60 permanent homes has been 7 MG, 7 MG, 8 MG, 9 MG, and 7 MG.

Peak bi-monthly consumption data for the past 5 years (from oldest to most recent): 4.3 MG, 4.5 MG, 4.4 MG, 4.2 MG, 4.1 MG. The peak bi-monthly consumption data of the 60 permanent homes has been 2.7 MG, 2.8 MG, 2.6 MG, 2.5 MG, and 2.5 MG.

Find: ADD, MDD, PHD, ERU_{ADD}, and ERU_{MDD} for the proposed expanding water system.

Solution:

1. Mostly confident in the water system's annual production and bi-monthly consumption data. Apply a FS = 1.10.
2. Annual production and consumption data does not reflect a trend toward lower withdrawal over time. Use 18 MG. 18 x 1.10 = 19.8 MG. Use 20 MG
3. DSL was calculated at 15 percent.
   a. DSL = 20 MG/yr x 0.15 = 3 MG/yr
4. Use annual consumption of 60 permanent homes to estimate ERU_{ADD}. Data does not reflect a trend toward lower peak bi-monthly consumption over time. Use 9 MG x 1.10 = 10 MG
   a. ERU_{ADD} = 10 MG ÷ 60 homes ÷ 365 days = 457 gpd
5. Use peak bi-monthly consumption of 60 permanent homes to estimate ERU_{MDD}. Data reflects a trend toward lower peak bi-monthly consumption over time. Use 2.5 MG and peaking factor of 1.7 (see Section 3.4.1).
   a. ERU_{MDD} = 2.5 MG x 1.10 ÷ 60 homes ÷ 60 days = 763 x 1.7 = 1,298 gpd. Use 1,300 gpd.
6. Translate existing DSL into ERUs. [3 MG/yr ÷ 365] ÷ 1,300 = 6.3 ERUs.
7. Assume DSL is proportional to size of the distribution system. Future DSL will be 1.6 times existing DSL because the distribution system is expanding by 60 percent. Future DSL ERUs = 6.3 x 1.6 = 10 ERUs
8. Future annual consumption:
   a. 100 existing single family homes: 457 gpd/ERU x 100 x 365 = 16.7 MG
   b. 200 new homes: Use a 20% reduction in ERU_{ADD} because of smaller lot size supported by analogous system data (meeting the analogous system data standards in Section 3.2.3): 200 x 457 gpd/ERU x 0.8 x 365 = 26.7 MG
   c. Community park irrigation: Use 180 gpd per 1,000 sf (Table 3-2) and assume 100 days of irrigation.
      i. 180 x (2 x 43,560) ÷ 1,000 = 15,681 gpd x 100 days = 1.57 MG
   d. RV park: 140 gpd/unit (see Table 3-2). Assume the RV park is 80% occupied during the spring, summer, and fall; closed during the winter months.
      i. 100 units x 140 gpd/unit x 0.80 (occupancy) x 365 x 0.75 = 3 MG
   e. DSL = 3 MG x 1.6 = 4.8 MG
   f. Total future annual production requirement = 51 MG per year
   g. Future system-wide ADD = 51 MG ÷ 365 = 145,000 gpd
9. Qa on the water right must provide at least 51 MG per year.
10. Future maximum daily consumptive demands:
    a. 100 existing single family homes: 1,300 gpd/ERU x 100 = 130,000 gpd
    b. 200 new homes: Use a 30% reduction in ERU_{MDD} because of smaller lot size supported by analogous system data (meeting the analogous system data standards in Section 3.2.3): 200 x 1,300 gpd/ERU x 0.7 = 182,000 gpd
    c. Park irrigation MDD is 15,681. Use 16,000 gpd
    d. RV park: 140 gpd/unit x 100 units = 14,000 gpd
    e. DSL = 4.8 MG ÷ 365 = 13,000 gpd
    f. Future system-wide MDD = 355,000 gpd. The system is predominantly residential.
11. Future number of ERUs:
    a. Existing homes = 100 ERUs

b. New homes = 182,000 ÷1,300 = 140 ERUs
c. Park = 16,000 ÷ 1,300 = 12.3 ERUs
d. RV Park = 14,000 ÷ 1,300 = 10.8 ERUs
e. DSL = [4.8 MG ÷ 365] ÷ 1,300 = 10 ERUs
f. Total future ERUs = 273 ERUs.

9. Design source pumping capacity to meet MDD in 20 hours (Section 3.10.4): 355,000 gpd/1200 min per day = 296 gpm. Use 300 gpm
13. Qi on the water right must provide at least 355,000 gpd ÷ 1440 = 246 gpm. Ideally, Qi is at least 300 gpm.
14. Use Equation 3-1 to determine PHD
    b. PHD = [(1,300 ÷ 1440) x (1.8 x 273 +125)] +18 = 575 gpm

3.12.5 Expanding Non-Community Water System

Known: A single 120-gpm well (no storage) supplies an existing 200-unit RV park, convenience store, service station, a water park, and approximately 1 acre of irrigated lawn and garden. The existing facilities have been in service for the past two years. The park owner states the park is closed November 1 through April 1 each year. The park is located in Spokane County. There are no residences. Average occupancy has been 80 percent during the 7 months the park is open.

Year 1 and year 2 annual water production was 3.0 MG and 3.5 MG, respectively. The one well is metered, and is equipped with a variable speed drive submersible pump. The system has only pressurized storage. There are no service meters.

Year 1 maximum monthly production was recorded as 0.6 MG, and year 2 was 0.7 MG. Anecdotally, the owner indicated that the system has always maintained at least 30 psi throughout the distribution system, even during extreme demand events.

The owner wants to add 150 additional RV spaces, each with water and sewer connections; a laundromat; and another 1 acre of grassy area.

Find: ADD, MDD, PHD

Solution:

1. Mostly uncertain about the water system's annual production and bi-monthly consumption data. Apply a FS = 1.4 due to the limited amount of data, and certain variables still untested over an extended timeframe.
2. The existing system's estimated ADD: [3.5 MG x 1.4] ÷ 213 days (7 months) = 23,000 gpd.
3. The existing system's MDD can be estimated as follows:
    a. [0.7 MG x 1.4] ÷ 31 = 31,600 gpd x 1.7 MMADD to MDD peaking factor for the RV park (see Section 3.4.1) = 54,000 gpd
4. The existing system's PHD is estimated as (54,000 ÷1440) x 2.8 peaking factor (see Figure 3-1 and Equation 3-2) = 105 gpm
5. Maximum daily demand of proposed new uses:
    a. MDD of each new RV space is estimated at 140 gpd (see Table 3-2) x 150 = 21,000 gpd (full occupancy).

b. MDD of laundromat based on an estimate that it will service 20% of RV occupants each day. MDD of the laundromat is 50 gals (See Table 3-2) x 0.20 x 300 RV spaces = 3,000 gpd.
c. Irrigation to 1-acre grassy park estimated at 180 gpd per 1,000 sf (See Table 3-2).
    i. [180 x 43,560] ÷ 1,000 = 7,840 gpd. Use 8,000 gpd

6. Future Annual Consumption:
    a. 23,000 x 213 days/yr = 5.5 MG
    b. 21,000 x 0.8 occupancy x 213 days/yr = 3.6 MG
    c. 3,000 x 0.8 occupancy x 213 days/yr = 0.5 MG
    d. 8,000 x 180 days irrigation/yr = 1.4 MG
    e. Total = 11 MG
7. Future MDD:
    a. 54,000 + 21,000 + 3,000 + 8,000 = 86,000 gpd
8. Future PHD is estimated as (86,000 ÷ 1440) x 2.5 peaking factor (see Figure 3-1 and Equation 3-2) = 150 gpm
9. Since there is no storage, source production must meet PHD. Design source pumping capacity must be at least 150 gpm.
10. Qi on the water right must provide at least 150 gpm.
11. Qa on the water right must provide at least 11 MG

3.12.6 Assessing Full- and Part-Time Residential Use

Known: The 200-home built-out community is known as a mixed primary and secondary home community. It is a summertime community, as indicated by comparing monthly source production data over the past 5 years. The peak month for production is July or August.

The water system is supplied by three wells. The system has a single reservoir with an external gauge indicating reservoir level. All three sources are metered, and source meters are read every month. All homes have service meters, and service meters are read every two months.

July 4th falls on a Saturday and Labor Day falls on September 7.

Find: (1) ERU_{MDD} based on full time occupancy.
(2) ERU_{ADD} based on full time occupancy.

Approach:

Estimate ERU_{MDD}:

1. Assist the purveyor in preparing a plan to read each source meter and the reservoir level on Friday, July 3 and on Friday, September 4 (pre-holiday condition)
2. Assist the purveyor in preparing a plan to read each source meter and the reservoir level on Monday, July 6 and on Tuesday, September 8 (post-holiday condition)
3. Assist the purveyor to identify a cohort of representative homes to do a drive-by survey during one of the afternoons and evenings of each study period. The number of homes surveyed should equal at least 25 percent of all homes. Determine the level of occupancy of the entire community based on these observations.
4. Calculate the 3-day system demand over July 4th weekend and 4-day system demand over Labor Day weekend. Select the highest average daily demand for these periods and apply a factor of safety to account for peaking during the days subject to data collection.

5. Calculate ERU_{MDD} based on the source production (plus or minus reservoir level) and occupancy.
   a. Multiply the number of homes (200) by the percent occupancy determined from the survey.
   b. Divide the value determined in #4 above by the adjusted number of homes (reflecting occupancy). The calculated ERU_{MDD} will reflect DSL since source production data was used.

Estimate ERU_{ADD}:

1. Review the bi-monthly service meter records of each customer and select customers that have two or more two-month periods of consumption that is less than 100 gpd per residence. These can be assumed to be occupied intermittently, or seasonally.
2. For all remaining homes, calculate the average daily consumption per residence. This is the $ERU_{ADD\text{-}FULLTIME}$ value based on consumption of the homes considered likely to be occupied full time.
3. Apply the $ERU_{ADD\text{-}FULLTIME}$ to all homes to determine system ADD.

From these ERU_{ADD} and ERU_{MDD} the design engineer can assess the capacity of the system based on full time occupancy.

Example Solution for $MDD_{FULLTIME}$:

1. The difference between July 6 and July 3 source meter readings was 320,000 gallons. The difference in the reservoir level was 5,000 gallons less on July 6. Among the 200 homes, 70 were surveyed. Among the 70 surveyed, 63 homes were observed to be occupied.
2. The difference between September 8 and September 4 source meter readings was 460,000 gallons. The difference in the reservoir level was 10,000 gallons more on September 8. Among the 200 homes, 60 were surveyed. Among the 60 surveyed, 48 homes were observed to be occupied.
3. July's 3-day period of use equates to
   a. 325,000 gal production ÷ [(63/70 occupancy rate) x 200 homes x 3 days] = 600 gpd per occupied residence.
4. Similarly, September's 4-day period of use equates to
   a. 450,000 gal production ÷ [(48/60 occupancy rate) x 200 homes x 4 days] = 700 gpd per occupied residence.
5. Select the higher calculated daily production per residence: 700 gpd per home
6. Mostly confident in the methodology and accuracy of the occupancy survey, reservoir level measurement, and source meter data. Apply a FS = 1.15.
7. 700 x 1.15 = 800 gpd per residence. This value is inclusive of DSL, since the primary data was source production.
8. MDD based on full-time occupancy is 800 gpd x 200 = 160,000 gpd

Example Solution for $ADD_{FULLTIME}$:

1. Total yearly production for 2011, 2012, and 2013 was 15 MG, 17 MG, and 20 MG
2. A review of 2011, 2012, and 2013 consumption data reveals the following:

Year	Homes < 100 gpd consumption 4+ months	Homes ≥ 100 gpd consumption for at least 8 months	Total consumption at homes ≥ 100 gpd for at least 8 months	ERU_{ADD} for homes ≥ consuming 100 gpd for at least 8 months
2011	115	85	10.8 MG	349
2012	105	95	10.6 MG	307
2013	90	110	14.5 MG	362

3. The results show no trend. Normal climatic conditions prevailed during this period. Select the higher calculated daily consumption per residence: 362 gpd per home
4. Mostly confident in the assumptions regarding threshold for fulltime occupancy (100 gpd per residence) and the accuracy of data collection. Apply a FS = 1.15.
5. $ADD_{FULLTIME}$ = 362 x 1.15 = 416 gpd per residence. This value excludes DSL, since the primary data was metered consumption.
6. Assume DSL at 10 percent of existing production. Therefore, DSL = 20 MG x 0.1 = 2 MG.
7. Estimate of total annual production requirement under fulltime occupancy = (416 x 365 x 200) + 2 MG = 32 MG

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Chapter 4: Water System Capacity Analysis

4.0 Introduction

Adequate service capacity to reliably meet consumer demands with safe drinking water should be the goal of every water system design. This chapter presents concepts and tools for determining the service capacity of a water system. Service capacity is based on the physical limitations of the water system and legal and contractual limitations such as water rights and intertie agreements.

Design engineers **must** assess the capacity of each system component such as source, treatment, storage, transmission, or distribution, individually and in combination with each other (WAC 246-290-222). The goal is to provide water of adequate quality, quantity, and pressure during minimum supply and maximum demand scenarios.

Chapter 4 relies heavily on information and guidance presented in Chapter 3 where we introduced the concept of the equivalent residential unit (ERU). Capacity evaluations consider how much water the system can reliably produce and how many connections it can reliably serve by that quantity of water, usually expressed in ERUs. While water system capacity is evaluated in ERUs, capacity is recorded on the Water Facilities Inventory form (WFI) as the total number of approved connections. See Attachment A at the end of this chapter for assumptions made in the conversion between excess capacity expressed as ERUs and approved connections.

Utilities are responsible for establishing and maintaining a water budget to monitor remaining service capacity, expressed in either gallons per day or ERUs, so that physical capacity and legal water use restrictions are not exceeded. The design engineer should be able to inform the water system's governing body and system operator how their system's service capacity was determined, identify the limiting factor(s), and provide guidance on appropriate methods to track service capacity as the type and number of connections change over time.

In this chapter, as in Chapter 3, we make a distinction between water systems that are predominantly residential and predominantly non-residential. Assessing physical capacity using ERUs is not applicable to predominantly non-residential systems. As such, this chapter is divided into two parts:

1. Capacity Analysis for Residential Systems – Focuses on capacity analysis for systems in which residential demands comprise a significant portion or all of the demand.
2. Capacity Analysis for Non-Residential Systems – Focuses on capacity analysis for systems in which residential demands comprise an insignificant portion of total demand.

The Drinking Water Operating Permit rules establish criteria to determine water system service adequacy including the maximum number of allowed service connections (Chapter 246-294 WAC). A water system that exceeds its maximum number of allowed service connections will see its operating permit status change to "category blue" (WAC 246-294-040(2)(c)).

Part 1 – Capacity Analysis for Residential Systems

4.1 General Expectations

Design engineers **must** analyze water system service capacity in planning documents (WAC 246-290-100) and in certain project reports and other engineering documents (WAC 246-290-110(4)(f)). The following are examples of when an engineering analysis of physical capacity is needed.

- An existing water system does not have our approval, and seeks a green operating permit.
- An existing water system, without an approved planning document, seeks an increase in the number of approved connections without the need for new source or storage infrastructure.
- Project construction is complete, but the constructed project components differ from the design approved by us.
- A non-expanding water system wishes to provide service to a type of connection not identified in its planning document, or any previous project approvals.

4.2 ERUs, Connections, and Population

4.2.1 ERUs

An ERU is defined as *a system-specific unit of measure used to express the amount of water <u>consumed</u> by a typical full-time single family residence* (WAC 246-290-010). In Chapter 3 we discuss how this unit value of demand is calculated. The physical capacity analysis **must** assess the water system's ability to supply the maximum day demand (MDD) for the entire water system and verify that adequate distribution system pressure can be maintained under peak hourly demand (PHD) and under MDD plus fire flow conditions where fire flow is provided (WAC 246-290-230). Physical capacity determinations for residential systems **must** be reported in ERUs (WAC 246-290-222(2)).

Many water systems serve a mixture of single and multi-family dwellings, commercial and industrial customers, and other uses. The ERU is a tool to translate non-single family residence demand into an equivalent value of demand on the system's infrastructure.

While establishing an appropriate ERU value is important, and water system service capacity must be expressed in ERUs, water system owners and operators generally think in terms of number of connections and are required to report connections on their Water Facilities Inventory form.

4.2.2 Connections Served

Each single-family home, each dwelling unit in a multifamily building, and each nonresidential building the water system serves is considered a connection.

An accessory dwelling unit (ADU) is not always considered a separate dwelling unit. We consider ADUs that are separate structures as additional dwelling units, but if the ADU is physically located within a larger residence, then the ADU is not considered a separate dwelling unit.

The following examples illustrate application of “connection”:

- A system serving eight duplexes and two single-family homes serves a total of eighteen dwelling units. Each dwelling unit is considered a connection.
- A system serving eight single-family homes, each with an accessory dwelling unit incorporated into the main structure of the home, serves a total of eight dwelling units, and therefore eight connections.
- A system serving eight single-family homes, each with an accessory dwelling unit built as a separate structure on the same parcel, serves a total of sixteen dwelling units, and therefore sixteen connections.
- For non-community water systems: Each recreational campsite, RV site, and overnight unit in a hotel or motel is considered a connection. For institutional facilities, commercial businesses, industrial, schools and other non-residential service connections, each building with water service is a connection.
- For community water systems: Each direct service connection to non-residential users such as campgrounds, RV parks, hotels, motels, businesses, and industrial parks.

4.2.3 Population Served

When designing water systems, the population engineers need to consider is the number of people that have *access* to piped water for human consumption. The population served is either residential (people living in a residence), or nonresidential (tourists, customers, employees) entering the premises and given the opportunity to access tap water.

For the purposes of design, 2.5 residents should be assigned to each dwelling unit. If a proposed system serves 10 or more dwelling units, we will review the design under the approval standards for Group A public water systems (chapter 246-290 WAC). For the purpose of design, each residence **must** be considered a full-time residence unless there are formal restrictions established that prevent full-time use (WAC 246-290-221 (2)).

4.3 Applying the Concept of Equivalent Residential Units in Design

Most small water systems serve at least some and often only single-family residences. Single-family residential customers have a typical demand pattern. Nonresidential customers (such as an industry or business) may have demand patterns significantly different from single family homes. See Section 3.6 for more information on estimating nonresidential demands.

Engineers **must** evaluate and design a water system by first translating nonresidential and multifamily consumer demands into an equivalent number of single family residences served by that system. A few simple examples that illustrate how to apply the concept of ERUs in service capacity analyses are given below.

Example 1: Basic ERU Conversion
A water system serves 100 single family homes, a school, and a small business. After analyzing metered consumptive data for full-time occupied single family homes, the ERU_{MDD} value for this system is determined to be 800 gpd. The MDD of the school during the same high demand period is estimated at 8,000 gpd, and the MDD for the business is estimated at 1,600 gpd.

As summarized below, the 102 service connections supplied by this system represent 112 ERUs.

- 100 homes = 100 ERUs
- School = 8,000 gpd/800 gpd per ERU = 10 ERUs
- Business = 1,600 gpd/800 gpd per = 2 ERUs

Example 2: ERUs Associated with Multifamily Residences
A water system serves 200 single family homes and 500 multi-family dwellings. After analyzing metered consumptive data for full-time occupied single family homes, the ERU_{MDD} value for this system is determined to be 700 gpd. This water system also serves 50 connections serving 500 multi-family dwellings. The MDD of the 50 multifamily connections during the same high demand period is estimated at 210,000 gpd.

As summarized below, the 250 service connections supplied by this system represent 500 ERUs.

- 200 homes = 200 ERUs
- Multi-family dwellings = 210,000 gpd/700 gpd per ERU = 300 ERUs

Example 3: ERUs Associated with Nonresidential Customers
A water system serves 1,000 single family homes, 50 small commercial businesses, and a food processing plant. After analyzing metered consumptive data for full-time occupied single family homes, the ERU_{MDD} value for this system is determined to be 600 gpd. The collective MDD of the 50 commercial businesses is estimated at 60,000 gpd, and the food processing plant MDD is estimated at 360,000 gpd during the same high demand period.

As summarized below, the 1,051 service connections supplied by this system represent 1,700 ERUs.

- 1,000 homes = 1,000 ERUs
- Commercial Businesses = 60,000 gpd/600 gpd per ERU = 100 ERUs
- Food Processor = 360,000 gpd/600 gpd per ERU = 600 ERUs

Note: The above examples do not include distribution system leakage (DSL), and therefore they do not include the number of ERUs associated with DSL since the primary demand data is from metered consumption. See Section 3.1 about the importance of factoring DSL into service capacity assessment.

4.4 Methodology to Determine Water System Capacity

The following steps represent an approach for determining a predominantly residential water system's physical capacity. Engineers may propose another approach supported by system-specific information and engineering justification.

4.4.1 Step 1: Water Demands

Design engineers **must** estimate water demands (WAC 246-290-221). For systems serving different customer classes, you should to create separate estimates for residential and nonresidential connections. For existing systems, design engineers should quantify MDD, ADD, ERU_{MDD} and ERU_{ADD} by using actual water consumption records. In designing a new water system water use from an analogous water system or the approach described in Appendix D may be used. PHD should be estimated using Equation 3-1.

4.4.2 Step 2: Source Capacity

All water systems **must** have sufficient source capacity to meet MDD (WAC 246-290-222(4)). If total consumptive demands plus DSL is greater than total permanent and seasonal source capacity, the equations shown later in this chapter should not be used, and the design engineer must perform an alternate analysis. Design engineers proposing to use storage to meet MDD should consult with the regional engineer on specific requirements before pursuing such a design.

In general, the total daily source capacity **must** be able to *reliably* provide sufficient water to meet the MDD for the water system (WAC 246-290-222(4)). If sources are not able to meet or exceed PHD, then equalizing storage **must** be provided to meet diurnal demands that exceed source capacity (WAC 246-290-235(2)). See Chapter 7 for equalizing storage requirements. Sources **must** also meet minimum reliability requirements (WAC 246-290-420). See Chapter 5 for design guidance for sources.

Total system source capacity is based on the capacity of permanent and seasonal sources (WAC 246-290-222). Design engineers **cannot** include emergency sources, including emergency interties, in the total source capacity calculations (WAC 246-290-222(3)).

Design engineers should document the approach used to assess source-based service capacity into the physical capacity analysis and all related assumptions: data acquired to determine full- and part-time residential, nonresidential, and non-revenue water demands; identifying DSL; and the source adequacy and reliability issues described in Section 3.10.

Design engineers should refer to the following information to determine source capacity. The most limiting element establishes the source-based service capacity.

- Water rights
- Pumping tests and groundwater reliability
- Surface water (watershed) reliability
- Installed pump capacity
- Intertie capacity (if used regularly to meet demands)
- Treatment capacity

Each of these source capacity limiting factors is described in more detail below.

4.4.2.1 Water Rights

This section recognizes that water rights have a legal bearing on water system capacity determinations. Engineers must consider this limitation because water rights may limit the annual withdrawal (Qa) and instantaneous withdrawal (Qi) of water that a water system can legally withdraw from drinking water sources. ADD is used in assessing water right limitations associated with Qa. MDD is used in assessing water right limitations associated with Qi.

Although water rights place a regulatory/legal limit rather than a physical limit on the amount of water legally available for service, design engineers **must** address this important issue. Engineers **must** complete a *Water Rights Self-Assessment Form* for all new sources and projects that increase water system physical capacity or the approved number of connections (WAC 246-290-110(4)(e) and 130(4)(a)).

We do not have the authority to evaluate water right documents. Design engineers should contact the Department of Ecology about issues related to water rights prior to submitting a capacity analysis to us. Contact information for Department of Ecology's Water Resources Program can be found in Appendix C.

4.4.2.2 Pumping Tests and Groundwater Reliability

A pumping test analysis determines the capacity of the well(s) to reliably supply the water needed to meet service demands. When evaluating well capacity, the design engineer should consider each of the following:

- Historical pumping records
- Water quality issues
- Pumping Test Procedures (Appendix E). If wells are close in proximity then it may be necessary to analyze the collective withdrawal capacity with multiple wells operating at once, to understand the level of interference between operating wells
- Seawater Intrusion (Section 5.5.4)

Engineers **must** provide their analysis of pump-test results to us (WAC 246-290-130).

4.4.2.3 Surface Water (Watershed) Reliability

Design engineers **must** analyze the reliability of a new surface water supply as part of source approval (WAC 246-290-130(3)(c)). Hydrologic models are often used for this purpose. The determination of the safe yield identifies the volume of water that can be expected during critical dry periods. In planning for drinking water supplies, the safe yield is usually defined as the one-in-50 year or one-in-100 year low flow, or a 98 or 99 percent level of annual water supply reliability (Prasifka 1988, Connecticut DPH 2006). Design engineers should define the expected inadequacy of supply during the return low-flow period, and address the expected water supply deficiency in a water shortage response plan. See Section 3.10.

A hydrologic assessment is an involved process described in more detail in several professional publications (AWWA 2007, Chow et al. 1988, Maidment 1993). In general, the safe yield analysis consists of many elements including:

- Developing a record of the spring or river flow – Multiple years of daily flow records are usually necessary.
- Diversions for other uses.
- Mandatory minimum flows for natural resource protection or other purposes.
- Reservoir storage that can be utilized given other competing demands such as flood control and hydroelectric power generation.
- Precipitation patterns.
- Evaporation and evapotranspiration rates.

Over time, changes in land use patterns, vegetation coverage, climate, and precipitation patterns may necessitate the reevaluation of the safe yield to ensure a reliable and adequate water supply in the future. Water systems **must** also identify how to manage their customer demand as part of an overall water shortage response plan (WAC 246-290-100(4)(f); WAC 246-290-420).

4.4.2.4 Installed Pump Capacity

Unless we approve in advance the use of finished water storage to meet MDD, source pumping capacity **must** be sufficient to meet MDD. Design engineers should consider each of the following when evaluating installed pump capacity:

- Metered source production and water system demand records
- System head conditions when pumping to storage or distribution, and when pumping from wells with significant seasonal changes in dynamic (pumping) water levels
- Pump curve(s)
- Pump controls and logic
- An engineering analysis that verifies pump performance under actual system head conditions

4.4.2.5 Interties

Non-emergency use interties with neighboring approved water systems can provide additional source capacity for the purpose of evaluating source-based service capacity. The engineer should evaluate each non-emergency intertie to determine its limitations. It is important to consider elements such as hydraulic limitations, water quality, and legal restrictions associated with water rights, or conditions on the purchase contract that define service restrictions. Section 5.9 provides further discussion and guidance on interties.

4.4.2.6 Treatment Capacity

A treatment capacity analysis determines whether any installed treatment processes limit the water system's source production capacity. When water treatment such as filtration or blending is applied to one or more sources, the net treatment plant production in combination with other untreated sources **must** be able to reliably supply at least the MDD while meeting all water quality performance requirements (WAC 246-290-222(5)).

Design engineers must translate nominal treatment plant capacity to net treatment plant capacity by deducting water volume and the time devoted to backwash and filter to waste (filtration), and other required limitations on production necessary to maintain treatment efficacy (blending).

4.4.2.7 Determining ERUs based on Source Capacity

The engineer needs to evaluate the capacity of each individual source a water system uses. The overall water system source-capacity is the sum of the reliable production capability from each source, excluding emergency sources.

The amount of water that any source may provide is the product of its delivery rate and the amount of time it is used for service.

Equation 4-1: Individual Source Capacity

$$V_j = (Q_j)(t_j)$$

Where:

V_j = Total volume for source "j" over a 24-hour period (excluding emergency sources)

Q_j = Delivery rate of source (gallons per unit time)

t_j = Time that flow (Q_j) was delivered from source "j" over a 24-hour period. We recommend assessing daily source capacity based on 20 hours of pumping per day (1,200 minutes per day). See Section 3.10.

Engineers should base the design flow-rate (Q_j) for each source on any limiting factor that might restrict the peak-flow rate during maximum demand periods (such as well, stream, or aquifer capacity; installed pumping capacity; intertie capacity; treatment limitations and net treatment production; and/ or legal limitations such as water right limits).

Engineers should base the time (t_j) for each individual source based on the time each respective source can be, or is, used over a 24-hour period during maximum demand periods. For example, a pump may be restricted to operate for only a designated amount of time each day, or a treatment plant may produce water for only certain periods each day.

Note: *To determine maximum source-production capacity, it is clear that pumping for the full 1,440 minutes a day will provide the highest estimate for water system capacity, expressed in ERUs. However, it may not be practical or advisable to operate source pumps continually for 24 hours, even during peak-demand periods.* ***We recommend assessing daily source capacity based on 20 hours (1200 minutes) of pumping in a 24-hour period, to provide a factor of safety to the assessment.***

When engineers know the specific delivery-rate and operation-time of delivery for each source, they can use Equation 4-2 to determine total source capacity (the total quantity of water available over a specified time period from all sources except emergency sources).

Equation 4-2: Total Source Capacity

$$V_T = \sum (Q_j)(tj) = \sum V_j$$

Where:

$\sum$ = Summation

j = Individual source designation, excluding emergency sources

$\mathbf{V}_T$ = Total volume of water delivered from all non-emergency sources over a 24-hour period.

Engineers can use Equation 4-3 determine ERUs based on source capacity, and Equations 4-4a and 4-4b to determine the service capacity based on water right limitations.

Equation 4-3: ERU capacity based on source capacity

$$N = \frac{V_T}{\mathrm{ERU_{MDD}}}$$

Where:

N = Number of ERUs based on the $\mathrm{ERU_{MDD}}$ value

Equation 4-4a: ERU capacity based on water rights (Qi)

$$N = \frac{Qi}{\mathrm{ERU_{MDD}}/1440}$$

Where:

N = Number of ERUs based on the $\mathrm{ERU_{MDD}}$ value

Equation 4-4b: Water system capacity based on water rights (Qa)

$$N = \frac{Qa}{[ERU_{ADD} \text{ x } 365]}$$

Where:

N = Number of ERUs based on the ERU_{ADD} value

Note: *Engineers may summarize their source capacity analysis information on Worksheet 4-1 (at the end of this chapter), or a similar form. Alternately, design engineers may estimate the average daily demand of each demand component separately (single family residential, multi-family, commercial, and so forth) plus DSL.*

4.4.3 Step 3: Capacity Based on Storage

The design engineer **must** consider each of the following storage elements when determining the total storage volume requirement (WAC 236-290-235), whether the element is directly related to system capacity or not:

- **Operational storage (OS).** Adequate OS is important for efficient and reliable operation of sources. The design engineer **must** identify and provide adequate OS (WAC 246-290-222), but OS is not a directly related to system capacity.

- **Equalizing storage (ES).** ES volume is based on PHD demand requirements. See Equation 4-6. ES volume is calculated based on the differential between operational source capacity and PHD multiplied by 150 minutes (2.5 hours). ES is directly related to system capacity.

- **Standby storage (SB).** SB allows a water system to maintain adequate pressure in the event of a mechanical, electrical or water quality issue with a source of supply, pumping, or treatment systems. See Equation 4-7. More detailed recommendations are in Section 7.1.1.3. SB volume is based on consumer expectations. ES and SB storage design calculations are exclusive of one another. SB is directly related to system capacity.

- **Fire suppression storage (FSS), if applicable.** FSS and fire flow rate and duration are established by the local fire authority and, in general, is defined by land use. FSS volume can be partially addressed through development of multiple sources (or multiple pumps in a single source, if applicable – see Section 5.11.2); emergency interties (see Section 5.9.2) and back-up power generation (see Section 5.11.1); and "nesting" SB and FSS (see Section 7.1.1.4). FSS is not directly related to system capacity.

- **Dead storage (DS).** DS is the volume of stored water not available to all consumers at the minimum design pressure (WAC 246-290-230(5) and (6)). DS is not directly related to system capacity.

The water system design **must** satisfy the minimum storage requirements of WAC 246-290-235. Elements that must be considered and used in defining minimum storage requirements include:

- The number of sources
- Source capacity
- ERU_{ADD}, ERU_{MDD}, and peak-hourly demands
- Local fire-suppression requirements
- The level of service and manner used to achieve reliability requirements described in (WAC 246-290-420)
- Adequacy of storage in specific pressure zones.
- Power grid reliability.
- Pressure requirements in the distribution system.

4.4.3.1 ERUs Based on Equalizing Storage

Equalizing storage **must** be available when determined necessary based on available, or designed, source pumping capacity (WAC 246-290-235(2)). Equation 4-5 is the result of combining Equation 3-1 with Equation 7-1, allowing a solution for needed ES given a number of ERUs (N value). Key concepts associated with the use of Equations 4-5 and 4-6:

- Applicable to water systems with significant residential demand.
- N is the number of ERUs supplied by all sources. DSL has an associated number of ERUs (see examples in Section 3.12 and Worksheet 4-1). Therefore, N includes DSL. "N" is the number of connections only if there is no distribution system leakage and all connections are single family homes.
- Check to be sure that ERU_{MDD} times "N" equals total maximum daily source production.
- The ERU value is ERU_{MDD}. It is not appropriate to apply the ERU_{ADD} value.

Equation 4-5:

$$ES = \left[\left(\frac{ERU_{MDD}}{1440}\right)\left[(C)(N)+F\right]+18-Qs\right](150)$$

Where	**C**	=	Coefficient associated with ranges of ERUs (see Section 3.4.2)
	N	=	Number of ERUs based on the ERU_{MDD} value
	F	=	Factor associated with ranges of ERUs (see Section 3.4.2)
	ERU_{MDD}	=	Maximum Day Demand per ERU (gallons per day)

Equation 4-5 is derived from combining these two equations:

Determine PHD (see Section 3.4.2)

PHD = (ERU_{MDD} /1440) [(C)(N) + F] + 18

Where

PHD	=	Peak Hourly Demand, total system (gallons per minute)
C	=	Coefficient Associated with Ranges of ERUs
N	=	Number of ERUs based on the ERU_{MDD} value
F	=	Factor Associated with Ranges of ERUs
ERU_{MDD}	=	Maximum Day Demand per ERU (gallons per day)

Determine ES (see Section 7.1.1.2)

ES = (PHD - Qs)(150 minutes)

Where:

ES	=	Equalizing storage component, in gallons
PHD	=	Peak hourly demand, in gpm
Q_S	=	Sum of all installed and active supply source capacities except emergency supply, in gpm.

If the calculated ES volume is greater than the available ES volume, then there is not sufficient ES for the number of ERUs.

Equation 4-6 allows design engineers to solve for the number of ERUs (N) that can be supplied by a given volume of ES. This approach applies to the most common method for controlling reservoir level, known as a “call-on-demand” system which calls on source(s) at a pre-set reservoir level(s).

Restating Equation 4-5 to solve for N, one can solve for N given available ES:

Equation 4-6:

$$N = \frac{1}{C}\left[\frac{(1440)}{(ERU_{MDD})}\left(\frac{ES}{150} + Qs - 18\right)\right] - F$$

Procedure for solving for N based on available ES (using Equation 4-6):

1. Calculate available ES. *The water system* ***must*** *maintain a 30-psi pressure in the distribution system under PHD when ES is depleted* (WAC 246-290-230(5)).
2. Determine the ERU_{MDD} and Qs.
3. Select the lowest N value calculated from among Equation 4-3 (source capacity) and Equation 4-4 (water rights).
4. From Table 3-1 select the appropriate values for both C and F for the N value determined in Step 3.
5. Use Equation 4-6 and solve for N.
6. If the resulting value of N lies outside the range associated with the C and F values selected in Step 4, repeat the calculation using Equation 4-6 by using a different set of values for C and F. Continue until the value for N lies within the range of ERUs associated with the values for C and F selected. After completing these iterative

calculations, the final value for N is the limiting level of ERUs for the amount of equalizing storage available.

4.4.3.2 ERUs Based on Standby Storage

Standby storage is a volume reserved in a water system's finished water storage system for the purpose of maintaining a certain level of service in the event one or more permanent or seasonal sources of supply becomes partially or completely unavailable for use.

Standby storage **must** be provided in an amount necessary to maintain reliable water service (WAC 246-290-235(3) and WAC 246-290-420). We recommend a baseline volume for SB equal to two times the system-wide ADD, subject to appropriate reductions based on redundant sources and other factors. In order to satisfy WAC 246-290-235 and -420, purveyors should provide a minimum value of 200 gpd per ERU regardless of such factors.

Under certain circumstances design engineers may consider SB nested within FSS. See Chapter 7 for more detail on recommended sizing of the SB storage component, particularly when a water system has multiple sources (or multiple pumps in a single source – see Section 5.11.2), an emergency intertie (see Section 5.9.2), and/or back-up power generation (see Section 5.11.1).

Equation 4-7:

$$N = \frac{SB}{(SB_i)(t_d)}$$

Where:

N = Number of ERUs based on the ERU_{MDD} value

SB = Total volume of water in standby storage component (gallons). See Section 7.1.1.3.

SB_i = Selected volume of standby storage to meet water system-determined standard of reliability in gallons per day per ERU (number of ERUs based on the ERU_{MDD} value)

t_d = Number of days selected to meet water system-determined standard of reliability.

Equation 4-7 can be manipulated to solve for SB to satisfy a given number of ERUs (N) when designing new or expanding water systems.

Engineers may enter tabulated results for Equations 4-6 and 4-7 on Worksheet 4-1, or a similar form.

4.4.4 Step 4: Capacity Based on Distribution Facilities

Design engineers **must** use a hydraulic analysis when evaluating distribution system capacity (WAC 246-290-230(1)). In most cases, a well-documented hydraulic model will be required. For some small, simple systems, manual calculations may be acceptable. Transmission and distribution system design is discussed in greater detail in Chapter 6.

Distribution systems are typically evaluated under two conditions:

- Peak hour demand (PHD) (WAC 246-290-230(5))
- Maximum day demand (MDD) plus fire flow, if applicable (WAC 246-290-230(6))

The hydraulic analysis is then used to determine if distribution system components are adequately sized to provide residual pressure at the customer meter or property line according to the water system's adopted standards, or the following minimum residual pressures, whichever is greater:

- At least 30 psi for new water systems or additions to existing water systems under PHD conditions (WAC 246-290-230(5))
- At the approved design pressure, but not less than 20 psi, under PHD for existing systems (WAC 246-290-420(2))
- At least 20 psi under demands that include MDD plus fire flow (WAC 246-290-230(6).

4.4.4.1 ERUs Based on Maintaining Adequate Residual Pressure Under PHD Conditions

A water system **must** be able to *reliably* provide PHD and (if applicable) maximum day demand plus needed fire flow while maintaining compliance with minimum pressure requirements throughout the distribution system (WAC 246-290-230(5)).

Distribution adequacy is determined on a pressure zone basis. ***The physical capacity of the distribution system is based solely on the ability of the water system to deliver the PHD while maintaining 30 psi or the approved design pressure throughout the system.*** When physical capacity limitations exist within a specific part of the distribution system, and until a solution is put in place, design engineers should:

- Ensure hydraulic deficiencies in PHD capacity are identified and reported to the water system and local planning and building department
- Prioritize capital improvements to address deficiencies in delivering PHD within a reasonable period of time
- Assist the water system in identifying and documenting operational steps that mitigate hydraulic deficiencies so that at no time at any service connection will the distribution system experience pressure less than 20 psi during PHD conditions

4.4.4.2 Fire Flow Impacts on Capacity

While distribution system facilities are frequently sized to meet fire flow demands, the number of connections that can be served by a water system is independent of fire flow demands. Therefore, inadequate fire flow capacity does not limit the number of ERUs or connections that can be served by a water system. Decisions on whether to restrict development based upon fire flow capacity are up to the local fire authority. As part of the design analysis, you should:

- Ensure hydraulic deficiencies in fire flow capacity are clearly identified and reported to the local fire authority.
- Prioritize capital improvements to meet potential fire flow demands within a reasonable period of time, and coordinate these improvements with the local fire authority.

- Assist the water system and local fire authority to prepare operational plans that reflect hydraulic deficiencies in providing fire flow.
- Distribution systems **must** be designed so that at least 20 psi can be maintained throughout the distribution system at all times under fire flow conditions (WAC 246-290-230(6)).

4.4.5 Step 5: Factoring DSL into Capacity Analysis

Distribution system leakage (see Section 3.8) exerts as real a demand on sources, storage, and distribution systems as actual customers. Evaluating water system capacity without considering the productive and distributive requirements associated with DSL will overstate water system capacity.

There are several ways DSL may be applied to the capacity analysis of a water system:

- Express DSL as a separate demand on the water system, and express DSL in terms of ERUs based on ERU_{MDD} (see Section 3.12 examples and Chapter 4 Attachment A). We recommend this option. ***It offers the advantage of making clear the number of ERUs unavailable to the water system because of DSL***, and so provides the basis to compare the cost and effort involved in increasing service capacity by (1) increasing infrastructure capacity; (2) decreasing consumer demands (ERU_{MDD} and ERU_{ADD}); and (3) reducing DSL.

- Spread out DSL equally among all customers on an ERU basis. Design engineers may take this approach if the existing system serves only (or almost only) full time single family homes. This approach inserts DSL into the derived ERU_{MDD} and ERU_{ADD} values.

4.4.6 Step 6: Determine Limiting Criteria and Water System Service Capacity in ERUs Capable of Being Served

Attachment A provides an example of determining system capacity for a mixed use, predominantly residential water system. The example presumes defined values for ERU_{ADD}, ERU_{MDD}, system-wide ADD and MDD, and PHD. Refer to Chapter 3 for guidance on determining water system demand values.

We recommend that design engineers summarize the element(s) that limit the total water system capacity as well as system capacity based on each system element (e.g., source production, water rights, storage, pumping). This information would inform a water system of the degree and sequence each element must be addressed to maintain water system capacity in support of their growing communities.

Part 2 – Capacity Analysis for Non-Residential Systems

Non-residential water systems do not lend themselves to analysis using ERUs as a common measure of demand. We recommend design engineers analyze non-residential water systems by identifying the composite ADD, MDD, and PHD for each customer or customer class. Section 3.6 provides guidance on determining water demand for non-residential systems.

4.5 Methodology to Determine Water System Capacity

The following steps represent an approach for determining a predominantly nonresidential water system's physical capacity. Engineers may propose another approach supported by system-specific information and engineering justification.

4.5.1 Step 1: Water Demands

Design engineers **must** estimate water demands (WAC 246-290-221). For systems serving different customer classes, you should create separate estimates for residential and nonresidential connections. For existing systems, design engineers should quantify MDD and ADD using actual water consumption records. In designing a new water system water use from an analogous water system or the approach described in Appendix D may be used. PHD should be estimated using Equation 3-1.

4.5.2 Step 2: Source Capacity

All water systems **must** have sufficient source capacity to meet MDD (WAC 246-290-222(4)). If total consumptive demands plus DSL is greater than total permanent and seasonal source capacity, the equations shown later in this chapter should not be used, and the design engineer must perform an alternate analysis. Design engineers proposing to use storage to meet MDD should consult with the regional engineer on specific requirements before pursuing such a design.

In general, the total daily source capacity **must** be able to *reliably* provide sufficient water to meet the MDD for the water system (WAC 246-290-222(4)). If sources are not able to meet or exceed PHD, then equalizing storage **must** be provided to meet diurnal demands that exceed source capacity (WAC 246-290-235(2)). See Chapter 7 for equalizing storage requirements. Sources **must** also meet minimum reliability requirements (WAC 246-290-420). See Chapter 5 for design guidance for sources.

Total system source capacity is based on the capacity of permanent and seasonal sources (WAC 246-290-222). Design engineers **cannot** include emergency sources, including emergency interties, in the total source capacity calculations (WAC 246-290-222(3)).

Design engineers should document the approach used to assess source-based service capacity into the physical capacity analysis and all related assumptions: data acquired to determine full- and part-time residential, nonresidential, and non-revenue water demands; identifying DSL; and the source adequacy and reliability issues described in Section 3.10.

Design engineers should refer to the following information to determine source capacity. The most limiting element establishes the source-based service capacity.

- Water rights
- Pumping tests and groundwater reliability
- Surface water (watershed) reliability
- Installed pump capacity
- Intertie capacity (if used regularly to meet demands)
- Treatment capacity

Each of these source capacity limiting factors is described in more detail below.

4.5.2.1 Water Rights

This section recognizes that water rights have a legal bearing on water system capacity determinations. Engineers must consider this limitation because water rights may limit the annual withdrawal (Qa) and instantaneous withdrawal (Qi) of water that a water system can legally withdraw from drinking water sources. ADD is used in assessing water right limitations associated with Qa. MDD is used in assessing water right limitations associated with Qi.

Although water rights place a regulatory/legal limit rather than a physical limit on the amount of water legally available for service, design engineers **must** address this important issue. Engineers **must** complete a *Water Rights Self-Assessment Form* for all new sources and projects that increase water system physical capacity or the approved number of connections (WAC 246-290-110(4)(e) and 130(4)(a)).

We do not have the authority to evaluate water right documents. Design engineers should contact the Department of Ecology about issues related to water rights prior to submitting a capacity analysis to us.

4.5.2.2 Pumping Tests and Groundwater Reliability

A pumping test analysis determines the capacity of the well(s) to reliably supply the water needed to meet service demands. When evaluating well capacity, the design engineer should consider each of the following:

- Historical pumping records
- Water quality issues
- Pumping Test Procedures (Appendix E). If wells are close in proximity then it may be necessary to analyze the collective withdrawal capacity with multiple wells operating at once, to understand the level of interference between operating wells
- Seawater Intrusion (Section 5.5.4)

Engineers **must** provide their analysis of pump-test results to us (WAC 246-290-130).

4.5.2.3 Surface Water (Watershed) Reliability

Design engineers **must** analyze the reliability of a new surface water supply as part of source approval (WAC 246-290-130(3)(c)). Hydrologic models are often used for this purpose. The determination of the safe yield identifies the volume of water that can be expected during critical dry periods. In planning for drinking water supplies, the safe yield is usually defined as the one-in-50 year or one-in-100 year low flow, or a 98 or 99 percent level of annual water supply reliability (Prasifka 1988, Connecticut DPH 2006). Design engineers should define the expected inadequacy of supply during the return low-flow period, and address the expected water supply deficiency in a water shortage response plan. See Section 3.10.2.

A hydrologic assessment is an involved process described in more detail in several professional publications (AWWA 2007, Chow et al. 1988, Maidment 1993). In general, the safe yield analysis consists of many elements including:

- Developing a record of the spring or river flow – Multiple years of daily flow records are usually necessary.
- Diversions for other uses.
- Mandatory minimum flows for natural resource protection or other purposes.
- Reservoir storage that can be utilized given other competing demands such as flood control and hydroelectric power generation.
- Precipitation patterns.
- Evaporation and evapotranspiration rates.

Over time, changes in land use patterns, vegetation coverage, climate, and precipitation patterns may necessitate the reevaluation of the safe yield to ensure a reliable and adequate water supply in the future. Water systems **must** also identify how to manage their customer demand as part of an overall water shortage response plan (WAC 246-290-100(4)(f); WAC 246-290-420).

4.5.2.4 Installed Pump Capacity

Unless we approve in advance the use of finished water storage to meet MDD, source pumping capacity **must** be sufficient to meet MDD. Design engineers should consider each of the following when evaluating installed pump capacity:

- Metered source production and water system demand records
- System head conditions when pumping to storage or distribution, and when pumping from wells with significant seasonal changes in dynamic (pumping) water levels
- Pump curve(s)
- Pump controls and logic
- An engineering analysis that verifies pump performance under actual system head conditions

4.5.2.5 Interties

Non-emergency use interties with neighboring approved water systems can provide additional source capacity for the purpose of evaluating source-based service capacity. The engineer should evaluate each non-emergency intertie to determine its limitations. It is important to consider elements such as hydraulic limitations, water quality, and legal restrictions associated with water rights, or conditions on the purchase contract that define service restrictions. Section 5.9 provides further discussion and guidance on interties.

4.5.2.6 Treatment Capacity

A treatment capacity analysis determines whether any installed treatment processes limit the water system's source production capacity. When water treatment such as filtration or blending is applied to one or more sources, the net treatment plant production in combination with other untreated sources **must** be able to reliably supply at least the MDD while meeting all water quality performance requirements (WAC 246-290-222(5)).

Design engineers must translate nominal treatment plant capacity to net treatment plant capacity by deducting water volume and the time devoted to backwash and filter to waste (filtration), and other required limitations on production necessary to maintain treatment efficacy (blending).

4.5.2.7 Determining Capacity based on Source of Supply

The engineer needs to evaluate the capacity of each individual source a water system uses. The overall water system source-capacity is the sum of the reliable production capability from each source, excluding emergency sources.

The amount of water that any source may provide is the product of its delivery rate and the amount of time it is used for service.

Equation 4-1: Individual Source Capacity

$$V_j = (Q_j)(t_j)$$

Where:

V_j = Total volume for source "j" over a 24-hour period (excluding emergency sources)

Q_j = Delivery rate of source (gallons per unit time)

t_j = Time that flow (Q_j) was delivered from source "j" over a 24-hour period. We recommend assessing daily source capacity based on 20 hours of pumping per day (1,200 minutes per day). See Section 3.10.

Engineers should base the design flow-rate (Q_j) for each source on any limiting factor that might restrict the peak-flow rate during maximum demand periods (such as well, stream, or aquifer capacity; installed pumping capacity; intertie capacity; treatment limitations and net treatment production; and/ or legal limitations such as water right limits).

Engineers should base the time (t_j) for each individual source based on the time each respective source can be, or is, used over a 24-hour period during maximum demand periods. For example, a pump may be restricted to operate for only a designated amount of time each day, or a treatment plant may produce water for only certain periods each day.

Note: *To determine maximum source-production capacity, it is clear that pumping for the full 1,440 minutes a day will provide the highest estimate for water system capacity. However, it may not be practical or advisable to operate source pumps continually for 24 hours, even during peak-demand periods.* ***We recommend assessing daily source capacity based on 20 hours (1200 minutes) of pumping in a 24-hour period, to provide a factor of safety to the assessment.***

When engineers know the specific delivery-rate and operation-time of delivery for each source, they can use Equation 4-2 to determine total source capacity (the total quantity of water available over a specified time period from all sources except emergency sources).

Equation 4-2: Total Source Capacity

$$V_T = \sum (Q_j)(tj) = \sum V_j$$

Where:

$\sum$	=	Summation
j	=	Individual source designation, excluding emergency sources
$\mathbf{V}_{\boldsymbol{T}}$	=	Total volume of water delivered from all non-emergency sources over a 24-hour period.

4.5.3 Step 3: Capacity Based on Storage

The design engineer **must** consider each of the following storage elements when determining the total storage volume requirement (WAC 236-290-235), whether the element is directly related to system capacity or not:

- **Operational storage (OS).** Adequate OS is important for efficient and reliable operation of sources. The design engineer **must** identify and provide adequate OS (WAC 246-290-222), but OS is not a directly related to system capacity.

- **Equalizing storage (ES).** ES volume is based on PHD demand requirements. See Equation 4-6. ES volume is calculated based on the differential between operational source capacity and PHD multiplied by 150 minutes (2.5 hours). ES is directly related to system capacity.

- **Standby storage (SB).** SB allows a water system to maintain adequate pressure in the event of a mechanical, electrical or water quality issue with a source of supply, pumping, or treatment systems. See Equation 4-7. More detailed recommendations are in Section 7.1.1.3. SB volume is based on consumer expectations. ES and SB storage design calculations are exclusive of one another. SB is directly related to system capacity.

- **Fire suppression storage (FSS), if applicable.** FSS and fire flow rate and duration are established by the local fire authority and, in general, are defined by land use. . FSS volume can be partially addressed through development of multiple sources (or multiple pumps in a single source, if applicable – see Section 5.11.2); emergency interties (see Section 5.9.2), and back-up power generation (see Section 5.11.1); and "nesting" SB and FSS (see Section 7.1.1.4). FSS is not directly related to system capacity.

- **Dead storage (DS).** DS is the volume of stored water not available to all consumers at the minimum design pressure (WAC 246-290-230(5) and (6)). DS is not directly related to system capacity.

The water system design **must** satisfy the minimum storage requirements of WAC 246-290-235. Elements that must be considered and used in defining minimum storage requirements include:

- The number of sources
- Source capacity
- MDD, ADD, and PHD
- Local fire-suppression requirements
- The level of service and manner used to achieve reliability requirements described in (WAC 246-290-420)
- Adequacy of storage in specific pressure zones.
- Power grid reliability.
- Pressure requirements in the distribution system.

4.5.3.1 Equalizing Storage

Equalizing storage **must** be available when determined necessary based on available, or designed, source pumping capacity (WAC 246-290-235(2)).

4.5.3.2 Standby Storage

Standby storage is a volume reserved in a water system's finished water storage system for the purpose of maintaining a certain level of service in the event one or more permanent or seasonal sources of supply becomes partially or completely unavailable for use. See Section 7.1.1.3 for recommendations for SB for certain types of nonresidential water systems.

4.5.4 Step 4: Capacity Based on Distribution Facilities

Design engineers **must** use a hydraulic analysis when evaluating distribution system capacity (WAC 246-290-230(1)). In most cases, a well-documented hydraulic model will be required. For some small, simple systems, manual calculations may be acceptable. Transmission and distribution system design is discussed in greater detail in Chapter 6.

Distribution systems are typically evaluated under two conditions:

- Peak hour demand (PHD) (WAC 246-290-230(5))

- Maximum day demand (MDD) plus fire flow, if applicable (WAC 246-290-230(6))

The hydraulic analysis is then used to determine if distribution system components are adequately sized to provide residual pressure at the customer meter or property line according to the water system's adopted standards, or the following minimum residual pressures, whichever is greater:

- At least 30 psi for new water systems or additions to existing water systems under PHD conditions (WAC 246-290-230(5))
- At the approved design pressure, but not less than 20 psi, under PHD for existing systems (WAC 246-290-420(2))
- At least 20 psi under demands that include MDD plus fire flow (WAC 246-290-230(6).

4.5.4.1 PHD Conditions

A water system **must** be able to *reliably* provide PHD and (if applicable) maximum day demand plus needed fire flow while maintaining compliance with minimum pressure requirements throughout the distribution system (WAC 246-290-230(5)).

Distribution adequacy is determined on a pressure zone basis. ***The physical capacity of the distribution system is based solely on the ability of the water system to deliver the PHD while maintaining 30 psi or the approved design pressure throughout the system.*** When physical capacity limitations exist within a specific part of the distribution system, and until a solution is put in place, design engineers should:

- Ensure hydraulic deficiencies in PHD capacity are identified and reported to the water system and local planning and building department
- Prioritize capital improvements to address deficiencies in delivering PHD within a reasonable period of time
- Assist the water system in identifying and documenting operational steps that mitigate hydraulic deficiencies so that at no time at any service connection will the distribution system experience pressure less than 20 psi during PHD conditions

4.5.4.2 Fire Flow Conditions

Design engineers must consider fire conditions, if applicable (WAC 246-290-230). Decisions on whether to restrict development based upon fire flow capacity are up to the local fire authority. As part of the design analysis, you should:

- Ensure hydraulic deficiencies in fire flow capacity are clearly identified and reported to the local fire authority.
- Prioritize capital improvements to meet potential fire flow demands within a reasonable period of time, and coordinate these improvements with the local fire authority.
- Assist the water system and local fire authority to prepare operational plans that reflect hydraulic deficiencies in providing fire flow.
- Distribution systems **must** be designed so that at least 20 psi can be maintained throughout the distribution system at all times under fire flow conditions (WAC 246-290-230(6)).

4.5.5 Step 5: Factoring DSL into Capacity Analysis

Distribution system leakage (see Section 3.8) exerts as real a demand on sources, storage, and distribution systems as actual customers. Evaluating water system capacity without considering the productive and distributive requirements associated with DSL will overstate water system capacity. DSL must be included in estimating system-wide ADD and MDD.

4.6 Documenting Nonresidential Water System Capacity

The capacity of non-residential systems should reflect existing customer demands (if any), and assumptions about future customers and their demand pattern. Capacity should be expressed as the ability to reliably supply MDD and PHD to a specific set of buildings and population – existing or planned - supplied by the non-residential system. We will hold the nonresidential water system owner accountable for operating within the approved capacity of the nonresidential system as documented in the approved design documents.

4.7 Examples of Nonresidential Water System Capacity Analysis

The following examples are provided to illustrate the concepts presented in this chapter. They are not meant to represent a recipe on how to complete a capacity assessment of a new, expanding, or existing non-expanding nonresidential water system.

Example 1 – Port District (expanding)

Given:

An existing water system, owned by a Port District, serves one industrial and four commercial customers. The system is approved to serve a total of five non-residential connections. The system is currently considered “built-out”.

The system is supplied by a single deep well turbine pump. The total source capacity is 6.0 MGD, based on 20 hours of pumping and pumping capacity of 5,000 gpm. Flow from the deep well turbine pump is modulated by a variable frequency drive to match system demand. The Port’s water right allows groundwater withdrawal of 5,000 gpm and 4,500 ac-ft per year. There is no storage. Distribution system piping is looped 12- and 16-inch ductile iron pipe. There is no treatment. The system supplies one industrial connection and four commercial connections. Water demand information is summarized in the table below:

User	ADD	MDD	PHD	Source of information
All	2.1	3.6	3,900	ADD and MDD taken from source meter readings. Therefore it includes DSL. Includes factor of safety.
Industrial	1.7	3.0	3,500	PHD based on information taken from a data logger installed at the industrial connection service meter – includes factor of safety
All Commercial	0.4	0.6	400	PHD estimated. Includes a factor of safety

The Port District receives notice from the industrial customer the facility's proposed process changes will increase its ADD by 0.2 MGD, MDD by 0.5 MGD, and PHD by 500 gpm. The Port agrees to amend its contract with the industrial customer.

At about the same time, the Port District Commissioners seize an opportunity to purchase adjacent land with the intention to increase the size of the service area and number of customers. The Port is willing to construct storage to improve reliability and provide fire suppression, but will not construct a new source of supply. The District's budget for water system improvements is $3.5 million. The purchased land may support a maximum up to new 10 industrial and commercial building sites.

The District intends to market the new industrial and commercial sites based on the availability of "an abundant water supply". The Commissioners want to maximize the number of new sites, and to promise prospective leasees a minimum MDD and PHD for each new site in keeping with the "abundance" marketing theme.

The local Fire Marshall determined that fire flow requirements are 3,000 gpm for 3 hours, and that fire suppression storage may be nested with standby storage.

Find:

The number of sites that can be developed and the volume/flow of water should be marketed and reserved for each new site.

Solution:

1. Revised ADD, MDD, and PHD of existing system after industrial customer changes:
    a. System ADD = 2.1 + 0.2 = 2.3 MGD
    b. System MDD = 3.6 + 0.5 = 4.1 MGD
    c. System PHD = 3,500 + 400 + 500 = 4,400 gpm
2. Remaining available water rights, ADD, and MDD:
    a. 5,000 – 4,400 = 600 gpm (Qi)
    b. [4,500 af/yr x 326,000 gallons per af] – 2.3 MGD x 365 = 628 MG per year (Qa)
    c. Remaining available ADD: 628 MG ÷ 365 = 1.7 MGD
    d. Remaining available MDD (20 hours pumping): 6.0 – 4.1 = 1.9 MGD
    e. Remaining available MDD (24 hours pumping): 7.2 – 4.1 = 3.1 MGD
3. Preliminary design and engineering cost estimate indicate a 0.9 MG elevated reservoir is the maximum size that can be designed and constructed for less than $2.5 million. The remaining $1 million is reserved for construction of new ductile iron water mains to serve the new industrial and commercial sites.
4. The following table explores options for new building site use and water supply allocation based on proposed commercial and industrial water supply contracts:

Option	Commercial			Industrial			Total		
	ADD	MDD	PHD	ADD	MDD	PHD	ADD	MDD	PHD
10 Commercial sites	1.0	1.6	1000	0	0	0	1.0	1.6	1000
1 Industrial & 8 Comm	0.8	1.3	800	0.7	1.2	2000	1.5	2.5	2800
2 Industrial & 3 Comm	0.3	0.5	300	1.4	2.4	4000	1.7	2.9	4300

5. Operational storage requirements are minimal, given the variable frequency drive on the source pump. Allocate 0.05 MG to OS. Remaining storage available for equalizing, standby, and fire suppression storage.
6. The following table summarizes the capacity evaluation for each option:

Option	Source Capacity	Water Right	Storage
10 Commercial sites	Adequate to supply MDD	Qa and Qi adequate	OS = 0.05 MG Fire flow = 0.54 MG ES = [(4,400 + 1,000) – 5,000] x 150 = 0.06 MG SB = Not promised in service contracts Storage needed: 0.65 MG
1 Industrial & 8 Comm	Adequate to supply MDD if pumped 22 hours per day	Qa and Qi adequate	OS = 0.05 MG Fire flow = 0.54 MG ES = [(4,400 + 2,800) – 5,000] x 150 = 0.33 MG SB = Not promised in service contracts Storage needed: 0.92 MG
2 Industrial & 3 Comm	Adequate to supply MDD if pumped 23 hours per day	Qa and Qi adequate	OS = 0.05 MG Fire flow = 0.54 MG ES = [(4,400 + 4,300) – 5,000] x 150 = 0.55 MG SB = Not promised in service contracts Storage needed: 1.15 MG

7. For each option, the existing and proposed 12- and 16-inch looped distribution system is not a limiting factor. At the bottom of ES during PHD pressure throughout the system was modeled at greater than 30 psi; at the bottom of fire suppression storage during MDD plus needed fire flow pressure throughout the system as modeled at greater than 20 psi.

8. Conclusions: The Port Commissioners may:
    a. Select the first option (10 new commercial connections), which leaves a cushion in supply and storage in the event customers exceed their allotted MDD and PHD water supply.
    b. Select the second option (1 new industrial and 8 new commercial connections), but this leaves no cushion in supply or storage, and assumes a 22-hour per day pumping duty to meet MDD. It would also require concurrence from the local Fire Marshall that the fire suppression volume in storage is slightly less than the full 3,000 gpm for a full three hours.

c. Not select the third option (2 industrial and 3 commercial connections) without changing the assumptions about contracted MDD and PHD for each customer, or increasing the budget to construct a larger reservoir.

9. To illustrate, assume the Port selects the second option. In that case, our approval letter will specifically reference the following information:
   a. Approved for 12 nonresidential connections allocated up to 100 gpm and 100,000 gpd each, plus 2 nonresidential connections allocated up to 4,000 gpm/3.5 MGD and 2,000 gpm/2.5 MGD, respectively.
   b. The Port District is responsible for operating within the approved design parameters.

<u>Example 2 – Youth Camp</u> (expanding and increasing use)

<u>Given:</u>

An existing youth camp supplied by:

- A single well producing 18 gpm with greensand filtration for iron/manganese removal that is 90% efficient
- A 20,000 gallon ground level storage tank
- A booster pump station consisting of two - 40 gpm pumps that can in parallel deliver about 70 gpm, but only at 25 psi at the recreation center (north end of system) and camp director’s home (south end of system) under peak demand

The camp water system:

- Was originally approved in 1969 for 15 cabins and the dining hall, on the basis of providing 20 psi during PHD. There have been no approvals since 1969.
- Currently has a “Blue” operating permit, since it is operating outside its design approval
- Consists of 2- and 3-inch PVC water mains and no service meters
- Is not required to provide fire suppression (per the local fire authority)
- Serves a dining hall, recreation center, lakeside bath house, individual homes for the camp director and the chief maintenance superintendent, common toilet and shower building, and 34 primitive cabins without water service, for a total of 40 structures
- Irrigates a ball field

The camp itself:

- Has a maximum approved occupancy, by local permit, of 220 persons
- Operates from late May until early September
- Holds a water right for 4 ac ft per year (Qa) and 20 gpm (Qi)

Existing water system demand information:

- Estimated MDD is 15,000 gpd, including DSL and a factor of safety, based on the last 3 years of source production information
- Estimated PHD is 40-50 gpm

- Annual source production is 0.75 MG, including a factor of safety, based on the last 3 years of source production information

Find:

The Board of Directors want to expand the period of operation, from early April until late October, to cater to other visitor groups on weekends during non-summer months. The camp also wants to increase the approved occupancy to 300, inclusive of campers, administrators, and camp counselors.

Determine whether the existing water system has the capacity to support the Directors' vision, and if not identify the improvements must be made.

Solution:

1. Source/treatment capacity is 18 gpm x 20 hours per day x 0.9 = 19,000 gpd
2. Remaining available water rights, MDD, and PHD:
    a. Remaining available Qa: [4 af x 326,000 gallons per af] – 0.75 MG = 0.55 MG
    b. Remaining available Qi: 20 gpm – 18 gpm = 2 gpm
    c. Remaining available MDD: 19,000 – 15,000 = 4,000 gpd
    d. Remaining available PHD: 70 gpm – 50 gpm = 20 gpm
3. Each additional overnight camper/counselor: 50 gpd per camper (Table 3.2)
4. 4,000 gpd excess MDD ÷ 50 gpd per camper = 80 new campers
5. 80 new campers will require construction of 10 new rustic cabins, conversion of the recreation center into a dual use dining and recreation facility, and construction of a second toilet and shower building, comprising a total 40 (existing) + 10 + 1 = 51 structures – 49 nonresidential and 2 residential.
6. PHD estimated at (19,000 ÷ 1440) x 4.5 = 59 gpm (eq. 3-2). Use 60 gpm.
    a. Existing booster pump capacity (in gpm) is considered sufficient
7. Storage analysis of expanded systems:
    a. OS = 20 minutes of source off to allow for filter backwash and well pump "off" cycle for pump protection = 20 minutes x 60 gpm = 1,200 gallons
    b. ES required = [60 gpm – (18 x 0.9)] x 150 = 6,600 gallons
    c. DS = 12 inches of tank bottom = 2,000 gallons
    d. Remaining SB available: 20,000 – 1,200 – 6,600 – 2,000 = 10,200 gallons. This provides for 35 gallons per camp occupant in the event the well pump or treatment process is temporarily removed from of service.
8. Annual withdrawal will increase by:
    a. 80 additional summer occupants x [(50 gpd per occupant) ÷ 0.9)] x 100 days = 0.44 MG
    b. 0.55 – 0.44 = 0.11 MG remains available for group weekend retreats in the spring and fall. Assume 16 weekends annually = 35 days per year total.
    c. 110,000 gallons = X occupants x [(50 gpd per occupant) ÷ 0.9)] x 35 days; X = 60 occupants, or about 1 - 2 occupants per cabin.
9. The two booster pumps must be replaced in order to deliver a PHD of at least 60 gpm and maintain at least 30 psi throughout the distribution system.
10. Our approval letter will specifically reference the following information:

a. Approval to serve 49 nonresidential and 2 residential structures, with a maximum occupancy of 300 persons
b. The camp is responsible for operating within the approved design parameters: Not to exceed 300 occupants during the summer camping season; not to exceed 50 occupants during the spring and fall; and managing supply requirements so it does not exceed 19,000 gpd during any time of year.

Example 3 – Concert and Catering Venue (expanding and increasing use)

Given:

A winery with a food service permit. The winery is supplied potable water by:

- A permit-exempt well with a rated capacity of 15 gpm at 100 ft total dynamic head (no water right and no source meter)
- A 5,000 gallon ground-level storage tank
- A variable frequency drive booster pump with a rated maximum capacity of 40 gpm at 30 psi (no meter on booster pump)

The winery potable water system supplies:

- A single family home
- A combined retail sales building and tasting room with approved occupancy up to 50 people
- Three production buildings with drinking fountains and wash sinks for 8 employees
- Picnic area (irrigated by non-potable irrigation system) with restroom, covered seating, and food prep sink facilities for up to 20 guests.
- A total of 6 nonresidential connections and 1 residential connection

Existing water system demand information:

- No measured production or consumption data available.
- Estimating MDD: Including the single family home, sales and tasting room based on maximum occupancy and two groups per day, employees, and picnic restroom/food prep results in an estimated MDD = 2,000 gpd. See Table 3.2.

The winery owner seeks to expand operations by developing an outdoor concert venue, a full-service RV park, and expanding food preparation from only pre-packaged items to also include hot and cold hors d'oeuvres.

Find:

The number of RV sites can be developed, and the total occupancy that can be accommodated at the new concert venue based on existing water system facilities.

Solution:

1. Remaining available maximum daily well withdrawal: 5,000 – 2,000 = 3,000 gallons

2. Using the fixture method, current PHD is estimated at 24 gpm
3. Remaining booster pumping capacity = 40-24 = 16 gpm
4. Storage analysis:
    a. OS is minimal. Use 400 gallons.
    b. DS is bottom 9 inches of reservoir = 600 gallons
    c. Remaining volume (4,000 gallons) allocated to ES. Maximum PHD supported by 4,000 gallons of ES. ES = 4,000 = (PHD – 15) x 150. PHD = 41 gpm.
    d. The owner decides no SB is necessary. The winery and concert venue would shut down if the well or booster pump fails.
5. Demand of proposed new uses:
    a. Each RV site is assumed to consume 140 gpd.
    b. Each concert goer/food patron is assumed to consume 10 gpd.
6. Remaining MDD = 3,000 = (RV x 140) + (Patron x 10). Possible solutions:
    a. 10 RVs and 150 patrons
    b. 15 RVs and 90 patrons
7. PHD, assuming the owner selects 10 RV sites and up to 160 additional patrons for concerts.
    a. Using Figure 3.1, Equation 3.2, and Table 3.2, the PHD for the 10 RV sites is estimated at 15 gpm. Total PHD = 24 + 15 = 39 gpm
8. Continue to assume the owner selects the first option. In that case, our approval letter will specifically reference the following information:
    a. Approved for 16 nonresidential connections and one residential connection, with a maximum occupancy at any one time of 200 visitors
    b. The winery owner is responsible for operating within the approved design parameters: Not to exceed 200 visitors (total) at any time; and managing supply requirements so it does not exceed 5,000 gpd (total).

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WORKSHEET 4-1
ERU Capacity Summary

Specific Single-Family Residential Connection Criteria (measured or estimated demands)

Average Day Demand (ADD): __________________ gpd/ERU

Maximum Day Demand (MDD) __________________ gpd/ERU

Water System Service Connections correlated to ERUs			
Service Classification	**Total MDD for the classification, gpd**	**Total # Connections in the classification**	**ERUs**
Residential			
Single-family			
Multifamily			
Nonresidential			
Industrial			
Commercial			
Governmental			
Agricultural			
Recreational			
Other (specify)			
DSL		N/A	
Other (identify)			
Total existing ERUs (Residential + Nonresidential + DSL + Other) =			______________

Service Capacity as ERUs and Gallons Per Day		
Water System Component (Facility)	**ERU Capacity for Each Component**	**GPD Capacity for Each Component**
Source(s)		
Treatment		
Equalizing Storage		
Standby Storage		
Transmission		
Water Rights (Qa and Qi)		
Other (specify)		
Water System Service Capacity (ERUs) = **(based on the limiting water system component shown above)**		

Notes:

1. *Capacity determinations are only for existing facilities that are operational for the water system.*
2. *Distribution system limitations (Section 4.5.4) on ERUs are not shown above because these are location-specific within the distribution system. These limits not expected to limit the ERU capacity of the entire water system.*

Attachment A: Documenting Capacity Evaluation (Example)

Below is an example of how a water system or consultant can document the results of a water system's service capacity evaluation. See Section 3.12 for examples of how these values are derived.

Average Day Demand ERU_{ADD} 425 gpd/ERU
Maximum Day Demand ERU_{MDD} 725 gpd/ERU

Water System Service Connections Correlated to ERUs			
Service Classification	**Total for the classification, gpd**	**Total # Connections in the classification**	**ERUs**
Residential			
Single-family	101,000 (MDD)	139	139
Multifamily	5,000 (MDD)	10	7
Non-residential			
Industrial	150,000 (MDD)	1	207
Commercial	20,000 (MDD)	3	28
Governmental	10,000 (MDD)	1	14
Agricultural	N/A	N/A	
Recreational	5,000 (MDD)	2	7
Other (specify)	N/A		
DSL	35,000 gpd	N/A	48
Totals	326,000 gpd	156 connections	450 ERUs

Service Capacity as ERUs and Gallons Per Day		
Water System Component (Facility)	**ERU Capacity for Each Component**	**Gallons/GPD Capacity for Each Component**
Source(s)	496	360,000 gpd
Treatment	No treatment provided	No treatment provided
Equalizing Storage	501	53,000 gallons
Standby Storage	600 Local standard = 300 gal/ERU	180,000 gallons
Transmission	>600	Determined ample
Water Rights (Qa and Qi)	705 for Qa 496 for Qi	330 ac-ft per year (0.3 MGD) 250 gpm (0.36 MGD)
Water System Service Capacity (ERUs) = 496 **(based on the limiting water system component shown above)**		

In this example, the water system demonstrates an excess capacity of 46 ERUs (496 – 450 = 46). We assume each future connection will use water consistent with an ERU. The example assumes that the addition of new connections will not result in an increase in the system's current DSL (i.e., build-out rather than expansion of the distribution system).

Following these two assumptions, we will translate and record the reported capacity as an approved number of connections, consistent with WAC 246-290-222.

Using the above example, we would record capacity information on the water system's WFI as shown in the table below. The "DOH Calc" service connection total (bottom line) includes single family residential connections, multifamily residential units, recreational service connections, and other types of connections. Total service connections do not include the multifamily buildings themselves (only each dwelling unit within such buildings).

The excess service capacity is reflected as the difference of the two numbers on the "total service connection" line (156 + 46 = 202).

Service Category	Current System Est.	DOH Calc.	DOH Approved
Total Residential Connections	0	149	202
Full-time Single Family Residences (Occupied >= 180 Days a Year)	139		
Part-time Single Family Residences (Occupied < 180 Days a Year)	0		
Multi-family Residential Buildings (Apartments, Condos, Barracks, Dorms, etc.)	1		
Full-time Residential Units (Occupied >= 180 Days a Year)	10		
Part-time Residential Units (Occupied < 180 Days a Year)	0		
Recreational Services and/or Transient Accommodations	2	2	0
Other Non-residential Services (Institutional, Industrial, Commercial, or Agricultural)	5	5	0
Total Service Connections		156	202

Our approval letter will include a statement such as:

The approved number of connections is based on an assumption that each future connection will use water consistent with an average single family residence supplied by the water system. Other types of new connections, such as apartments, businesses, or parks may use more or less water than an average single family residence.

You are responsible for permitting additional new connections in a manner which recognizes all new connections added and the water demands associated with each new connection. Your process must ensure an accurate assessment of the remaining service capacity available, expressed as ERUs, so that physical capacity and water right limitations are not exceeded.

Our approval of your (water system plan or project report) does not confer or guarantee any right to a specific quantity of water. The approved number of service connections is based on your representation of available water quantity. If the Department of Ecology, a local planning agency, or other authority responsible for determining water rights and water system adequacy determines that you have use of less water than you represented, the number of approved connections may be reduced commensurate with the actual amount of water and your legal right to use it.

Chapter 5: Source of Supply

5.0 Introduction

Safe and reliable drinking water begins with the source of supply. Each public water system has an obligation to secure drinking water from the highest quality source feasible (WAC 246-290-130(1)) with sufficient capacity to meet customer demands (WAC 246-290-222(4) and WAC 246-290-420). Meeting this obligation is challenging given our state's limited water resources and the variety of potential contaminant sources in our environment.

Water resources in Washington present a significant challenge to drinking water systems seeking to develop new or expanding supplies. We urge design engineers to consult with the Department of Ecology before investing significant time and resources into water supply planning and design. In some instances water right limitations may preclude the development of a new drinking water source. See Section 5.12 for guidance on water resource issues related to new or expanding sources of supply.

Our natural and man-made environments pose significant risks of source water contamination. Technology makes it possible to provide treatment to virtually any water source and produce high quality drinking water. However, we believe complex technology in ill-equipped hands creates an unacceptable level of public health risk. When evaluating a new source of supply – especially one that needs treatment - design engineers should consider the long-term technical, managerial, and financial capacity of the water system to maintain and operate the facility to adequately protect the health of their customers.

This chapter offers guidance for the evaluation of different drinking water supplies and presents topics that design engineers must consider when designing a new or modified drinking water source.

5.1 Drinking Water Contaminants

All sources used as a public drinking water supply **must** meet water quality standards, known as maximum contaminant levels (MCLs), set by EPA or the state (WAC 246-290-310). If despite a thorough alternative analysis the selected source water requires treatment, then the source design **must** include the appropriate treatment needed to reliably meet applicable water quality standards (WAC 246-290-130(5), 250, and Part 6). See Chapter 10 for guidance on treatment plant design.

Primary drinking water contaminants can affect human health. Acute contaminants such as viruses, *E. coli, Giardia*, *Cryptosporidium* (microbial), and nitrate and nitrite (chemical) may cause illness with short-term exposure. Depending on the concentration, chronic contaminants such as organic, most inorganic, and radionuclides may cause illness after longer-term exposure. Each regulated chemical and radiological contaminant has a drinking water MCL. Microbial drinking water contaminants have an assigned drinking water treatment technique requirement (disinfection or disinfection and filtration) instead of a MCL.

Secondary contaminants are regulated for aesthetic reasons. They impart an unwanted taste, odor, or appearance to the water, but are not currently regulated based on a risk to human health. There is on-going research into human health effects associated with certain secondary contaminants, such as manganese, which may someday lead to establishing a new enforceable primary drinking water standard.

5.1.1 Initial Sampling Requirements for New Sources

New drinking water sources **must** meet the applicable water quality standards for acute and chronic contaminants prior to entry to the distribution system (WAC 246-290-130(3)(g)). The minimum initial water quality testing varies by:

- Type of water system.
- Location within the state.
- Type of source (groundwater or surface water).
- Susceptibility of the source to contamination.

Initial water quality samples should be collected from new ground water sources at the end of the pump test (see Section 5.5.3) so that samples reflect the water quality in the aquifer, and are not an artifact of well construction. Sampling other water quality parameters during the pump test may be necessary; for example, monitoring chloride levels during the pump test when a well is considered at risk for seawater intrusion (see Section 5.5.4).

The minimum initial required source water quality testing described under WAC 246-290-130(3)(g) includes the following.

- **Bacteriological:** All new source approval requires collection of a coliform sample. We may require additional bacteriological sampling for some groundwater sources and all surface water sources prior to source approval (WAC 246-290-300(a); 300(e); -630(16)(b); -640).
- **Inorganic chemicals (IOCs) and physical parameters:** All new source approval requires collection of a complete IOC sample.
- **Volatile organic chemicals (VOCs):** All new source approval requires collection of a complete VOC sample.
- **Radionuclides:** New source approval for community systems requires collection of a gross alpha and radium 228 sample. Additional radionuclide testing for radium 226 and uranium may be required depending upon the initial results of gross alpha and radium 228.
- **Synthetic organic chemicals (SOCs) and soil fumigants:** New source approval for community and non-transient non-community systems may require collection of SOCs depending on the vulnerability of sources in certain parts of the state. These are usually in

agricultural areas, where SOCs and soil fumigants are or were used. Our staff will inform the water system if source approval requires SOC or soil fumigant sampling.

5.1.2 Detecting Primary Drinking Water Contaminants

Initial sampling may reveal concentrations of one or more primary contaminants close to, but not exceeding the contaminant MCL. Based on decades of observing water quality trends in existing ground water sources, we know that water quality can change with time due to natural and man-made influences (declining aquifer, farming and fertilizer practices, development). Two of our state's most common primary contaminants, nitrate and arsenic, vary seasonally or over multiple years for some sources.

For this reason, when a primary contaminant is measured at 75 percent or more of the MCL, the design engineer should address in the new source project report (see Chapter 2) how the water system will address exceeding a primary drinking water standard. We expect the design engineer's project report to include careful consideration of the following issues:

- Space for treatment facilities
- Disposal of treatment waste
- Construction and operational costs
- Operating requirements, including level of operator certification

5.1.3 Exceeding Primary Drinking Water MCL

If an initial sample from a new source indicates a primary chemical or radiological contaminant above the MCL, we will typically require resampling of the source until follow-up results are definitive with respect to compliance with the MCL. Because of the potential for seasonality the required follow-up sampling may occur over a period of months.

If test results confirm an MCL exceedance and the water system intends to develop the source, the new source design submittal must include physical treatment, blending, or other mitigation to ensure that the water entering the distribution system meets the MCL except as noted below:

> For contaminants that have MCLs based upon chronic health risks, such as arsenic, treatment will not be required for transient non-community water systems unless we determine that the level of the contaminant poses an unacceptable risk to those served by the water system.

See Chapter 10 for treatment guidance.

Coliform contamination detected at a ground water source will trigger a requirement for source treatment. Microbial contaminants may be introduced in development of a new ground water source or when modifying an existing one. To ensure representative source sampling, design engineers should specify best practices in well development, well completion, and in the handling and disinfection of drilling equipment. In addition, water systems **must** ensure proper disinfection and testing is performed before the well is put into service (WAC 246-290-451). New or modified sources should be purged, disinfected, flushed, and sampled according to the latest AWWA C654 specification for well disinfection.

5.1.4 Secondary Contaminants

Secondary contaminants, such as iron and manganese, are currently regulated for aesthetic reasons. We consider it a best practice for all new sources to meet secondary drinking water standards. If an initial sample from a new source indicates a secondary contaminant above the secondary MCL, we may require resampling of the source until follow-up results are definitive with respect to compliance with the secondary MCL. Because of the potential for seasonality, required follow-up sampling may occur over a period of months.

If test results confirm a secondary MCL exceedance, the system intends to develop the source, and the source supplies a new community or non-transient non-community water system, the new source design submittal **must** include physical treatment, blending or other actions to ensure that the water entering the distribution system meets the MCL if (WAC 246-290-320(3)(d)). See Chapter 10 for treatment guidance.

Existing water systems may avoid treatment for exceeding a secondary MCL if they document that consumers accept the water quality impacts and reject the cost of meeting the secondary standard. The design engineer must document construction and operating costs and document consumer acceptance in order to avoid the requirement for treatment. See Appendix F for additional guidance on secondary contaminants.

5.1.5 Ground Water Source Construction-Related Contaminants

Inadequate well development and flushing following construction may result in high turbidity or detection of chemical residuals used in the well drilling process. Collecting initial water quality samples at the end of the pump test will help ensure that sample results reflect the water quality in the aquifer, and are not the result of well construction.

The design engineer should be aware of the risk certain construction materials and practices pose to detection of organic chemicals. Low level detection of organic contaminants may be the result of residuals associated with well development and construction. It is possible to introduce organic contaminants such as tetrahydrofuran and 2-butanone (components in PVC glue) and toluene (component in lubricants) during well construction. Such construction-related contamination, even in very small concentrations, can result in a significant increase in organic chemical monitoring requirements.

High turbidity in a new well or spring is often an indicator of one or more consequential issues, including:

- Poor source development. Inadequate well cleaning following construction may result in high turbidity, and indicate the need to re-develop the source. Collect initial water quality samples at the end of the pump test so that samples reflect the water quality in the aquifer, and are not a result of well construction.

- Iron or manganese. These commonly occurring inorganic contaminants will cause turbidity and, in most cases, require treatment to remove them from newly developed sources if they exceed the secondary MCL.

- Groundwater under the direct influence of surface water (GWI). High turbidity measured in wells developed close to lakes and rivers and in springs may indicate direct surface water influence. Conduct additional testing of these types of sources to determine if there is a significant microbial risk from surface water contaminants. See Section 5.7 for additional requirements.

5.1.6 Source Sample Taps

Proper location of sample taps is very important to successful operations and the ability to demonstrate compliance with drinking water quality standards. The attributes of a good sample tap include:

- The sample tap outlet faces downward.
- Located in a clean, secure, accessible location.
- Located at least 12 inches above the floor or ground level. When taps are lower than that, water containing coliform bacteria is more likely to backsplash into the sample bottle.
- Located where the volume of water from flushing the tap for 5 minutes can easily drain away.
- Smooth-nosed, without internal or external threads (Ten State Standards 2012).

For more information on good sample tap design and location see DOH 331-436.

All drinking water sources **must** have sample taps to meet the water quality monitoring requirements of WAC 246-290-300. The location of sample taps is required at:

- The source, prior to any treatment. Install the sample tap as close to the source as practical.
- After treatment, before entering the distribution system. If there are multiple treatment processes, install sample taps after each unit process. Sample taps should be installed to allow for adequate mixing between any chemical addition and the sampling location.

Sampling requirements for reservoirs and distribution systems are discussed in Chapters 6 and 7 and WAC 246-290-300.

5.2 Source Protection

Location is a key factor in securing the highest quality source feasible. In analyzing a source location, design engineers should consider the measures necessary to establish and maintain sanitary or watershed control, physical protection, and barriers to contamination (e.g., surface intake depth, completed well and annular seal depth). Careful consideration of alternatives will reduce, and in many cases eliminate current and future treatment requirements. See DOH 331-106.

The design engineer must describe and document the adequacy of source water protection in each source approval submittal. These requirements are intended to protect against existing or potential sources of contamination (WAC 246-290-135(2)). Required documentation includes the following. For a more complete listing of groundwater submittal requirements, refer to

Appendix A.3.2. For watershed protection requirements for new surface water supplies, refer to Chapter 11.

5.3 Distribution Water Quality Impacts from New or Modified Sources

Water systems **must** review how proposed projects could potentially affect water quality in the distribution system (WAC 246-290-110(4)(d)). Blending a new source with existing sources, constructing an intertie, installing treatment, or replacing an existing source with a new one can create water quality problems in the distribution system. These distribution system water quality impacts may include:

- Release of accumulated inorganic contaminants from pipe walls
- Increased corrosion of metals
- Impacts on the ability to maintain a disinfection residual (Taylor et al. 2005; Kippin et al. 2001).

For example, iron or manganese can precipitate when water with higher dissolved oxygen levels is introduced from a new or modified source. An increase in hardness and silica levels can lead to water quality issues in the distribution system, such as staining and etching. Changes in flow direction and water age can release scales and accumulated sediments from pipes in the distribution system (Friedman et al. 2010; Hill et al. 2010). These mobilized scales and sediments can contain metals such as arsenic, chromium, and lead (Peng et al. 2012). Design engineers should identify how the water system will address such issues. Commonly-used approaches include:

- Bench scale studies
- Pipe loop studies with new or existing distribution system materials
- Enhanced monitoring and flushing programs.

Decreasing water's oxidation-reduction potential (ORP) increases instability of pipe scales, making them more prone to release into the distribution system. Factors which tend to lower ORP, and thus increase pipe scale instability include:

- Decreased disinfectant residual
- Decreased dissolved oxygen
- Decreased alkalinity
- Decreased pH

Design engineers **must** specifically address how a change in treatment or introduction of a new source will affect compliance with the Lead and Copper Rule (40 CFR 141.81(b)(3)(iii); 141.86(d)(4); and 141.90(a)(3)). Examples of treatment changes that could increase corrosivity and lead/copper solubility as well as other distribution system issues include:

- Introduction of a disinfectant. Disinfectants can affect corrosivity, metal release, or both (Schock and Lytle 2011). For example, the initial introduction of chlorine can increase the release of copper (Stone et al. 1987). Corrosion rates of both mild steel and copper were found to be higher in the presence of free chlorine than in its absence (Pisigan and Singley 1987).

- Changes in residual disinfectants. For example, switching from free chlorine to chloramine as a residual disinfectant can lead to significant lead release (Edwards and Dudi 2004; Boyd et al. 2008). Change the oxidation-reduction potential, and thus increasing the risk of destabilizing tubercles and the release of bound and harbored biological and inorganic compounds.

- Switching coagulant chemicals. Changing coagulants can change the chloride to sulfate mass ratio can cause significant lead release (Edward and Triantafyllidou 2007; Nguyen et al. 2010).

- Installation of additional treatment. For example, the installation of ion exchange can change the pH, alkalinity, and chloride to sulfate mass ratio increasing lead and copper release at the tap (Nguyen et al. 2010).

Water systems that change treatment or introduce a news source of supply may be required to re-start lead and copper distribution system monitoring (40 CFR 141.86(d)(4)(vii)).

5.4 Source Water Quantity and Reliability

The source or sources for a water system **must** be able to meet the water system's maximum day demand (MDD) (WAC 246-290-222(4)). For reliability purposes, supply sources should be able to replenish depleted fire suppression storage within 72 hours (or sooner if required by the local fire authority) while concurrently supplying the MDD of the water system. For wells and other pumped sources, we recommend establishing source water quantity on the basis of pumping no more than 20 hours per day. See Sections 3.10 and 4.4.2 additional guidance.

After deciding the new source's capacity, design engineers should check to be sure reservoir overflow capacity is sufficient to safely discharge the maximum combined source of supply available to the zone served by the reservoir (e.g., all wells, interties, pressure reducing valves, booster pump stations), without damage to property or surcharging the reservoir structure.

5.5 Wells

The vast majority of source development projects in our state are drilled wells and well fields (see the *Well Field Designation Guideline* in Appendix B.1). See Chapter 2 and Appendix A for guidance on preparing a project report and the construction documents for a new well. See Appendix E for well pumping test standards. See Policy M.01 for requirements on pitless units and well caps.

The design engineer should use a pump test or hydrogeologic analysis to determine how reliable groundwater or aquifers will be over time. We expect pumping tests to run at a flow rate greater than or equal to the maximum design pumping rate (WAC 246-290-130(3)(c)(iii)).

Design engineers may reference well source development data from nearby sources as supplemental evidence that water quantity is adequate to meet design criteria. If the aquifer setting is well characterized and quantified and hydrogeologic information is adequate to establish a sustainable pumping rate, then the engineer may submit a hydrogeologic report to justify the proposed pumping rate. The report should be prepared by a licensed hydrogeologist, and include a detailed analysis of the well, aquifer, and local conditions including impacts on nearby sources of supply.

5.5.1 Steps to Take Prior to Drilling a Well

There are a number of steps that should or must be taken prior to drilling a well that are intended to assure development of the highest quality source feasible and to facilitate a quick and efficient design review process.

Prior to drilling a well, the applicant **must**:

- Obtain a notice of intent to construct a well from the Department of Ecology (WAC 173-160-151).
- Ensure that a licensed well driller will drill the well and that the well will be constructed according to chapter 173-160 WAC.
- Obtain a well site inspection by state or local health jurisdiction staff (WAC 173-160-171(3)(c)).

Prior to drilling a well, the applicant should:

- Prepare and submit for DOH review and endorsement the well pumping test plan. In developing the pump testing plan refer to Appendix E.

- Evaluate the possibility of obtaining alternate sources of supply through interties with neighboring utilities or wells already in existence.

- Conduct a preliminary hydrogeologic assessment, which includes preparing a contaminant source inventory a Wellhead Protection Potential Contaminant Source Inventory.

- Contact ODW to learn the parameters used to delineate groundwater under the direct influence of surface water (GWI). A determination of the GWI status is required prior to source approval. If a well meets ODW's criteria for a potential GWI (for example, less than 50 feet deep and within 200 feet of a surface water body), data will be required to determine whether the source is hydraulically connected to surface water and to what extent. ODW will be involved with approval of the plan for the data to evaluate a source as GWI.

- Obtain a legal right through an ownership option or recorded covenant to prevent potential sources of contamination from being located within the standard sanitary control area (normally a 100-foot radius around the well).

Before drilling, it is prudent to ensure that the water system has the ability to control the sanitary control area (SCA) (100' radius around well) through a protective covenant or other mechanism. Concern about controlling the entire SCA, or considering a reduction in the size of the SCA, should be raised to the ODW representative at the time of the preliminary site inspection prior to the well being drilled. This ensures an opportunity to evaluate siting conditions and allows for the identification of possible mitigation measures, such as a deeper surface seal during well construction.

5.5.2 Documenting Wellhead Protection

The SCA must have a radius of 100 feet, unless justification demonstrates that a smaller area can provide an equivalent level of source water protection. The justification should address geological and hydrological data, well construction details, mitigation measures and other relevant factors necessary to ensure adequate source water quality protection. Major factors influencing a decision to allow a smaller than standard SCA include depth of the screened interval and "confinedness" of the water bearing zone being used. ODW may require a larger SCA, additional mitigation measures, or both if land use, geological or hydrological data support such a decision.

Prior to receiving ODW approval of the proposed source the purveyor must be able to provide the dimensions, location, and legal documentation of the SCA. The purveyor is required to "control" the SCA to prevent any potential source of contamination from being constructed, stored, disposed of, or applied within the sanitary control area. To ensure the purveyor can control the SCA, the purveyor is required to either own the SCA outright, or the purveyor must have the right to exercise complete sanitary control of the land through other legal provisions, such as a duly recorded declaration of covenant, restricting the use of the land.

To evaluate the appropriateness of a proposed drilling location, a preliminary susceptibility assessment, preliminary WHPA delineation and initial contaminant inventory should be conducted. Findings should be mapped.

A preliminary susceptibility assessment provides the applicant, and ODW, information prior to drilling a well that helps evaluate the suitability of the proposed well site before a significant expenditure of resources occurs. It may be helpful to contact ODW for guidance regarding conduct of a preliminary susceptibility assessment before selection of a potential well site. Elements that are unknown until after drilling occurs should have their values estimated based on site specific conditions and best professional judgment (length of screen, confined vs. unconfined aquifer, and other parameters).

The initial WHPA delineation can be done using the "Calculated Fixed Radius" method. More sophisticated and accurate methods of delineation, such as analytical or numerical modeling, are encouraged to ensure a higher level of source protection. See ODW *Wellhead Protection Program Guidance Document* (331-018) for further explanation of these methods. The WHPA delineation should identify the six-month, one-, five- and ten-year time of travel boundaries. A survey for potential sources of groundwater /source water contaminant should be conducted in the WHPA area. Additional information on source water protection is available on our Source Water Protection web page.

5.5.3 Ground Water Pumping Tests

Design engineers can use pump tests to assess the reliability of groundwater to meet the demands of a projected population over time. See Appendix E for detailed guidance on pumping tests. The design engineer must ensure the pumping test provides sufficient data to achieve its objectives.

The objective of the pumping test is to acquire data identifying the source's safe yield and maximum design pumping rate, to establish well pump depth setting, and to size and select the well pump. Specific reliability concerns include low-flow conditions, fracture-flow conditions, aquifer of limited areal extent, seawater intrusion, effects of concurrently pumping multiple wells, and seasonal variability.

The timing of a pump test may be more or less important to determining safe yield, depending on the aquifer setting. Groundwater is less dependent than surface water on seasonal and annual climate conditions, except in sensitive settings such as shallow alluvium aquifers and in some areas where localized recharge "lenses" occur. Design engineers should identify if a proposed groundwater source is located in a sensitive setting and design the pump test accordingly.

After the pumping test, design engineers **must** compile the following data into a project report, and submit it to us as part of the source approval documentation (WAC 246-290-130(3)(c)):

- All items for source approval, if applicable.
- A time-drawdown graph (on standard and semi-log paper).
- An analysis and discussion of applicable hydraulic parameters (such as transmissivity, hydraulic conductivity, storativity), as appropriate, to support the objectives of the pumping test.
- A map and description (1/4, 1/4, section, township, range) accurately indicating the well location and the land surface elevation to the nearest foot. Locate observation wells with distances to the nearest foot.
- A well report.

The end of the aquifer pump test is a good time to collect the water quality samples required for source approval.

5.5.4 Seawater Intrusion

Wells or well fields developed close to seawater may be vulnerable to seawater intrusion. Seawater intrusion caused by over-pumping the basal (freshwater) lens degrades groundwater quality by drawing in chloride (from seawater) into the remaining fresh groundwater. Department of Ecology rules prohibit degradation of the state's groundwater. We recommend that water systems have a hydrogeologist or qualified engineer assess the potential for seawater intrusion and oversee well testing. See Appendix E for guidance on developing sources vulnerable to seawater intrusion. Wells at risk for intrusion include wells located:

- Within ½ mile of the shoreline and pump water from a depth below sea level.
- Within ½ mile of a groundwater source with chloride concentrations over 100 mg/L.

The design engineer should avoid supply sources at risk for seawater intrusion. Ecology may condition water right permits to provide for reduced pumping rates or may even require a water system to abandon sources if seawater intrusion threatens senior water right permits. In addition, several counties have policies or ordinances affecting water systems in areas vulnerable to seawater intrusion. We recommend that the design engineer contact Ecology and the local health jurisdiction for current policies or rules on developing wells where seawater intrusion may be a concern.

5.6 Spring Sources

A design engineer submitting a spring source for approval **must** identify (WAC 246-290-130(3) and -135):

- The safe yield of the spring.
- Water quality meets all applicable drinking water standards.
- Measures to protect the sanitary control area and water quality of the spring supply from contamination at all times.

5.6.1 Spring Source Safe Yield

Methods for determining the safe yield of a spring's flows may be difficult to apply. Pumping test procedures do not generally apply to springs because the recharge is unidirectional and associated only with the delivery of flow at the ground-surface interface. Therefore, to measure spring-flow quantity, design engineers should use actual flow records (with weirs or other mechanisms capable of measuring surface flows) during high and low flow conditions over a variety of seasonal conditions. At a minimum, spring flows should be measured at least monthly for at least 12 months (Meuli & Wehrle, 2001).

Because drought conditions often affect spring flows, it is appropriate to estimate the flows that would prevail during drought conditions. Precipitation data should be collected along with measured spring flows and compared with precipitation data from previous years to estimate the safe yield and minimum flows from the spring. Appendix C includes climatological organizations (NOAA, Office of State Climatologist, Western Regional Climate Center) whose data may assist with determining how current-year precipitation compares with the historic weather patterns.

We recommend that the design engineer use the 50-year low-rainfall level to estimate the safe yield from the spring. Spring flows are inherently uncertain and can be sharply affected by changes in precipitation patterns, so it also is appropriate to apply a safety factor to any flow quantity derived from measurements. We recommend a safety factor of no more than 0.5 times the estimated minimum flow when determining the spring's safe yield.

5.6.2 Spring Source Water Quality

Because springs are potential GWI sources (see Section 5.7), the design engineer **must** establish whether the spring source is GWI (WAC 246-290-130(3) and -640). We expect the content of the project report for a spring to comply with either ground water or surface water source approval, depending on the outcome of the GWI determination.

Spring sources are at significant risk to surface contamination; the greater the degree of seasonal fluctuation in flow, the higher the risk of contamination (Meuli & Wehrle, 2001). The design of the spring catchment and conveyance systems must minimize the risk of direct surface water infiltration; otherwise, the spring source will be considered a surface water supply subject to all applicable sections of WAC 246-290 Part 6.

In many cases, springs not otherwise subject to the requirements of the surface water treatment rule will be required to provide CT6 disinfection treatment (4-log virus inactivation) before the first connection (WAC 246-290-451(4)) and WAC 246-290-640(4)). We may also require sanitary protection of the source beyond the minimum 200 feet required in WAC 246-290-135.

In general, unique geological conditions will dictate the steps design engineers will follow when developing a spring source. Design engineers should tailor their design and construction activities to protect the spring, and the areas above the spring, from surface contamination.

- Construction materials **must not** create an opportunity for water quality problems (WAC 246-290-220).
- Surface water runoff diversions should be provided.
- Designs for spring collectors and catchment facilities must prevent infiltration of contamination.
- Protection from vandalism should be provided (fencing, lockable hatches, and other security measures).
- Requirements for screening vents or other openings appropriate to the spring are similar to those for distribution reservoirs (see WAC 246-290-235).

The design engineer can get guidance and specific details on spring development, sanitary protection, and water quality considerations in the references listed at the end of this chapter (AWWA 1999; USEPA 1991; Meuli & Wehrle, 2001).

5.7 Groundwater Under the Direct Influence of Surface Water

Groundwater under the direct influence of surface water (GWI) is any water beneath the surface of the ground with:

1. Significant occurrence of insects or other macro organisms, algae or large diameter pathogens such as *Giardia lamblia*, or

2. Significant and relatively rapid shifts in water characteristics such as turbidity, temperature, conductivity, or pH, which closely correlate to climatological or surface water conditions (WAC 246-290-010).

Water systems with sources confirmed to be GWI **must** comply with the filtration and disinfection requirements for surface water sources (WAC 246-290, Part 6).

Water systems **must** evaluate all potential GWI sources to determine if additional treatment is necessary (WAC 246-290-640). Potential GWI sources include (WAC 246-290-010):

- Wells with a first open interval less than 50 feet below the ground surface and are located within 200 feet of surface water
- Springs
- Infiltration galleries
- Ranney wells

During review of source approval information, we may determine a source other than that listed above is a potential GWI source, and subject to the GWI evaluation process. An example of such a source is a well located on a bluff above a nearby stream. The depth to the first open interval may be greater than 50 feet as measured from the top of casing, but it less than 50 feet as measured from the stream's high water elevation.

We will not approve a new potential GWI source before a proper evaluation. Figure 5-1 at the end of this chapter outlines the evaluation process for potential GWI sources. The project report must document the details of the GWI evaluation, including how anticipated source withdrawal conditions will be simulated for new (not yet in service) potential GWI sources. For planning purposes, design engineers should schedule at least18 months to complete the GWI evaluation process.

Potential GWI sources not determined to be GWI are not required to meet the treatment requirements for surface water sources. However, potential GWI sources determined to be in hydraulic connection with surface water **must** provide minimum CT6 disinfection prior to entry to the distribution system (WAC 246-290-451(4), WAC 246-290-640(4), and Policy F.12)). Additional guidance on evaluating potential GWI sources is available from references at the end of this chapter (WSDOH 2003a; WSDOH 2003b).

5.8 New Surface Water Supplies

Development of a new surface water supply can take many years since it may be difficult to secure the necessary permits from natural resource agencies and other involved parties. For example, new surface water sources trigger the requirement for detailed environmental review under the State Environmental Protection Act (SEPA).

In addition to treatment, the design engineer should consider the unique features of surface sources when evaluating them for the drinking water supply. Often, several competing beneficial uses (agriculture, fisheries, and other resource demands) affect the long-term reliability of surface sources. Water rights may be very difficult to secure, and they may be so restricted during some periods that little or no portion of the source can be used to supply drinking water (see Interruptible Water Rights, Section 5.12.2). The reliability of this type of source is inherently subject to greater degrees of uncertainty because of its association with annual precipitation levels (rain and snow), and regulation of withdrawal due to low stream flow.

Surface water sources are also more vulnerable to contamination than protected groundwater supplies. The vulnerability of the source to natural disasters (floods, wildfires, and landslides) and human caused contamination (waste disposal, spills, and runoff from agricultural activities) should be thoroughly assessed. One specific requirement is that new surface sources **must** conduct detailed source water monitoring to assess the degree of microbial risk. High risk sources require a higher level surface water treatment (WAC 246-290-630(16)(b); 40 CFR 141.702(f)).

Surface water supplies normally require treatment by conventional, direct, slow sand, diatomaceous earth filtration, or an approved alternative technology and **must** comply with the Surface Water Treatment Rule and WAC 246-290, Part 6. Detailed design criteria are in WAC 246-290, Chapter 10 of this manual, and the *Recommended Standards for Water Works* (Ten State Standards 2012). The introduction of a new surface water supply may cause water quality changes in the distribution system, which must be evaluated in a project report (See Section 5.3).

Design engineers planning to submit new or modified surface water treatment designs **must** first perform a pilot study to evaluate alternatives (WAC 246-290-250; WAC 246-290-676(3)). Section 10.3 provides detailed guidance on treatment pre-design and pilot studies. In some cases, design engineers may need DOH and natural resource agencies to approve the intake facilities before initiating the pilot study.

Design engineers can help to ensure an efficient and orderly review of their surface water treatment proposals by meeting with our regional office staff to establish specific design requirements.

Water systems should develop surface sources with full knowledge that some reductions in service capacity may result over time as low rainfall years, low snow-pack years, or drought conditions occur. Water systems will need to compare historic hydrological data against customer service expectations to gauge the adequacy of the source.

5.8.1 Surface Water Safe Yield

Surface water source yield depends on climatic influences from year to year. Design engineers can use flow measurements and hydrologic assessments with an appropriate factor of safety to measure surface water source safe yield. However, when defining expectations for long-term service, design engineers should base the reliability of flow from these sources on years with the lowest precipitation levels.

In general, the safe yield of a surface water reservoir is the reliable withdrawal rate of water that can be provided by a watershed through the critical drought period. We recommend using a 98 percent level of reliability, equivalent to a 50-year drought, for surface supplies. This basis of design was developed from various references (Prasifka 1998; HDR 2001; City of Seattle, 2013).

Instream flow reservations and other natural resource impacts need to be accommodated when assessing the safe yield of the water resource during drought.

Appendix C includes climatological organizations whose data may assist with determining how current-year precipitation compares with the historic weather patterns. Appendix C also includes a link to Department of Ecology's stream flow data base.

5.9 Purchased Water and Emergency Interties

Interconnections (interties) between water systems are an alternative to developing new supply sources. Interties can help provide a level of reliability difficult to secure otherwise.

A design engineer considering an intertie to augment supply sources **must** satisfy the requirements of WAC 246-290-100 and -132. These requirements exist to ensure the wholesaling and consecutive systems have the physical and legal capacity to sell and purchase the expected volume and flow of water. Planning and engineering documents submitted to support constructing a new or expanding consecutive system **must** include the intertie agreement. There are different standards for emergency interties and purchased water (non-emergency interties).

5.9.1 Reliability of Purchased Water (Non-Emergency Interties)

New and existing public water systems may be supplied in whole or in part by purchased water from another utility. The design engineer is responsible for assessing the reliability of a purchased water agreement, and for demonstrating how the intertie improves overall system reliability (WAC 246-290-132 (3)(a)(v)). Water systems **must** satisfy the requirement to provide an adequate quantity and quality of water in a reliable manner (WAC 246-290-420 (1)).

A design engineer submitting a new and renewed wholesale water agreement that raises reliability concerns (see list below) **must** submit a viable plan identifying an alternative water supply which will satisfy the requirement to provide an adequate quantity and quality of water in a reliable manner (WAC 246-290-420 (1)) if/when the agreement is terminated. With the agreement, the design engineer **must** submit evidence that the wholesaler's full allocation of water, storage, and/or booster pumping capacity to the consecutive system is reflected in the wholesaler's service capacity assessment (WAC 246-290-222).

Termination of the water supply, done in accordance with provisions written into a mutually-agreed upon purchased wholesale water agreement, <u>is not</u> considered an abnormal operating condition. To improve reliability, we believe wholesale water agreements should not be subject to termination except for customary reasons (e.g. failure to pay). The criteria described below reflect this principle.

Recommended attributes of a wholesale water agreement:

- How and when charges are calculated and billed
- When payment is due, and what happens if payment is past-due
- Description of whether or how much standby storage and/or fire suppression storage is available to the consecutive system

- Adherence to cross connection control requirements at the point of service
- How the water rate(s) is adjusted over time
- Absolute and seasonal limits on instantaneous flow and annual volume
- Limits on type and place of use
- Impact of a declared emergency or natural disasters

Attributes of a wholesale water agreement that raises reliability concerns:

- Date-based termination clause (e.g., "this agreement is valid for ten years")
- Needs-based termination clause (e.g., "this agreement may be terminated at any time due to unforeseen circumstances that results in a limited water supply that must be allocated to in-city customers")
- Short-term unilateral termination clause (e.g., "this agreement may be terminated after 30 days' notice by either party")

A consecutive system purchasing water through an intertie may need to treat the purchased water in order to maintain compliance with drinking water standards (e.g., maintaining a free chlorine residual in the consecutive system's distribution system) or avoid distribution system water quality impacts such as those described in Section 5.3. A consecutive system may also incur additional sampling requirements, such as for disinfection byproducts and Groundwater Rule triggered sampling.

We may approve planning and engineering documents based on the following types of purchased water agreements. These agreements fall into a spectrum of risk to interruption, from practically no risk of interruption to near certainty of termination.

- <u>Regional Water Supply Agreement (Very reliable)</u>. A consortium of water systems receives their water supply from source and transmission infrastructure held in common and proportional ownership. This shared infrastructure is operated and maintained by the member systems. In these cases, there is little to no increased risk of interruption in supply to any individual member.

- <u>Bought-in Wholesale Capacity Agreement (Very reliable)</u>. A single utility wholesaler permits one or more other utilities to buy-in to the wholesaler's supply and transmission infrastructure, similar to a retail water customer paying a system development charge to a utility for the privilege of receiving water service from that utility. This investment assures the participating utilities a proportional or fixed share of the supply, but does not provide them with a say in the operation or maintenance of those supplies. In these cases, there is little to no increased risk of interruption in supply to any of the participating utilities.

- <u>Purchased Wholesale Water Supply Agreement (Poor reliability)</u>. A single utility wholesaler agrees to sell water to one or more consecutive utilities. There is no ownership stake held by any of the consecutive systems. The wholesale agreement that may not be renewable and/or may be terminated at the option of the wholesaler (such contracts are common). As a result, there is a greater risk of interruption in supply.

- Reserve Infrastructure Agreement (Variable reliability). A portion of the capacity of the wholesaler (source water, stored water) is made available for use as a reserve source to a consecutive system, thus saving the consecutive system from the full cost of investing in its own standby infrastructure. The wholesaler should account for the transfer of standby capacity in its own planning. The intertie valve should be automatic. The risk of interruption in standby reserve capacity (e.g., the provision of fire suppression flow or storage; provision of standby storage) depends on the content of each agreement.

5.9.2 Emergency Interties

An emergency intertie may often be a cost-effective means of reducing the risk of supply interruption. Because of the difference in design approval requirements, it's important to establish how an emergency intertie is different from a non-emergency intertie. Satisfying all of the following criteria identifies an intertie as an emergency intertie:

- The consecutive system's own source(s) of supply, booster pumps, and reservoirs are capable of meeting the maximum daily demand (MDD) and peak hourly demand (PHD) while maintaining the design standards of WAC 246-290-230 without supplemental supply delivered through the intertie; and

- The events intended to be addressed by the emergency intertie, and documented in the intertie agreement between the two parties, are limited to one or more of the following:
    - Temporary failure of one or more non-emergency sources where the remaining sources of supply cannot maintain 20 pounds per square inch (psi) during PHD throughout the consecutive system's distribution system.
    - Fire where the fire suppression requirement (flow rate and duration) combined with MDD cannot be met by the consecutive system's own system while maintaining 20 psi throughout the distribution system.
    - Water quality emergency

The original circumstances and associated design intent of the intertie may change with time. In the transition from an "emergency use intertie" to a "non-emergency intertie," the design engineer should make certain that the purveyor meets all the applicable requirements of WAC 246-290-132.

RCW 90.03.383 addresses intertie approvals intended to resolve emergent public health concerns, short-term emergencies, and drought emergencies. RCW 90.03.383 (2) states an "emergency-use intertie" does not trigger a requirement to change the upstream water system's water right, and does not require a place-of-use change. As stated in Section 4.4.2, designers **cannot** include emergency sources, including emergency interties, in the total source capacity calculations (WAC 246-290-222(3)).

5.10 Unconventional Sources

Many watersheds are limited in their capacity to supply water for growth and development while maintaining sufficient stream flow. Consequently, design engineers may look to unconventional sources such as rainwater collection and seawater desalination to develop into drinking water supplies. Before approving an unconventional source, we expect the design engineer to complete a thorough alternative analysis to determine the highest quality source feasible for the water system.

5.10.1 Rainwater Collection

We consider rainwater surface water, subject to all the requirements of the surface water treatment rule. Collected rainwater often has significant fecal contamination and as well as other contaminants (Birks et al. 2004; Lye 2002; Osterholt et al. 2007; Hoque et al. 2003). Any public drinking water system that uses collected rainwater **must** provide treatment, including filtration and disinfection in compliance with the Surface Water Treatment Rule. Surface water treatment design requirements, ongoing operations and maintenance requirements, and daily monitoring and monthly reporting requirements are described in WAC 246-290 Part 6. Design submittal requirements for rainfall catchment are significant. See Appendix F.7.

Rainwater is slightly acidic and low in dissolved minerals. These qualities make it corrosive to metals and other materials. The rooftop collection material and coating systems **must** meet ANSI/NSF Standard 61 or NSF Standard Protocol P151 to reduce the risk of chemical contaminants. In addition, the water system may have to install corrosion control treatment to overcome rainwater's natural corrosivity.

Rainwater collection systems intended for non-potable uses are a high cross-connection control hazard, especially if the rainwater system is delivered through dedicated internal plumbing. Therefore, any water system providing service to a building with a rainwater collection system **must** protect the water distribution system from contamination by cross connections (WAC 246-290-490).

Reliance on rainwater is problematic due to drought and extended dry periods that occur even in the wettest parts of this state. In Washington, there is less rainfall during the summer, which is the period of greatest water demand for most water systems. The design engineer for a rainfall collection system **must** evaluate rainfall, usage patterns, and water storage thoroughly to ensure a reliable supply (WAC 246-290-130(3)). See Appendix F.8 for guidance on assessing adequacy of rainfall collection as a drinking water supply.

As with any surface water supply, the safe yield of a rainfall catchment system is the reliable withdrawal rate of water from the cistern that can be sustained by a catchment surface through the critical drought period. We recommend using a 98 percent level of reliability, equivalent to a 50-year drought, for rainfall catchment systems. Rainfall varies in Washington State on an annual, seasonal, and regional basis. This variability in supply makes reliance on rainwater collection as the sole source of supply impractical for nearly all public water systems.

5.10.2 Trucked and Hauled Water

We will not approve trucked or hauled water as a permanent drinking water supply to new or existing public water systems. Trucked water is not considered a reliable permanent source of supply, and the handling of trucked water under non-pressurized conditions poses an increased health risk. We acknowledge the use of trucked or hauled water as a temporary, last-resort measure to meet basic public health requirements in response to an emergency for the period a public water system lacks access to an adequate and safe drinking water supply. See DOH publication 331-063.

5.10.3 Desalination

Desalination of seawater or brackish groundwater is technically feasible and may be the only option available in some situations. Design engineers should consult with us before initiating a desalination project, and should contact other county, state, and federal agencies early in the design process to identify potential permitting issues. See Chapter 10 and Appendix F.6 for design guidance associated with brackish water and seawater desalination.

5.10.4 Aquifer Storage and Recovery

Aquifer storage and recovery (ASR) increases existing groundwater supplies by artificially recharging and storing groundwater. ASR is a water resource strategy designed to take water when it is available and store it in an aquifer deemed appropriate for later withdrawal and beneficial use. This water supply strategy is being implemented by several water utilities in Washington State, such as the Sammamish Plateau Water and Sewer District and the Cities of Walla Walla, Yakima, and Kennewick.

We do not have primary responsibility to oversee or approve ASR projects. The Department of Ecology implements the ASR permitting process under the authority of Chapter 90.03 RCW with standards for review established by Chapter 173-157 WAC. The permitting process is challenging, and requires applicants to provide documentation, including demonstration through actual and/or modeled hydrogeologic conditions:

- The percentage of water pumped into the aquifer that is recoverable. An adequate recoverable volume of water is typically critical to the economic feasibility of the project for the applicant. Successful ASR applicants own the right to use the recoverable portion of the water they store underground.

- Groundwater quality will not be degraded by ASR. There are potentially negative impacts associated with pumping treated surface water or reclaimed water into an aquifer and ASR projects **must** be compliant with state water quality standards (Chapter 173-200 WAC).

ASR projects intended to augment an aquifer's capacity to supply drinking water must not only meet Ecology's ASR permitting requirements but must eventually satisfy public drinking water source requirements. Design engineers pursuing a drinking water-related ASR project should concurrently satisfy all applicable sections of WAC 246-290.

5.11 General Source Design Considerations

We consider several other design elements associated with new or existing drinking water sources when reviewing design documents and subsequent reports.

5.11.1 Power Supply Reliability

To avoid the attendant risk of backflow contamination from depressurization of some or all of the distribution system, design engineers **must** consider the reliability of the power supply grid if the proposed system has no provision to maintain pressure in the event of a power outage (WAC 246-290-001, -200).

In Sections 3.10, 5.4, and 5.8 we summarized our recommendations for sources to reliably supply MDD. Below we provide guidelines on what we consider the minimum acceptable level of utility power supply reliability. We consider a reliable power supply as defined below:

- Power outages average three or less per year based on data for the three previous years with no more than six outages in a single year. Power loss for 30 minutes or more qualifies as an outage.

- Outage duration averages less than four hours based on data for the three previous years, with no more than one outage during the three previous year period exceeding eight hours.

If a power supply to a source, pump station, or treatment plant cannot meet the minimum standards described above, then we consider the power supply unreliable and further reliability measures **must** be taken (WAC 246-290-420(4)). Section 3.2 of the *Recommended Standards for Water Works* contains additional design guidelines on this issue.

If the utility power supply is unreliable, the design engineer should consider one or more of the following measures:

- In-place auxiliary power available (auto transfer capable)
- Two or more sources each connected to a different electrical substation
- Construct and maintain adequate gravity standby storage (see Chapter 7)
- Power connections to two independent primary public power sources

5.11.2 Criteria for Multiple Sources or Multiple Pumps per Source

We encourage water systems to have multiple supply sources to better ensure operational reliability in the event of mechanical, electrical, treatment, or structural failure of a single source. Multiple sources may offset recommended standby storage (SB) volumes.

We recognize that multiple pumps for a single source may be more reliable than a single-pump source. However, we do not consider a single source with multiple-pumps to be as reliable as multiple sources. Design engineers should address the following criteria in seeking approval to consider multiple pumps in a single well as equivalent to multiple sources for the purpose of evaluating a reduction in SB volume (see Section 7.1.1.3).

- The pumps are set in the well such that each pump can be taken out of service, replaced or repaired, without the need to de-pressurize the water system. Water systems should consider establishing an emergency on-call service contract with a qualified repair or service entity to minimize down time.
- The well design includes an alarm to signal a pump has failed.

- The submittal includes an operational plan to address repairs and minimize downtime (WAC 246-290-415).
- The well(s) should be easy to access for repairs and pump removal.

For further consideration, bear in mind the risk of contamination of a single well with multiple pumps. Mechanical failure of a single pump may be overcome by the second pump, but two pumps cannot overcome contamination of the source. Consider the need for an expanded, more robust sanitary control area for a multiple pump source.

5.12 Water Resource Analysis and Water Rights

As part of the source approval process, the engineer **must** do a water resource analysis to consider and address water rights issues (WAC 246-290-130(3)). A water resource analysis **must** evaluate opportunities to obtain or optimize the use of sources already developed, or other methods to meet water needs (WAC 246-290-100 (4)(f) and 110(4)(c)).

Water supplies for Group A water systems **must** conform to state water right laws (WAC 246-290-130(3)(b) and (4)(a)). Water systems that submit new source development or other growth-related projects for our review and approval (WAC 246-290-100(4) or 110(4)(e)) **must** include a *Water Rights Self-Assessment Form* (WAC 246-290-130(4)(a)). The department reviews the information provided on this form to ensure the water system has adequate water rights to meet the projected increased ability to provide service.

5.12.1 Temporary Water Rights

Temporary water rights are generally issued by the Department of Ecology under one of two scenarios:

- For short-term use with an associated expiration date; or
- For the use of water during the pendency of application review and final decision for issuance of a traditional water right permit

We will not increase an existing system's service capacity solely because the system secures a temporary water right. We may approve a new public water system based on a source of supply with a temporary water right if all three of the following conditions are met:

1. The water system operating under the temporary water right does not and will not provide water service to any permanent structure or permanent use.

2. The design submittal/project report reflects the expiration and non-renewal of the temporary water right in its financial planning section, and the local government land use decision reflects and supports the temporary use.

3. The title of the property served by the temporary water right reflects the limitations and attributes of the temporary water right, and includes a disclaimer approved by us.

5.12.2 Interruptible Water Rights

Design engineers considering a new or expanding public water system dependent on interruptible water rights **must** demonstrate the water system has access to an uninterruptible instantaneous supply sufficient to meet basic maximum daily demand requirements, and an uninterruptible volumetric supply sufficient to satisfy the system's non-discretionary average daily water demands (WAC 246-290-230). We consider access to an adequate uninterruptible supply to include:

- An uninterruptible instantaneous supply equal to at least 350 gpd/ERU (WAC 246-290-221 (4)) (See *Baseline Residential Water Demand* in Appendix D).
- A volumetric supply sufficient to supply at least 200 gallons per day (gpd) per ERU during the entire period of interruption.

During the entire period of interruption, the water system **must** remain capable of providing needed fire flow as determined by the local fire control authority (WAC 246-290-221(5)).

The water system **must** submit a plan for temporary demand curtailment during the design period of interruption (WAC 246-290-420 (1)). If the temporary demand curtailment plan does not describe a credible plan to limit demand, we may require a greater maximum daily demand (MDD) and/or average daily demand (ADD) be used in the design of the water system.

We recommend planning to mitigate the maximum duration of an interruption based on a 50-year return period. Design engineers should request from Ecology the estimated duration of interruption on the basis of 50-year low flow in the source(s) of supply

5.12.3 Leased Water Rights

In general, leased water right contracts are not considered a reliable source of supply since such arrangements risks a permanent water supply interruption if the lease is revoked, not renewed, or a permanent right cannot be obtained prior to expiration of the lease contract. Federal leased water, such as Bureau of Reclamation leases, is assumed to be non-revocable and renewable in perpetuity. Consequently, federal leases are considered reliable and appropriate for approval of a new or expanding water system.

In most circumstances, the design engineer **must** demonstrate service capacity based on the ability to supply 350 gallons per day per ERU under a non-leased water right(s) (WAC 246-290-221 (4) and 246-290-420 (1)). In addition, the water system **must** submit a plan for permanent demand curtailment (WAC 246-290-420 (1)). If the permanent demand curtailment plan does not describe a credible plan to limit demand, we may require a greater maximum daily demand (MDD) and/or average daily demand (ADD) be used in the design of the water system.

5.13 Placing a New or Modified Source into Service

Before a new or modified source can be placed into service, it **must** be properly inspected, disinfected and tested (WAC 246-290-120(4) and -451(1)). The WSDOT/APWA standard specifications (Division 7) and *AWWA C651 - Standard for Disinfecting Water Mains* are commonly used to define pressure, leakage and disinfection standard practices (WSDOT/APWA 2016; AWWA 2014) for pipelines installed as part of a new or modified source. AWWA C654 – *Standard for Disinfection of Wells* is the disinfection and testing standard for new or modified wells (WAC 246-290-451(1)) (AWWA 2013). The specific standards used for the project should clearly identify:

- Inspection and flushing requirements
- Pressure and leakage testing methods
- Disinfection and bacteriological testing methods

Only after a new or modified source has been properly disinfected, testing results show that the water from it is safe to drink, and the engineer in charge of the project submits to the department a Construction Completion Form may the source be placed into service (WAC 246-290-120(5); WAC 246-290-125(2)(b)). Design engineers **must** submit a completed *Construction Completion Form* (DOH 331-121) to the department.

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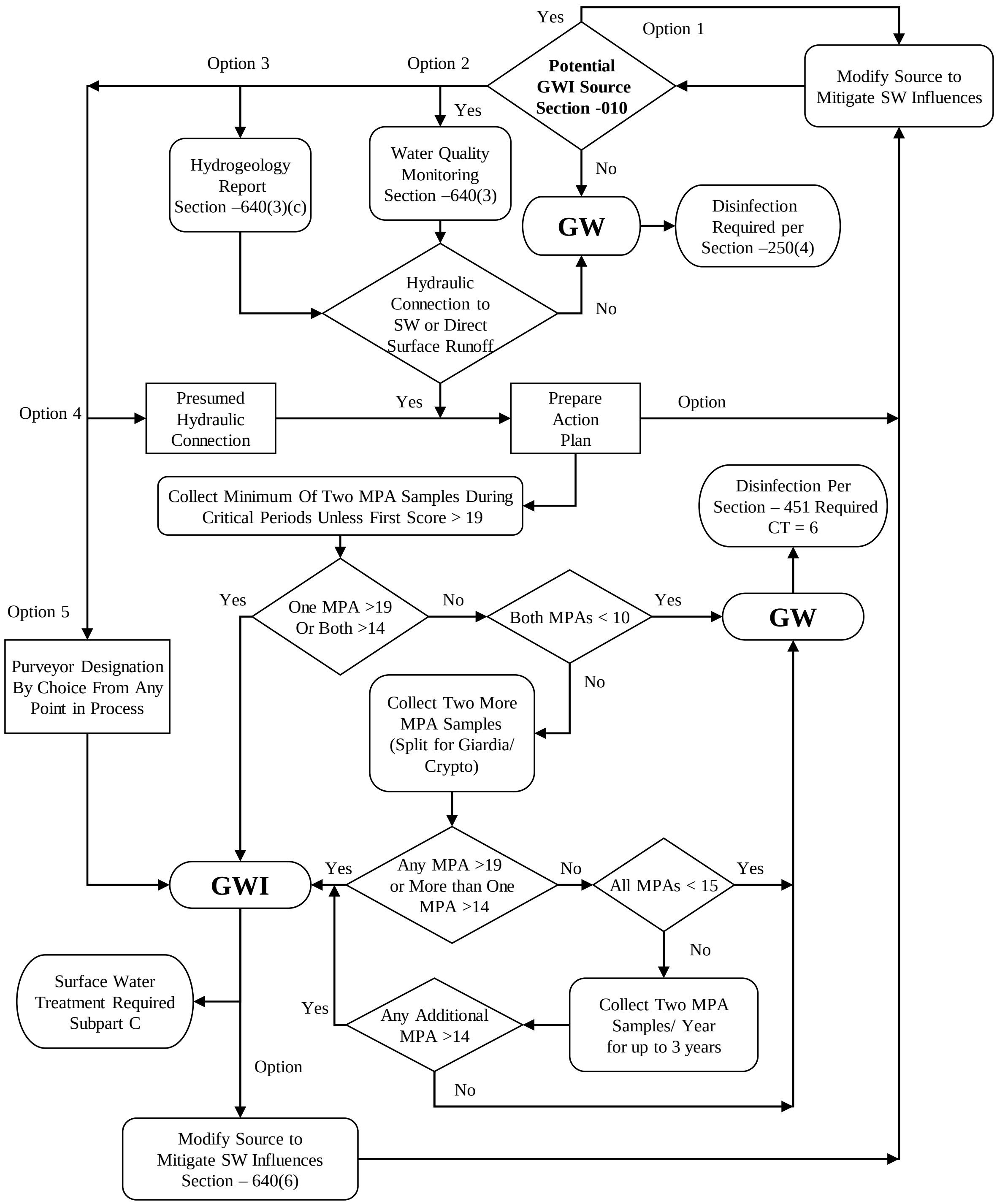
Acronym Key for Figure 5-1

CT	Chlorine Concentration x Time
GW	Groundwater
GWI	Groundwater under the direct influence of surface water
MPA	Microscopic Particulate Analysis
SW	Surface water

Figure 5-1 Evaluating Potential GWI Sources WAC 246-290-640



Chapter 6: Transmission and Distribution Main Design

6.0 Introduction

The proper design of transmission and distribution mains is vitally important to ensure reliable and efficient conveyance of safe drinking water to consumers. Proper assessment of hydraulic capacity under a variety of flow conditions, facilitating best management practices in the operation of the piping system, and providing for the buried infrastructure’s long-term physical resiliency are the essential objectives for the design of transmission and distribution systems.

Good design begins with good planning. Chapter 3 provides the basis for assessing water demands, and Chapter 4 describes the relationship between water infrastructure, water supply requirements, and water system capacity. The design of the transmission and distribution system must be integrated into the water system’s overall design.

The responsibility of the design engineer does not end with ensuring the system will be capable of delivering the design flow at useful pressure to consumers immediately after construction is complete. Water systems will count on the continuous use of these new pipes for many decades. To promote such longevity, the design engineer should carefully consider selection of pipe material, size, location, and bedding; and evaluate the need for measures such as:

- Cathodic protection and polyethylene encasement to protect against corrosion
- Vacuum relief and surge mitigation to protect against destructive surge forces
- Restrained flexible joint pipe to provide resiliency against ground movement
- Cross connection control at point of service to high health hazard premises (see WAC 246-290-490)

Operators are naturally interested in applying best management practices in the operation and maintenance of their distribution systems. This means that pipeline design should facilitate operators’ efforts, including the provision of:

- Sampling stations at representative locations in the distribution system.
- Isolation valves and looped pipe to facilitate maintenance while minimizing service disruption.
- Flushing facilities and looped pipe needed to minimize water age and enable clearing the distribution system of settled particulates.

This chapter provides guidance on the size, materials, facility location, and other design factors required to achieve the design objectives listed above. Briefly, the following definitions apply to this chapter:

Transmission mains convey water from the source, treatment, or storage facilities to the distribution system.

Distribution mains deliver water to individual customer service lines and provide water for fire protection through fire hydrants, if applicable.

6.1 Hydraulic Analysis

Design engineers **must** use a hydraulic analysis to size and evaluate a new or expanding distribution system (WAC 246-290-230(1)). Typically, all but the simplest distribution systems require a computer model for an accurate assessment. Hydraulic analyses take four steps (Cesario 1995; AWWA 2012):

1. **Collect data.** Hydraulic analysis data include physical data on pipes, pumps, reservoirs and valves, and operational data on flows and facility operations.
2. **Develop the model.** Use the data collected to develop a hydraulic model.
3. **Calibrate the model.** Calibration involves comparing model results with field observations. It is an essential step in developing a useful model (Walski 2000).
4. **Analyze the distribution system.** Use the calibrated model to analyze the distribution system to determine if the existing system can meet minimum pressure requirements; if not, modify the pipe network to determine the improvements needed to meet minimum requirements. Distinct pressure zones should be analyzed separately.

Besides assessing a pipe network's capacity to deliver required flowrates while meeting minimum and maximum pressure standards, a hydraulic model can be used to assess:

- Unidirectional flushing
- Water quality in the distribution system
- Water age
- Water velocity
- Hydraulic transients (water hammer)
- Reservoir siting and optimal geometry

We **require** a detailed hydraulic analysis as part of a system's water system plan (WAC 246-290-100). We may also ask a water system for an analysis on an "as needed" basis (to resolve an operating problem for example) (WAC 246-290-110(2) and (4)(f)). In all cases, minimum pressures **must** be maintained (Chapter 246-290 WAC, Part 3).

6.1.1 Data Collection

The following information is needed to initially construct a model:

- Actual diameter and length of each pipe used in the model. See discussion of "skeletonization" below.
- Pipe type and age (to determine "C" roughness coefficient).
- Parameters that vary with time (pump rates, reservoir levels, discharge pressures, and demand patterns).
- Reservoir geometry and design levels (operating, equalizing, standby, fire suppression, and dead storage elevations) and whether a pressure zone is "open" or "closed" as defined in Chapter 8.
- Schematic of key distribution system elements such as reservoirs and booster pump stations to identify the scenarios that need to be analyzed.

- Pump curves used in hydraulic simulations should represent the actual pump characteristics of the unit. Over time pump impellers wear and can change the pump characteristics. Design engineers should determine if the pump curves are still representative of the installed pumps, or if the curves should be redrawn based on in-service pump flow and pressure testing.
- Operational rules for all major water system components. For example, get answers to these questions:
    - Under what conditions do operators turn on a pump, open or close a control valve, or adjust a pressure-regulating valve?
    - Do reservoir level switches or pressure switches control the pumps?
    - What are the corresponding on-off levels or pressures? Do pumping schedules change to minimize power costs?
    - Are all facilities available, or are some off-line for maintenance or repair?

  These answers are especially critical when running extended-period simulations.
- Node elevations taken from the best source possible (topographic survey, Google Earth). You should usually use ground level elevations at nodes, rather than pipe elevations, since pressure measurements will usually be taken close to ground level. In steep terrain accurately locating the node is critical to an accurate elevation, and therefore model results. A 10-foot variance from actual elevation will result in a nearly 5 psi inaccuracy in results before any other modeling inaccuracies come into effort.

Skeletonization is the deliberate exclusion of distribution piping from the model. This is typically done to simplify model construction and to speed up analysis. With increased computational power, skeletonized models are rarely developed (Speight et al. 2010). Detailed information on skeletonized model criteria can be found in other guidelines (AWWA 2012, EPA 2006). Design engineers should state whether the computer model is skeletonized, and if so provide justification and assessment of the skeletonized model in relation to the needed level of accuracy.

6.1.2 Hydraulic Model Development

Hydraulic model development involves a combination of:

1. Defining the physical attributes – sources, pipes, reservoirs, valves, and pump stations – that comprise the distribution system. See Section 6.1.1.

2. Identifying and allocating customer demand.

Allocating customer demand is probably the most important and difficult part of modeling (Speight et al. 2010). There are many sources of data that are typically used to estimate current and future demand allocation including customer usage records, especially for large customers, distribution system leakage estimates, zoning information, projected land use, and fire flow requirements. From this information, it is possible to estimate maximum day demands, peak hour demands, and fire flow demands and allocate these demands to nodes within the model. For extended period simulation models, diurnal demand curves need to be developed for each pressure zone.

6.1.3 Hydraulic Model Calibration

Calibration is an essential part of developing a useful hydraulic model. The calibration process involves comparing modeled or predicted results with field measurements. This process is necessary for the computer model to provide accurate and reliable results. Calibration is often a trial and error process of adjusting the physical attributes and other information until there is satisfactory agreement between the field data and modeled results. Several reasons for discrepancies between field data and modeled results are outlined below such as:

- Erroneous model parameters (pipe roughness values and node demand distribution).
- Erroneous network data (pipe diameters and lengths).
- Incorrect network geometry (pipes connected to the wrong nodes).
- Errors in boundary conditions (incorrect pressure-regulating valve settings, unknown closed valves, tank water levels, and pump curves).
- Errors in historical operating records (pumps starting and stopping at incorrect times).
- Equipment measurement errors (improperly calibrated pressure gauges).
- Measurement error (reading the wrong values from measurement instruments).
- Field data collection error (moving too quickly from one field point to another without allowing the water system to stabilize between readings).

The level of effort to calibrate the model varies depending upon the end use. The design engineer should identify the end use of the computer model's output and confirm that calibration/accuracy is sufficient for that use. A poorly calibrated model may result in inadequate fire flow, pressure problems, incorrect pipe sizing, or others issues that have significant repercussions.

Design engineers may use various criteria to evaluate model accuracy. The most common are:

- **Absolute pressure difference**. Measured in psi.
- **Relative pressure difference**. Measured as the ratio of the absolute-pressure difference to the average-pressure difference across the water system.

Relative pressure difference is the preferred criterion. Simulations over extended periods involve comparing predicted to observed flow rates, pressures, and tank water levels.

It often takes a repetitive process to eliminate errors, especially when modeling larger water systems. It is most difficult to calibrate very old and corroded distribution systems, and water systems with little or no information.

There are no standard national or industry-adopted criteria for calibrating a hydraulic network model. Engineers can use the references and guidelines in Table 6-1 in the calibration of hydraulic models. See the end of this chapter for recommended references on calibrating network distribution models (AWWA 2012; Bhave 1988; Cesario, 1995; Ormsbee and Lingireddy 1997; Speight et al. 2010; Walski 2000).

When calibrating extended-period simulation (EPS) models, the engineer should start with a steady-state hydraulic analysis for pipe roughness, elevations and demand distribution (Walski et al. 2003). As part of developing an EPS model, engineers will need to develop a diurnal demand curve for the water system or pressure zone(s) they are analyzing. See Section 6.1.5 for information on EPS modeling.

Table 6-1: Industry Criteria for Calibrating Hydraulic Models

Accuracy of Readings	Accuracy of Flow Readings	Reference
Hydraulic grade line of model is within 5 to10 ft. of field data. Water levels within 3 to 6 feet.	N/A	AWWA 2012
± 5% of maximum headloss for 85% of readings ± 7.5% of maximum headloss for 90% of readings	± 5%, where flow >10 % of the total demand	WRc 1989
Predict the hydraulic grade line to within 5-10 ft at model calibration points during peak demands, such as fire flows	N/A	Walski et al. 2003

Note: 2.31 feet of head is equal to 1 psi.

Here are some data issues design engineers ("modelers") should consider in calibrating hydraulic models:

1. Water systems that use a supervisory control and data acquisition (SCADA) system should confirm the calibration of devices used to measure tank levels, pressures, and flows for selected locations.

2. Without a SCADA system, water systems should manually track reservoir levels during flow and pressure tests, paying particular attention to the time they take each level reading.

3. Re-set demand allocation. For small water systems, it may be possible to gather source and individual meter readings before and after flow tests, and estimate the volume used during the tests. A reasonable way to check the model is to impose actual water system demand and flow test data in a simulation, and then compare predicted residual pressures to those actually measured.

4. Because there may be uncertainty about pipe-roughness values in older distribution systems, we recommend that engineers adjust operational, consumption, and network data before they adjust pipe-roughness values. Adjust pipe roughness values for whole classes of pipe (e.g., change C for all cast iron pipe 40 years and older to 75). If

known, the effective pipe diameter (as opposed to the nominal diameter) can be used in very tuberculated pipe. Pipe diameter can vary significantly in older pipe, even within the same pipe, and may be irregular and random due to build-up (tuberculation) or corrosion.

5. Pipe-roughness values significantly affect water system flows and pressures during peak hour demands and fire flows. Procedures for hydrant flow tests are in *Installation, Testing and Maintenance of Fire Hydrants* (AWWA 2006a). Criteria to identify deficiencies in pipe segments are in *Computer Modeling of Water Distribution Systems* (AWWA 2012). They include:
   - Velocities greater than 5 fps
   - Head losses greater than 10 feet per 1,000 feet
   - Large-diameter pipes (16 inches or more) with head losses greater than 3 feet per 1,000 feet

The accuracy of the calibrated model declines as changes occur in the actual water distribution network. The engineer should recalibrate the model whenever major new facilities are added to the network system, the peak hour demand or maximum daily demand exceeds that used in the model, or operational procedures change significantly.

6.1.4 Hydraulic Model Analysis

Engineers can use the calibrated model to analyze the existing distribution system and various scenarios of proposed improvements to arrive at the most cost efficient, effective solution. The hydraulic analysis should clearly identify how the model was developed and calibrated, and summarize the output. The following items should be in the hydraulic model discussion. These items are also in the hydraulic analysis checklist in Appendix A.3.4

1. Develop a diagram showing all nodes (junctions) used and a corresponding written summary of assumed supply and demand flows for each condition that must be evaluated. Larger scale diagram sheets may be necessary to accurately show proper location and functions of all control valves and pump station facilities.
2. Explain all assumptions used for the model, including friction factors for the pipes and operating conditions of sources, storage reservoirs, booster pumps, and valves. For additions to existing water systems, also provide evidence that the computer model results were compared to actual field measurements, and that the model was calibrated accordingly.
3. Using a system contour map, identify the minimum pressure results found at the highest elevations and other critical areas in each pressure zone of the system under flow conditions found in item 5 (below).
4. Enter pump curves for the proposed source and booster pumps into the program to indicate how the system will respond to varying flow conditions.
5. Steady-state flow conditions **must** include each of the following (see WAC 246-290-230(5) and (6)):

a. PHD in each pressure zone and throughout the water system, under conditions that deplete all equalizing storage volume and assume all sources are operating. The resulting pressures **must** meet the requirements listed in Section 6.2.2.

b. Highest demand fire suppression flows, such as commercial zones or industrial complexes (>1,000 gpm fire flows, for example), during MDD. The design engineer **must** evaluate the water system and each pressure zone under conditions that deplete designed fire suppression volume and equalizing storage. The resulting pressures must conform to Section 6.2.2 with respect to values and locations. Systems and zones subject to the Water System Coordination Act **must** be evaluated using the assumption that the largest capacity pump is out of service (WAC 246-293-640; WAC 246-293-660).

   WAC 246-290-230 requires that distribution system design provides adequate capacity under a variety of flow conditions. WAC 246-290-420 requires water systems to operate their distribution systems so that 20 psi is available at the flowing hydrant(s) and positive pressure is maintained at all times under fire flow conditions. This means that design engineers should assess the flow available from a hydrant operating down to 20 psi – even if that flow is greater than required by the local fire authority – on residual pressure elsewhere in the distribution system.

   A fire department may not know the impact it can have at distant points from the flowing hydrant when drafting it down to 20 psi. Design engineers should evaluate the potential that firefighting equipment may cause very low water system pressure at sites distant from the hydrant(s) in use. These low pressures may present a public health concern due to an increased risk for contamination from cross-connections and pathogen intrusion. Options may include discussing water system constraints with fire protection authorities, color-coding fire hydrants to indicate limitations, placing orifice plates or other devices that restrict flow rates, following stringent disinfection O&M procedures after similar events, and informing users of precautions they can take to provide additional protection after a fire flow event.

6. Provide an explanatory narrative to accompany graphic figures. The narrative and figures should address:

   a. Low and high-pressure areas in each pressure zone
   b. Identify whether each zone has adequate equalizing and fire suppression storage
   c. Identify capacity limitations within pressure zones. See Sections 4.4.4.1 and 4.5.4.1.
   d. Corrective measures and demonstration that corrective measures resolve deficiencies identified in the analysis.

6.1.5 Extended Period Simulation

Larger, more complex water systems should consider doing extended period simulation (EPS) (typically a multiple of 24 hours), using model conditions such as ADD, MDD, and a worst-case fire flow event with appropriate hourly peaking factors during the day. These simulations also may be warranted for water systems with limited source capacity and greater reliance on storage facilities to meet demand. Water systems need EPS to understand the effects of changing water usage over time, cycles of draining and filling storage tanks, or the way pumps or valves respond to changes in demand.

As part of developing an EPS model, it is necessary to develop a diurnal demand curve for the water system or pressure zone being analyzed (Cesario 1995). The shape of the diurnal demand curve will vary between water systems and even between pressure zones within a water system. It isn't appropriate to take a diurnal demand curve from a textbook and apply it to an EPS model. Several publications explain how to develop a diurnal demand curve (AWWA 2012; Walski 2003).

6.1.6 Hydraulic Transients (Water Hammer)

If the conceptual design or simple manual calculations do not make the engineer confident that the water system is safe from excessive water hammer conditions, the water system should be further modeled. Furthermore, transmission mains designed to operate at velocities greater than 10 feet per second (10 fps) **must** have a hydraulic transient analysis in conjunction with the hydraulic analysis described above (WAC 246-290-230(9)). Factors that make distribution systems vulnerable to hydraulic transients include:

- Long dead-end pipe segments
- High velocity (greater than 5 ft/sec)
- Pipeline profiles with sharp changes in slope that create high points (AWWA 2012).

There are various computer programs available to the designer. Many programs designed to perform hydraulic analysis also do transient analyses. It is important to select a model that matches the complexity of the facility. During facility start-up, the engineer should verify modeled results by gradually generating more and more severe conditions. This approach can show the water system works, as predicted, prior to generating the worst-case design conditions.

6.2 Sizing Pipelines

When sizing water system mains, engineers should consider many factors including pumping costs, future water system demands, land use, friction losses, flow velocities, and water quality. These factors interrelate, so designers should recognize the influence of each when selecting optimum piping arrangements. Engineers **must** design transmission lines, distribution facilities, water sources, pumping facilities and storage facilities so that, together, they meet minimum demand – including needed fire flow if applicable - and pressure requirements as described in WAC 246-290-230.

6.2.1 Sizing Procedures

Many engineering textbooks, reference books, and design manuals convey procedures for sizing water system distribution and transmission lines. There also are many common computer programs available to aid in the design of complex water systems. We expect engineers to use design procedures consistent with those the professional civil engineering discipline applies and accepts as good engineering practice.

6.2.2 Minimum Size

Engineers **must** use a hydraulic analysis to determine the minimum size of a transmission or distribution main (WAC 246-290-230(1) and (9)). The hydraulic analysis must address the parameters outlined in Section 6.1. In general, the main sizes must be able to provide the flow rates required to serve the anticipated land use near the water system as characterized in the water system plan and the local land use plan. All new and replaced distribution mains **must** be at least 6 inches in diameter, unless a hydraulic analysis justifies another size (WAC 246-290-230(2)).

Any pipeline designed to provide fire flow **must** be at least 6 inches in diameter (WAC 246-290-230(3)). Design engineers **must** consider at least two demand scenarios when using a hydraulic analysis to size water mains and other water system facilities (WAC 246-290-230(5) and (6)).

- **First**, the water system **must** be able to deliver the peak hourly demand at the required pressure of 30 psi at every existing and proposed service connection.
- **Second**, if the water system provides fire flow, the distribution pipelines **must** be able to deliver the maximum day demand (MDD) rate, in addition to the needed fire flow, at the required pressure of 20 psi throughout the distribution system.

Design of transmission mains must provide a minimum of 5 psi at the ground surface above the pipeline under maximum design flow conditions, except when the transmission main is located adjacent to a reservoir and the normal operation of the reservoir provides less than 5 psi.

6.2.3 Peak Hourly Demand

Distribution pipelines **must** be able to deliver enough water to meet peak hourly demand (PHD) at 30 psi at every existing and proposed service (WAC 246-290-230(5)). PHD is the maximum rate of water use expected to occur in a defined service area, excluding fire flow. Unless there are accurate water demand records identifying PHD, the designer should use the equations in Chapter 3 to estimate PHD. If there is more than one pressure zone, the engineer must estimate PHD separately for each zone.

6.2.4 Fire "Suppression" Flow

The local fire protection authority or county fire marshal usually determines minimum fire flow requirements (WAC 246-290-221(5)). The design engineer should always confirm the fire suppression requirements associated with a given water system design with the local fire protection authority. Where fire suppression is required, it is usually fire suppression flow plus maximum daily demand that controls the sizing and layout of distribution systems. That's why it's so important to confirm the fire flow requirements.

Typically, the fire protection authority is the town or city fire chief, or county fire marshal in unincorporated areas. Some incorporated areas may contract for fire protection services with a district or the county. Local fire protection agencies may reference standards established by the Insurance Service Office (ISO). As an example of these standards, ISO's 2014 *Guide for Determination of Needed Fire Flow* standards for 1- and 2-family dwellings not exceeding 2 stories in height for a duration of 1 hour:

DISTANCE BETWEEN BUILDINGS	NEEDED FIRE FLOW
More than 30 feet	500 gpm
21 – 30 feet	750 gpm
11 – 20 feet	1,000 gpm
0 – 10 feet	1,500 gpm

The design engineer should discuss optimal fire hydrant spacing with the utility and local fire officials. Office of the State Fire Marshall and Washington Surveying and Rating Bureau contact information can be found in Appendix C.

6.2.5 Minimum Distribution System Pressure

New water systems and additions to existing water systems **must** be able to provide PHD at no less than 30 psi at all service connections throughout the distribution system when all equalizing storage is depleted (WAC 246-290-230(5)). The water system **must** meet this minimum pressure at all existing and proposed service meters or along property lines adjacent to mains if no meters exist.

Many utilities recognize the 30 psi standard is not optimal for modern appliances and sprinkler systems. Design engineers should check performance standards with the local water utility, as local standards may be more stringent. At 10 gallons per minute, the friction loss through a typical 5/8-inch meter and ¾-inch service line from water main to a home (assuming the total distance is 60 feet) is over 10 psi. Assuming 5-10 feet of elevation gain from the water main to the first floor of the home, only about half of the 30 psi at the water main is available for use inside the home.

During fire suppression events, the water system **must** be able to provide 20-psi minimum pressure at ground level at all points throughout the distribution system. The water system **must** be able to provide this minimum pressure under fire-flow conditions plus the MDD rate when all equalizing and fire flow storage is depleted (WAC 246-290-230(6)).

We may allow water systems to use individual-service booster pumps that are owned and controlled by the system as an interim solution to provide minimum design pressure, but they are not acceptable as a permanent design feature. See Chapter 8 for specific design guidelines on individual-service booster pump stations.

6.2.6 Maximum Velocity

We recommend a maximum velocity of no more than 8-feet per second (fps) under PHD conditions, unless the pipe manufacturer specifies otherwise. Maximum velocities greater than 8 fps may occur under fire flow conditions, for short main sections, or piping in pump and valve station facilities. Excessive velocities may reduce pipe service life, cause excessive energy consumption, and increase the risk of damaging hydraulic transients. In addition, unplanned high velocity flow may scour interior pipe surfaces and cause water quality problems for consumers.

Engineers should conduct a hydraulic transient (water hammer) analysis for distribution piping designed to exceed 5 fps during PHD or fire flow conditions (AWWA 2012), and **must** do so when a transmission main is designed to operate at 10 fps or more (WAC 246-290-230(9)). See

Section 6.1.6 for a discussion on modeling transient conditions and Section 6.2.8 for surge control.

6.2.7 Excess Pressure

When designing a water main, it is important to consider the type of pipe used and the pressure needs of the water system. Excessive water system pressure can increase the risk of pipe failure and result in increased distribution system leakage. Distribution system pressure should not exceed 80 psi, unless the design engineer can justify the need for higher pressure (to reduce pumping costs, increase fire flow reliability, or for other reasons), and verify that the pipe material is appropriate for this use. See Section 6.4.8 for recommendations on individual pressure-reducing valves.

6.2.8 Surge and Transient Control

Many factors influence hydraulic surges and transient conditions (water hammer), including main size, length, profile and construction materials. See Section 6.1.6 for an analysis of transient conditions. Engineers should base pipe pressure tests and thrust restraint on the maximum transient conditions, including an appropriate safety factor.

We recommend water systems consider installing facilities that enable real-time monitoring of distribution system pressures to better understand the occurrence of hydraulic transients and the associated risk of contamination via backsiphonage. Continuous pressure monitoring and SCADA reporting can be done from booster pump stations, reservoir vaults, and PRV vaults.

There are many ways to provide surge control, including:

- Open surge tanks and pressurized surge tanks.
- Surge anticipator valves, vacuum relief valves, and regulated air release valves.
- Optimize the main size and alignment.
- Electric soft-start or -stop and variable speed drives for pumps.
- Electric interlocks to prevent more than one pump from starting at the same time.
- Slow opening and closing valves.
- Increase the polar moment of inertia of the rotating pump or motor assembly.

It may be necessary to combine methods. Engineers should take care to avoid a hydraulic –prevention strategy from causing a secondary water hammer equal to or worse than the original design.

Reliability of the surge protection facility is important. If appropriate, the design should provide redundancy for essential equipment such as vacuum relief valves. Surge tanks and similar components should have early warning alarms to notify operators. The design should not allow the pumping system to operate if the surge protection facilities are not operable.

6.2.9 Assessing Water Quality Impacts on the Existing Distribution System

Changes in the physical or chemical environment in a distribution system may destabilize tubercles and introduce their chemical and biological communities into the water column. In Chapter 5 (Section 5.3) and Chapter 10 (Section 10.1.3) we describe the need to assess the impact of adding a new source or treatment process on distribution system water quality due to concerns that chemical changes to the water in the distribution system (pH, dissolved oxygen, oxidation/reduction potential) may destabilize the tubercles, thus suspending corrosion byproducts, adsorbed metal ions, and biofilm/microbial contaminants.

For the same reasons, design engineers should assess the impact associated with replacement or addition of distribution system water mains on flow direction and velocity. These physical changes can shear tubercles from pipeline inner walls, resulting in suspension of chemical and biological contaminants into the distribution system. Existing pipe segments found to be vulnerable to tuberculation shearing should be cleaned prior to activating the new pipeline(s).

6.3 General Design Considerations for Mains

The location, depth, pipe material, and bedding must be carefully considered from the perspective of perpetual access, resiliency, and physical protection of the pipeline. These basic design elements are all the more important since the pipeline is buried and out of sight for its entire service life.

6.3.1 Installation

Pipelines should be laid in a public corridor and installed according to established standards such as those from the Washington State Department of Transportation (WSDOT/APWA Division 7). You can also use the information in and reference latest edition of AWWA Standards such as C651 (Disinfection); C600 (DI pipe installation); C605 (PVC pipe installation); and ASTM standards such as F2620 (HDPE joining).

You may also find the following AWWA Manuals of Practice useful in preparing construction documents for the installation of transmission and distribution mains:

- AWWA M9 - Concrete Pressure Pipe (AWWA 2008)
- AWWA M11 - Steel Water Pipe – A Guide for Design and Installation (AWWA 2004)
- AWWA M23 – PVC Pipe Design and Installation (AWWA 2002)
- AWWA M41 – Ductile-Iron Pipe and Fittings (AWWA 2009a)
- AWWA M55 – PE Pipe Design and Installation (AWWA 2006b)

6.3.2 Depth of Pipe Burial

Pipes should be buried below the frost line seen in the most severe winters; otherwise, they should be protected against freezing. When determining proper depth, engineers should evaluate temperature variations in the area, especially in Eastern Washington and mountainous areas. The minimum fill depth over the top of the pipe is usually 36 inches. The design engineer may justify another depth (for example, to avoid underground obstructions or rocky conditions).

If providing less than 36 inches of cover, the following should be considered and documented in the project report:

- Pipe load and pipe strength
- Freeze protection
- Vulnerability to damage from future excavation

6.3.3 Special Design Considerations

The design should protect pipes above ground from freezing (such as bridge crossings) and secure pipes at river crossings or subject to tidal action. The engineer should consider:

- **Pipe thrust restraints** whenever pipelines leave the soil.
- **Underground thrust blocking** whenever a pipe changes direction (such as a bend) or unbalanced thrust forces exist (pressure and momentum).

6.3.4 Separation from Nonpotable Conveyance Systems

Appropriate separation between potable and non-potable pipelines protects public health and safety. Pipeline failure or leaks can result in pipeline contamination that increases risks public health and safety. Non-potable conveyance systems include piping that carries:

- Sanitary or industrial sewage
- Reclaimed water
- Irrigation supply from non-potable sources
- Storm drains
- Petroleum products (oil, refined products)
- Natural gas

We do not consider a driveway culvert a non-potable pipeline requiring special consideration.

Backflow of leaked content from a non-potable conveyance system, or complete pipe failure of non-potable piping can result in contamination of the potable water system. Pipeline designers can increase potable pipeline reliability through the proper selection of pipe materials, wall thickness, pipe joint and thrust restraint systems, pipe bedding, and internal and external corrosion control, and adequate separation between pipelines

Adequate separation minimizes incidental damage during the repair of potable water lines and other pipelines during repair or replacement of either one. Adequate separation also assures sufficient room to repair leaks and replace broken sections. Finally, separation reduces the potential for pipeline failure caused by a leak or failure of its neighboring pipeline.

The following recommendations apply to pipelines of 24-inch diameter or less. Larger pipelines create more concerns and, therefore, require additional consideration.

Standard industry guidance calls for a minimum 10-foot horizontal separation between the outer walls of potable and non-potable pipelines in parallel installations, and a minimum 18-inch vertical separation (potable water line above) between the invert of the potable water line and the crown of the non-potable line at pipeline crossings (10-State Standards, WAC 246-290-200).

We recognize that actual conditions can make it impossible to comply with these standards. If the design of new or replacement water main cannot provide the standard 10-foot horizontal separation, design engineers should consult *Pipeline Separation Design and Installation Reference Guide* (WSDOE and DOH 2006). Provisions allow for parallel potable and non-potable installations to be as close as 4-feet horizontally provided certain conditions are met. We expect design engineers to provide justification, and demonstrate that the conditions for a pipe separation less than 10-feet are met in the project report.

If the potable line is closer than 18 vertical inches from the nonpotable line at the point of crossing, or the potable line must cross under the nonpotable line, the potable line should be encased with ductile iron or steel pipe designed to withstand a minimum static pressure of 150 psi extending at least 10 feet to either side of the crossing, with the annular space within the casing pipe pressure-grouted.

If the water utility receives permission to do so, mitigation of close parallel or crossing installations may be applied to the non-potable line. This may include encasement of the non-potable pipe with pressure grouting as described above, or placement of concrete or controlled density fill encasement of the non-potable line. Project-specific information and justification, permission to work with the non-potable pipe, and appropriate direction in the utility's pipe repair SOPs should be included in the project report.

6.3.5 Separation from Other Potential Sources of Contamination

Design engineers should thoroughly evaluate water main installations on a case-by-case basis if they are near other potential sources of contamination. This may include a facility if a failure at the facility would subject the water in the main to toxic or pathogenic contamination. Other potential sources of contamination include storage ponds, land disposal sites for wastewater or industrial process water containing toxic materials or pathogenic organisms, and solid waste disposal sites.

Water mains should not be installed closer than 10 feet from a septic tank, drain field, or any other on-site wastewater treatment/disposal component. The measurement should be made from the outer wall of the water main to the outer boundary of the drain field bed or exterior face of any other on-site component.

Design engineers and water systems should take precautions before selecting materials for a pipeline in an area with soils known or suspected to be contaminated by lower-molecular-weight organic solvents or petroleum products. Certain pipe materials, especially polyvinyl chloride (PVC), polyethylene (PE and HDPE), and polybutylene (PB), are susceptible to permeation by such contaminants (Holsen et al. 1991; Ong et al. 2008, Cheng et al. 2012). Elastomeric gaskets made of ethylene propylene diene monomer (EPDM) used to join ductile iron pipe are susceptible to permeation as well. However, nitrile-butadiene rubber (NBR) is resistant to permeation by organic solvents and petroleum products, so ductile iron pipe with these types of gaskets should be used if potential permeation is an issue (Cheng et al. 2012).

Designers and water systems should ask the pipe manufacturer about the risk of permeation of pipe walls and jointing material in such areas.

6.3.6 Pipe Materials

Various materials are available for distribution and transmission pipes. Engineers base their material selection on factors such as life-cycle cost (capital and maintenance), reliability, special design considerations, utility preference or familiarity, conformance with existing materials, and certification under ANSI/NSF Standard 61. The design engineer **must** use established standards, such as AWWA or the American Society for Testing and Materials (ASTM), when justifying the class of pipe selected (WAC 246-290-200(1)).

Any selected material that will have substantial contact with drinking water supplies **must** be certified to meet ANSI/NSF Standard 61 (WAC 246-290-220(1)). This applies to coatings, liners, or any joining materials used. "Substantial contact" means the potential for contaminants to enter the drinking water. Factors to consider are the total area of exposure, volume, length of time water is in contact with material, and level of public health risk.

6.3.7 External Corrosion Control

Engineers should consider protection from external corrosion in areas where corrosive soils are prevalent or when pipelines are exposed. This protection is especially true for bridge crossings in salt-water (coastal) environments or other harsh environments. This protection may also be necessary in colder locations where salt is used to de-ice roads.

Engineers should also evaluate and, if appropriate, protect metal pipes from corrosion due to stray electrical currents in the soil. This usually occurs when metal pipes are near or cross major oil or natural gas pipelines protected by impressed current.

The AWWA Manual of Practice M27 on external corrosion control has detailed information on:

- Assessing the potential for corrosion of buried and exposed water mains
- Protection of buried water mains
- Material selection of coating systems for exposed pipes (AWWA 2014a)

6.3.8 Location of Pipes in Geologically Vulnerable Areas

Earthquakes and landslides have caused water mains to fail leading to depressurization of distribution systems, boil water advisories, and significant service disruptions (Tanaka 1995; Ballantyne et al. 2009; WMD-EMD 2014). To meet state and local requirements, engineers **must** address geologic risk (seismic and unstable slopes) when designing water systems (WAC 246-290-200) in both the assessment of existing water mains and installation of new water mains.

Priority should be given to establishing earthquake-resilient transmission and distribution systems supplying medical facilities; power plants; fuel refining, storage, and distribution facilities; food production, storage, and distribution facilities; emergency response command and communication centers; and emergency shelters.

Seismic risk may be reduced or mitigated by:

- Installation of water mains constructed with seismically resistant pipe and pipe joints, enabling substantive pipe joint deflection and capacity to longitudinally expand and contract without joint or pipe failure
- Knowledge of soil liquefaction potential
- Redundant facilities and/or looped piping
- Appropriate valving to isolate vulnerable areas
- Installing pipe within a reinforced pipe tunnel
- Polyethylene encasement

The Washington State Department of Natural Resources has geologic hazard maps that can be used to identify seismic and other geologic hazards. Some pipes, such as butt-fused HDPE (AWWA 2015a), molecularly oriented PVC (AWWA 2009b), and seismic joint ductile iron pipe, are much less prone to failure in earthquakes and landslides (Water Supply Forum 2015). You can also consider using specialized, flexible expansion joints that can accommodate significant ground motion, especially near where water mains enter structures such as reservoirs and booster pump stations.

There are design guidelines available from the American Lifelines Alliance that can be useful in selecting and designing water mains in areas with significant geologic risk (ALA 2001; ALA 2005). In areas with the potential for significant ground motion, you may need to seek the services of a qualified geotechnical engineer or other professional qualified to assist in the selection of materials and other aspects of design.

6.3.9 Layout of Mains

Water mains should be planned and designed in segmented grids and loops located in the established right-of-way or utility easement. Distribution mains should be looped, if possible, to avoid as many dead ends as possible. Looping may not always be practical due to topography, geology, pressure-zone boundaries, unavailable easements, or locations of users. If water systems cannot avoid dead ends, they should provide blow-offs to allow adequate flushing of those mains. See Section 6.4.3.

6.3.10 Protection Against Cross-Connection

Water systems are required to protect their distribution systems from contamination through cross-connections with any source of non-potable liquid, solid, or gas that could contaminate the potable water supply by backflow. Design engineers should incorporate provisions that enable their water system clients to meet their regulatory obligations and to follow best management practices upon completion of the project. Backflow assemblies should be installed so that they can be readily tested, inspected, and maintained. Reduced pressure assemblies should not be installed in places that are vulnerable to flooding such as underground vaults. Definitions, descriptions, and best practices for cross-connection control (CCC) are described in detail in the following manuals;

- *Manual of Cross-Connection Control (USC FCCCHR 2009).* See Appendix C for USC Foundation for CCC contact information
- *Cross Connection Control: Accepted Procedure and Practice Manual (*PNWS-AWWA, 1996).
- *Cross-Connection Control for Small Water Systems* (WSDOH 2004).
- *Recommended Practice for Backflow Prevention and Cross-Connection Control* (AWWA 2015b).

It is important to determine CCC requirements during the planning phase of any water system project to avoid the expense and difficulty of retrofitting an existing facility or device to accept backflow protection. The water system's Cross-Connection Control Specialist (CCS) should be consulted to be sure CCC requirements are properly addressed in the design.

Basic CCC design considerations include:

1. Increased headloss. Backflow prevention assemblies increase headloss. The headloss curve for a backflow prevention assembly can be found in the manufacturers' assembly specifications.

2. Premises listed in Table 9 in WAC 246-290-490(4). These facilities are considered high health cross-connection hazards. The design engineer should consult with the water system to determine if any existing Table 9 premises will be connected to a new water main and, if so, consult with the CCS on the appropriate CCC strategy for each such prospective service connection.

3. Cross connection requirements within water treatment facilities (see Section 10.9).

4. Premises served by rainwater collection systems, private wells, reclaimed water, or any other non-potable supply (see Section 5.10.1).

5. Overflow and drain pipes from storage tanks (see Section 7.4.3 and 7.4.4).

6. Pump-to-waste and air-vacuum relief valve discharge. Pump to waste must be fitted with an appropriate air gap.

7. Individual booster pumps. Design engineers must ensure that the appropriate facilities are designed to ensure individual booster pumps do not distribution system water quality (see Section 8.4.1).

All backflow assemblies relied upon to protect the public water system must be listed on the department's approved assemblies list developed by the USC Foundation for Cross-Connection Control and Hydraulic Research.

6.4 Appurtenant Design Considerations

The selection and location of pipeline appurtenances such as valves, instrumentation, flushing and sampling stations enable operators to optimally manage the distribution system and maximize consumer value. Engineers should consider the following as part of the overall distribution and transmission main appurtenant design.

The State Building Code Council (SBCC) administers our state's code adoption process. Contact information for SBCC can be found in Appendix C.

6.4.1 Valves

The placement of valves should be sufficient to minimize the number of customers out of service when the water system must isolate a location for maintenance, repair, replacement, or additions. Spacing of distribution system isolation valves should be limited to 800 feet (AWWA 2008) unless the grid geometry or low density justify greater spacing.

6.4.2 Vacuum Relief and Air Release Valves

The engineer **must** ensure the distribution system is protected from backflow contamination as a result of the intended operation of the vacuum relief or combination vacuum/air relief valve (WAC 246-290-490). The design should not provide a pathway for distribution system contamination; for example, via backsiphonage from an air-vacuum relief valve whose vent is located inside an un-drained pit or located in a pump house drain line. The vent on these valves should be equipped with an appropriate air gap above the highest possible water level. Pathways for introduction of contaminated water through back siphonage must not be created.

High points of distribution or transmission lines where air can accumulate should have a means of venting the air. Venting options include an automatic air release valve, combination vacuum relief/air release valve, or manually operated devices. The use of a manual air relief valve or other manual means of venting air (fire hydrant, flushing hydrant, some types of service connections) is recommended wherever possible in lieu of an automatic valve. (See 10-State Standards). The discharge from any air relief must be protected from back siphonage by an appropriate air gap.

Vacuum relief may be necessary at any point along a water pipeline where column separation is possible as a result of a negative pipeline transient pressure wave. See Sections 6.1.6 and 6.2.8. Such events may be caused by a sudden increase or decrease in flow. The location where a vacuum relief valve is needed may be near or far from the cause of the transient wave (e.g., sudden booster pump station or source pump failure, rapidly closed valve, large and sudden pipeline break).

Vacuum relief may be necessary near a reservoir. Isolation of a system's or pressure zone's only reservoir may necessitate the installation of a vacuum relief valve on the system side of the reservoir isolation valve. See Chapter 7. Vacuum relief is also necessary in support of vertical turbine pump operations. Shut down of a vertical turbine pump results in a fall of water out of the pump column, and air must be introduced on the pump side of the pump check valve to allow the pump column to return to atmospheric pressure.

If a valve is installed in a vault, the vault should be rated for appropriate vehicular loading whenever there is any possibility of traffic around the vault. The interior of the vault should provide at least a two-foot clearance around the valve. The air inlet and discharge vents should be located outside the valve vault at least 18 inches above finished grade.

Each vent (certain valves may have multiple vents) should have a screened downward-facing vent opening. Proper drainage away from the vent outlets is necessary. During valve operation some water discharge will occur through the vent. If the internal valve does not seat properly, there will be continuous water discharge. If the design calls for any vent opening inside the vault, including a drilled weep hole on the vent riser and the vent pipe outlet, the following criteria applies to the vault and the location of the vent opening.

1. The vault has a daylight drain that can be bore sighted to a discharge point above grade and above the maximum flood level. The drain must be sized to discharge the flow of water that potentially could be discharged from the relief valve port.

2. Each vent opening is installed at least two vent pipe diameters above the crown of the vault daylight drain.

3. Groundwater is prevented from entering the vault.

6.4.3 Flushing Valves, Blow-offs and Hydrants

To allow sufficient flushing and proper disinfection of distribution mains, engineers should install blow-offs, automatic flushing stations, or hydrants at low points and dead-ends in the distribution system. They should be designed to achieve a minimum velocity of 3.0 fps in the main for scouring purposes. To meet these criteria, small water systems with larger pipes may need to consider design allowances that enable them to add temporary pumping or storage facilities.

6.4.4 Fire Hydrants

You should check with the local fire protection authority to make sure that the make and model of any proposed fire hydrant is acceptable since there are multiple different types of dry barrel fire hydrants (AWWA 2005b; AWWA 2006a). The Water System Coordination Act defines standard fire hydrants (WAC 246-293-650(3)):

> "All fire hydrants shall conform to American Water Works Association specifications for dry barrel fire hydrants. Each hydrant shall have at least two hose connections of 2½ inches diameter each and one pumper connection."

New hydrants must be installed off mains at least 6-inches in diameter. Existing hydrants connected to a water main less than 6 inches must be approved by, and the hydraulic analysis must demonstrate the flow available at the hydrant meets the standards of, the local fire protection authority (WAC 246-290-230(4)). Residual pressure requirements described in WAC 246-290-230 (6) apply to hydrants installed on undersized water mains.

Other types of “hydrants” not designed to provide fire flows, such as flush valves, standpipes, blow-offs, or nonstandard, smaller volume hydrants without pumper ports may be placed on smaller mains (less than 6 inches in diameter).

Designers should provide all fire hydrants with their own auxiliary gate valve. Auxiliary gate valves are a safety item on hydrants, and most, if not all, utilities require them.

6.4.5 Sampling Stations

Every purveyor is required to develop and follow a coliform monitoring plan. See coliform monitoring plan guidance: DOH 331-036 and DOH 331-240. Design engineers should consider the location of sampling stations that provide the purveyor the ability to collect representative water quality samples from the distribution systems. In general, we recommend the following sampling station features:

1. Use distribution piping, not household plumbing.
2. The sampling location should be in an active part of the distribution system.
3. The water system should have control (ownership) of the location or sample station.
4. The sample tap should be located in a lockable enclosure and be otherwise protected from the weather and tampering.
5. A dedicated standpipe with a smooth-nosed sample tap is preferable.

To protect the sample against potential contamination and false positives, engineers should not use stop-to-waste designs without first considering operations and maintenance, drainage, and security. Designs should provide adequate protection from freezing such as the use of connections for a hand evacuation pump or the use of continuous flow sampling stations. See Section 5.2.6 for more information on desired sample tap attributes.

6.4.6 Yard Hydrants

Water systems may not install yard hydrants under their direct control that drain the riser into the ground for any purpose without appropriate cross connection control installed to protect the distribution system from contamination. The riser weep hole drain presents a risk of contamination to the distribution system through a cross connection with contaminated groundwater. If you choose to use a yard hydrant without cross connection control protection, the *Uniform Plumbing Code* requires use of a model that does not drain into the ground. Yard hydrants that conform to *American Society of Sanitary Engineering Standard 1057* (ASSE 2012) are acceptable because they do not drain into the ground.

6.4.7 Angle, Curb or Meter Stops

Separate angle, curb, or meter stops should be installed for each individual service connection. They allow water systems to close individual customer connections temporarily without interrupting service to other customers. Supplying multiple water service connections from a single tap on the main is not recommended.

6.4.8 Individual Pressure-Reducing Valves

When a water system anticipates pressure in the mains will exceed 80 psi, the system is responsible for recommending that customers install and maintain an individual pressure-reducing valve (PRV) as described in the *Uniform Plumbing Code*. Water systems should not install a PRV for an individual customer unless they have a written agreement with the customer showing who is responsible for required PRV maintenance, repair, or replacement. The design engineer should check for local ordinances or service agreements on PRV use.

6.4.9 Automatic Control Valves

The most common type of automatic control valve used in a distribution system is a pressure reducing valve. Other types include altitude valves (usually associated with an atmospheric reservoir) and combined pressure reducing/pressure sustaining valve (used when maintenance of upstream pressure to a pre-determined minimum is more important than delivering all the flow needed downstream of the valve).

These valves are typically installed in underground vaults that are not drained, and therefore may be subject to flooding from high groundwater or storm water infiltration. Designers should ensure installations that include any atmospheric vents installed as part of the valve's hydraulic control system are protected from backsiphonage. See Section 6.4.2 for vent, vault, and drain standards.

6.5 Construction Documents for Pipelines

In most instances, construction documents – comprised of construction drawings and specifications - intended for construction of water system infrastructure **must** be submitted to us for review and approval before construction begins (WAC 246-290-120). See Chapter 2. Construction documents comprise all the information the contractor needs to construct the improvements. Nothing of consequence should be left out of construction documents and left to the assumptions of the contractor. We will not approve construction documents unless they provide a sufficient level of detail to inform the contractor of his requirements with respect to:

- Approved materials
- Sequence, location, and orientation of construction
- Testing requirements
- Submittal requirements for owner's approval

More specific guidance on information that should be provided in construction documents for pipelines is included in the checklists in Appendix A.3.3.

Observation of construction by the water system's representative, including pipe and appurtenance handling; trench excavation, preparation, and backfilling; separation from other utilities; thrust restraint; disinfection; and testing are important to ensuring successful operation of the finished pipeline. Upon completion of construction, the water system is obligated to submit to ODW certification by a licensed professional engineer – typically the design engineer

– that construction was completed in accordance with the approved construction documents as described in Section 6.6.

6.5.1 Construction Specifications for Pipelines

Construction specifications **must** meet commonly accepted technical standards such as AWWA, WSDOT/APWA specifications or equivalent (WAC 246-290-200(1)(d)). Attention to detail is important to ensure the identified specifications include all required information. The referenced specifications may require some case-by-case determinations. Specifications **must** thoroughly describe the materials, means, and methods for satisfying the requirements and conditions of the project (WAC 246-290-120(1)). Department of Transportation contact information can be found in Appendix C.

Water systems may include standard construction specifications in their water system plan or make them available as a separate document. The standard specifications should include materials and construction/installation details the utility considers standard for water system construction and maintenance.

6.6 Placing a Water Main into Service

Before a new transmission or distribution main can be placed into service, it **must** be properly inspected, disinfected and tested (WAC 246-290-120(4)). The WSDOT/APWA standard specifications (Division 7) and *AWWA C651 - Standard for Disinfecting Water Mains* are commonly used to define pressure, leakage and disinfection standard practices (WSDOT/APWA 2016; AWWA 2014b). The specific standards used for the project should clearly identify:

- Inspection and flushing requirements
- Pressure and leakage testing methods
- Disinfection and bacteriological testing methods

Only after a water main has been flushed and properly disinfected, testing results show that the water from it is safe to drink, and the engineer in charge of the project submits to the department a Construction Completion Form may it be placed into service (WAC 246-290-120(5); WAC 246-290-125(2)(b)). To ensure that meaningful bacteriological results are obtained, collect coliform samples after the water main has been flushed and chlorine residuals have returned to background levels. For water system with a current, approved water system plan that includes standard construction specifications for distribution mains, design engineers can use the *Construction Completion Form for Distribution Main Projects* (DOH 331-147) and keep it on file. For all other projects, design engineers **must** submit a complete *Construction Completion Form* (DOH 331-121) to the department.

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Chapter 7: Reservoir Design and Storage Volume

7.0 Introduction

Adequate finished water storage provides multiple advantages to a water system. Storage can reduce excessive pump cycling; reduce sizing requirements for sources of supply, treatment works, and transmission/distribution piping; and provide a reserve for fire-fighting and continued pressurization of the system despite a temporary loss of supply. Water reservoirs discussed in this chapter operate at atmospheric pressure. See Chapter 9 for pressurized storage guidance.

The objective of reservoir design is to provide the water system with adequate and resilient water storage facilities that protect the quality of stored water. Public health protection depends on a thorough consideration of each reservoir design element, adhering to appropriate construction standards, and implementing best reservoir management and operating practices.

Washington's sanitary surveys have historically revealed defects in reservoir design, construction, or maintenance that threaten the safety of the drinking water supply. The same observations have been made throughout the country. In 2015, EPA indicated that they may amend the Revised Total Coliform Rule with specific finished water storage facility inspection requirements (USEPA 2015). At this time, the issue of regulating storage reservoir maintenance remains undecided, but the driving force behind it is clear: Reservoirs that are poorly designed, constructed, and maintained present a significant risk for distribution system contamination.

This chapter is intended to provide guidance for:

- Reservoir Sizing (Section 7.1)
- Geometry, elevation, and integration with existing and future facilities (Section 7.2)
- Location and site considerations (Section 7.3)
- Construction materials and design elements (Section 7.4)
- Operational constraints and considerations (Section 7.5)
- Reservoir water quality and sampling access (Section 7.6)
- Placing a reservoir into service (Section 7.7)

We do not intend to establish any particular reservoir design approach. See the references at the end of the chapter for more information on reservoir design (AWWA 2008, 2013; Ten State Standards 2012; Kirmeyer et al. 1999; Martel et al. 2002; Walski 2000).

7.1 Reservoir Sizing

Appropriate planning for changes in population and land use ensures the reservoir under design provides the level of service expected from the community for the life of the facility. Storage facilities are designed to serve the needs of the community for a planned number of years, or to accommodate full water system build-out (for a particular subdivision, planned development, or as a condition of plat approval). The design life for properly maintained concrete and steel storage tanks is at least 50 years.

The following design guidance applies to the specific area(s) directly served by the reservoir under design. This area may be the entire water system where only one reservoir is intended to serve all customers, or it may be a discrete pressure zone. For the purpose of this section, “supply pumps,” “supply,” and “source” are defined in Section 7.1.3.

7.1.1 Storage Components

The design engineer **must** consider the five storage components discussed in Section 4.4.3 and listed below (WAC 246-290-235(3)):

1. Operational storage (OS).
2. Equalizing storage (ES).
3. Standby storage (SB).
4. Fire suppression storage (FSS), if applicable.
5. Dead storage (DS), if applicable.

Figure 7-1 illustrates and Table 7-1 describes a typical cross-section of reservoir storage components.

7.1.1.1 Operational Storage

Operational storage (OS) supplies the water system while the pumps supplying the reservoir are in “off” status (WAC 246-290-010). This volume will vary according to:

1. The sensitivity of the water level sensors controlling the supply pumps.
2. The geometry of the reservoir between the designated pump-off and pump-on water level set points.

Various water level sensors can be used to signal pump-off and pump-on levels, including float switches, ultrasonic sensors, and pressure switches. Some can detect water level changes as small as a fraction of an inch. Others require more than a foot. Tank designers must account for the type of level sensor they used to determine the vertical dimension needed for proper operation of the device.

The OS volume should be sufficient to avoid source of supply pump cycling in excess of the pump motor manufacturer's recommendation. In general, design engineers should limit the supply pump motors to no more than six starts per hour unless the pump motor manufacturer permits more frequent cycling. To limit pump starts to no more than six per hour, OS volume can be conservatively calculated as the pump supply capacity (in gpm) times 2.5 minutes. Typically OS is substantially smaller than the remaining volume of the tank.

Operational storage volume is not applicable to:

- Service capacity analysis (see Chapter 3).
- Water systems operating under a continuous supply mode (see Section 7.1.1.2).

Other general design considerations for operational storage:

- To reduce disinfection by-product formation or address other impacts of extended water age, it may be necessary to adjust pump-on and pump-off levels and/or expand OS as a percentage of total reservoir operational volume.

- To reduce the potential for ice formation, it may be necessary to increase OS to improve circulation. An ice cap inside a reservoir can cause significant structural damage, damage to the internal coating system, and destruction of the level control system.

7.1.1.2 Equalizing Storage

The water system **must** provide equalizing storage (ES) when source pumping capacity cannot meet the peak hourly demand (WAC 246-290-235(4)). New water systems and additions to existing water systems **must** be able to provide PHD at no less than 30 psi at all service connections throughout the distribution system when all equalizing storage is depleted (WAC 246-290-230(5)). The water system **must** meet this minimum pressure at all existing and proposed service meters or along property lines adjacent to mains if no meters exist.

Many utilities recognize the 30 psi standard is not optimal for modern appliances and sprinkler systems. Design engineers should check performance standards with the local water utility, as local standards may be more stringent.

Several factors influence the ES volume, including demand, source capacity, and the mode of supply. Two modes of supply operation are listed below.

1. **Call-on-Demand (common)**
   This mode of operation is most commonly used for estimating ES. For call-on-demand operations, the ES is used to supply the daily peak period of demand. Engineers should use Equation 7-1 to estimate minimum ES requirements unless actual water use records indicate a more applicable volume. Water systems with multiple sources may need to provide ES in excess of Equation 7-1 depending on the mode of operation.

Equation 7-1:

$$\mathbf{ES = (PHD - Q_S)(150\ minutes)}\text{, but in no case less than zero}$$

Where:

ES	=	Equalizing storage component, in gallons
PHD	=	Peak hourly demand, in gpm, as defined in Chapter 3 of this manual
Q_S	=	Sum of all installed and active supply source capacities except emergency supply, in gpm. See Section 7.1.3 for definition of sources as it applies to this equation.

2. **Continuous Supply (unusual)**
   This approach applies to situations in which the reservoir is filled continuously over a period of time that does not necessarily coincide with the peak demand period such as filling a reservoir during the night when the cost of energy can be lower. The volume of ES using this mode of operation can be significant since it is used to meet longer periods of demand. ES sizing with continuous source pumping will require developing a maximum day demand (MDD) diurnal curve for the pressure zone(s) supplied by the reservoir. Diurnal demand varies due to the pressure zone size, season, and type of demand (residential, commercial, industrial, and recreational). After developing the MDD diurnal curve, the design engineer can calculate the required ES by determining the difference between supply and demand. As a general guideline, the volume of ES needed using constant pumping is about 10 to 25 percent of the MDD (Walski 2000).

The engineer may eliminate ES only if the combined capacity of the supply sources meets or exceeds the PHD for the water system, or pressure zone, with 30-psi pressure provided at each existing and proposed service connection.

7.1.1.3 Standby Storage

SB volume is intended to provide continued water supply during abnormal operating conditions such as structural, electrical, mechanical, or treatment process failure; or source contamination (WAC 246-290-420). These standby storage guidelines should be used for community water systems and non-community water systems such as schools and health care facilities where service disruptions would have significant impacts on those served.

The degree to which standby storage is incorporated into reservoir design is a direct reflection of the consumers' expectations of water service during abnormal operating conditions. The water system governing body, representing the interests of the community, is in a position to determine the cost and benefit associated with providing a given level of reliability. SB volume should be sized based on locally-adopted standards for water supply under emergency or abnormal operating conditions as outlined in Equation 7-2.

Equation 7-2:

$$\mathbf{SB = (N)(SB_i)(T_d)}$$

Where:

SB	=	Total standby storage component, or its equivalent, in gallons.
N	=	Number of ERUs based on the ERU_{MDD} value.
SB_i	=	Locally-adopted unit SB volume in gallons per day per ERU (number of ERUs based on the ERU_{MDD} value).
T_d	=	Number of days selected to meet water system-determined standard of reliability.

The lower elevation of the SB component should produce no less than 20 psi at all existing and proposed service connections throughout the distribution system during PHD conditions.

Our recommendation to governing boards and developers is to consider SB volume equal the MDD for the pressure zone(s) served (i.e., T_d =1 day) and adjust SB volume based on factors listed below.

Water systems with a single source

Special consideration of SB volume should be made for water systems with only one source. Temporary loss of a water system's single source leaves only storage as a back-up supply. Design engineers should consider SB volume greater than MDD if the system's only source is vulnerable to flooding or other extreme weather events, extended power failures, or source treatment process failure; or if the transmission main from source to distribution is vulnerable to natural hazards (see Section 7.3.1). Design engineers considering the storage needs of water systems with only one source of supply should perform an all-hazards assessment. This assessment should evaluate the probability and duration of events that may lead to temporary loss of the system's only source of supply.

Water systems with multiple sources

It may be appropriate for design engineers to consider SB volume less than MDD if multiple sources provide mechanical, electrical, treatment, and transmission redundancy, and resilience to a single contamination event. Section 7.1.3 lists what we consider to be continuously available sources of supply for reservoir design purposes.

Design engineers may justify a reduction of SB volume based on one or more of the following:

1. The water system and the local fire authority allow for nesting SB and FSS volumes, where the FSS volume is greater than the SB volume. See Section 7.1.1.4.

2. Two or more sources have permanent on-site auxiliary power that starts automatically when the primary power feed is disrupted. With the largest of these sources out of service, the remaining sources plus SB volume are capable of maintaining at least 20 psi throughout the distribution system under PHD conditions.

3. Sources are located in different watersheds, wellhead protection areas, or aquifers.

4. Converting dead storage to standby storage by providing mechanically redundant booster pumping capacity with permanent on-site auxiliary power that starts automatically when the primary power feed is disrupted.

Even for pressure zones served by multiple sources of supply and with other reliability measures as outlined above, we recommend SB volume of at least 200 gallons per ERU.

Water systems serving non-critical uses

If a loss of water-supply event occurs, certain types of non-community water systems could shut down without affecting public health and welfare. See Section 7.1.5 for additional design guidance for such systems.

7.1.1.4 Fire Suppression Storage

The local fire protection authority or county fire marshal determines a fire flow requirement for water systems. This fire suppression storage (FSS) level depends on the maximum flow *rate* and *duration*. Water systems required to provide fire flow by the local fire authority **must** build and maintain facilities, including storage reservoirs, capable of meeting fire flow requirements while maintaining 20 psi pressure throughout the distribution system (WAC 246-290-221(5)).

For water systems supplied through gravity storage, the bottom of the FSS component **must** be at an elevation that produces no less than 20 psi at all points throughout the distribution system under the MDD rate plus fire flow conditions (WAC 246-290-230(6)).

Water systems with a single source

The minimum FSS volume for water systems served by a single source of supply is the product of the required flow rate (expressed in gpm) multiplied by the flow duration (expressed in minutes). See Equation 7-3.

Equation 7-3:

$$\mathbf{FSS = (FF)(\,t_m\,)}$$

Where:

FF = Required fire flow rate, expressed in gpm, as specified by fire protection authority

$\mathbf{t_m}$ = Duration of FF rate, expressed in minutes, as specified by fire protection authority

Water systems with multiple sources

Design engineers may justify a reduction of FSS volume based on meeting all of the following conditions:

1. Exclude the capacity of the largest producing supply source from the calculations.

2. Each source of supply (excluding largest source) is:
    a. Supplied by permanent on-site auxiliary power that starts automatically when the primary power feed is disrupted.
    b. Capable of operating for the full duration of the maximum fire at the source's designated flow rate.

3. Maintain at least 20 psi under needed fire flow plus MDD conditions throughout the distribution system for the full duration of the maximum fire.

4. The engineer obtains the written consent of the local fire protection authority for the design approach taken.

Consolidating Standby and Fire Suppression Storage (nesting)

SB and FSS volumes may be consolidated – or nested - with the larger of the two volumes being the minimum available, provided the local fire protection authority does not require them to be additive (see WAC 246-290-235(4)). The reservoir project report must include the written consent of the local fire protection authority.

Stand Alone Fire Suppression Storage

Supply to a dedicated, stand-alone fire suppression storage tank **must** be fitted with an approved air gap (WAC 246-290-490). All components downstream of the air gap are considered non-potable and may not be interconnected with the potable water system without appropriate cross connection control. Design engineers should consult a certified cross connection control specialist. See Section 6.3.10.

7.1.1.5 Dead Storage

Dead storage (DS) is the volume of stored water not available to all consumers at the minimum design pressure (WAC 246-290-230(5) and (6)). The reservoir and water system capacity analysis should clearly identify the DS volume. Dead storage is never included in a capacity analysis. DS is always below the top of the outlet pipe silt stop. DS in pumped-storage reservoirs includes the volume below the top of the pump suction pipe or the net positive suction head requirement of the withdrawal pumps, whichever is higher in elevation.

7.1.2 Storage Used for Treatment Purposes

Storage near or adjacent to a source of supply is occasionally used for public health protection and treatment efficacy including:

- Disinfection contact time
- Filter backwashing
- Other treatment purposes

As a result, certain minimum water volumes may need to be maintained in the reservoir for the treatment to be effective. These minimum volumes required for treatment purposes cannot be used in capacity determinations. They are, in effect, dead storage. In general, we recommend that storage for treatment purposes be provided by treatment reservoirs (clearwells) that operate independent of distribution storage.

In cases when a reservoir is used for both distribution storage and disinfection contact time, the OS and ES volumes cannot be used in the determining contact time since these volumes are routinely used on a daily basis. Design engineers should also consider the risk that SB and FSS volumes will not be available for contact time. When a reservoir is used for both storage and disinfection contact time the design should clearly identify and justify the basis for the volume of water used for disinfection contact time, and any associated operational/control requirements and constraints (e.g., level control and alarms).

7.1.3 Source Definition Used in Sizing New Reservoirs

Engineers may consider any source classified as “permanent” or “seasonal” when designing new reservoir facilities if the source is **continuously available** to the water system and meets, at a minimum, all primary drinking water standards (WAC 246-290-010, 222(3), and 420(2) and (5)).

"Continuously available to the system" means all of the following:

1. The source is equipped with functional pumping equipment (and treatment equipment, if required).
2. The equipment is exercised regularly to ensure its integrity.
3. Water is available from the source year round.
4. The source activates automatically based on pre-set parameters (reservoir level, water system pressure, or other conditions).

For designing new reservoir facilities, we consider the following as sources:

1. Each pump in a booster pump station (pumps installed in parallel, not series) pumping into the zone served by that particular reservoir.
2. Each independent, parallel treatment train in a water treatment facility.
3. Each well, or well field comprised of wells, constructed according to the Minimum Standards for Construction and Maintenance of Wells (chapter 173-160 WAC) and capable of pumping concurrently as justified by actual pump test records.
4. Each pump installed in a large capacity, large diameter well if the water system can take each pump out of service without interrupting the operation of any other pump.
5. An emergency intertie, if all the following conditions are met:
   - It is equipped with an automatic valve.
   - There is an intertie agreement that specifically includes provision of SB, FSS, or both.
   - The supplying and receiving distribution systems have sufficient hydraulic capacity to deliver the allocated flow at no less than the minimum pressure required by WAC 246-290-230. If the intertie requires booster-pumping facilities, then each pump installed in parallel constitutes a source.
6. A pressure reducing valve between pressure zones within the same water system if both:
   - Adequate volume is available in the upper zone's storage facilities; and
   - The distribution system (from the upper zone through the PRV to the end use in the lower zone) has the hydraulic capacity to deliver the allocated flows to meet or augment peak hour flows or fire flows, at no less than the minimum pressure required by WAC 246-290-230.

Design engineers should use the actual installed capacity of the facilities and equipment when determining physical capacity based on storage requirements.

7.1.4 Storage for Consecutive Water Systems

A "consecutive water system" purchases some or all of its water supply from another regulated water system (see Section 5.9). A consecutive water system may use the storage available from the supplying water system to satisfy the requirements of Chapter 7 if these conditions are met:

1. The wholesale water agreement between the supplying water system and the consecutive water system defines the quantity of ES, SB, and FSS the supplying water system specifically reserved for the consecutive water system. See Section 5.9.1.
2. The engineer can demonstrate that both the supplying and consecutive water systems can satisfy the hydraulic design criteria described in Sections 6.2.2 and 6.2.5.
3. The local fire protection authority approves the amount of FSS allocated for each system.

7.1.5 Storage for Non-Community Water Systems

In many cases, the storage volumes needed for certain kinds of non-community water systems may be significantly less than those for community water systems. These types of systems include:

- RV parks
- Campgrounds
- Fair grounds
- Outdoor concert grounds
- Restaurants
- Non-critical commercial and institutional uses

For equalizing storage (ES), the approaches outlined in Section 7.1.1.2 should be followed in order to ensure 30 psi is maintained during PHD under normal operating conditions. The guidance in Section 7.1.1 should be followed for operational storage (OS), fire suppression storage (FSS), and dead storage (DS).

If a source failure, power failure, or similar loss of water-supply event occurs, these water systems could shut down without affecting public health and welfare. As a result, we have no specific design guidelines for standby storage.

The source capacity for a public water system should be able to satisfy the maximum day demand (MDD) with no more than 20 hours of pumping. We strongly recommend against constructing and reserving finished water storage to compensate for supply capacity less than MDD. However, in rare cases applicable to transient non-community water systems with relatively few days of demand in excess of source capacity, it may be appropriate to use storage to meet MDD.

Design engineers should consider the impacts of a relatively large storage volume held for a long period to compensate for supply capacity less than MDD. Such large volumes may lead to water quality issues associated with stagnant water including the loss of chlorine residual, biological growth in the reservoir, and the formation of disinfection byproducts. See Section 7.6.

7.2 Geometry, Elevation, and Integration with Existing and Future Facilities

The operation of a finished water storage reservoir must be compatible with the unique features presented by the water system's sources, booster pumps, transmission and distribution piping, and service area topography. Incorrectly siting a reservoir may result in the full reservoir capacity being unavailable to the system (e.g., operating elevation too high and so it doesn't fill, or too low and an altitude valve must hydraulically isolate the tank from the water system).

Selecting tank geometry is also important to maintaining water quality. Improper selection of tank geometry may prevent adequate mixing or promote thermal stratification (e.g., tanks that are much taller than they are wide with a single inlet/outlet demonstrate significant stratification. See Section 7.6.

7.2.1 Establishing Overflow Elevations

Considerations for establishing overflow elevations for reservoirs designed to provide gravity water service include:

1. **Consistency with other facilities and plans**
   The tank overflow elevation should be consistent with other storage facilities the water system uses or plans to use. The design engineer should also consider the overflow elevation of existing or proposed facilities at other nearby water systems if there are or might be gravity interties.
2. **Consistency with pressure requirements and limits**
   The tank overflow elevation should be consistent with pressure requirements and pressure limitations within the existing and future water-service area. The design engineer should consult elevation data in addition to information received from the water system hydraulic analysis described in Section 6.1.
3. **Consistency with source capacity**
   Design engineers should evaluate tank elevation and tank geometry with source equipment discharge-head characteristics to ensure sources meet our source capacity requirements.
4. **Maintaining levels**
   Use altitude valves to prevent over-filling reservoirs constructed with different overflow elevations within the same pressure zone.

The overflow elevation should be located sufficiently above the pump-off control level so that there is no risk of regular overflow occurring during routine reservoir operations. Similarly, there should be sufficient "freeboard" provided between the maximum water surface elevation during the design peak overflow and the wall-to-roof joint.

Reservoirs in very cold climates face the risk of ice formation. The overflow should be located sufficiently above the pump-off level so that ice cannot block its function, especially since ice formation may impact the pump – reservoir level control system. See Section 7.4.4 for additional overflow design guidance.

7.3 Location and Site Considerations

Deciding where to construct a new reservoir can be a difficult design consideration. Any number of competing factors and interests may come into play, with each influencing cost, operability, and maintainability. Some may even influence project feasibility. The project report should adequately discuss site considerations, including:

1. Parcel size sufficient to build and maintain the facility, and to construct future storage to meet projected growth if needed.
2. Zoning compliance, building code compliance, and community acceptance.
3. Distance to the existing distribution and transmission system.
4. Integration/connectivity with existing SCADA system.
5. SEPA analysis (if over 0.5 million gallons).
6. Need for new distribution and transmission pipelines to meet pressure standards.
7. Existing ground-surface elevation and site drainage.
8. Site vehicle access.
9. Disposal of reservoir overflow.
10. Geotechnical engineering field investigations including:
    a. Site drainage
    b. Foundation design requirements
    c. Soil type and soil-bearing strength
    d. Groundwater table elevation
    e. Soil stability/liquefaction/slope failure analysis
11. Availability of Power

7.3.1 Natural Hazard Considerations

Natural hazards/disasters have the potential to damage reservoirs and may even cause catastrophic failure. Reservoirs should be located to minimize their vulnerability to damage in the event of natural disasters such as:

- Avalanche
- Earthquake
- Flood
- Landslide
- Tree fall
- Tsunami
- Windstorm

To meet state and local requirements, engineers **must** address geologic risk (seismic and unstable slopes) when designing reservoirs (WAC 246-290-200). The Washington State Department of Natural Resources (DNR) has geologic hazard maps that can be used to identify seismic and other natural hazards. DNR contact information is provided in Appendix C.

Seismic risk may be reduced or mitigated by:

- Knowledge of soil liquefaction potential
- Appropriate valving to isolate the reservoir
- Proper anchorage and foundation design
- Proper structural design
- Flexible couplings and pipe that can accommodate significant ground movement.

Various design guidelines highlight the multiple seismic vulnerabilities of reservoirs (ALA 2001; AWWA 2011b). In areas with the potential for significant ground motion, design engineers may need to seek the services of a qualified geotechnical engineer or other professional qualified to assist in the design.

7.4 Construction Materials and Design Elements

The basic design concept (standpipe versus in-ground; vendor-purchased plastic product versus constructed in-place) and the materials chosen to construct the reservoir directly impact the function, reliability, operability, and integrity of the facility. The vast majority of reservoirs are constructed of reinforced concrete or steel, and there are detailed standards from AWWA for steel and reinforced concrete reservoirs including:

- D-100: Welded Carbon Steel Tanks for Water Storage (AWWA 2011b)
- D-103: Factory-Coated Carbon Steel Tanks for Water Storage (AWWA 2009)
- D-110: Wire- and Strand-Wound Circular Prestressed Concrete Water Tanks (AWWA 2004)
- D-115: Tendon-Prestressed Concrete Water Tanks (AWWA 2006)

A new reservoir is expensive to build and, depending on the type of reservoir, can be expensive to maintain. Water systems will count on the reservoir for reliable service for decades. Design engineers **must** identify operations and maintenance requirements and their associated cost over the life of the reservoir when evaluating design alternatives (WAC 246-290-110(4)). The least expensive alternative to construct may require a significant level of maintenance and a short asset life, resulting in the costliest alternative on a life-cycle cost basis. The design engineer **must** evaluate the water system's technical, managerial, and financial capacity to properly operate and maintain the new reservoir (WAC 246-290-110(4)), and ensure the new reservoir continuously functions to provide safe and reliable drinking water to the public.

Certain reservoir design concepts, such as those listed below, pose specific contaminant risks including lack of resilience, construction gaps, bird and animal infestation, contamination by chemical wood preservatives, joint and seal failure, cracking, and embrittlement (e.g., ultraviolet light, heat/cold, or chemical degradation). In addition, these designs may not lend themselves to installing all the proper appurtenances necessary for effective reservoir operation and maintenance (see Section 7.4.2). We recommend against pursuing any of the following design concepts for new reservoir designs.

- Wood stave tanks
- Corrugated or other thin wall metal silos commonly used for grain storage
- Concrete basin – wood truss roof tanks
- Floating roof/covers
- Pre-cast panels used as finished storage roofing
- Plastic/polyethylene tanks
- Retrofitting of existing reservoirs with plastic, interior liners.

Regardless of the construction material and design concept, all reservoir submittals **must** include the site-specific design information required by chapter 246-290 WAC, Part 3. Some circumstances justify the design engineer to call for the submittal of vendor/contractor shop drawings for various construction and/or appurtenant details. Such shop drawings must be submitted to the department for review and approval before approval by the engineer and water system. Additional guidance on site-specific design requirements appears throughout this chapter.

7.4.1 Partially Buried and Underground Reservoirs

Special design considerations for partially buried and below-grade reservoirs improve water system reliability and prevent contamination of stored water. Engineers should consider backup power supplies, grading surrounding soils, and other design aspects described in the following sections.
The following recommendations apply to partially buried and underground reservoirs:

1. Locate outside the 100-year flood plain.
2. Water systems should grade the area to a distance of at least 50 feet surrounding a partially buried or below-grade reservoir to prevent standing water near the reservoir.
3. When the reservoir bottom is below the normal ground surface, it should be above the groundwater table. If this is not possible, special design considerations should include providing perimeter foundation drains to daylight and exterior tank sealants. These are necessary to keep groundwater from entering the tank and to protect the reservoir from potential flotation forces when the tank is empty.
4. Partially buried or underground reservoirs should be at least 50 feet from sanitary sewers, drains, standing water, and similar sources of possible contamination. If gravity sewers are within 50 feet of the reservoir, engineers should use the same type of pipe used for water mains. These pipelines should be pressure tested in accordance with AWWA or WSDOT/APWA standards for water mains.
5. Nearby trees and large vegetation should be removed to a distance of at least 50 feet for buried or partially-buried concrete reservoirs to prevent root penetration. Easements to allow grounds maintenance to allow periodic tree removal should be secured.

6. The top of the reservoir should be at least 2 feet above normal ground surface, unless special design considerations address maintenance issues and prevent surface contamination.

7.4.2 Piping and Appurtenances - General

All reservoir appurtenances should be designed to be water tight and protected against freezing and ice damage, which will interfere with proper functioning (such as tank level controls, riser pipes, overflows, and atmospheric vents). These appurtenances **must** be designed to prevent entry by birds, animals, insects, excessive dust, and other potential sources of external contamination (WAC 246-290-235(1)).

Engineers should use seismically appropriate pipe materials and pipe joints for all pipes located within the reservoir and directly below and within 20 feet outside of the reservoir foundation. These pipelines should be evaluated for corrosion potential, and corrosion mitigation should be included as appropriate. These pipelines will be difficult and expensive to repair or replace after the reservoir is in place. The location of the inlet and outlet pipes can also affect the quality of the stored water (See Section 7.6).

Design engineers are expected to consider, and where required by regulation provide, design information for reservoir appurtenances. See Appendix A.3.5 for reservoir design submittal checklist.

7.4.3 Reservoir Drains

Reservoir designs **must** include drain facilities that drain to daylight or an approved alternative that is adequate to prevent cross-connection contamination (WAC 246-290-235(1)). The facility should be able to drain the full contents of the tank without water entering the distribution system or causing erosion at the drainage outlet. Any connection to storm sewers or sanitary sewers must have a properly designed air gap or other feature to prevent cross contamination. Drain lines may discharge directly to a dedicated dry well if the drywell design and construction protect against backflow into the reservoir or distribution system.

Other design considerations:

- Drainage discharge must not threaten the integrity of the reservoir foundation.
- If the topography makes a drain to daylight unrealistic, the reservoir design must include the means of completely emptying the reservoir such as a sump that can be pumped out to empty the reservoir completely.
- The reservoir drain should be separate from the outlet pipe to minimize the risk of a cross connection and prevent sediment from entering the distribution system.
- Establish an easement for drainage path (if applicable).

7.4.4 Reservoir Overflows

The reservoir overflow must be capable of discharging the full inlet supply potential without surcharging the reservoir roof (see Section 7.2.1). The following factors will determine the height of water reached above the overflow invert or weir elevation:

- The design overflow rate
- Size, location, and configuration of the reservoir overflow inlet
- Overflow pipe diameter, length, slope
- Overflow outlet facilities

Every reservoir design **must** include an overflow pipe with atmospheric discharge and suitable means to prevent cross-connection contamination (WAC 246-290-235(1)). Poorly protected and maintained overflow pipes are often a route for contamination of reservoirs (AWWA and EES, Inc. 2002; NRC 2005). Key design features for overflow pipes to minimize the risk that birds, insects, and other sources of contaminants enter reservoirs include:

1. **Properly screened or otherwise secured**. Acceptable design approaches include covering the overflow outlet with a 4-mesh corrosion-resistant screen or a securely closing mechanical device such a duckbill valve, or both. To provide structural strength to the screen, we recommend the 4-mesh screen material be at least 0.047 inches in diameter and constructed of stainless steel. In addition to preventing animals from entering the reservoir, protection of overflows must take into account protection from vandalism, hydraulic restrictions on mechanical devices such as duckbill valves, clogging by debris and ice, and the force of hydraulic loads on the screening or mechanical device the event of an overflow event.

2. **Easy to observe and maintain.** Overflow lines should extend downward to an elevation of 12 to 24 inches above ground level and discharge into a splash plate, rocked area, or suitably above the grate of a catch basin.

3. **Protected against cross connections.** Any connection to a storm drain or sanitary sewer must include an air gap or other feature to prevent a cross contamination.

Other design considerations:

- Overflow discharge must not threaten the integrity of the reservoir foundation.
- The overflow and reservoir drain may share a single discharge.
- Establish an easement for overflow drainage path (if applicable)

7.4.5 Reservoir Atmospheric Vents

Every reservoir design **must** include an atmospheric vent (WAC 246-290-235(1)). An overflow may not serve as an atmospheric vent. Poorly designed and maintained vents and screens are often a route for contamination of reservoirs (AWWA and EES, Inc. 2002; NRC 2005). Key design features for reservoir vents include:

1. Proper screening to prevent entry of contaminants
2. Properly secured and sealed to the structure to prevent entry of contaminants
3. Properly hooded to prevent entry of contaminants
4. Easy to observe and maintain
5. Maintain acceptable internal tank pressure under all possible operating conditions

Properly screened

Acceptable design approaches include covering the screened area with a 4-mesh corrosion-resistant screen backed with a 24-mesh corrosion-resistant insect screen, or approved equal. To provide structural strength to the screen, we recommend the 4-mesh screen material be at least 0.047 inches in diameter and constructed of stainless steel or other non-corrodible metal.

Properly secured

The vent-roof connection and the vent structure itself should be strong enough to withstand the design wind speed for which the overall reservoir structure was designed. We recommend connection of the vent to the roof by a bolted pipe flange or welded saddle. Vent openings should never be used to facilitate water level measurement or be installed as an integral part of the roof hatch access structure.

Properly hooded

Vent design **must** prevent the entry of precipitation that has come into contact with any surface, in the event that the surface itself is contaminated (e.g., bird feces) (WAC 246-290-235). Examples of unacceptable vent designs include those with the potential for allowing:

- Animals nesting directly on the vent screen itself
- Rain splatter off the roof to enter the reservoir vent (e.g., roof vent with screened opening only a few inches above the roof without an adequate "hood" over the vented area)
- Roof run-off entering the vent (e.g., screened opening at the roofline)
- Rain falling through the vent opening itself (e.g., turbine roof ventilators)

Eliminating the risk of precipitation entering the reservoir may provide some basic protection against vandalism. Certain reservoir vent designs provide strong barriers against vandalism, such as those with extensive and resilient hoods, multiple screens, and rigid structural design.

Easy to observe, access, and maintain

Screened openings should be observable, to confirm the integrity of the screen fully protecting the reservoir from contamination. To reduce maintenance, all vent components should be constructed of corrosion-resistant materials.

Maintain acceptable internal tank pressure

In addition to preventing contamination, the screened vent opening must be located sufficiently above the roof to prevent blockage by accumulated ice and snow. We recommend every part of the screened vent opening be located at least 24 inches above the roof (or covering earth) for partially buried and underground reservoirs, and at least 12 inches above the roof for an elevated tank with controlled access.

The design engineer must ensure enough vent capacity to limit the pressure drop (during tank draw) and pressure increase (during tank fill) under all operating conditions so that internal tank pressure remains within the manufacturer's design limits. Large welded steel tanks are most vulnerable to structural damage from inadequate venting. But the design engineer must ensure all tank types including rigid bolted steel, fiberglass, and concrete tanks have adequate ventilation under fill and draw conditions to avoid the risk of drawing in groundwater or stagnant roof water.

The design engineer should obtain from the vent manufacturer a flow rate versus pressure drop (in inches of water) curve. This curve should be used to estimate the pressure drop at the worst case outflow condition (e.g., broken transmission main). The resulting drop in internal pressure should be within the design limits for the tank. Where there are no such design limits specified by the tank manufacturer (e.g., cast-in-place concrete reservoir), we recommend a design pressure drop of no more than one inch of water (0.033 psi or about 5 psf).

A pressure vacuum screened vent or a separate pressure vacuum relief mechanism should be provided on tanks vulnerable to structural damage (e.g., steel tanks) in the event the screen is blocked (e.g., blockage by snow, ice, frost) (AWWA, 2011b).

See *Sanitary Protection of Reservoirs - Vents* DOH 331-250 for guidance on reservoir vents.

7.4.6 Access Hatches

All reservoirs **must** be equipped with a weather-tight hatch sized for human entry (WAC 246-290-235(1)(c)). Except for reservoirs that can be isolated from the distribution system without disrupting consumer service, the access hatch should be installed on the roof, thus allowing access while the tank remains in service. A roof hatch should be framed at least four inches above the surface of the roof at the opening, fitted with a solid water weather-tight cover which overlaps the frame opening and extends down around the frame at least two inches, hinged on one side, and lockable from the side (not top) of the cover. A durable gasket should be installed at the point of contact between the hatch cover and hatch frame.

For partially-buried and underground reservoirs, the roof access hatch should be constructed at least 24 inches above the top of the roof or covering earth, whichever is higher. See 10-State Standards, WAC 246-290-200.

Designers are cautioned on the use of "gutter" style hatches. These are typically constructed on a raised concrete curb, and have an internal gutter located beneath the cover. The cover system itself is not water-proof. The hatch gutter is drained to an external outlet. A non-corrodible screen should cover the drain outlet, to prevent animals or insects from entering the internal gutter. The gutter and screen should be cleaned on a regular basis; otherwise drainage may back up into the internal gutter and spill over into the reservoir. These types of hatches must be well constructed and maintained in order to minimize the risk of contamination.

See *Sanitary Protection of Reservoirs - Hatches* DOH 331-249 for guidance on reservoir hatches.

7.4.7 Roof Drainage

The reservoir roof should be well drained. The reservoir roof should slope at least 2 percent (¼-vertical-inch per horizontal foot). To avoid possible contamination, downspout pipes **must not** enter or pass through the reservoir (WAC 246-290-490).

7.4.8 Reservoir Security

Design engineers should apply a multi-layered strategy to protect reservoirs and other water system facilities:

1. **Deter:** Perimeter fencing is a common means of deterrence. If the reservoir site looks hard to break into, a trespasser will generally move on to a more-easily accessible site.
2. **Detect:** Video surveillance, intrusion monitors, and other sensors signal unauthorized access to a facility.
3. **Delay:** Layers of gates, locks, and perimeter fencing make it more difficult for an unauthorized person to gain entrance.
4. **Respond:** Detection and delay technology must talk to each other, and to the responders. Security guards and local law enforcement often ***respond*** to unauthorized intrusions; having a solid relationship with responders, so that they understand the importance of water system facilities, is critical to effective response.

More detailed guidelines on specific physical security features are available elsewhere (AWWA 2009b, Oregon Health Authority 2009). These guidance documents provide design recommendations to improve security at reservoirs and other water system facilities.

7.5 Operational Constraints and Considerations

All new reservoir designs should meet all applicable Occupational Safety and Health Act (OSHA) and Washington Industrial Safety and Health Act (WISHA) requirements, notably fall protection issues such as ladders, guardrails, and safety devices. In addition, engineers should consider the following reservoir construction and operational issues:

1. Disposal of chlorinated water after construction and disinfection.
2. Disposal of tank drain-line outflow and tank overflow stream.
3. Effect on water system operation when the new reservoir is taken off-line for maintenance or cleaning.

The Department of Labor and Industries (L&I) is the state agency responsible for implementing WISHA workplace safety standards. L&I contact information can be found in Appendix C.

7.5.1 Reservoir Valves

The reservoir design **must** include a way to isolate the tank for maintenance (WAC 246-290-235(1)). Engineers can meet this requirement by providing an isolation valve(s) on the reservoir inlet and outlet piping. In addition, there **must** be a combination air-release/vacuum-relief valve on the distribution side of the outlet piping isolation valve if there is no other atmospheric reservoir in the pressure zone to prevent development of negative pressure in the distribution system (WAC 246-290-490) when the outlet valve is closed.

7.5.2 Reservoir Level Control

All new reservoirs should have a control system to maintain reservoir water levels within a pre-set operating range (OS). Design engineers should include the normal high- and low-water surface elevations that define this operating range in the design. The water system should install a high- and low-level alarm system to notify operation personnel directly.

Cable-supported float switches are vulnerable to ice damage which can render them inoperable. Where potential freezing conditions exist, design engineers should evaluate alternate ways to control and monitor the tank level.

7.5.3 Backup Power Facilities

We recommend that water systems operating pumped storage reservoirs (reservoirs that can only supply a distribution system in whole or in part through a booster pump station) have onsite backup power facilities. See Chapter 8 for booster-pump design guidelines. We recommend backup power facilities that start through an automatic transfer switch if a utility power supply interruption occurs. Manual transfer may be sufficient if it can occur within a reasonable time according to established operating procedures. Maintaining pressurized conditions in the distribution system during a power outage minimizes the risk of backflow/cross connection contamination.

7.6 Reservoir Water Quality and Sampling Access

Long detention times and inadequate mixing can degrade water quality in reservoirs. Stagnant conditions provide an opportunity for chemical and microbial contamination of the stored water. Therefore, engineers **must** design distribution reservoirs to maintain water circulation, prevent stagnation and, in some cases, provide disinfection contact time (WAC 246-290-235(1)).

For reservoirs with a nominal residence time of 3 to 5 days during the summer, design engineers should conduct a mixing and water age analysis of the proposed reservoir design, such as computational fluid dynamic modeling. Such modeling will guide on the design of inlet-outlet piping and valves, as well as setting operational levels. Chemical contamination also can occur in newly constructed reservoirs and those with protective coatings. See Appendix G.

Reservoirs **must** include access for water quality monitoring (WAC 246-290-235(1)(c)). At a minimum, this access should include a smooth-nosed sample tap on the reservoir side of the reservoir isolation valve(s) to facilitate sample collection after construction and maintenance, as part of system assessment after detection of contaminants, or vandalism event. In addition, design engineers should consider providing the means to conduct water quality monitoring at the inlet and within the reservoir itself, including on-line measurement of chlorine residual, pH, and temperature. Temperature probes and piping to collect samples at various reservoir depths provide operational capacity to monitor reservoir water quality (Friedman et al. 2005).

We recommend installing a sampling tap in the valve vault on the tank side of the isolation valve.

7.6.1 Water Circulation and Stagnation

Poor water circulation and long detention times in reservoirs can lead to loss of disinfectant residual, microbial growth, sediment accumulation, formation of disinfection byproducts, taste and odor problems, and other water quality issues (AWWA and EES 2002; NRC 2005). A properly designed reservoir can minimize the potential for these problems.

Engineers should evaluate the following design features to improve reservoir water quality:

1. **Orient inlet and outlet to promote mixing.** Poorly mixed reservoirs can lead to stagnant zones where the water age exceeds the average water age in the facility. A properly designed inlet promotes mixing. Water entering the reservoir can create a jet that entrains ambient water effectively mixing the reservoir (Grayman and Kirmeyer 2000). For effective mixing, the inlet flow must be turbulent and have a long enough path for mixing to develop. You should consider the ability to provide long fill and draw cycles, and thus promote reservoir mixing, as part of the design process.

   Reservoirs that float on the water system, especially those with single inlet-outlet designs, probably won't have sufficient inflow to mix the reservoir adequately. Special valve arrangements, using one or more check valves on a single inlet-outlet pipe, can be used to promote mixing. Some reservoirs may need specialty mixers to prevent stagnation.
2. **Minimize temperature differences in the reservoir.** Temperature differences as small as 1°C can cause thermal stratification, especially in tall tanks with large diameter inlets located near the bottom. To decrease the potential for thermal stratification, locate the inlet off the bottom of the reservoir and increase the inlet momentum (defined as velocity times flow rate). To increase inlet momentum, decrease the diameter of the inlet pipe. Longer fill cycles also promote mixing by increasing the time for circulation patterns to develop.
3. **Increase the frequency of reservoir turnover.** Although not an absolute standard for stored water, there is a high risk for water quality problems to develop when reservoir turnover time exceeds five days, especially in warmer parts of the year. As a starting point, complete turnover of reservoir water should occur at least every three to five days (Kirmeyer et al. 1999).
4. **Site reservoir to promote turnover**. Reservoirs located at the edge of a pressure zone, or beyond, have longer detention times than those within the pressure zone (Edwards and Maher 2008). Distribution system models that evaluate water age, as well as water system hydraulics, can be useful in evaluating reservoir sites.
5. **Evaluate other engineering considerations**. Temperature gradients in the stored water cause thermal stratification. For this reason, some water systems apply light or reflective protective coatings to the tops of their reservoirs. Tall, narrow standpipes are more prone to thermal stratification than reservoirs with roughly equal height and diameter (Grayman and Kirmeyer 2000).

7.6.2 Tank Materials in Contact with Potable Water

All additives, coatings and compounds that will substantially contact drinking water, such as those listed below, **must** have ANSI/NSF Standard 61 certification (WAC 246-290-220). These materials must be applied carefully, according the manufacturer's recommendations. To avoid unnecessary public health concerns and consumer complaints on aesthetic qualities, the design engineer should address the following concerns:

1. For concrete tanks, use appropriate form-release agents, concrete surface sealants, and admixtures. See Appendix G for guidance on water quality concerns associated with concrete in contact with potable water.
2. For steel tanks, consider the materials used to prepare the surface of the tank, as well as the painting or coating water systems used to protect against corrosion. Cathodic protection should be provided as necessary (especially for underground or partially buried tank installations).
3. Reservoir membrane liners, plastic tanks, fiberglass tanks, or other materials that substantially contact drinking water **must** be ANSI/NSF Standard 61 certified.
4. It is important to follow the manufacturer's instructions when applying protective coatings. Temperature, ventilation, and the thickness of the applied layers affect the time required to cure coatings and the potential for contaminants to leach into the water. If there is any concern over the curing of the coatings and materials, or leaching from the reservoir liner, we may require additional water quality monitoring from the reservoir before it goes into service. Appendix G includes additional guidance on testing materials that leach.

7.7 Placing a Reservoir into Service

Before a reservoir can be placed into service, it **must** be properly tested, inspected, and disinfected (WAC 246-290-120(4)). The specifications for the reservoir design should clearly identify:

1. **Curing of coatings**. All coatings in contact with potable water must be certified under ANSI/NSF Standard 61. In some cases, it may be necessary to allow the coating to cure for multiple days before disinfecting it. Following disinfection, additional water quality testing should be conducted (AWWA 2011a; Ten State Standards 2012). This water quality testing includes analysis for taste, odors, VOCs, pH, and conductivity to make sure the water is palatable and meets drinking water standards before it is served to customers.

2. **Disinfection and bacteriological testing requirements**. There are a few different standard approaches for disinfecting a reservoir including filling of the reservoir with chlorinated water so that at the end of the soak period, a chlorine residual of at least 10 mg/L is maintained or spraying all surfaces with a solution that contains at least 200 mg/L of available chlorine as described in *AWWA C652 Standard for Disinfection of Water Storage Facilities* (AWWA 2011a). At the end of the disinfection period and after chlorine residuals have returned to concentrations acceptable for distribution, a coliform

sample must be collected and analyzed. If coliform are present, additional disinfection and bacteriological testing will be necessary.

3. **Leakage testing.** There are various standards for leak testing of reservoirs and reservoir roofs depending upon the type of material used to construct the reservoir (AWWA 2004; AWWA 2006; AWWA 2009a; AWWA 2011b). Regardless of the materials used in construction, specific methods for testing and identification of criteria for passing should be identified.

Only after the reservoir has been cleaned, tested, and disinfected, and testing results shows that the water quality from it is acceptable may it be placed into service. Purveyors **must** submit a *Construction Completion Report Form* (DOH 331-121) to DOH within 60 days after they complete a reservoir project and before they place the reservoir into service (WAC 246-290-120(5)).

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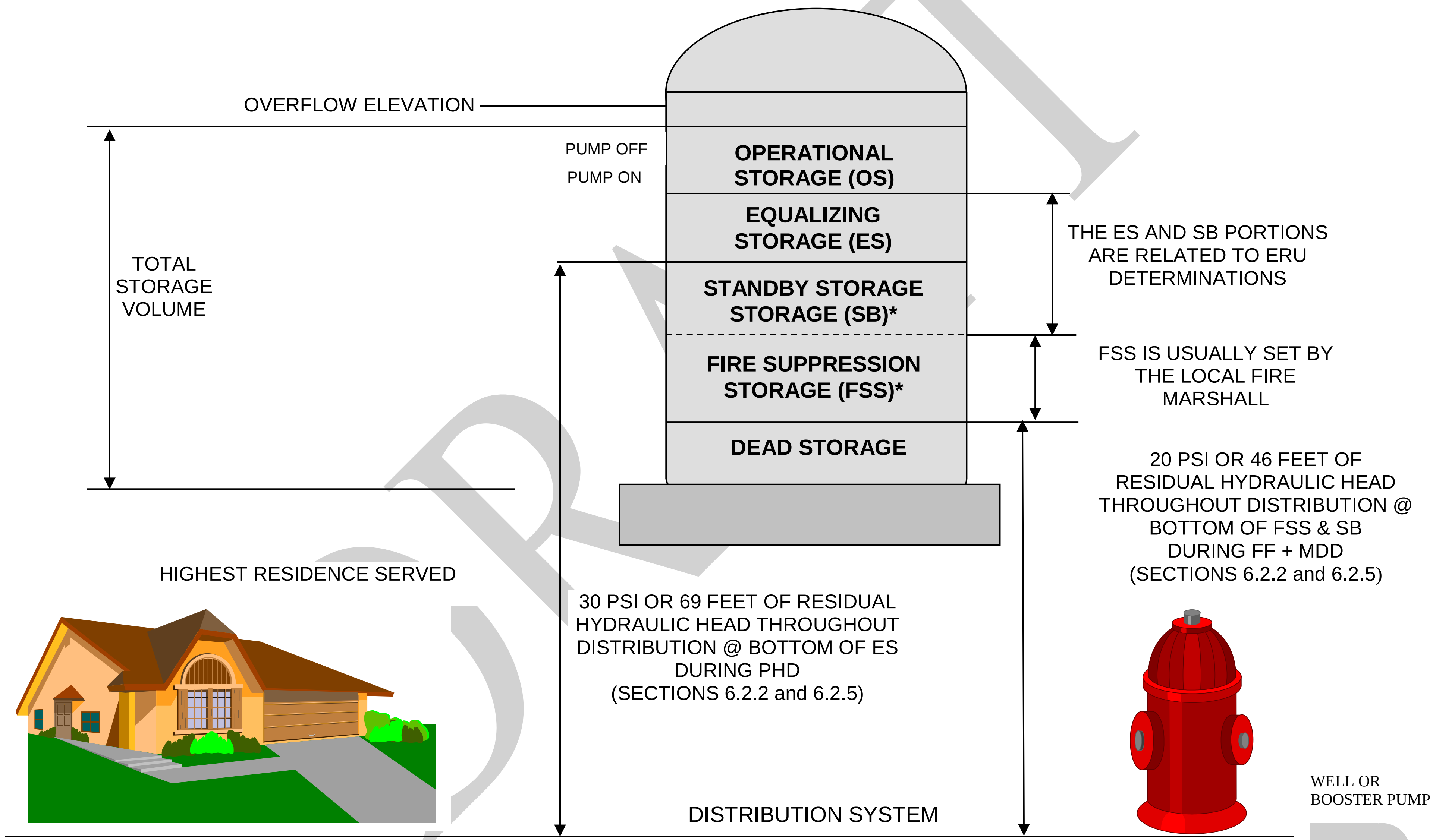
Table 7-1: Reservoir Storage Component Cross-Section Diagram

High Level Alarm. Overflow above ***pump off*** elevation

Pump(s) Off Pump(s) On	**Operational Storage (OS) Component** Not part of ES. Not applicable for continuous pumping systems. Minimum OS volume for pump protection can be conservatively calculated as the pump supply capacity (in gpm) times 2.5 minutes. OS = Operational storage component (gallons).
Maintain 30 psi (required)	**Equalizing Storage (ES) Component** For call-on-demand: ***ES* = *(PHD* - *Q_S)(150 min.),*** but in no case less than zero. *ES* = Equalizing storage component (gallons). *PHD* = Peak hourly demand (gpm). *Q_S* = Total of all permanent and seasonal sources (gpm). See Section 7.1.1.2 for sizing criteria for continuous pumping operations.
Low Level Alarm Maintain 20 psi (required)	**Fire Suppression Storage (FSS) Component** For Single Sources: ***FSS = (FF)(t_m)*** FSS = Fire suppression storage component (gallons). FF = Needed fire flow rate, expressed in gpm as specified by fire authority or the Coordination Act, whichever is greater. t_m = Duration of FF rate, expressed in minutes as specified by fire authority.
Maintain 20 psi (recommended)	**Standby Storage (SB) Component** ***SB = (N)(SB_i)(T_d)*** SB = Total standby storage component, or its equivalent, in gallons. N = Number of ERUs based on the ERU_{MDD} value SB_i = Locally-adopted unit SB volume in gallons per day per ERU (number of ERUs based on the ERU_{MDD} value) T_d = Number of days selected to meet water system-determined standard of reliability We recommend SB volume sufficient to provide at least 200 gallons per ERU.
	Dead Storage (DS) Portion of a gravity reservoir that does not provide required minimum pressure.

Figure 7-1: Reservoir Storage Components



* WAC 246-290-235(4) ALLOWS CONSOLIDATION OF THESE COMPONENTS WITH APPROVAL OF LOCAL FIRE PROTECTION AUTHORITY. IF CONSOLIDATION (“NESTING”) IS APPROVED, APPLY WHICHEVER THE GREATER OF SB OR FSS.

Chapter 8: Booster Pump Station Design

8.0 Introduction

Many water systems, because of treatment, topography, or high design flows need booster pumping facilities. In these water systems the booster pumps are an integral component of the distribution system – like the water mains themselves – that must be adequate in capacity and reliable. Inadequate or unreliable booster pumping facilities leave a water system vulnerable to inadequate pressure, customer complaints, and place the distribution system at risk of contamination.

Booster pumps work with pressure tanks, atmospheric storage tanks, variable frequency drives, and control valves to maintain a consistent pressure range in the distribution system. This chapter describes requirements for minimum design pressures and reliability standards (WAC 246-290-230 and 420) and offers design guidance on booster pump station:

- Pumping system capacity
- Location and site considerations
- Material selection, piping, and appurtenances

The objective of booster pump station design is to provide the water system with adequate and resilient water pumping facilities that protect the quality of water in the distribution system while delivering needed supply to consumers over a wide range of operating conditions. Thorough consideration of each booster pump design element, following appropriate construction standards, and implementing best management/operating practices is important to public health protection.

8.1 Booster Pump Station Capacity

In general, the booster pump station, in addition to any other supplies to the zone – wells, other pressure zones, and storage reservoirs - **must** be able to meet minimum demand and pressure requirements (WAC 246-290-230(5) and (6)). The demand conditions include the maximum daily demand (MDD), peak hourly demand (PHD), and fire flow for the area served by the pump station, as well as the supplying pressure zone(s). See Chapter 3 for guidance on estimating demand.

One of the key factors in sizing a booster pump station is the availability of storage in the pressure zone served by the booster pump station. If there is gravity storage in the pressure zone, the zone is referred to as an **open system** in this manual because there is a water surface open to the atmosphere. If there is no finished water reservoir in the pressure zone, this part of the distribution system is called a **closed system**. The sizing of booster pump stations for open and closed systems is slightly different as described in the following subsections.

In general, ODW recommends that engineers design booster pumping facilities to accommodate at least the next 10 years of water system development, and preferably the period associated with full water system build-out for its service area. Variable frequency drive pumps are particularly well suited to accommodate growth (see Appendix B.3).

After assessing needed booster pump station capacity, design engineers should check to be sure reservoir overflow capacity is sufficient to safely discharge the maximum combined pumping capacity of the booster pump station, and any other source of supply available to the zone served by the reservoir (e.g., all wells, interties, other pump stations, pressure reducing valves) without damage to property or surcharging the reservoir structure.

8.1.1 Open System Booster Pump Station Sizing Guidelines

For open systems with adequate equalizing storage, the minimum discharge capacity of the booster pump station(s) – plus the supply from other sources - is at least the MDD of the pressure zone and any sequential zones served (WAC 246-290-230). In addition, a booster pump station cannot create low pressure in any supplying zone during peak demand periods such as when fighting fires. The specific design requirements are summarized in Table 8-1.

Table 8-1

Summary of Hydraulic Analysis Conditions for Open System Booster Pump Stations

Scenario	Supplying Pressure Zone(s)		Pump Station Discharge	Discharge Pressure Zone	
	Demand Conditions	Pressure Requirement		Demand Conditions	Pressure Requirements
1	PHD	Maintain 30 psi min.	MDD[1]	MDD	Maintain 30 psi min.
2	MDD + FF[2]	Maintain 20 psi min.	MDD[1]	MDD	Maintain 30 psi min.
3	MDD	Maintain 30 psi min.	MDD + FF[1, 2]	MDD + FF	Maintain 20 psi min.

1. Or pump station operating capacity, whichever is greater.
2. FF (Fire flow) - The largest pump must be assumed to be out of service for water systems located within a Public Water System Coordination Act area.

If the booster pump station constitutes a critical part of the water system, the engineer should consider additional capacity or redundancy for purposes of expansion or reliability. At a minimum, the booster pump station must be able to meet the MDD of the pressure zone(s) with all pumps in service and should be able to supply the average day demand (ADD) with the largest pump out of service in conjunction with any other sources of the supply.

8.1.2 Closed System Booster Pump Station Sizing Guidelines

The pumps in a closed system booster pump station supply the entire flow and pressure required by the service area. Because state rules require the water system to provide PHD at no less than 30 psi at all service connections throughout the distribution system, the engineer **must** design a closed system pump station to meet this requirement (WAC 246-290-230(5)).

For reliability purposes, the booster pump station should be able to meet the PHD when the largest capacity booster pump is out of service. Because the service area of a closed system pump station depends entirely on the continuing operation of the pump station, the engineer **must** consider standby power facilities (WAC 246-290-420). In addition, a closed system pump station **must** also be capable of meeting fire flow requirements as defined by the local fire marshal. Where fire flow is required, the pumping system **must** be capable of maintaining a minimum of 20 psi at ground level at all points in the distribution system while supplying MDD plus needed fire flow (WAC 246-290-230(6)). These design requirements are summarized in Table 8-2.

Table 8-2

Summary of Hydraulic Analysis Conditions for Closed System Booster Pump Stations

Scenario	Supplying Pressure Zone		Pump Station Discharge	Discharge Pressure Zone	
	Demand Conditions	Pressure Requirement		Demand Conditions	Pressure Requirements
1	PHD	Maintain 30 psi min.	PHD[1]	PHD	Maintain 30 psi min.
2	MDD + FF[2]	Maintain 20 psi min.	PHD[1]	PHD	Maintain 30 psi min.
3	MDD	Maintain 30 psi min.	MDD + FF[1, 2]	MDD + FF	Maintain 20 psi min.

1. Or pump station operating capacity, whichever is greater.
2. FF (Fire flow) - The largest pump must be assumed to be out of service for water systems located within a Public Water System Coordination Act area.

8.1.3 Fire Flow Requirements for Pump Stations- Public Water System Coordination Act Areas

For new booster pump stations in an area governed by the Public Water System Coordination Act (Chapter 246-293 WAC), fire flow **must** be met with the largest capacity booster pump out of service (WAC 246-293-660(1)). For open system booster pump stations, reservoir storage in the pressure zone supplied can be used to meet fire flow requirements. The remaining pumps, in conjunction with supply from the reservoir, **must** be capable of maintaining a minimum of 20 psi at ground level at all points in the distribution system while supplying MDD plus needed fire flow (WAC 246-290-230(6)).

In general, you should be cautious using a closed booster pump station if fire flow is required since a closed booster pump station is less reliable than gravity storage. A new booster pump station serving a closed system should be designed with back-up power operated by an automatic transfer switch if power outages exceed the threshold standards in WAC 246-293-660(1).

8.1.4 Flow Control for Booster Pump Stations

There are often wide variations in diurnal demand in the pressure zone(s) served by a pump station. There are several ways that these demand fluctuations can be met including:

Constant speed pumps with pressure tanks. This design approach is most often used for small, closed pressure zones. In these pump stations, the booster pumps are controlled by pressure switches which start and stop the pumps. Pressure tanks are used to maintain system pressure within a fixed range and to limit pump cycling. Additional information on sizing pressure tanks with constant speed pumps is in Chapter 9.

Variable frequency drives (VFDs). This design approach offers many advantages, including energy savings, improved pressure and flow control, and elimination of pressure transients associated with abrupt start/stop of single-speed pumps. Flow control is provided through a feedback loop from a pressure sensor on the pump discharge to the VFD controller governing pump motor rotational speed (and therefore flow). Additional information on VFDs is provided in Appendix B.3. For closed booster pump stations, a pressure tank is still necessary to minimize pump cycling under very low flow conditions.

A jockey pump for low flow conditions. A small pump, commonly referred to as a jockey pump, can be used to meet demand during low flow conditions. The jockey pump is used in conjunction with larger pumps to meet peak demands. For closed booster pump stations, a pressure tank is still necessary to minimize pump cycling.

Cycle control valves with a pressure tank. These specialized pressure regulating valves keep a constant downstream pressure over a wide range of flows. A pressure tank is still required for closed pressure zones served by pumps stations that use these valves to accommodate the need for pump cycling control under very low flow conditions. Additional information on cycle control valves is provided in Appendix B.2.

8.2 General Booster Pump Station Site Considerations

In general, booster pump station designs **must** comply with state or locally adopted building, mechanical, electrical, and land use codes (WAC 246-290-200(b)). The contents of these codes, not to mention local ordinances, is beyond the scope of this manual. Overall, you should review locally adopted codes and ordinances that could impact the siting and design of a booster pump station in a project report. These considerations include, but are not limited to:

- Zoning compliance, building code compliance, and community acceptance.
    - Noise can be an issue for pump stations located near parks or residences. The maximum permitted sound level can be as low as 45 dBA in residential areas at night (WAC 173-60-040).
- Operator access, equipment maintenance, and safety. The pumps and other mechanical equipment need periodic maintenance. As such, the pump station should be designed to make it easy to inspect, operate, and maintain the equipment including:
    - Be aware of creating permit required confined spaces – Booster pump stations in below grade vaults or other permit required confined spaces as defined in Chapter 296-809 WAC can create operations and maintenance issues. Department of Labor and Industries contact information is provided in Appendix C.
    - Provide adequate space around mechanical equipment and electrical equipment - In general, at least 36 inches clearance is recommended between piping, pumps

and other mechanical equipment. The minimum clearance in front of electrical panels is governed by electrical codes, though is at least 36 inches and can be 60 inches or more for high voltage panels (Sanks et al. 1998; AWWA/ASCE 2012).

    - Facilitate removal and installation of heavy valves and equipment – In general, any piece of equipment that weighs more than 100 pounds should be accessible by crane or have other lifting assistance. Other means of access include large doorways or roof hatches to facilitate removal of heavy equipment directly into a truck. Areas where the operator will walk or maintenance will be performed should be clear of overhead obstructions to a height of at least 7 feet (Sanks et al.1998).
    - Hearing protection and other measures to protect people in the pump station are required when the noise exceeds 85 dBA (Chapter 296-817 WAC).
- Geotechnical engineering field investigations including:
    - Site drainage
    - Soil type and soil-bearing strength
    - Groundwater table elevation
    - Soil stability/liquefaction/slope failure analysis
- Electrical power supply
    - Reliability – The reliability of the power supply should be assessed along with the necessity of standby generators. See Section 5.11.1.
    - Sizing – In some cases, such as for very large pump stations and ones located in rural areas, the capacity of the electrical grid may limit the suitability of some sites or require upgrades in the local electrical service that will need to be factored into siting decisions.

There are many other items that should be addressed as part of pump station design. Some additional items are highlighted in Checklist A.3.6 in Appendix A.

8.2.1 Natural Hazard Considerations

Natural disasters have the potential to damage pump stations to the point that they may fail to operate. Pump stations should be designed and located to minimize their vulnerability to damage in the event of natural disasters such as:

- Avalanches
- Earthquakes
- Floods
- Landslides
- Tree falls
- Tsunamis
- Windstorms

To meet state and local requirements, engineers **must** address geologic risk (seismic and unstable slopes) when designing pump stations (WAC 246-290-200). The Washington State Department of Natural Resources (DNR) has geologic hazard maps that can be used to identify seismic and other natural hazards. DNR contact information can be found in Appendix C.

Seismic risk may be reduced or mitigated by:

- Knowledge of soil liquefaction potential
- Proper anchorage and foundation design
- Proper structural design
- Flexible couplings and pipe that can accommodate significant ground movement.

Various design guidelines highlight the multiple seismic vulnerabilities of piping and large mechanical equipment in some pump stations (ALA 2002; ALA 2004). In areas with the potential for significant ground motion, you may need to seek the services of a professional qualified to assist in the design of pipe bracing, equipment support, and other aspects of design.

8.3 Booster Pump Station Design Details

In general, the design of a booster pump station **must** comply with state and locally adopted national model codes (WAC 246-290-200(b)). The details of these building, electrical and mechanical codes are beyond the scope of this manual. The design of all but the simplest pump stations may require the involvement of licensed professionals with detailed knowledge and experience with the codes (WAC 197-27A-020(2)). This section and Checklist A.3.6 in Appendix A provide further guidance on basic pump station design elements.

Meters and gauges
To help ensure that pumps perform as designed, each pump should have:

- A pressure gauge between the pump and the discharge check valve
- A compound gauge on its suction side; and
- A way to meter the discharge.

Each booster pump station should also have a meter capable of measuring the total water pumped and pumping rate.

Valves
Each pumps should have valves adequate to permit satisfactory operation, maintenance, and equipment repair. There should be an isolation valve on the suction and discharge side of each booster pump. Other appurtenances should include:

- A check valve on the discharge side of each booster pump.
- End connections for booster pumps, pressure vessels, and large equipment should have flexible flanged coupling adapters for larger units and threaded unions for smaller units. They will simplify maintenance and provide flexibility in installation.

- Pump control valves and surge anticipation valves, as needed, to prevent destructive hydraulic transients during normal and emergency pump starts or stops.
- Air relief valves at any high points in the piping.

Controls and Alarms
The pump station should be designed to alert the operator in the event of a pump failure or abnormally high or low pressure. One approach is to have a visible external alarm light (with a battery backup). If practical, the pump station alarm system should be connected to an auto-dialer to notify the operator, water system owner, and other key personnel of any unusual conditions or unauthorized entry.

Piping Material
The strength, stiffness, ductility and resistance to water hammer/pump cycling make steel and ductile iron the most suitable choices for exposed piping in pump stations (Sanks et al. 1998). Plastic pipe such as PVC and HDPE are prone to fatigue failure from pump cycling, become brittle at low temperatures, or lose strength at temperatures that can occur normally in pump stations. For these reasons, the use of PVC or HDPE pipe inside a booster pump station should be approached with caution and only with the approval of the water system owner. The design should also address special anchoring or support requirements for equipment and piping.

Piping Connections
Engineers should use seismically appropriate pipe materials and connections for all pipes located within the pump station, directly below it, and within 20 feet of the pump station foundation. These pipes should be evaluated for corrosion potential, and corrosion mitigation should be included as appropriate since these pipes will be difficult and expensive to repair or replace if they fail after the pump station is in place.

Taps on the discharge piping
Booster pump stations are convenient places to provide water quality monitoring and, if necessary, provide booster chlorination or other water quality adjustments. You should consider installing at least two taps on the common discharge line:

- A sample tap to allow for monitoring water quality
- A tap to allow for booster disinfection in an emergency

Access for pipe cleaning and condition assessment tools
As distribution pipes age, they can gradually accumulate solids and suffer from corrosion. As a result, it may be useful to install a pig-launch or other access point on the pump station discharge piping.

8.4 Individual Booster Pumps

An individual booster pump station may **not** be installed to serve a property on a new water system or an addition to an existing water system (WAC 246-290-230(5)).

For existing water systems, there may be a need to install individual service booster pumps to meet minimum pressure requirements for specific connections. Such designs **must** be submitted to the department for approval (WAC 246-290-125). Individual booster pumps installed because the minimum 30 psi standard in WAC 246-290-230(5) cannot be met are the responsibility of the purveyor, not the consumer. Water systems may only use individual booster pumps on an interim basis, typically less than 10 years, and they **must** manage and control any individual booster pumps (WAC 246-290-230(8)). The water system should evaluate vulnerabilities in the distribution system until upgrades can be made that eliminate such low pressure areas and the associated need for individual booster pumps.

If the pressure in the distribution line meets the minimum requirements of WAC 246-290-230(5), a purveyor may allow installation of individual booster pumps to serve customers who want additional pressure. For example, developers may install booster pumps to serve structures built at significant elevations above the service meters. The purveyor should approve the design, installation, and operation of such individual booster pumps. Moreover, the purveyor **must** ensure the booster pumps do not adversely affect pressure in the rest of the distribution system (WAC 246-290-230 and 420), and address all cross-connection control concerns (WAC 246-290-490). Booster pumps installed where the minimum 30 psi standard in WAC 246-290-230(5) is met are the responsibility of the building owner and may be allowed on a permanent basis.

8.4.1 Cross-Connection Control for Individual Booster Pumps

When designing or installing an individual service booster pump, the engineer should recognize that the location the individual booster pump will serve is a **cross-connection hazard.** Under normal circumstances, the pressure on the downstream side of the individual service booster pump is higher than system pressure. However, the check valve could fail or leak, causing water from the premises to backflow through the pump and into the distribution main. Therefore, a cross connection control specialist **must** assess the degree of hazard for facilities that use booster pumps and approve the installation of an acceptable backflow assembly (WAC 246-290-490(4)(e)(iii)).

Special consideration should be given for booster pumps within multistory buildings given the higher pressures in these structures relative to the distribution system, internal storage in some cases, and greater potential for a multitude of uses, including high health cross connection hazards that require premises isolation (PNWS-AWWA 1996).

Water that enters the consumer's premises is "used water." Therefore, any piping arrangement that allows pressure relief **must not** be directed back into the distribution system (WAC 246-290-490(2)(k)).

8.5 Placing a Booster Pump Station into Service

Field testing of pumps should be considered to ensure that pumps are properly installed and can deliver their rated performance. A field pump test consists of measuring the pump discharge, pressure or head, power input, and speed. This information can then be used to determine if there are operational issues with the pumps as outlined in the *AWWA E103- Standard for Horizontal and Vertical Line-Shaft Pumps* (AWWA 2007).

Before a booster pump station can be placed into service, it **must** be properly tested, inspected, and disinfected (WAC 246-290-120(4)). The specifications for the pump station should clearly identify the disinfection and bacteriological testing requirements. The WSDOT/APWA standard specifications (Division 7) and *AWWA C651 - Standard for Disinfecting Water Mains* can be used for this purpose (WSDOT/APWA 2016; AWWA 2014). Purveyors **must** submit a *Construction Completion Report Form* (DOH 331-121) to DOH within 60 days after they complete a pump station project and before they place the pump station into service (WAC 246-290-120(5)).

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Chapter 9: Pressure Tanks

9.0 Introduction

A pressure tank contains pressurized air and water. The compressed air acts as a cushion to exert or absorb pressure as needed. There are two types of pressure tanks. ***Bladder tanks*** have some type of membrane separating the air from the water. ***Hydropneumatic tanks*** allow air-water contact.

Pressure tanks work with pumps in closed systems (see Chapter 8) to maintain pressure within a selected range without requiring continuous pump operation. This chapter offers design guidance on:

- Pressure tank sizing
- Department of Labor and Industries standards
- Pressure tank type selection and appurtenances

The objectives of pressure tank design are to avoid premature pump failure due to excessive cycling and to protect the quality of water in the distribution system by reliably maintaining distribution system pressure within the design operating range. The needed number and size of pressure tanks depends on how the pump discharge rate is controlled. Control options include:

- Variable frequency drives
- Single-speed pumps with on-off pressure switches
- Cycle control valve with downstream pressure set point

Pressure tanks are not appropriate for providing equalizing, standby, or fire protection storage. If such storage is required, design engineers should select ground or elevated storage as described in Chapter 7.

9.1 Pressure Tank Sizing

The portion of pressure-tank volume that can be usefully withdrawn between pumping cycles while maintaining 30 psi pressure throughout the distribution system under peak hour demand (PHD) conditions (WAC 246-290-230(5)) is referred herein as ***withdrawal capacity.*** The procedure for selecting or sizing bladder tanks differs from that used for hydropneumatic tanks.

9.1.1 Bladder Tank Sizing

Bladder tank sizing depends on the number of "selected-size" tanks needed to provide pump protection. Bladder tanks are assumed to be pre-charged with air to a pressure of about 5 psi below the low operating (pump-on) pressure for the system. Design engineers need to call out this stipulation in the design specifications.

Design engineers may use Equation 9-1 to determine the number of bladder tanks of a certain gross volume, based on the pump-on and pump-off pressure settings for single speed pumps controlled by a pressure switch. See Section 9.1.5 for an alternative pressure tank design approach based on use of a cycle control valve or variable frequency drive pumping system.

Equation 9-1: $$T \geq \frac{(R)(Q_p)}{(N_c)(V_B)}$$

Where:

R	=	$\frac{15(P_1 + 14.7)(P_2 + 14.7)}{(P_1 - P_2)(P_2 + 9.7)}$ (or refer to Table 9.1)
V_B	=	The gross volume of an individual bladder tank in gallons ("86-gallon tank," for example).
T_s	=	The number of bladder tanks of gross volume $\mathbf{V_B}$
P_1,P_2	=	Pressures selected for water system operation in psig (gauge pressures). $\mathbf{P_1}$ corresponds to the pump-off pressure and $\mathbf{P_2}$ to the pump-on pressure
Nc	=	Number of pump operating cycles per hour. This should be the maximum number of pump motor starts per hour as recommended and documented by the pump or motor manufacturer. Without such information, design engineers should use no more than six cycles per hour.
Q_p	=	Pump delivery capacity in gallons per minute at a midpoint of the selected pressure range. Determine this by examining pump curves or tables. If this value is not used, the designer should use the Q_p that occurs at P_2 (pump-on).

9.1.2 Bladder Tank Design Procedures

The following is a step-by-step procedure for designing bladder tank pressurized storage systems used in connection with single-speed pumps with on-off pressure switches. See application of these procedures in Example 9-1 in Section 9.6.

1. Based on water system hydraulic requirements, select the operating range of pressure, P_1 (pump-off) and P_2 (pump-on). P_2 pressure **must** satisfy minimum system pressure requirements (WAC 246-290-230).
2. Select the operating cycles per hour, N_c. The value for N_c should not exceed six cycles per hour unless the pump manufacturer justifies a larger value. For multiple pump installations, N_c may be increased if an automatic pump switchover system is installed to automatically alternate pumps.
3. Determine the delivery capacity, Q_p, for the midpoint of the operating pressure range [$(P_1 + P_2)/2$]. The pump capacity at P_2 pressure **must** meet system demand and pressure requirements (WAC 246-290-230 and 420).

4. Select an appropriate gross volume, V_B , for each bladder tank (bladder tank size). This volume should be available from bladder tank manufacturers. We recommend limiting individual bladder tank sizes to no more than 220 gallons gross volume.
5. Calculate the value of ***R***. For convenience, Table 9.1 gives ***R***-values for several commonly used pressure ranges.
6. Use Equation 9-1 (see above).
7. Round up the value determined in Step 6 to the nearest whole number. This is the number of tanks, each with the selected volume, V_B , to be used for pump protection.
8. See Appendix A.3.7 for further design recommendations.

Table 9-1
R Values for Various Pressure Tank Ranges

	P_1 pump-off pressure				
P_2 pump-on pressure	**55 psi**	**60 psi**	**65 psi**	**70 psi**	**80 psi**
35 psi	58.1	49.8	44.3		
40 psi	76.7	61.7	52.6	46.6	
45 psi		81.5	65.2	55.5	
50 psi			86.4	68.8	51.3
60 psi					76.1

9.1.3 Hydropneumatic Tank Sizing Equations (bottom outlet)

Horizontally-Oriented Tanks

Design engineers may use Equation 9-2 to determine the gross volume of a hydropneumatic tank to be installed horizontally, based on the pump-on and pump-off pressure settings for single speed pumps controlled by a pressure switch.

See Section 9.1.5 for an alternative pressure tank design approach based on use of a cycle control valve or variable frequency drive pumping system.

Equation 9-2:

$$V_t = \frac{P_1 + 14.7}{P_1 - P_2} x \frac{15\, Q_p\, (MF)}{Nc}$$

Vertically-Oriented Tanks

Design engineers may use Equation 9-3 to determine the gross volume of a hydropneumatic tank to be installed vertically, based on the pump-on and pump-off pressure settings for single speed pumps controlled by a pressure switch.

Equation 9-3:

$$V_t = \frac{P_1 + 14.7}{P_1 - P_2} x \frac{15\, Q_p\, (MF)}{Nc} + 0.0204\, D^2$$

Where:

- **V_t** = Total tank volume in gallons.
- **P_1, P_2** = Pressures selected for water system operation in psig (not absolute pressures). **P_1** corresponds to the pump-off pressure and **P_2** to the pump-on pressure.
- **Nc** = Number of pump operating cycles per hour. This should be the maximum number of pump motor starts per hour as recommended and documented by the pump or motor manufacturer. Without such information, design engineers should use no more than six cycles per hour.
- **Q_p** = Pump delivery capacity in gallons per minute at the midpoint of the selected pressure range. Determine this by examining pump curves or tables. If this value is not used, the Q_p that occurs at P_2 (pump-on) should be used.
- **D** = Tank diameter in inches.
- **MF** = A multiplying factor related to tank diameter to include the volume needed to maintain a six-inch water seal above the tank inlet/outlet installed at the bottom of the tank. See Table 9-3. Use this factor only for sizing a horizontal tank. MF for vertically-oriented tanks equals 1.

9.1.4 Hydropneumatic Tank Design Procedures

The following is a step-by-step procedure for designing horizontally or vertically oriented hydropneumatic pressurized storage systems used in connection with single-speed pumps with on-off pressure switches. See application of these procedures in Example 9-2 in Section 9.6.

1. Based on water system hydraulic requirements, select the operating range of pressure, $\boldsymbol{P_1}$ (pump-off) and $\boldsymbol{P_2}$ (pump-on). $\boldsymbol{P_2}$ pressure **must** satisfy minimum system pressure requirements (WAC 246-290-230).

2. Select the operating cycles per hour, $\boldsymbol{N_c}$. The value for $\boldsymbol{N_c}$ should not exceed six cycles per hour unless the pump manufacturer justifies a larger value. For multiple pump installations, $\boldsymbol{N_c}$ may be increased if an automatic pump switchover system is installed to automatically alternate pumps.

3. Determine the delivery capacity, $\boldsymbol{Q_p}$, for the midpoint of the operating pressure range [$\boldsymbol{(P_1 + P_2)/2}$]. The pump capacity at $\boldsymbol{P_2}$ pressure **must** meet system demand and pressure requirements (WAC 246-290-230 and 420).

Note: *When multiple pumps will be pumping through a pressure tank, the* $\boldsymbol{Q_p}$ *can be based on the largest pump.*

4. For either vertical or horizontal tanks, select a tank diameter (in inches) that suits the space available in the pump house.
5. For a **horizontal tank**, refer to Table 9-3 for the multiplying factor, ***MF***, needed to accommodate the required water seal. The MF in this table is calculated to provide a six-inch water seal above the tank inlet/outlet installed at the bottom of the tank. If a **vertical tank** is to be used, the additive value for the water seal volume can be calculated directly and is already included in Equation 9-3.
6. Calculate the necessary tank volume by incorporating the parameters above into the appropriate sizing equation. The tank is subject to the American Society of Mechanical Engineers (ASME) code construction requirements identified in Section 9.2.
7. Check the calculated volume requirement with any commercial tank size table (see Table 9-2) to see if a tank that meets the necessary volume at the selected diameter is available. If a tank that provides the necessary volume at the diameter selected is not available, or cannot be fabricated, select another tank diameter and repeat the sizing calculations until the design is satisfied. This may also be necessary if the pump house layout will not accommodate the length needed.
8. See Appendix A.3.7 for further design recommendations.

Table 9-2
Pressure Tank Dimensions[1]

Dimensions, Capacities and Tappings							
		Dimensions, Inches			Tappings, FPT[2]		
Tank Model Number	Capacity Gallons	Outside Diameter A	Shell Length B	Approximate Overall Length C	Relief[3] R	Blowdown[4] S	Water In & Out[5] W
144	36	14	48	58	3/4		1
145	44	14	60	70	3/4		1
164	48	16	48	59	3/4		1
165	58	16	60	71	3/4		1
166	69	16	72	83	3/4		1
184	62	18	48	60	3/4		1-1/4
185	75	18	60	72	3/4		1-1/4
186	88	18	72	84	3/4		1-1/4
204	77	20	48	62	1		1-1/4
205	93	20	60	74	1		1-1/4
206	109	20	72	86	1		1-1/4
244	113	24	48	64	1		1-1/4
245	137	24	60	76	1		1-1/4
246	160	24	72	88	1		1-1/4
247	184	24	84	100	1		1-1/4
304	186	30	48	67	1		2
305	223	30	60	79	1		2
306	260	30	72	91	1		2
307	296	30	84	103	1		2
308	333	30	96	115	1		2
309	370	30	108	127	1		2
365	330	36	60	82	1		2
366	383	36	72	94	1		2
367	436	36	84	106	1		2
368	489	36	96	118	1		2
369	542	36	108	130	1		2
3610	594	36	120	142	1		2
426	533	42	72	96	1-1/4	2	2
427	605	42	84	108	1-1/4	2	2
428	677	42	96	120	1-1/4	2	2
429	749	42	108	132	1-1/4	2	2
4210	821	42	120	144	1-1/4	2	2
4211	893	42	132	156	1-1/4	2	2
4212	965	42	144	168	1-1/4	2	2
4213	1037	42	156	180	1-1/4	2	2
4214	1110	42	168	192	1-1/4	2	2
486	712	48	72	100	1-1/4	2	3
487	806	48	84	112	1-1/4	2	3
488	900	48	96	124	1-1/4	2	3
489	994	48	108	135	1-1/4	2	3
4810	1089	48	120	148	1-1/4	2	3
4811	1183	48	132	160	1-1/4	2	3
4812	1277	48	144	172	1-1/4	2	3
4813	1371	48	156	184	1-1/4	2	3
4814	1465	48	168	196	1-1/4	2	3
548	1160	54	96	126	1-1/2	2	3
5410	1398	54	120	150	1-1/2	2	3
5411	1517	54	132	162	1-1/2	2	3
5412	1636	54	144	174	1-1/2	2	3

Dimensions, Capacities and Tappings							
		Dimensions, Inches			Tappings, FPT[2]		
Tank Model Number	Capacity Gallons	Outside Diameter A	Shell Length B	Approximate Overall Length C	Relief[3] R	Blowdown[4] S	Water In & Out[5] W
5413	1755	54	156	186	1-1/2	2	3
5414	1874	54	168	198	1-1/2	2	3
5415	1993	54	180	210	1-1/2	2	3
5416	2112	54	192	222	1-1/2	2	3
6010	1750	60	120	154	1-1/2	2	3
6012	2044	60	144	178	1-1/2	2	3
6014	2338	60	168	202	1-1/2	2	3
6016	2632	60	192	226	1-1/2	2	3
7210	2609	72	120	160	1-1/2	3	4
7212	3032	72	144	184	1-1/2	3	4
7214	3455	72	168	208	1-1/2	3	4
7216	3878	72	192	232	1-1/2	3	4
Above data is based on use of Elliptical Heads with 2" max SF.							

1. Table furnished for example only. Any commercial table may be used.
2. "FPT" means female pipe thread.
3. "Relief" means size of FPT provided for installation of pressure relief valve.
4. "Blowdown" means size of FPT provided for tank drain.
5. "Water in and out" means size of FPT provided for water inlet and outlet connections to tank.

Table 9-3
Multiplying Factors Ensuring a 6-inch Water Seal Depth in a Horizontal Pressure Tank
(Use with Equation 9-2)

Tank Nominal Diameter, inches	Multiplying Factor MF = $V_t/(V_t-V_6)$
12	2.00
16	1.52
20	1.34
24	1.24
30	1.17
36	1.12
48	1.08
54	1.06
60	1.05
72	1.04
84	1.03
96	1.03
120	1.02

Note: Use *linear interpolation to determine MF values for diameters between those shown. Use an MF of 1.02 for horizontal tanks with diameters of 120 inches, or more*

9.1.5 Reduced Pressure Tank Sizing

Designs using variable frequency drive (VFD) pumping systems or pump cycle control valves (CCV) will result in a reduction in pressurized storage needed to protect pumps from over-cycling while maintaining adequate pressure in the distribution system. The criteria used to size pressure tanks serving a closed pumping system employing a VFD or CCV differs from the approach described in Sections 9.1.1 through 9.1.4.

Given that CCVs and VFDs deliver water within the controlled pressure range at flow rates much less than would be required under a standard design approach (see Sections 9.1.1 through 9.1.4) the size and/or number of pressure tanks for water systems using a CCV or VFD will be lower than those required by single speed pumps with on-off pressure switches. For additional information on CCVs and VFDs refer to Appendices B.2 and B.3.

Cycle Control Valves

A pump cycle control valve (CCV) may be used to control the pressure in a distribution system. The CCV is intended to extend run time with minimal pressurized storage. It will maintain constant downstream pressure (i.e., the valve's set point) until demand downstream of the valve falls below the valve's prescribed low flow level, at which point the pressure will rise to the pressure switch pump-off set point. The valve is mechanically prevented from restricting flow past its preset minimum.

Depending on the model used, the control valve will **stop** pump operation at a pre-set threshold flow of as little as 1 gpm or 2 gpm. At flows higher than this threshold, the valve will open or close in response to water system demands while the pump operates continuously. Design engineers who choose to use a CCV should include the head loss through the valve when determining the friction loss within the pump house.

The CCV is designed to keep the pump operating nearly all the time. For most water systems water demand will be very low during nighttime hours, resulting in prolonged pump operation at the upper end of its pump curve. If the manufacturer did not design the pump and motor for prolonged operation at that point on the pump curve, the pump will operate at low energy efficiency and at risk of premature failure. We recommend design engineers consult directly with the pump vendor or manufacturer to make sure the pump and motor are compatible with the intended operating conditions. Refer to Appendix B.2 for additional design information.

Variable Frequency Drives

A VFD is an electronic controller that adjusts the pump motor speed by modulating frequency and voltage. VFDs match motor speed and therefore pump output to specific water demand through a pressure control feedback loop to the variable frequency controller. Refer to Appendix B.3 for additional design information.

9.2 Labor and Industries Standards for Pressure Tanks

As of 2016, RCW 70.79.080 (5) requires that pressure vessels including bladder tanks greater than 37.5 gallons in gross volume must be constructed in accordance with ASME standards. The ASME standard is intended to promote a safe environment and protect against property damage, injury and death caused by an abrupt failure of the tank.

General Agreement

In 2011, Washington Department of Labor and Industries (L&I) added to a list of proposed changes to RCW 70.79.080 an exemption for non-ASME bladder tanks used in public water systems. When legislation is in conflict with practices that meet the intent of the rule (in this case, safe operation of bladder tanks used in public water systems), L&I can enter into a general agreement with another agency until such time as the legislation is changed. Design engineers are responsible for addressing all applicable L&I requirements at the time of pressure tank design. Refer to current L&I rules and legislation.

A General Agreement between L&I and ODW requires that design of non-ASME bladder tank systems conform to the standards shown in DOH publication 331-429. The General Agreement does not apply to hydropneumatic tanks. All hydropneumatic tanks **must** be constructed in accordance with the latest ASME specification code (RCW 70.79.080), regardless of size.

All pressure tanks greater than 37.5 gallons gross volume **must** have a properly sized and installed ASME Section VIII pressure relief valve (PRV) (WAC 296-104-316). Pressure tanks smaller than 37.5 gallons gross volume **must** have a properly sized and installed pressure relief device manufactured in accordance with a recognized national standard, the specifications and certification of which must be provided. We strongly recommend the use of an ASME Section VIII PRV for pressure tanks smaller than 37.5 gallons gross volume. PRVs protect a pressure vessel from over-pressurization due to a failure in the pump control system, over or intense heating of the water (e.g., during a fire), and pressure surge.

No isolation valves should be located between the PRV and the pressure tank. The potential for closure of the isolation valve during normal operations would negate the intended function of the PRV. For other design requirements and guidance, see DOH publication 331-429 *Pressure Relief Valves on Pressure Tanks* (DOH 331-429) online at https://fortress.wa.gov/doh/eh/dw/publications/publications.cfm.

The maximum allowable working pressure for a tank is on the nameplate attached to the tank. For nonstandard pressure vessels, engineers can determine the maximum allowable working pressure with the L&I formula in WAC 296-104-405. A properly sized ASME PRV should have a relieving capacity sufficient to prevent pressure in the vessel from rising more than 10 percent or 3 psi above the maximum design set pressure of the PRV, whichever is greater.

L&I contact information is provided in Appendix C.

9.3 Locating Pressure Tanks

Pressure tanks should be located above normal ground surface and be completely housed. Buried pressure tanks are subject to floatation by high groundwater, and allow external corrosion to go undetected. L&I standards require at least 18 inches of clearance around the tanks for proper inspection, maintenance, and repair access (WAC 296-104-260). In some cases, it may not be practical to provide this much clearance all the way around a pressure tank. Therefore, L&I developed a *Boiler/Pressure Vessel Clearance Variance Request* form (F620-041-000). It is available from the L&I Boiler/Pressure Vessel Web site (http://www.lni.wa.gov/forms/pdf/F620-041-000.pdf).

9.4 Piping

Pressure tanks should have bypass piping to permit the water system to operate while it is being repaired or painted. Process control elements such as a pressure switch or hydraulic valves should not be located such that they are isolated during bypassed operations. Sampling taps should be provided before and after the pressure tank(s).

9.5 Hydropneumatic Pressure Tank Appurtenances

Good engineering design includes the following appurtenances:

- An automatic pressure relief valve safely discharging to a building drain (with installed air gap) or outside of the building.
- No valves may be between the PRV and the pressure tank.
- Float switch controlling water surface elevation (needed to avoid water logging the tank).
- Air compressor and air filter. If the compressor is oil lubricated, only food-grade oil may be used as a lubricant. The air compressor should be located and air filter should be specified to ensure capture and compression of healthy air quality and ease of air filter inspection, maintenance, and replacement.
- Sight glass or other tank level indicator.
- Tank drain, pressure gauge, and pressure switch.
- Vertical and lateral support appropriate for soil conditions and seismic risk.
- Access hatch 24 inches in diameter allowing inspection of the interior, with clearance of at least 5 feet between hatch and adjacent structures (WAC 296-104-260).

9.6 Pressure Tank Sizing - Examples

Below are two examples illustrating design guidance provided in Sections 9.1.1 through 9.1.4.

Example 9-1 – Bladder Tank Sizing

For a mid-pressure range pumping rate, Q_p, of 40 gpm, a selected cycling of six cycles per hour, a bladder tank gross volume of 86 gallons, and a selected pressure range of 60/80, the number of 86-gallon tanks required is determined as follows:

$Q_p = 40$; $N = 6$; $V_B = 86$

Using Table 9-1 for $P_2/P_1 = 60/80$, $R = 76.1$

Using Equation 9-1:

$$T_s > \frac{(R)(Q_p)}{(N_c)(V_B)}$$

$$T_s > \frac{(76.1)(40)}{(6)(86)} = 5.9$$

Select six 86-gallon bladder tanks for pump protection, pre-charged to 55 psi (5 psi below pump-on pressure).

Example 9-2 – Horizontal Hydropneumatic Tank Sizing

1. Assume a small water system with the following:
   a. 50 connections.
   b. Peak hourly demand (from water system meter information) = 103 gpm.
   c. Well capacity is 60 gpm.
   d. Booster pump draws from ground level atmospheric storage and pumps into horizontal pressure tank with bottom outlet.
   e. Desired pressure range is 40/60 psig (minimum/maximum).
   f. Booster pump capacity is 110 gpm at 40 psig and 55 gpm at 60 psig.
   g. Booster pump capacity is 96 gpm at 50 psig [$(P_1 + P_2)/2$] as determined from the manufacturer's pump curve.
2. The pump cycling will be limited to no more than six cycles per hour.
3. Minimum water seal of 6 inches is required.
4. Pertinent data summarized:

   $P_1 = 60$ **$P_2 = 40$** **$Q_p = 96$** **$N_c = 6$**

5. Select a trial tank diameter of 42 inches. Using Table 9-3, the multiplying factor, MF, is 1.10 (by interpolation between the 36-inch and 48-inch tank sizes).
6. Substituting these values in the horizontal tank equation, Equation 9-2,

$$V_t = \frac{P_1 + 14.7}{P_1 - P_2} x \frac{15\,Qp\,(MF)}{Nc}$$

$$V_t = \frac{60 + 14.7}{20} x \frac{15\,(96)\,(1.10)}{6}$$

$$\mathbf{V_t \quad = \quad 986\ Gallons}$$

This is the minimum volume that will satisfy the 6-inch seal-depth requirement for a 42-inch diameter vessel. The tank selected from commercial charts will need to be equal to or greater than this volume.

7. A commercial tank table (see Table 9-2) shows there is a 42-inch tank with a volume of 965 gallons. This volume is close to the required 986 gallons, but it will not give a 6-inch water seal under the operating conditions stipulated. Therefore, if a tank with a 42-inch diameter is to be used, the next larger tank of 1,037 gallons is the one to select.

Note: *If a 48-inch diameter tank had been selected, a minimum volume of 968 gallons would be calculated. For this example, Table 9-2 shows a 994-gallon tank is available and acceptable. Note that the 48-inch tank would be about four feet shorter than the 42-inch tank. That may be an important consideration when placing a tank in limited space.*

Chapter 10: General Water Treatment

10.0 Introduction

Effective and reliable water treatment is essential to protect public health and promotes consumers' confidence in the water they receive. This chapter includes general information on treating groundwater, seawater, and surface water. Since there are many unique aspects and regulatory requirements associated with the design of surface water treatment facilities, detailed guidance on surface water treatment is provided in Chapter 11.

Enforceable drinking water standards are described in regulation as treatment techniques, action levels, or maximum contaminant levels (MCL). These standards form the minimum treatment objectives for any water treatment design. Consumer and water system expectations may go beyond minimum regulatory requirements. As such, design engineers are encouraged to develop designs that focus on providing a greater level of public health protection than just meeting regulatory standards. Providing this higher level of public health protection is called optimization. Voluntary programs that focus on optimized treatment include the AWWA Partnership for Safe Water and EPA Area Wide Optimization Program. These programs, and similar ones adopted in Washington State, have developed water quality optimization goals related to arsenic, disinfection, and surface water treatment.

Public notification that explains potential health risks to consumers may be required when treatment disruptions occur (WAC 246-290-71001). When evaluating water treatment alternatives, design engineers should consider the full range of source water characteristics, availability of skilled operators, capital and operational costs, and water system acceptance of the treatment technology.

Considering these factors, non-treatment alternatives such as consolidating with a nearby system, improving source water protection, or abandoning and replacing the contaminated source are often a better long-term approach to protect public health than constructing a treatment facility. If treatment is the best long-term solution, this chapter is structured to help design engineers select and design a treatment process that is appropriate for the community's needs and resources, protects public health, and supports consumer confidence in their water utility.

The overall structure of the chapter is as follows:

- Alternative Analysis (Section 10.1)
- Treatment Technologies (Section 10.2) including:
    - Disinfection
    - Fluoridation
    - Corrosion Control
    - pH Adjustment
    - Chemical and Radiological Contaminants
- Predesign Studies (Section 10.3) including pilot studies

- Project Reports (Section 10.4) including identifying
  - Design Criteria
  - Process Control -Monitoring, Instrumentation, and Alarms
  - Start-up, Testing Procedures and Operations
- Construction Documents (Section 10.5)
- Treatment Chemicals (Section 10.6)
- Cross-Connection Control for Water Treatment Facilities (Section 10.7)
- Waste Residuals Management (Section 10.8)
- Placing a Water Treatment Plant into Service (Section 10.9)

The water treatment design process is usually more involved than the design of reservoirs, pump stations, and other types of projects. Figure 2-3 outlines the general design and review process for water treatment projects. In most cases, the design process begins with an assessment of treatment alternatives.

10.1 Alternative Analysis

A water treatment facility is a major capital investment with high life-cycle costs and a potential risk to the public if the treatment processes fail to operate as intended. Therefore, the engineer **must** evaluate all appropriate and applicable alternatives, and justify the selected option in reports submitted to ODW for approval (WAC 246-290-110(4)(c)).

This chapter includes information design engineers can use to screen potential treatment alternatives prior to undertaking an in-depth analysis. The engineer should cover the following items in the analysis of alternatives with more detailed guidance provided in professional references (AWWA/ASCE 2012a; Kawamura 2000b):

- Current and future capacity needs
- Source water quality
- Secondary impacts of treatment
- Operations and maintenance considerations
- Waste disposal and management
- Life cycle costs
- Site considerations

Point-of-use (POU) and point-of-entry (POE) treatment is not a viable option for compliance with drinking water standards in Washington. ODW limits the use of POU and POE treatment because their application is incompatible with existing regulatory requirements (WSDOH 2007). A limited exception to this restriction applies to non-community water systems that use a POE treatment device to treat all the water entering a single-building water system.

10.1.1 Source Water Quantity

The finished water quantity objectives are closely tied to the utility's expectations of future capacity requirements. See Chapter 4 for further details on estimating future water system capacity needs. Design engineers should clearly define future water supply expectations before beginning preliminary evaluation of water supply and treatment alternatives.

The safe yield of any potential water source is the quantity of water – annual, seasonal, and daily – reliably available to the treatment facility. Water rights are another limiting factor. See Chapter 5 for recommendations on establishing an appropriate value for a supply's safe yield.
The design engineer should carefully consider the efficiency of any proposed treatment process against supply limitations and expected supply needs. Some water treatment processes, especially adsorption and filtration processes, need to be periodically backwashed or otherwise regenerated. The amount of water required by the treatment process to backwash, regenerate, rinse, and/or filter to waste needs to be considered when determining the maximum daily treatment capacity and raw water supply requirements.

10.1.2 Source Water Quality

In general, water systems **must** use the highest quality sources feasible (WAC 246-290-130(1)). Source water and finished water quality objectives form the basis for selecting treatment process alternatives for evaluation. The extent and availability of source water data may affect preliminary screening of alternatives and the duration of the pre-design study. ODW recommends that the design engineer contact one of our regional offices to discuss source water monitoring needs in light of available source water quality data. The design engineer should collect the necessary data to evaluate the efficacy of viable treatment technologies.

Engineers can use a limited source water sampling program to characterize groundwater wells. However, we caution design engineers against relying on a single source water sample because doing so may fail to reveal important changes in water quality over the course of a year. For example

- Nitrate in groundwater may fluctuate seasonally due to rainfall, irrigation practices, and other seasonal land use practices.

- Arsenic concentration in groundwater supplies may vary due to seasonal changes in aquifer level.

- Determining the potential for disinfection byproduct formation associated with addition of chlorine in coastal groundwater may require analysis of an extended suite of unregulated water quality parameters such as ammonia, bromide, total organic carbon, dissolved organic carbon, and chloride over a period of months due to seasonality.

Design engineers should consider an extended source water sampling program to characterize surface water. Surface water quality is highly variable due to changing weather conditions; primarily rainfall patterns and snowmelt conditions. Chapter 11 (Section 11.1.3) and Appendix F provide further guidance on collection of source water quality in support of evaluating surface water and groundwater treatment process alternatives, respectively.

10.1.3 Secondary Impacts of Water Treatment

Water systems **must** review how proposed projects could potentially affect water quality in the distribution system (WAC 246-290-110(4)(d)). Source changes, or a new or significantly modified treatment process can affect distribution system water quality. These water quality impacts could include:

- Release of accumulated organic and inorganic contaminants from pipe walls
- Increased corrosion of metals
- Impacts on the ability to maintain a disinfection residual (Taylor et al. 2005; Kippin et al. 2001)

Design engineers should identify how the water system will address such issues. Commonly-used approaches include:

- Bench scale studies
- Pipe loop studies with new or existing distribution system materials
- Enhanced monitoring and flushing programs

Design engineers **must** specifically address how a change in treatment may affect compliance with the Lead and Copper Rule (40 CFR 141.81(b)(3)(iii); 141.86(d)(4); and 141.90(a)(3)). Examples of treatment changes that could increase corrosivity and lead/copper solubility as well as other distribution system issues include:

- **Introduction of a disinfectant.** Disinfectants can affect corrosivity, metal release, or both (Schock and Lytle 2011). For example, the initial introduction of chlorine can increase the release of copper (Stone et al. 1987). Corrosion rates of both mild steel and copper were found to be higher in the presence of free chlorine than in its absence (Pisigan and Singley 1987).

- **Changes in residual disinfectants.** For example, switching from free chlorine to chloramine as a residual disinfectant can lead to significant lead release (Edwards and Dudi 2004; Boyd et al. 2008). A change to the oxidation-reduction potential can increase the risk of destabilizing tubercles and the release of bound and harbored biological and inorganic compounds.

- **Switching coagulant chemicals.** Changing coagulants can increase the chloride to sulfate mass ratio, causing an increase in lead release (Edwards and Triantafyllidou 2007; Nguyen et al. 2010).

- **Installation of additional treatment.** For example, the installation of ion exchange can change the pH, alkalinity, and chloride to sulfate mass ratio increasing lead and copper release at the tap (Nguyen et al. 2010).

Water systems that change treatment or introduce a new source of supply may be required to complete additional rounds of lead and copper tap sampling (40 CFR 141.86(d)(4)(vii)).

10.1.4 Operations and Maintenance Considerations

During the analysis of treatment alternatives, the design engineer should consider:

- **Expected operational capability of the water system.** This depends on the system's size. Smaller water systems often can't provide the same level of operational capability as large water systems. Engineers should select technology appropriate for the anticipated level of expertise, time, and resources the water system will devote to operating the treatment facility.
- **Operator certification.** Water systems **must** be able to meet certification requirements that apply to the proposed treatment technology (WAC 246-292-050). The design engineer should make a preliminary determination of the operator certification level for any proposed treatment facility using ODW's Purification Plant Rating worksheet. Include operator staffing for multiple shift operations as appropriate.
- **Operator safety.** Design engineers should minimize, to the extent practical, the use of hazardous materials (caustic soda, gaseous chlorine, and acid), confined spaces, and fall hazards.
- **Reliability and Ease of Operation.** Some treatment process are simpler and inherently more reliable than others, and require less work by operators. The treatment process stability and complexity should be considered, especially for smaller systems with part-time or contract operators.

10.1.5 Treatment Plant Waste Disposal

Water treatment plants that remove contaminants will have backwash water that needs to be disposed of properly. The Department of Ecology (Ecology) is the lead agency in permitting all water treatment plant wastewater discharges such as sludge, backwash water discharged to waste, ion exchange waste streams, and membrane reject water. Most water treatment plant waste discharges are permitted under either a National Pollutant Discharge Elimination System (NPDES) permit (general or individual), or a state waste discharge permit.

Water systems developing water treatment proposals should evaluate waste-product issues early because they could significantly affect the cost or feasibility of a proposed approach or technology. See Section 10.8 for more detailed information on waste disposal considerations.

10.1.6 Life Cycle Cost Analysis

Estimated capital and annual operating costs **must** be included in the project report for any proposed treatment project (WAC 246-290-110). Preliminary cost estimates should have the detail and accuracy purveyors need to make decisions about treatment system alternatives. Preliminary construction cost estimates should approach an accuracy of plus-or-minus 30 percent (AWWA 2012b).

Capital costs: Location, capacity, site constraints, water system hydraulics, and source water quality all affect capital costs. They can vary significantly from facility to facility. Engineers can sometimes use cost curves to develop preliminary construction costs for specific treatment processes and then adjust them for inflation and local conditions. Engineers can also use price quotes from equipment manufacturers, local construction experience, and information from similar projects to develop preliminary construction costs.

Operations and maintenance costs: A water treatment facility's operations and maintenance (O&M) costs include labor, power, maintenance, repair, supplies, and services. Engineers can prepare preliminary O&M cost estimates with methods similar to those used for construction costs. However, lack of published data may make detailed component cost assessments necessary to evaluate alternative treatment methods for small water systems.

10.1.7 General Water Treatment Plant Site Considerations

In general, water treatment plant designs **must** comply with state or locally adopted building, mechanical, electrical, and land use codes (WAC 246-290-200(b)). The contents of these codes, not to mention local ordinances, is beyond the scope of this manual. Overall, you should review locally adopted codes and ordinances that could impact the siting and design of a treatment facility in a project report. These considerations include, but are not limited to:

- Zoning compliance, building code compliance, and community acceptance.
    - Noise can be an issue for treatment plants located near parks or residences. The maximum permitted sound level can be as low as 45 dBA in residential areas at night (WAC 173-60-040).
- Operator access, equipment maintenance, and safety. The pumps and other mechanical equipment need periodic maintenance. As such, the treatment plant should be designed to make it easy to inspect, operate, and maintain the equipment including:
    - Be aware of creating permit required confined spaces. Treatment plants located in below grade vaults or other permit-required confined spaces as defined in Chapter 296-809 WAC can create operations and maintenance issues.
    - Provide adequate space around mechanical equipment and electrical equipment. In general, at least 36 inches clearance is recommended between piping, pumps and other mechanical equipment. The minimum clearance in front of electrical panels is governed by electrical codes, though is at least 36 inches and can be 60 inches or more for high voltage panels (Sanks et al. 1998; AWWA/ASCE 2012f).
    - Facilitate removal and installation of heavy valves and equipment. In general, any piece of equipment that weighs more than 100 pounds should be accessible by crane or have other lifting assistance. Other means of access include large doorways or roof hatches to facilitate removal of heavy equipment directly into a truck. Areas where the operator will walk or maintenance will be performed should be clear of overhead obstructions to a height of at least 7 feet (Sanks et al. 1998).
    - Hearing protection and other measures to protect people in the pump station are required when the noise exceeds 85 dBA (Chapter 296-817 WAC).
- Geotechnical engineering field investigations including:
    - Site drainage
    - Soil type and soil-bearing strength

    - Groundwater table elevation
    - Soil stability/liquefaction/slope failure analysis
- Electrical power supply
    - Reliability – The reliability of the power supply should be assessed along with the necessity of standby generators. See Section 5.11.1.
    - Sizing – In some cases, such as for very large pump stations and ones located in rural areas, the capacity of the electrical grid may limit the suitability of some sites or require upgrades in the local electrical service that will need to be factored into siting decisions.

10.1.7.1 Natural Hazard Considerations

Natural disasters have the potential to damage treatment plants to the point that they may fail to operate. Treatment plants should be designed and located to minimize their vulnerability to damage in the event of natural disasters such as:

- Avalanches
- Earthquakes
- Floods
- Landslides
- Tree falls
- Tsunamis
- Windstorms

To meet state and local requirements, engineers **must** address geologic risk (seismic and unstable slopes) when designing treatment plants (WAC 246-290-200). The Washington State Department of Natural Resources has geologic hazard maps that can be used to identify seismic and other natural hazards.

Seismic risk may be reduced or mitigated by:

- Knowledge of soil liquefaction potential
- Proper anchorage and foundation design
- Proper structural design
- Flexible couplings and pipe that can accommodate significant ground movement

Various design guidelines highlight the multiple seismic vulnerabilities of piping and large mechanical equipment in some treatment plants (ALA 2002; ALA 2004). In areas with the potential for significant ground motion, you may need to seek the services of a professional qualified to assist in the design of pipe bracing, equipment support, and other aspects of design.

10.1.8 Variances

Under a set of very specific criteria, a water system may qualify for a variance from compliance with certain drinking water standards. A variance allows a water system serving fewer than 10,000 people to install treatment and be considered in compliance while at the same time exceeding a drinking water standard. ODW will not consider variances for compliance with coliform MCL or treatment technique requirement, or consider a variance for any surface water treatment requirement (WAC 246-290-060(2)).

As of 2018, ODW hasn't granted a variance from a primary MCL or treatment technique. A small system may receive a variance under a particular national primary drinking water regulation only if:

1. The contaminant was not regulated prior to January 1, 1986 (See 40 CFR 142.304).
2. EPA has identified an applicable variance technology.
3. The water system actually installs, operates, and maintains the specific technology in question.
4. The water system provides extensive public notification as described in federal regulations (See 40 CFR 142.308).

10.2 Treatment Technologies

This section and the tables at the end of this chapter provide information about many of the more commonly used treatment technologies for:

- Disinfection (Section 10.2.1; Table 10-7)
- Disinfection byproducts (DBPs) control (Section 10.2.2; Table 10-11)
- Fluoridation (Section 10.2.3)
- Corrosion control (Section 10.2.4; Table 10-8)
- pH Adjustment (Section 10.2.5)
- Inorganic chemical (IOC) removal (Section 10.2.6; Table 10-9)
- Volatile organic chemical (VOC) and synthetic organic chemical (SOC) removal (Section 10.2.7; Table 10-10)
- Radionuclides removal (Section 10.2.8)
- Emerging contaminants (Section 10.2.9)

Treatment technologies for surface water are covered in Chapter 11.

As an alternative to physically removing a contaminant, in some cases it may be appropriate to blend sources to achieve compliance with a drinking water standard. ODW considers blending a form of treatment. A project report and construction documents are required for any blending project. The engineering analysis for blending should include:

- **An analysis of the water quality** from the sources being consider including any seasonal changes in water quality that may affect the blending strategy.

- **A description of the blending strategy** including calculations of flow and finished water quality, monitoring, controls, and alarm or shutdown conditions.

- **A preliminary design and schematic** that shows piping, control valves, monitoring points, and other important features.

Blending is most often used to address inorganic and organic contaminants. In some cases, both blending and physical treatment are used to address contaminants of concern. Blending cannot be used to address microbial risks.

Most of the treatment technologies in Tables 10-7 through 10-11 are established technologies widely used with established control and monitoring strategies. Alternative technologies without such extensive design and operating experience will usually require more analysis and pilot testing to establish the design parameters, process control, and technology-specific monitoring strategies. Given the additional engineering and analysis that may be required, design engineers should check with one of our regional offices before proceeding with an alternative technology.

10.2.1 Disinfection

The design approach to disinfection treatment will differ depending on the intended purpose of the application. In some cases, disinfection is designed to address microbial risk in the source water, also referred to as primary disinfection. In other cases, it is designed to address microbial risks in the distribution system, referred to as secondary disinfection. Some systems, including all surface water systems, must provide both primary and secondary disinfection treatment. Disinfection requirements for surface water sources are covered in Chapter 11.

Disinfection is the most common form of potable water treatment in Washington. Over half of the state's 8,000 Group A drinking water sources are treated with a disinfectant. Of these sources, more than 90 percent are treated with hypochlorite, commonly called free chlorine. For this reason, this section is mostly focused on the use of free chlorine as a primary or secondary disinfectant. Free chlorine is also used as an oxidant in processes that remove inorganic contaminants. See Section 10.2.1.3 for more information on the use of chlorine as an oxidant.

Adding or changing a chemical disinfectant will change water chemistry and may generate secondary impacts beyond DBP formation. These secondary impacts may result in significant water quality changes in the distribution system such as release of corrosion byproducts due to changes in oxidation-reduction potential (ORP). See Sections 5.3 and 10.1.3 for further details.

Additional information on disinfectants is presented in Table 10-7. Additional performance criteria for groundwater and seawater disinfection are in Chapter 246-290 WAC, Part 5. Information specific to primary and secondary disinfection of surface water supplies is covered in Chapter 11.

10.2.1.1 Source Water Quality

Design engineers **must** evaluate source water quality in the design of any primary or secondary disinfection system (WAC 246-290-110). Some water quality parameters, such as iron, manganese, natural organic matter, dissolved organic carbon (DOC), ammonia, and sulfide exert a chlorine demand. If natural ammonia is present in the source water, it can present multiple challenges including difficulty in maintaining a free chlorine residual and creating taste and odor problems. Chemical disinfectants such as ozone, chlorine, chlorine dioxide, and chloramines produce disinfectant byproducts (DBPs). Some source waters, especially those with high concentrations of natural organic matter, can lead to the formation of DBPs at levels of public health concern. In addition to understanding chlorine or other disinfectant demand of the source water, design engineers should assess the disinfectant byproduct formation potential when adding or changing a chemical disinfectant.

When designing treatment that includes the addition of chlorine as a disinfectant or oxidant, the following source water quality parameters should be analyzed:

- Ammonia, to assess the efficacy of chlorine as a disinfectant and oxidant

- DOC, to assess HAA and THM formation potential. We found values less than 1.0 mg/l DOC are unlikely to result in exceedances of the TTHM MCL

- Bromide and chloride (in coastal ground water sources), to assess brominated HAA and THM formation. The highest proportions of brominated HAA and THM formation can be expected from sources with relatively low DOC concentrations and higher $[Br^-]$ levels (i.e., high $[Br^-]/[DOC]$ ratios).

We found that total formation potential tests for THMs and HAA5 do not correlate with regulatory TTHM and HAA5 test results in distribution systems studied at length, and therefore recommend design engineers use discrete DOC and bromide test results in assessing future DBP formation.

For additional guidance on source water information useful in the design of a new chlorination system refer to Appendix F.1.

10.2.1.2 Primary Disinfection of Groundwater Sources

Unanticipated environmental conditions, or other factors beyond the control of the utility, may adversely affect source water quality at any time. Engineers should account for primary disinfection in the design of each groundwater source. After-the-fact retrofitting an operational source, pump house, or transmission facilities can be expensive and disruptive to utility operations.

If the department determines that a groundwater source is vulnerable to microbiological contamination or water quality data confirms *E. coli* contamination, then a water system **must** provide continuous primary disinfection of the source (WAC 246-290-451 or -453, respectively).

The design of primary disinfection with free chlorine **must** provide a CT equal to or greater than 6 without exceeding the total chlorine maximum residual disinfectant level (MRDL) of 4 mg/L. The CT value is calculated by multiplying the free chlorine residual concentration (“C”, in mg/l) by the chlorine contact time (“T”, in minutes). CT6 is applicable where the treated water temperature is greater than or equal to 10°C and the pH is in the 6 to 9 range. If the temperature is less than 10°C, the minimum required CT is 8. For ground water outside the range of 6 – 9, contact your regional office.

For primary disinfection with other disinfectants, see CT requirements for 4-log virus inactivation in Section 10.2.1.5 and Table 10-1.

Contact time (T) is calculated at peak hourly flow between the point of chlorine injection and the residual chlorine monitoring location. The residual disinfectant monitoring location must be at or before the first customer. Assessing the contact time ("T", in minutes) can be a challenging part of the design process since contact time depends upon the baffling efficiency of the tanks, pipes, and reservoirs through which the water flows. More detailed information on estimating the baffling efficiency of various structures and contact time is in Chapter 11. For additional guidance on disinfection by free chlorine refer to Appendix F.1; on ozone disinfection refer to Appendix F.5; and on UV disinfection refer to Appendix I.

10.2.1.3 Secondary Disinfection of Distribution Systems

Secondary disinfection is the addition of a chemical disinfectant at a source or pump station to maintain a distribution system residual in order to establish microbiological control throughout the distribution system. If the treatment is limited to secondary disinfection, CT6 and 4-log design and operational monitoring requirements don't apply. However, considerations for disinfection residual monitoring in the distribution system **must** be included in the design so the water system can to demonstrate maintenance of a detectable residual in all active parts of the distribution system (WAC 246-290-451(7)).

10.2.1.4 Monitoring Plans

Monitoring is a key part of the design of any disinfection system. Delivering water continuously treated with free chlorine, chloramines, chlorine dioxide, or ozone triggers disinfection residual monitoring requirements (WAC 246-290-300(6)(c); -451; -453). The addition of a chemical disinfectant to a source **must** include pre-treatment and post-treatment source sample taps for monitoring water quality (WAC 246-290-300(3)(h); -451(6)(c)). A DBP monitoring plan should accompany the design of a new disinfection system.

The addition of a disinfectant or change in disinfection practice can also affect coliform monitoring practices. A system adding a disinfectant **must** measure and record the residual disinfectant concentration in samples collected at the same time and location that routine and repeat coliform samples are collected (WAC 246-290-451(9)). Adding a disinfectant may change a system's contingency planning for detection of *E. coli*. For this reason, every water system **must** update their coliform monitoring plan as part of any change in disinfection practice (WAC 246-290-300(3)(b)). Refer to our coliform monitoring plan guidance (DOH Publications 331-036 and 331-240) for additional information.

Chlorine or another disinfectant used solely as an oxidant can affect the water quality served to customers. Design engineers **must** still address issues such as:

- Disinfection byproducts monitoring
- Monitoring and maintenance of chlorine residual in the distribution system (WAC 246-290-451(7) and (9); -300(6))

10.2.1.5 Disinfection of Seawater or Brackish Water Source

Open seawater sources **must** be treated to a minimum CT6 prior to the first connection. (WAC 246-290-451(4)). This level of primary disinfection provides an additional barrier in case the integrity of a membrane or another water system component is compromised. Refer to our design checklist for desalination of seawater sources in Appendix F.6.

Design engineers should assess the requirement for primary disinfection of a saline well treated with reverse osmosis the same as any other groundwater supply. Considerations should include:

- Construction of the well
- Degree of wellhead protection
- Aquifer characteristics
- Bacteriological history of the source

10.2.1.6 Alternative Disinfectants

Chlorine dioxide, chloramines, ozone and ultraviolet (UV) disinfection can be used instead of or along with chlorine, to address microbial risks, though each disinfection approach has limitations. Use of iodine is strongly discouraged and has very limited permitted use as a drinking water additive. See Policy F.01.

For primary disinfection of groundwater, free chlorine is usually a more practical choice than alternative disinfectants based upon the required CT values compiled in Table 10-1. This table also includes the maximum residual disinfectant levels (MRDLs) for these disinfectants. Initial design references for these alternative disinfectants are available elsewhere (USEPA 1999; USEPA 2006; Ten State Standards 2012).

Table 10-1
Disinfection Requirements

Disinfectant [1,2]	CT required for 4-log virus inactivation At 5°C (mg-min/L)	At 10°C (mg-min/L)	MRDL (mg/L)
Chlorine	8	6	4.0 as Cl_2
Chloramines	1988	1491	4.0 as Cl_2
Chlorine Dioxide	33.4	25.1	0.8[3]
Ozone	1.2	1.0	Not applicable
UV	186 mJ/cm^2	186 mJ/cm^2	Not applicable

Notes:

1. Chemical disinfection requirements are from Appendix E of the *Guidance Manual for Compliance with the Filtration and Disinfection Requirements for Public Water Systems Using Surface Sources* (USEPA 1990)
2. UV disinfection requirements for viruses are in 40CFR 141.720(d)
3. If a violation of the MRDL for chlorine dioxide occurs, it is a Tier 1 violation requiring public notification soon as possible and no later than 24 hours after the violation occurs.

Ozone

Ozone is a very strong oxidant and highly reactive. In drinking water, it is used as an oxidant and for primary disinfection of surface water sources. Because it is so reactive, ozone is not feasible for secondary disinfection, and special materials and equipment are required in the design of ozonation facilities. In addition to monitoring for THMs and HAA5, water systems that use ozone **must** also monitor for bromate at the entry point to the distribution system (WAC 246-290-300(6)(b)(iii)). Any water system planning to use ozone should carefully review fire code and worker safety requirements associated with its use. For additional guidance refer to Appendix F.5.

Chlorine Dioxide

Chlorine dioxide is a strong disinfectant that may be used for both primary and secondary disinfection. To date, it has had limited use in Washington State. The use of chlorine dioxide involves special monitoring and reporting requirements. The monitoring requirements include daily measurements for chlorine dioxide and chlorite at the entry point to the distribution system (WAC 246-290-300(6)(b)(ii)). There are also special requirements for monitoring chlorine dioxide and chlorite in the distribution system. If the concentration of chlorine dioxide in the distribution system exceeds 0.8 mg/L, consumers **must** be notified immediately (WAC 246-290-300(6)(b)(ii)). Some end users such as hospitals, kidney dialysis centers, and customers with fish tanks should be notified as these end users can be especially sensitive to chlorine dioxide and its byproducts.

Chloramines

Chloramines are formed when free chlorine reacts with ammonia. Monochloramine is the main disinfecting agent and it is considered a weak disinfectant. As a result, it is generally used solely as a secondary disinfectant. Special concerns associated with the use of chloramines include the sensitivity of some customers, such as hospitals, kidney dialysis centers, and customers with a fish tank, and the potential for increased corrosion and damage to some materials in the distribution system. See Section 10.1.3 and Table 10-7.

UV Disinfection

UV is very effective at relatively low doses for the inactivation of *Giardia lamblia* and Cryptosporidium, pathogens present in surface water supplies. The UV dose required for inactivation of viruses is significantly higher, which limits the use of UV as a primary disinfectant for groundwater. UV reactors also do not provide a residual disinfectant. Appendix I includes additional guidance on the application of UV disinfection.

Dichlor and Trichlor

Dichloroisocyanuric acid (dichlor) and trichloroisocyanuric acid (trichlor) are strong biocides and oxidizers produced in granular or tablet (solid) form. When dissolved in water they produce hypochlorous acid, (free chlorine) and chloroisocyanuiric acid. Residual chloroisocyanuiric acid interferes with standard methods for the analysis of free chlorine. This interference overstates the actual level of free chlorine. As a result, we will not approve designs for primary or secondary disinfection that use dichlor or trichlor as the source of free chlorine.

10.2.2 Disinfection Byproducts

Disinfection byproducts (DBPs) can form whenever a chemical disinfectant is added to drinking water. Therefore, all community and non-transient non-community (NTNC) water systems that distribute water to which a disinfectant has been added **must** monitor for DBPs and have a DBP monitoring plan (WAC 246-290-300(6)). All affected water systems **must** monitor for total trihalomethanes (TTHMs) and the five currently regulated haloacetic acids (HAA5) with special monitoring requirements for water systems that use chlorine dioxide or ozone (WAC 246-290-300(6)(b)). If a DBP MCL violation occurs, corrective action including removal of DBP precursors and/or operational changes (treatment, storage, distribution) to limit DBP formation will be required (WAC 246-290-320).

Certain unregulated water quality parameters can significantly influence the potential for DBP formation following chlorination. Dissolved organic carbon (DOC) in the source water is the primary factor affecting DBP formation in chlorinated groundwater. Source water DOC concentration over multiple seasons should be measured to assess the likelihood of excessive HAA and THM concentration in water treated with chlorine. Ammonia exerts a strong demand for free chlorine. Bromide and chloride should be measured in coastal ground water sources when considering the addition of chlorine.

Water systems may be able to limit DBP formation by minimizing the contact between chemical disinfectants and DBP precursors such as natural organic matter. If DBP results are of concern, design engineers and water systems should initially consider simple operational changes. These operational changes include:

- **Modifying disinfection practices –** Filtration typically removes DBP precursors. Therefore, adjusting pre- and post-filtration practices can reduce the formation of DBPs.

- **Decreasing water age or stagnation –** In most cases DBPs gradually increase in the distribution system with water age. Ways to minimize water age and decrease stagnation include manual and automatic flushing, reservoir mixing, and adjusting storage volumes.

- **In-reservoir aeration** – Some THMs are can be removed through aeration processes including in-reservoir aeration. In-reservoir aeration usually involves retrofitting the reservoir inlet with spray nozzles, and possibly other modifications. With aeration, there will likely be some loss of chlorine residual, so downstream adjustments may be needed.

Some of these changes may require submittal of a project report and construction documents. Please contact your regional office to determine if a submittal is required.

If operational changes are insufficient in addressing DBP issues, other actions may be necessary. Common treatment technologies used to minimize DBP formation include:

- **Alternative oxidants and disinfectants –** If an oxidant is needed, oxidants such as permanganate, air, and oxidizing media that do not form DBPs can be used instead of free chlorine. If disinfection is required, alternative disinfectants such as monochloramine, chlorine dioxide and UV disinfection can be evaluated. Some changes in disinfection practice can cause increased lead and copper release. See Section 10.1.3. Alternative disinfectants are covered in more detail in Section 10.2.1.6.

- **Granular media filtration –** Some types of filtration can be used to remove natural organic matter. These processes include rapid rate filtration commonly used to treat surface water, and pressure filtration for arsenic, iron, and manganese removal. In these processes, the physical removal of DBP precursors is dependent upon the dose of aluminum or iron based coagulants (or amount of naturally occurring iron), polymer addition, and physical parameters such as filtration rate, media type, and depth.

- **Biological filtration –** Slow sand filtration, which combines both biological and physical/chemical processes, and biologically active rapid rate filtration, where the biological activity in rapid rate filters is enhanced, can provide DBP precursor removal that may be greater than purely physical/chemical removal processes. Low temperatures reduce biological activity, so more extensive pilot testing may be necessary to assess the biological aspects of the filtration process and resulting impacts on DBP removal.

- **Membrane filtration –** Low-pressure membrane filtration, such as microfiltration and ultrafiltration, by themselves, are ineffective at removing DBP precursors. However, if a coagulant, such as aluminum chlorohydrate (ACH) is added prior to filtration, effective DBP precursor removal can be achieved. Pilot testing is needed to determine the appropriate coagulant dose. Too much coagulant can lead to excessive membrane fouling. Too little coagulant can result in poor DBP precursor removal and other operational issues.

- **Granular activated carbon (GAC) –** GAC is typically installed in a contactor either before or after filtration, or as a cap on rapid rate filters. Pilot testing is necessary to determine how long the GAC media will be effective before it needs to be replaced and the associated operational costs. GAC removes dissolved organic carbon (DOC). DOC is a precursor in the formation of DBPs. As long as GAC remains effective at removing DOC there should be little DBPs formed. Measurement of UV 254 absorbance is a surrogate for the presence of DOC. UV 254 absorbance is a useful surrogate for the on-going effectiveness of GAC to remove DOC. An increase in UV 254 absorbance is an indicator the carbon filter is exhausted and DBP levels will likely increase. Weekly UV 254 absorbance field testing should accompany DBP treatment by GAC. If GAC is also used as filtration media, its effectiveness for pathogen and other contaminant removal needs to be evaluated through pilot testing.

- **Powdered activated carbon (PAC) –** PAC is fed prior to filtration and removed in the filtration process. Key initial design parameters include the PAC characteristics, effective dose, and contact time. Bench and pilot testing is usually necessary to determine the appropriate design parameters.

- **Anion exchange –** Both fixed bed anion exchange systems and recirculating ion exchange systems such as MIEX® can be considered for the removal of DBP precursors.

Fouling of the ion exchange resin over time, pH effects, and waste disposal of the salt brine used to regenerate the ion exchange resin are common design concerns.

Additional information on these and other technologies are in Table 10-11 at the end of this chapter. EPA also has a number of manuals on DBP removal and monitoring (USEPA 1999a; USEPA 1999b; USEPA 2001).

10.2.3 Fluoridation

The Department of Health supports community water fluoridation as a sound, population-based public health measure. The decision to add fluoride to a public water system is made by the local community.

The optimal treated water fluoride concentration is 0.7 mg/L. Water systems that fluoridate **must** maintain a fluoride concentration between 0.5 mg/L and 0.9 mg/L in the distribution system (WAC 246-290-460). This requirement ensures fluoridation is tightly controlled, effective, and reliable.

Technologies used for fluoridating drinking water include liquid and dry feed systems. This manual does not contain specific recommendations for fluoridation technologies, which are available in references such as *Water Fluoridation: A Manual for Engineers and Technicians* (Reeves 1986), the *Recommended Standards for Water Works* (Ten State Standards 2012), and *Water Fluoridation Principles and Practices* (AWWA 2016). Appendix F.2 includes a checklist for the design of sodium fluoride saturators. ODW does not require pilot studies for fluoridation facilities.

Fluoride design recommendations include:

- The maximum chemical feed rate attainable from the chemical metering pump(s) or feeder(s) should not exceed twice the recommended dose.

- There should be two interlocks on any fluoride feed equipment to ensure that it does not start until after water is flowing past the fluoride feed point.

- For liquid fluoride chemical feeders, anti-siphon check valves should be installed in the solution line at two locations: 1) at the pump head, and 2) at the point of injection.

- Where applicable, the plugs or fill ports for fluoride liquid storage tanks should be clearly labeled and of a unique size or shape to reduce the risk of inadvertent addition of any other chemical to a fluoride storage tank.

- The potable water connection to any fluoride saturator **must** be equipped with either an air gap or a properly installed and tested reduced pressure backflow assembly (WAC 246-290-490(4)(b)).

- Fluoride feed-rate design assumptions should be verified during the start-up, testing, and operation period.

- Prior to start-up, operators of fluoridated systems should receive at least six-hours of DOH-approved “fluoridation basics” training for water operators.

See Section 10.6 for additional information on safe chemical handling and storage.

10.2.4 Corrosion Control

Water systems exceeding the lead or copper action level and all large water systems (serving more than 50,000 people) **must** implement optimal corrosion control treatment (chapter 246-290 WAC and 40 CFR 141.80 through 141.90). Large systems must conduct a corrosion control study. Systems serving less than 50,000 people can submit a corrosion control recommendation report unless direct to complete a corrosion control study. Engineers can use the *Optimal Corrosion Control Treatment Evaluation Technical Recommendations for Primacy Agencies and Public Water Systems* (USEPA 2016) to identify appropriate technologies for water systems that exceed the lead or copper action level. Water systems that exceed an action level should contact one of our regional offices. Table 10-8 cites commonly used corrosion control technologies and identifies issues associated with them.

We recommend design engineers use pipe-loop or other pilot scale work to evaluate actual corrosion or corrosion rates using a proposed treatment approach. ODW recommends that bench or pilot scale testing for selected technologies (aeration, calcite contactors, and pH adjustment) be used to verify that a proposed design dose-rate will meet treatment objectives (target pH or alkalinity). These studies may be oriented toward ensuring that target pH/alkalinity goals are met rather than measuring resulting corrosion rates. In some cases, water systems have had difficulty matching full-scale results to bench scale data. Purveyors proposing to use aeration or air stripping should conduct a pilot test to confirm the ability of the process to adequately increase the treated water pH.

Many corrosion control approaches include chemical feed facilities to adjust the treated water quality to make it less corrosive to distribution, plumbing, and service lines materials. As such, protection from treatment chemical overfeed is typically included to minimize the risk to public health. See Section 10.6.1 and Table 10-3 for more information on design features to decrease the risk of chemical overfeed.

Lime and soda ash feed systems may be operator intensive because of the potential for plugging feed equipment and piping. Engineers should size chemical metering pumps to provide for potential differences in demand, and compare bench scale results to theoretical water chemistry expectations. More detailed guidance on chemical feed systems is in Section 10.6.

See Section 10.1.3 for information on other treatment process changes that can affect corrosion control.

10.2.5 pH Adjustment

In addition to its role in corrosion control treatment as discussed in the previous section, pH impacts many other treatment processes. For example, chlorine disinfection is more effective at lower pH. Removal processes for many inorganic chemicals such as arsenic, iron and manganese work best within specific pH ranges as discussed in Section 10.2.6. Likewise, performance of coagulation chemicals used in surface water treatment can vary with pH.

USEPA has established a secondary (aesthetic) standard for pH of 6.5 to 8.5. Lower pH conditions can cause water to have a bitter metallic taste and be corrosive to plumbing materials. High pH can make water feel slippery, giving it an unpleasant taste or cause deposits in plumbing systems.

Most approaches for adjusting pH involve chemical feed facilities to inject an acid or base. Common chemicals used to lower pH are carbon dioxide (gas), citric acid, and phosphoric acid. Common chemicals used to raise pH are lime, sodium carbonate (soda ash) and sodium hydroxide (caustic). All chemicals **must** be used within their ANSI/NSF 60 approved doses (WAC 246-290-220(3)).

Changes in pH (and alkalinity) can impact distribution water quality. Secondary impacts of treatment **must** be considered as described in Sections 10.1.3 and 10.2.4.

Strong acids and bases require careful selection, storage and handling to protect worker safety. For example, a 50 percent caustic solution solidifies at 58ºF, which can plug piping and even cause injury if valves or piping fail as a result. For this reason, we recommend using a more dilute solution of 25 percent or less. Section 10.6.2 has more information on chemical storage and handling. When feeding concentrated acids and bases, include design features to lower the risk of chemical overfeed. See Section 10.6.1 for more information.

Effective pH adjustment requires appropriate process control through monitoring, instrumentation and alarms. Continuous monitoring of pH should be provided upstream of the chemical injection point and downstream after the chemical is completely mixed. Section 10.4.2 has more information on process control.

10.2.6 Inorganic Chemicals

There are primary (health-based) or secondary (aesthetic) water quality standards for more than a dozen inorganic chemicals (IOCs) (chapter 246-290 WAC). The IOCs most frequently detected above their MCLs are arsenic (As), fluoride (F), nitrate (NO_3), iron (Fe), and manganese (Mn). Table 10-9 summarizes treatment options for these contaminants.

Chloride and conductivity are secondary contaminants that may indicate seawater intrusion. Seawater intrusion itself indicates that a source water quantity issues exists. ODW and local health departments may require additional action when seawater intrusion threatens the reliability of the water supply. See Section 5.5.4 for more information on seawater intrusion.

10.2.6.1 Arsenic

The arsenic MCL of 10 parts per billion (0.010 mg/L) was established in 2001 based on chronic health concerns, including carcinogenic and cardiovascular risks. Water systems developing a new well with arsenic over the MCL, or operating an existing seasonal or permanent source exceeding the arsenic MCL on a running annual average, must initiate design of remedial measures in order to comply with the arsenic standard.

Measures may include physical removal, blending, or a non-treatment alternative such as drilling a new well or connecting to a nearby water system. Non-treatment alternatives should be considered first, especially if arsenic in the source exceeds 0.050 mg/L, as treatment of this high a concentration of arsenic can be challenging and expensive. In addition, when the concentration of arsenic is this high, a treatment failure can present an acute health risk. The simplicity of operations and availability of qualified operators are key considerations in the selection of a long-term solution to arsenic contamination.

Raw water quality parameters such as pH, iron, manganese, ammonia, phosphate and silica are important considerations in the selection of any arsenic treatment technology. Effective treatment is dependent upon arsenic being present as arsenate or As(5), the oxidized form of inorganic arsenic. Effective treatment is also dependent on knowing and addressing how other water quality parameters can affect the treatment outcome. Ammonia, if present, can prevent effective oxidation of arsenic to As(5). Silica, especially in conjunction with pH greater than 8.0, reduces the ability of treatment processes to remove arsenic. Commonly used treatment approaches include:

- **Adsorbents:** There are a number of iron oxide and other metal based adsorbents that can be used to bind arsenic. Eventually, all the binding sites in the adsorbent are used up and the adsorbent needs to be replaced. The replacement period for adsorbents can vary widely, and can be much shorter than estimates from suppliers because of interference by other ions (e.g., silica). Adsorbents should be evaluated for the specific source water quality prior to selecting an adsorbent or detailed design. If the pH of the water is adjusted to increase the effective life of the adsorbent, continuous pH monitoring and looped process control is necessary to prevent the arsenic from being released from the adsorbent. Inadequate evaluation of adsorbents has led to impractically short replacement periods and subsequent abandonment of the adsorbent treatment process.

- **Anion exchange:** Since arsenate (As(5))is a negatively charged ion at the pH of most natural waters, anion exchange can be used to remove arsenic. The anion resin is then periodically regenerated using a salt brine solution. One of the key design parameters is the volume of water that can be treated prior to regenerating the system. Any estimate of the volume of water that can be treated prior to regeneration should be confirmed through pilot testing. Anion exchange treatment also initially decreases the pH and alkalinity of the water, which can make it more corrosive to lead, copper, and other metals. As a result, corrosion control treatment may be needed as part of the design.

- **Coagulation-Filtration:** For source water with insufficient iron, iron and aluminum based coagulants can be added to the raw water to bind with arsenic for subsequent removal by filtration. A pre-oxidant such as free chlorine or permanganate is usually necessary for effective treatment, and should be added so that there is at least 20 seconds of contact time prior to coagulant addition. The filtration media, depth, loading rate, and backwash frequency and duration are other key design parameters.

- **Oxidation-Filtration:** For source water with sufficient iron, oxidizing the iron will bind arsenic and remove both iron and arsenic through filtration. The ratio of iron to arsenic necessary for effective treatment is usually at least 20:1 on a mass basis, and may need to be greater than 100:1 in some cases. As with coagulation filtration, the filtration media, depth, loading rate, and backwash frequency and duration are important design parameters.

Additional information about these treatment approaches is available in Table 10-9 and guidelines in Appendix F.3, F.4, and F.5.

There are many guidance documents from the USEPA and Water Research Foundation (formerly AWWARF) that may be useful is evaluating arsenic treatment alternatives (USEPA 2003; Hoffman 2006). In addition, the USEPA Arsenic Treatment Technology Demonstrations website contains many reports and detailed case studies that can be useful in the design of a treatment system.

10.2.6.2 Nitrate and Nitrite

Nitrate and nitrite are acute contaminants for susceptible individuals (primarily infants less than 12 months old and pregnant women). A single exposure can negatively affect the health of these susceptible individuals. The primary sources for nitrate and nitrite are agricultural activities and septic tank effluent.

Design engineers should explore non-treatment alternatives to resolve nitrate contamination of a groundwater supply, including abandonment of the source and developing an alternate groundwater supply or intertie with an adjacent water system. If there are no feasible non-treatment alternatives, consider the following:

- **Blending** – The mass-balance of nitrate from both sources is the basic design parameter. Because of the variability of nitrate in groundwater over time, a significant factor of safety should be applied when determining the mixing rate of the two (or more) sources.

- **Anion exchange** – Since nitrate is a negatively charged ion anion exchange can be used to remove nitrate. The anion resin is then periodically regenerated using a salt brine solution. One of the key design parameters is the volume of water that can be treated prior to regenerating the system. Any estimate of the volume of water that can be treated prior to regeneration should be confirmed through pilot testing. Another important design parameter is the concentration of other ions which preferentially compete for exchange sites on the resin, such as sulfate. If not regenerated in time preferred ions (i.e sulfate) will displace less preferred ions (nitrate) resulting chromatographic peaking of the less preferred ion in the treated effluent. Anion exchange treatment also initially decreases the pH and alkalinity of the water, which can make it more corrosive to lead, copper, and other metals. As a result, corrosion control treatment may be needed as part of the design. For additional guidance see Appendix F.12.

- **Reverse osmosis** – This treatment process is mainly used for very small flow applications due to the high cost associated with larger RO treatment systems and high proportion of reject water. RO treatment rejects most dissolved minerals, thereby changing the conductivity of the water. For this reason, continuous conductivity monitoring can be considered as an alternative to frequent nitrate monitoring.

Additional notes about these technologies are in Table 10-9 at the end of this chapter.

Information on nitrate occurrence in Washington State and a discussion of treatment and non-treatment alternatives for nitrate is in the ODW guidance document *Nitrate Treatment Alternatives for Small Water Systems* (DOH 331-309). More detailed information about ways to address nitrate contamination of groundwater can be found in other guidance manuals (Jensen et al. 2012; Seidel et al. 2011).

10.2.6.3 Iron and Manganese

There are several contaminants that have regulatory standards based upon aesthetics such as taste, color, and staining of plumbing fixtures. Of the chemicals with secondary maximum contaminant levels (SMCLs), iron (Fe) and manganese (Mn) are the two most commonly found in untreated water sources. We recommend water systems treat each source exceeding the Fe or Mn SMCL. Since Mn can accumulate in the distribution system and later be released during changes in flow or water chemistry, current industry guidance recommends that source waters be treated so not to exceed 0.020 mg/L of Mn at entry to the distribution system (Kohl and Medlar 2006).

The requirement to comply with the Fe and Mn SMCLs vary depending upon whether the source and water system are new or existing.

- An existing water system whose other sources do not exceed the Fe and Mn SMCL **shall** provide treatment for a new source exceeding the Fe or Mn SMCL (WAC 246-290-130 (3)(g)).

- A new community or new nontransient noncommunity water system without active consumers **shall** provide treatment for a new source exceeding the Fe or Mn SMCL (WAC 246-290-320(3)(d)).

- An existing system with one or more existing sources exceeding the Fe and Mn SMCL should submit life cycle treatment cost information and a copy of a resolution by the governing board or owner of the water system that the community/consumers accepts current water quality problems and the proposed new source will not add to existing water quality problems.

Water systems may use an existing emergency source that exceeds a secondary MCL during an emergency without the need for an engineering report. There are a number of other conditions that must be met before an emergency source can be safely placed into service, including prior sampling for acute drinking water contaminants and/or public notification. See WAC 246-290-131.

Iron and manganese frequently occur in groundwater at concentrations above their SMCLs. Oxidation combined with filtration is the most common treatment process used to remove iron and manganese from drinking water. Oxidants include air, chlorine, potassium permanganate

($KMnO_4$), and ozone. Use of ozone or chlorine will require disinfection byproduct monitoring. Aeration may not provide sufficient oxidation. For that reason, $KMnO_4$ is considered the oxidant of choice, and is effective over a wide range of pH. Fe and Mn oxidation/filtration removal is most effective at pH 7.5 and above.

Ion exchange technologies can also be used for Fe/Mn removal. With these methods, special care must be taken to ensure that the iron and manganese is not oxidized before application through the exchange media. Fouling of the exchange bed can occur if the iron or manganese is not maintained in a chemically reduced state.

The design engineer should be aware of water quality characteristics such as total organic carbon, pH, and competing ions that can adversely affect treatment performance. The limitations of treatment options for iron and manganese are in Table 10-9 and other texts (HDR 2001; Sommerfeld 1999; Faust and Aly 1998; AWWA/ASCE 1990). Additional guidance on iron and manganese treatment is in Appendix F.10 and F.11.

10.2.6.4 Fluoride Removal

Fluoride can naturally occur in source waters at concentrations greater than the primary MCL of 4.0 mg/L. Bone char and activated alumina are the two most common treatment technologies to remove excess naturally occurring fluoride from drinking water. Detailed design guidance for fluoride removal can be found elsewhere (Fawell et al. 2006; AWWA/ASCE 2012b).

10.2.7 Volatile Organic Chemicals and Synthetic Organic Chemicals

A list of treatment technologies acceptable for removing volatile organic chemicals (VOCs) and synthetic organic chemicals (SOCs) is in Table 10-10. In addition to specific technologies, this table identifies selected issues the engineer should consider. In most cases, due to the complexity of treatment processes for specific organic contaminants, the engineer will have to use pre-design studies and pilot tests to determine whether a treatment process is appropriate to a particular source.

10.2.8 Radionuclides

There are primary MCLs for radium 226 and radium 228 (5 picocuries/L combined), gross alpha particle radioactivity (15 picocuries/L), beta particles, photon emitters, and uranium (30 ug/L). Naturally occurring radionuclides are associated with granitic and metasedimentary rock and younger sedimentary formations in northeastern Washington.

Purveyors **must** use pre-design studies and pilot tests to determine treatment and waste disposal options appropriate for their specific situations (WAC 246-290-110(4)). Water systems can remove radium and uranium from drinking water by using properly designed ion exchange treatment processes (Clifford 1999). Reviews of other treatment processes and the waste disposal issues related to them are available elsewhere (USEPA 2006b).

10.2.9 Emerging and Unregulated Contaminants

Currently, there are more than 90 regulated contaminants in drinking water with some contaminants serving as indicators for others. With increasing awareness and improving laboratory detection limits, new water quality contaminants of concern continue to emerge. Many of these are under review by EPA and other public health professionals. Some contaminants have established health advisory limits and may be regulated in the future. Recent examples include cyanotoxins, hexavalent chromium, perchlorate, and perfluoroalkyl substances such as perfluorooctanic acid (PFOA).

While treatment for an unregulated contaminant may not currently be enforceable under federal drinking water regulations, ODW may require a water system with an unregulated contaminant exceeding an established health advisory level to issue public notice and cooperate with ODW in educating consumers about the health risk. Detection of unregulated contaminants may prompt local health officials, community leaders, and/or the public to demand a treatment solution. Design engineers can refer to the EPA Drinking Water Treatability Database and Health Advisory Tables (USEPA 2012) for more information, as well as many other resources on emerging and unregulated contaminants. Please contact your regional office early in the process if you are considering treatment for an unregulated contaminant.

10.3 Predesign Studies

Predesign studies, including pilot studies as appropriate, are **required** for proposed treatment projects (WAC 246-290-250). The goal of the predesign study is to establish the most effective treatment approach, considering life-cycle costs, to produce treated water that meets all regulatory requirements. A predesign study should precede the project report. As such, the information from the predesign study **must** be included in the project report (WAC 246-290-250). Pre-design study options include desktop, bench, and pilot studies.

Desktop studies involve reviewing detailed water quality data, guidance documents, technical publications, and other information to select a treatment approach for further evaluation or full-scale design. Desktop studies usually require significant amounts of water quality data to guide the selection of an appropriate treatment alternative. Desktop studies can be useful in evaluating distribution system impacts (e.g., corrosion) associated with any new or modified treatment process.

Bench-scale studies include jar testing to identify an initial coagulant dose, initial chemical dosages for iron and manganese sequestering, and estimates of disinfection demand and decay.

Pilot studies often follow desktop and/or bench-scale studies so engineers can identify design parameters and decide how reliable a treatment process will be over the range of source water quality conditions.

10.3.1 Pilot Studies

Inadequate pilot testing may result in treatment process performance inefficiencies or outright failure, delayed implementation of effective treatment, and costly retrofitting or replacement of treatment facilities. For these reasons, pilot studies are generally **required** for proposed treatment projects (WAC 246-290-250). Treatment approaches where a pilot study may not be needed include:

- Simple disinfection for small groundwater sources. Large water systems should perform bench-scale tests on proposed disinfection methods to evaluate the potential for generating regulated disinfection byproducts. See Sections 10.2.1 and 10.2.2.
- Simple in-reservoir aeration for pH adjustment, aesthetic reasons or removal of trihalomethanes.
- Fluoridation.
- When a corrosion control desktop study clearly points to a particular corrosion control treatment approach consistent with source water quality, operator capacity, and distribution system conditions.
- Identical treatment processes are applied to nearly identical source waters, such as reverse osmosis on well-circulated seawater.

Pilot studies attempt to replicate as closely as possible the operating conditions and treatment results expected at full scale. Pilot plants are scaled-down versions of a proposed process, and may be skid or trailer mounted. Engineers use pilot plant testing to ensure treatment is effective, determine final design parameters, and estimate construction and operation costs.

It is impractical to transfer pilot results from one proprietary design to another. Equipment for proprietary processes is usually so specialized that pilot testing results are unique to a specific equipment design (e.g., differences in low-pressure membrane filtration).

Sections 10.3.2 through 10.3.4 discuss the recommended pilot study duration, content of a pilot study plan, and final pilot study report. Section 10.3.5 includes a discussion of full-scale pilot testing.

10.3.2 Pilot Study Duration

Pilot studies should be of long enough duration to demonstrate the effectiveness, stability, and reliability of the proposed treatment system. The testing should include the period of most challenging water quality for the piloted treatment technology. If the pilot study is too short or misses important seasonal changes in source water quality, the process may not work as designed or incur higher than expected operational costs. Pilot studies often are shorter for groundwater than surface water treatment because groundwater quality is usually more stable.

The number of samples collected and study duration can vary widely depending on the type of source, amount of historical data, water quality, and the proposed treatment technology (Logsdon et al. 1996; Ford et al. 2001; AWWA/ASCE 2012d). In some cases, engineers can use bench-scale testing to determine the initial operational parameters for pilot testing and possibly decrease the duration of the pilot study. See Table 10-2 for guidance on the duration and objectives of pilot studies for a variety of treatment processes.

10.3.3 Pilot Study Plan (Protocol)

A pilot study plan is necessary to establish an implementation strategy for evaluating a proposed treatment alternative, or alternatives. The pilot-study plan establishes pilot study goals, the monitoring program, operational requirements, equipment needs, layout, duration, and cost.

Several of the elements discussed below are appropriate for desktop or bench-scale studies. Engineers should address them in the protocol they submit to ODW for approval.

Pilot Study Goals: Engineers should use the following goals to determine the scope of a pilot study and for pilot study planning and operational decisions:

- Determine the operational feasibility of a selected technology.
- Establish full-scale water-treatment design criteria.
- Develop more refined cost estimates.
- Provide hands-on operator training for water system personnel.
- Determine projected hydraulic impacts on the water system.
- Select an appropriate treatment technology.
- Determine waste disposal requirements and constraints.

Monitoring Program: Pilot study monitoring programs vary significantly depending on the treatment device, finished water requirements, and the specific contaminants in the source water. Engineers can use Table 10-2 to develop monitoring programs for the treatment technologies listed. For additional guidance contact one of our regional offices.

Most pilot study monitoring programs should include:

- Water quality parameters.
- Monitoring frequency for each parameter.
- Monitoring equipment and calibration standards.
- Personnel or outside laboratories responsible for monitoring activities.

Table 10-2
Pilot Study Duration and Objectives

Treatment	Purpose	Minimum Recommended Duration	Objectives	References
Adsorption	DBP precursors, IOCs, VOCs, SOCs	6-12 months[1]	Run length, hydraulic loading rate, empty bed contact time, finished water quality.	Ford et al. 2001; Cummings and Summers 1994; Westerhoff et al. 2003
Ion Exchange	IOCs	2-12 months	Regeneration frequency, leakage, resin stability, potential for chromatographic peaking, pH/corrosion control, finished water quality.	Liang et al. 1999; Clifford and Liu 1993
Oxidation/ Filtration	IOCs	1-6 weeks	Oxidant demand and dose, coagulant dose, hydraulic loading rate, filter run length, finished water quality.	Gehling et al. 2003; HDR 2001

Reverse Osmosis	Desalination	2-7 months	Pretreatment required, flux rate and stability, back flush parameters, chemical dose(s), cleaning frequency, finished water quality.	Kumar et al. 2006; USEPA 2005
Various	Surface Water	Up to 12 months[2]	See Chapter 11 and Appendix F, H, and I for further details.	

Notes:

1. Engineers can decrease the pilot test period to a few weeks if rapid small-scale column tests (RSSCTs) are used.
2. See Chapter 11, Table 10-3.

Equipment Needs, Layout, and Calculations: The pilot study plan should include a schematic of the process or processes under consideration and the detailed drawings necessary to construct the pilot facilities. The schematic and the pilot facility design are integral to the overall project design and should include unit processes, pipe sizes, pipe connections, flow direction, chemicals and application points, monitoring points, flow-control devices, monitoring equipment or gauges, and various process elements (such as intakes, pumps and blowers).

Operational Requirements: Pilot study plans should identify the operational requirements necessary to ensure water system personnel understand their role and responsibility to provide routine O&M and data collection. ODW recommends that the design engineer prepare a schedule to clarify routine pilot study activities for water system personnel and others that may be involved with the study.

Pilot Study Costs: Engineers should develop equipment rental, testing, and operation costs as part of the pilot study plan. Engineers can estimate these costs after they develop the goals, duration, and monitoring program for the pilot study.

10.3.4 Pilot Study Report

A pilot study helps to define the design and operational parameters for a treatment process. Therefore, when a pilot study is required, engineers **must** prepare a pilot study report that evaluates pilot-study data and determines whether the treatment option is feasible for full-scale implementation (WAC 246-290-250 and 676(3)).

General pilot-study evaluation criteria include:

- Tabular data for each measured parameter.
- Graphical data showing relationships between measured parameters.
- Narrative on the relationships between measured parameters.
- Cost projections for full-scale operation (yearly, monthly, and per customer).
- Final design and operational parameters.
- Recommendations for full-scale implementation.

- Comparison of recommended design and operational parameters to design goals, water quality goals, and other performance benchmarks.

For pilot studies in excess of 4 months duration, ODW recommends the design engineer submit an interim status report once every two months. These reports will serve as useful checks on pilot study objectives, costs, progress, and findings, and determine if the pilot study should continue as planned, amend the pilot study protocol, or end it in favor of pursuing a different treatment approach.

10.3.5 Full Scale Pilot Study

Some water systems are so small that the capacity of a commonly used pilot plant is equal to the capacity of the full-scale treatment unit. In such a case, the purveyor and designer should approach the pilot facility design as if it will be a permanent facility in the future. As such, a project report and construction documents **must** be prepared (See Sections 10.4 and 10.5).

Final ODW acceptance of the full-scale pilot facility depends on a successful demonstration as determined by an approved pilot-study report (see Section 10.3.4). The purveyor and engineer take on significant risk when designing a pilot to full-scale since treatment efficiency or operation costs may not match pre-design expectations, and either major modification or complete abandonment of the approach may be required. Submittals for full-scale pilot testing should identify actions the engineer will take if it is necessary to make major modifications or abandon the project.

A water system may deliver water for public consumption produced from a full-scale pilot plant if all of the following conditions are met. See Section 10.9 for additional information regarding placing a water treatment plant into service.

1. Plans and specifications (construction documents), start-up, testing, and operation procedures are approved as part of a pilot-study plan before the full-scale pilot plant is constructed.
2. Source capacity limitations on an existing water system do not provide adequate source capacity for the combined demand of the existing customers and the pilot study.
3. The design engineer demonstrates to ODW's satisfaction that the treatment process will not increase risk to consumers.
4. Treatment plant facilities must be pressure tested, flushed, disinfected, and tested for all applicable drinking water contaminants.
5. A properly certified operator is available to operate the treatment equipment.
6. The treatment system technology:
    a. Is an approved alternative filtration technology (if a surface water application).
    b. Is constructed of components listed under ANSI/NSF Standard 61.

10.4 Project Reports

ODW **must** approve project reports before the purveyor installs any new or expanded treatment facilities (WAC 246-290-110). The engineer should submit a final project report for treatment facilities before submitting construction documents. Project reports for treatment facilities should reference all planning, design, and applicable pilot study reports for the proposed facility. They **must** include:

- Detailed design criteria and calculations
- Process control information
- Proposed methods and schedules for start-up, testing, and operating the completed treatment facility
- Operator training and certification requirements
- Reliability

See Chapter 2 for guidance on preparing project reports.

10.4.1 Design Criteria and Facility Design

Project reports **must** include design criteria for all major treatment-facility project elements (WAC 246-290-110(4)(h)). Project design criteria and calculations should include the following:

- **Overall process design.** The design engineer should create a detailed narrative of design concepts, design calculations, and supporting information for the treatment process(es), process piping and equipment, process control, and waste disposal.
- **Other project design elements.** The design engineer should outline the general design aspects, such as siting issues, ingress or egress access, roads, sidewalks, parking, earthwork, drainage facilities, building layout and design, special structural requirements or constraints, heating, ventilation, fire suppression features, general utilities, electrical supply, chemical storage and feed system(s), all-hazards assessment, and operator safety.
- **Cost and financing.** The engineer **must** include construction cost estimates, O&M cost estimates, and the proposed financing method(s) if not already covered in a current approved water system plan (WAC 246-290-110(4)(b)(vii)). At this stage, the accuracy of the projected cost depends on how well the construction documents are completed, but should be within 10 percent of the actual cost (AWWA/ASCE 2012g). The engineer should identify the cost-estimation method and compare the final cost estimates to the estimates in the financial program of the water system plan, if applicable.

10.4.2 Process Control - Monitoring, Instrumentation and Alarms

Overall process control is important to ensure that the treatment process can function safely and reliably at all times, especially when unattended, and **must** be addressed in project reports for water treatment facilities (WAC 246-290-110(4)(h)). The description of process control can range from the simple interconnection of a chemical feed pump with the well pump controls to detailing specific process monitoring and control set-points. For more complex treatment facilities, the description and analysis of overall process control should include monitoring, instrumentation and alarms.

Monitoring

All treatment processes should be monitored. Monitoring ensures that the treatment plant meets applicable treatment technique or maximum contaminant levels. This monitoring can include water quality measurements like chlorine residual and pH; and physical parameters such as flow, pressure and tank level. The means, methods, and frequency for monitoring water quality and physical parameters **must** be clearly identified for each treatment process in the project report (WAC 246-290-110(4)(h)). For treatment processes that remove regulated primary chemical contaminants, the treated water **must** be analyzed by a certified lab at least monthly (WAC 246-290-455). Additional monitoring may be required based on the complexity or size of the treatment process.

Draft monitoring procedures and forms should be developed early in the design process to support monitoring of the treatment process once it is completed. Some treatment process monitoring forms are available on the ODW Drinking Water Forms website. If a standard form cannot be found on the website, please contact your Regional Engineer to identify the monitoring and reporting requirements needed to help ensure that the treatment process is operated as designed.

Sample taps **must** be provided before treatment to assess the source water quality, and after treatment but prior to the entry to the distribution system (WAC 246-290-300; WAC 246-290-320(2)(g); WAC 246-290-451). Additional sample taps should be installed at intermittent points in more complex treatment plants to help in process control, verify on-line analyzers, and assess specific treatment processes.

Source sample taps should be placed far enough upstream so they are not influenced by downstream chemical injection. Sample taps for treated and partially treated water should be located after added chemicals have been allowed to completely mix. Since turbulent flow conditions can dislodge pipe scale or entrain air, avoid sample taps in turbulent flow locations, such as near valves, ells, tees and flanges. Also, avoid tapping the bottom or top of the pipe, which can introduce sediment or air. We recommend using a sample probe or quill to sample from the center of the pipe.

Sample taps should be smooth nosed without any internal or external threads to reduce the risk of microbial contamination or aeration of the sample. Aeration can change the pH or result in loss of chlorine residual so that the sample is not representative of the water in the pipe.

Instrumentation

On-line instrumentation is often required to ensure that a treatment process is working well. Instrumentation can minimize the risk of customer complaints as well as improve public health protection. The design engineer should identify the benchtop equipment needed or other ways that the operator can verify the readings from the on-line instrumentation in the project report. This benchtop equipment will allow checking of on-line instrumentation periodically as components age or become fouled with time.

There are several design considerations that should be addressed in locating and installing on-line instrumentation. These include:

- **Operator safety and ease of access –** Instrumentation should be located in safe, non-corrosive environments where the operator has easy access and ample clearance for servicing the device.

- **Calibration, verification, and testing –** All instrumentation requires periodic calibration and often frequent verification as outlined by the manufacturer or in approved methods. In addition, means for physically testing alarms associated with instrumentation should be considered, such as the ability to spike the sample upstream of the instrumentation.

- **Sample piping –** Sample lines should be kept as short as possible, and small diameter non-translucent piping/tubing should be used. In general, the sample delay should be less than 2 minutes between the pipe and instrumentation.

- **Flow and pressure control –** There should be a means of measuring and controlling the pressure and flow to the instrument. Such control is necessary to ensure that the flow rate is within manufacturer specifications and pressure fluctuations do not affect the instrumentation.

- **Power supply –** Instruments should be protected with surge protectors and uninterruptible power supplies, and not located near equipment with large electrical motors. Some instruments, such as pH analyzers, are especially sensitive to small changes in the power supply.

Guidance on the design of plumbing to process analyzers is included in Appendix F 9.

Alarms

Alarm conditions should be identified, especially for critical process components where very high or very low levels could lead to unsafe water being delivered to customers. Critical alarm conditions for water treatment facilities can include both water quality and physical parameters such as:

- **Flow rate –** The maximum flow rate through a treatment process should not be exceeded as doing so may cause the process to fail or lead to a treatment technique violation.

- **Water level –** A minimum level of water in a clearwell or reservoir is needed to ensure that adequate disinfection is provided. In other cases, there may be a critical level of stored water needed to backwash filtration equipment.

- **Pressure –** For filtration, it is often necessary to closely monitor the differential pressure across the treatment process (headloss) to ensure that the equipment does not exceed regulatory or manufacturer acceptable thresholds.

- **Turbidity –** While primarily used to monitor the effectiveness of surface water treatment processes, it can also be used to monitor groundwater filtration processes to minimize the risk of failure and customer complaints.

- **Disinfectant residual** – Chlorine residual analyzers are commonly used to addresses microbial health concerns and should be considered in other applications to minimize the risk of process failures.

- **Inorganic chemicals** – When treatment process failure can lead to an acute health risk from a regulated inorganic chemical such as nitrate, continuous monitoring for the chemical should be considered. During the three-year period 2014-2016 there were 32 instances of a nitrate system treatment plant failure resulting in an acute MCL violation and a bottled water advisory to the public.

- **pH –** A significant change in pH, either too low or too high, can cause treatment to be ineffective, cause water quality impacts in the distribution systems, lead to treatment technique violations, and, in extreme cases, place public health at risk.

- **Conductivity** – Is most often used to monitor high-pressure membranes such as reverse osmosis and nanofiltration units, to confirm that the membranes are intact.

You can find additional information on the design of alarms for water treatment plants in *Testing Critical Alarms* (DOH 331-472) and the "Policy Statement on Automated/Unattended Operation of Surface Water Treatment Plants" in the *Recommended Standards for Water Works* (Ten State Standards, 2012). While these references are primarily intended for surface water treatment plants, information in them is applicable in many other situations such as where corrosion control, pH adjustment, or continuous source disinfection is installed.

Alarm conditions normally trigger notification to the operator and other personnel through audible or visual means. When a water treatment facility is often unattended, an autodialer, as well as shutdown of the treatment plant itself, should be considered for treatment process failures that may result in an acute health risk. Once a treatment plant is automatically shut down, it should not be allowed to restart until an operator is physically present at the treatment plant itself. In some cases, treatment plants that have automatically restarted have led to process failures putting the health of consumers at risk.

10.4.3 Start-up, Testing and Operations

Start-up of a water treatment plant can be very challenging and may have unintended consequences if not closely managed and planned well in advance. Therefore, start-up and testing should be evaluated early in the design process.

Start-up and Testing

The start-up and testing of a treatment plant is a complex operation. Project reports **must** include proposed methods and schedules for start-up and testing (WAC 246-290-110(4)(h)). These methods and schedules may be revised as needed prior to construction completion. Schedules should include the anticipated start-up date and proposed testing duration. Methods should identify specific standards and the persons involved. The methods and schedules can be general in the project report and refined in the construction documents.

Final start-up and testing plans should identify the persons involved in start-up and testing and their specific roles. These plans should also identify specific criteria that will be used to determine that it is safe to serve the treated water to consumers including testing of the treatment equipment, proof of reliable operations, and water quality standards. The criteria will be project specific and could range from simple grab samples for chemical addition processes to detailed evaluation criteria for multiple processes in complex treatment facilities.

Prior to start-up, meetings involving the design engineers, contractors, and others should occur to help ensure a smooth and successful start-up of a new treatment plant. See Section 10.9 (Placing a Water Treatment Plant into Service) for additional information on start-up requirements and recommendations. If the project is funded by our drinking water state revolving loan fund, provisions for such meetings must be made in the project specifications and contract documents.

For large or complex water treatment plants, a start-up and testing plan should be submitted separately from the project report and construction documents.

Operations

Project reports for treatment facilities **must** address operation of the completed project (WAC 246-290-110(4)(h)). At this stage, the design engineer should identify the organization or people responsible for operating the finished facility and their required qualifications. The level of operator certification is dependent upon the population served, treatment capacity, and complexity of the treatment process. The design engineer should include a draft "Purification Plant Rating Criteria Worksheet", which is available on our Operator Certification webpage, as part of the project report to help identify the correct treatment plant classification. Upon review ODW may revise the design engineer's draft determination of operator certification requirement.

See Chapter 11 for operations and monitoring requirements for surface water treatment facilities and Section 10.9 (Placing a Water Treatment Plant into Service) for additional information that **must** be addressed prior to use of any new or modified water treatment facility.

10.4.4 Treatment System Reliability

Engineers **must** design water treatment facilities to meet minimum water quality standards at all times, except where otherwise noted (WAC 246-290-420(1)). "Treatment reliability" means the failure of any single component will not result in delivery of unsatisfactory drinking water to consumers. Information on treatment-process reliability is in other published design references (AWWA/ASCE 2012e; Ten State Standards 2012).

Reliability is especially important when a treatment process failure can present an acute health risk (groundwater requiring disinfection, pH adjustment for corrosion control, nitrate removal, and surface water). In addition, there are some general approaches that can increase the reliability of other treatment process through the use of continuous analyzers and alarms.

Nitrate: As noted in Section 2.9 of the *Recommended Standards for Water Works* (Ten State Standards 2012), treatment plants for nitrate removal should have an on-line finished water nitrate analyzer. Design engineers that do not propose to install a nitrate on-line analyzer should indicate the means of providing equivalent process monitoring or assurance of public health protection in the event of a treatment process failure.

Surface water treatment: Surface water treatment plants **must** have certain reliability features (WAC 246-290-678). See Chapter 11 for further details on reliability for surface water treatment plants.

10.5 Construction Documents

Before water systems install new or expanded treatment facilities ODW **must** approve the final construction documents (WAC 246-290-120). The construction document submittal **must** include detailed drawings and specifications. Some small projects may include relevant specifications on the construction drawings if applicable. Chapter 3 summarizes the information that must be in all construction document submittals. Design engineers should review the checklists in Appendix A to confirm they meet the minimum submittal requirements.

10.6 Treatment Chemicals

All treatment chemicals **must** be used within their maximum application dosages listed under NSF/ANSI Standard 60 (WAC 246-290-220(3)). In water treatment facilities, the improper storage and application of treatment chemicals may present a potential hazard similar to that at an industrial or chemical plant. Treatment plants and some distribution treatment facilities store large quantities of chemicals such as chlorine gas, hypochlorite compounds, aluminum sulfate, caustic soda, fluoride compounds, potassium permanganate, ammonia, and numerous proprietary organic polymers. Typical treatment practices feed these products directly into treated potable water or water being processed into potable water.

Chemical addition, when uncontrolled, can result in a dangerous overfeed due to improper design, operation, or maintenance of feed equipment. Specific risks include component failure, and/or backflow/backsiphonage. Design manuals such as the *Recommended Standards for Water Works* (Ten State Standards 2012) or *Water Fluoridation Principles and Practices* (AWWA 2016) provide information and recommendations for preventing these types of failures. Important considerations in the design of the chemical feed system include:

- Chemical overfeed prevention
- Safe storage and handling
- Cross-connection control

10.6.1 Chemical Overfeed Prevention and Feed Systems

Injecting chemicals into the water supply always poses some potential of overfeed if equipment is not designed, installed, operated, or maintained properly. Overfeeds of ammonia, chlorine, sodium hydroxide, and fluoride have been reported (Brender et al. 1998; AWWA 1993; AWWA 2016; Lee et al. 2002).

Operation and maintenance errors, design flaws, mechanical failure, installation errors, or a combination of factors can cause these failures. Documented failures include:

- **Ammonia Overfeed.** An ammonia injection point was moved downstream in the process to increase free chlorine contact time prior to chloramine formation, and the hydraulic head on the bulk storage tank was sufficient to allow ammonia to flow into the main without pumping. The anti-siphon valve designed to prevent overfeed failed, allowing the full bulk storage tank to empty into the water system. Operators failed to recognize the problem despite unusually high pH values and unusually low chlorine residuals.

- **Sodium Hydroxide (Caustic Soda) Overfeed.** To control corrosion, the water system treated a well supply with sodium hydroxide. When operators closed the distribution system valves to complete a main repair, the pressure at the well increased significantly, reducing well production from 450 gpm to less than 85 gpm. The caustic feed system was not flow paced. As a result, the pH of the water eventually reached 13. Two people who drank water from a nearby public fountain received mouth and throat burns. The pressure and pH build-up occurred over a two-day period; daily inspection of the well and treatment system would have caught the problem sooner.

- **Fluoride Overfeed.** In 1992, an incident in Hooper Bay, Alaska caused 1 death and about 262 illnesses. An incorrectly wired circuit for the fluoride feed pump (in parallel instead of in series) allowed fluoride solution to pump into the water system even though the source wasn't operating. This "slug" of fluoride (up to 150 mg/L) was delivered to customers.

- **Chlorine Overfeed.** A computer controller card on a rate-of-flow controller malfunctioned, failing to shut down the chlorination circuit when the well sources (controlled by reservoir levels) shut off. Nearby customers noticed the continued injection of chlorinated water when the well sources were called on again, and the water was delivered to the distribution system.

Design elements and appropriate Standard Operating Procedures (SOPs) can minimize the potential for overfeed. Below are some design considerations to minimize the risk of overfeeding. See Appendix F.2 for design elements to reduce the potential for fluoride overfeed.

1. Day tanks should be included when the use of large bulk volumes of treatment chemicals is necessary. These day tanks should be sized to store no more than 30 hours of supply and be designed to fill in a controlled manner by an operator (Ten State Standards, 2012). These tanks promote daily inspection of the feed systems, and reduce the magnitude of an overfeed.
2. Evaluate the failure modes of the equipment, and add redundant safeguards if needed. In the chlorine and fluoride overfeed examples cited above, a redundant flow switch wired in series with the feed pumps would have stopped the chemical injection system after it detected a lack of treated water flow. Alternately, engineers should consider the feasibility of installing flow-based chemical feed control.

3. Select chemical injection points to minimize the potential for siphoning or hydraulically draining chemical storage tanks, even if their design includes anti-siphon features.
4. Include continuous monitoring equipment (pH, chlorine, fluoride) with integrated alarms. In some cases, redundant monitoring equipment should be provided. It is appropriate for these alarms to automatically shut down the equipment (see Section 10.4.2).
5. Take into account the capacity of the operator(s) to properly operate, control, and maintain the water treatment plant facilities. Operator error, or operator inattention caused or aggravated several of the overfeed incidents described above. SOPs should tell operators how to react to unexpected changes in water quality parameters (increasing or decreasing pH, values outside "normal" ranges, and other issues).
6. Focus SOPs on routine equipment maintenance. For example, water systems should periodically inspect their anti-siphon valves and replace them as needed.
7. Engineers should provide appropriate cross-connection control (see Section 10.7).

10.6.2 Safe Chemical Storage and Handling

The delivery and storage of chemicals should be carefully considered in the design of water treatment plants. Improper delivery, storage, or use may result in a toxic or explosive environment such as that which occurs when sodium hypochlorite is mixed with alum (releasing chlorine gas) or mixing calcium hypochlorite with gasoline, grease, or fatty or oily substances. The design engineer should consult with the local fire marshal, building code official and other authorities responsible for implementing regulations as part of the design process regarding the safe use and storage of treatment chemicals.

General design considerations should include:

- Operator safety including the provision of eyewash and shower stations with tempered water that are accessible to operators and delivery personnel.
- Containment around chemical storage and feed facilities.
- Seismic bracing, supports, and pipe design to prevent damage to chemical storage and handling facilities in an earthquake.
- Separate delivery, storage, and feed facilities for strong oxidants, such as gaseous chlorine or calcium hypochlorite.
- Clear labeling of chemical fill ports, piping, and storage tanks with the chemical used.
- Locks on every chemical fill port to prevent access without the operator being present.
- Covered spill containment around chemical fill ports.
- Equipment for containment and scrubbing of chlorine gas.
- Egress/ingress requirements for rooms or areas with chemical storage and feed facilities.
- Compliance with all applicable OSHA/WISHA standards, such as signage, safety gear, training, atmospheric monitoring devices, ventilation, and eye wash and safety shower location.
- USEPA requirements under the Risk Management Program if large volumes of chemicals are stored, such as 2,500 pounds or more of chlorine gas.

Hypochlorite is the very commonly used for disinfection and other water treatment purposes, so you may find the following information useful.

Sodium Hypochlorite. Concentrated sodium hypochlorite solution (5.25% to 12.5%) is a corrosive liquid with a pH of greater than 11. Therefore, typical precautions for handling corrosive materials such as avoiding contact with metals, including stainless steel, should be used. Spill containment must be provided for the sodium hypochlorite storage tanks. Typical spill containment structures include containment for the entire contents of the largest tank (plus freeboard for rainfall or fire sprinklers), no uncontrolled floor drains, and separate containment areas for each incompatible chemical. Bulk sodium hypochlorite should be stored at a concentration of no more than 3 percent chlorine by weight.

Calcium Hypochlorite. Calcium hypochlorite is an oxidant and as such should be stored separately from organic materials that can be readily oxidized. It should also be stored away from sources of heat. Improperly stored calcium hypochlorite has caused spontaneous combustion fires.

Other: Other chemical commonly used in water treatment plants may require special design and handling considerations, including:

- Primary coagulants such as alum and ferric chloride
- Strong oxidants such as hydrogen peroxide, potassium permanganate, and ozone
- Strong bases such as sodium hydroxide
- Strong acids such as citric, fluorosilicic, and phosphoric acids

There is extensive information on the safe storage, handling, material compatibility, and use of common water treatment chemicals in such sources as:

- *Recommended Standards for Water Works - Part 5* (Ten State Standards, 2012)
- *Integrated Design and Operation of Water Treatment Facilities* (Kawamura, 2000a)
- *Water Treatment Plant Design* - *Chapter 20 Chemical Systems* (AWWA/ASCE 2012e)

10.7 Cross-Connection Control for Water Treatment Facilities

Protecting drinking water from contamination starts at the source and continues through treatment facilities designed to improve water quality. In water treatment facilities, this protection requires engineers to incorporate safeguards into their designs.

In water treatment facilities, the improper storage and application of chemicals presents a potential cross-connection hazard similar to that found in an industrial or chemical plant. Treatment plants and some distribution treatment facilities store large quantities of hazardous materials such as chlorine, hypochlorite compounds, aluminum sulfate, caustic soda, potassium permanganate, and numerous proprietary organic polymers. In typical treatment practice, these products are fed either directly into treated potable water or into water being processed into potable water. Most treatment works also contain significant quantities of raw, or incompletely treated water.

Potable water in a treatment facility is often at atmospheric pressure increasing the potential for cross contamination, particularly due to backflow or backsiphonage. This may involve backflow or backsiphonage of treatment chemicals, raw, or partially treated water.

10.7.1 Premises Isolation

Premises isolation protects the water supply by installing backflow prevention assemblies, typically at or near the point where water enters a building or facility. Because the chemical hazards in a waterworks facility can be identical to those in industrial facilities, the potable water service line(s) to a water treatment plants **must** be equipped with an air gap or reduced pressure backflow assembly (RPBA) (WAC 246-290-490).

Because premises isolation does not protect against backflow within the plant, the treatment facility should provide separate “potable” and “process” supply systems within the building(s). Many water treatment facilities require a separate potable water supply system to supply lavatory and/or kitchen sinks, eye wash and shower facilities, drinking fountains, and other human consumptive uses. Process water uses in a water treatment plant may include diluting treatment chemicals, carrying concentrated feed solutions, driving eductors, mixing, and supplying surface washers.

10.7.2 Cross-Connection Control inside the Water Treatment Plant

Each potable water connection to the plant process water supply system **must** be equipped with either an approved air gap or reduce pressure backflow assembly, or both, in order to protect the quality and safety of the plant’s potable supply (WAC 246-290-490). Design engineers should limit potable water to supply unit treatment processes from no more than two discrete points. See Table 4-4 of *Cross Connection Control: Accepted Procedures and Practice Manual* (PNWS-AWWA, 1996). In addition, wherever the plant process water supply system supplies multiple processes, each process (“fixture”) should also be protected against backflow in order to protect the integrity of the process water system.

For example, the process water supply to an upflow fluoride saturator (see Appendix F.2) should be equipped with a reduced pressure backflow assembly or approved air gap in order to prevent backflow of fluoride into the process supply to the gas chlorine eductor. This approach is known as “fixture” protection. In water treatment plant design, cross connection control should incorporate both premise isolation and fixture protection principles. Table 10-3 lists backflow prevention assemblies for water treatment plant equipment and processes.

To facilitate identification of piping, we recommend design engineers use a piping color code to identify potable and process water lines such as described in *Recommended Standards for Water Works* (Ten State Standards 2012).

Some common backflow situations, and their associated requirements, are discussed in more detail below.

1. Discharge to waste: An approved air gap must be provided between all process water and waste path, and all finished water and waste path.

2. Gaseous chlorinators and ammonia feed: A reduced pressure backflow assembly (RPBA) is required between the potable water supply and the gas chlorine/gas ammonia -water mixing point ("eductor"). There shall be no branching of solution lines downstream of a gas eductor to water at different stages of treatment (e.g., raw water, pre-filter, post filter) since these branch lines are potential cross connections between inferior and superior quality water.

3. Sample lines to process monitoring instruments: There shall be no branching of sample lines to water at different stages of treatment (e.g., raw water, pre-filter, post filter) since these branch lines are potential cross connections between inferior and superior quality water.

4. Upflow saturators (for all chemical compounds, including sodium fluoride): A RPBA is required between the potable water supply and the upflow saturation tank.

5. Down-flow saturators and other hard-plumbed piping to chemical feed solution tanks: An air gap or RPBA is required between the potable water supply and the chemical solution tank.

6. Erosion feed systems: When used in combination with a secondary tank, an RPBA or air gap is required to protect the potable water supply from any holding tank. Evaluate erosion feeders on a case-by-case bases to ensure adequate backflow prevention is installed.

7. Solution tanks without approved air gaps (i.e., filled by a hose): A RPBA is required between the drinking water supply and the hose bib(s) used to fill the solution tank(s). All hose bibs should have a hose bib vacuum breaker.

Treatment heads on ion exchange reactors relying on brine regeneration are typically equipped with a common line for filling and withdrawing water to create and use the brine solution. These installations do not require backflow protection. The design of the treatment head precludes the possibility of backflow conditions during normal operation.

10.7.3 Other Design Considerations

Control at the point of chemical injection into the public drinking water supply is necessary to protect against overfeed as a result of backsiphonage. We recommend applying the following standards. Other ways the design can minimize the risk of chemical overfeed are in Part 5 of the *Recommended Standards for Water Works* (Ten State Standards, 2012).

1. All liquid chemical feed systems: Install one diaphragm-type (diaphragm spring-loaded in the closed position) anti-siphon device at the head of the metering pump. In the case of fluoride, hydroxide, and acid addition, a second anti-siphon device should be installed at the injection point.

2. Potable water as transport carrier solution (e.g., gas chlorine; ammonia): Install one diaphragm-type (diaphragm spring-loaded in the closed position) anti-siphon device at the point of injection into the potable water carrier pipe.

10.7.4 Common Wall Construction in Treatment Facilities

Engineers must avoid the cross-connection contamination risk associated with failure of a common wall between untreated or partially treated (non-potable) and finished water (filtered). Common walls may be structural or piping.

1. Structural walls: Although this is a greater concern for package-filtration treatment plants, it could apply to any treatment process designed with adjacent walls between various unit processes. In surface water treatment applications, design engineers should design double-wall separation (providing an air space) between unfiltered water, such as flocculation and sedimentation basins, and filtered water (underdrains and clearwell for filtered water) so operators can check for fractures in either wall's integrity (the air space will fill with water). See Chapter 11 for further details.

2. Piping through filter media: There shall be no air supply or other piping installed vertically through filter media. The pipe represents a direct conduit between unfiltered and filtered water.

Table 10-3
Recommended Protection at Fixtures and Equipment In Water Treatment Plants *

AG	Air Gap	PVBA	Pressure Vacuum Breaker Assembly
AVB	Atmospheric Vacuum Breaker	RPBA	Reduced Pressure Backflow Assembly
DCVA	Double Check Valve Assembly		

Description of Fixture, Equipment, or Use	Minimum Protection at Fixture
Untreated and process water tanks	Provide AG on overflow and drain
Bulk liquid chemical storage tanks	1. Provide AG on dilution water supply line, and 2. AG on overflow and drain, and 3. Provide day tank
Day tanks	1. Provide AG on dilution water supply line, and 2. AG on overflow and drain, and 3. Size to hold no more than a 30-hour supply
Dry chemical feeder solution tanks	1. Provide AG on fill supply line, and 2. AG on overflow and drain
Saturators	1. Provide AG or RPBA on fill supply line, and 2. AG on overflow and drain
Chemical feed pumps, general	1. Assure discharge at point of positive pressure, or 2. Provide vacuum relief (anti-siphon device), or 3. Provide other suitable means or combinations as necessary to control siphoning 4. No pump priming or flushing line
Chemical feed pumps, fluoride compounds	In addition to general recommendations for chemical feed pumps, these precautions apply: 1. Provide dedicated electrical connection interlocked with well or service pump, and 2. Provide two diaphragm-type anti siphon devices (one at pump head, one at injection point), and 3. Size pumps to operate at 30 percent-70 percent of capacity.
Chemical carrying line, chemical injection line, eductor line	Provide RPBA
Chemical injection line in common between potable water and nonpotable water	Provide RPBA or manifold chlorine gas rather than chlorine solution, eliminating cross-connection (use separate injectors for untreated and filtered water, if applicable)
Surface washers	Provide AVB or PVBA or DCVA
Filter backwash waste discharge	Provide AG to waste
Filter-to-waste	Provide AG to waste, or AG to process stream ahead of filters
Membrane clean-in-place systems	Provide physical disconnect
Sample lines to monitoring equipment	Provide AG or AVB
Monitoring equipment for untreated and potable water that is used in common	Provide AG or physical disconnect
Hose bibb connections	Provide hose bibb vacuum breaker (at hose outlet)

*These recommendations assume isolation of process water supply from the potable water supply. Air gaps to treated or partially treated water must be screened

10.8 Water Treatment Plant Wastewater Disposal

Design engineers should evaluate waste-product issues early because they could significantly affect the cost or feasibility of a proposed treatment approach or technology. There are a variety of liquid waste streams that can be generated from water treatment processes including:

- Backwash waste from filtration processes
- Brine waste from ion exchange treatment
- Concentrate high in dissolved solids from reverse osmosis and nanofiltration processes
- Filter-to-waste streams from filtration processes
- Waste chemicals from membrane cleaning processes (clean-in-place waste)
- Waste streams from on-line instrumentation such as chlorine residual analyzers, pH analyzers, and turbidimeters
- Other concentrate streams from clarification and sedimentation processes, usually only found in plants that treat surface water.

There is detailed guidance on treating, processing, and otherwise managing these liquid waste streams in a number of professional references (AWWA/ASCE 2012c; Kawamura 2000a; AWWA 1999; Ten State Standards 2012).

In some cases, recycling of water from the backwash process may be considered. However, the recycled water may contain higher levels of turbidity, pathogens, or treatment chemicals than the source water so additional design details should be considered. These special design considerations include:

- **Backwash holding tank sanitary integrity** – When backwash recycling is used for groundwater sources, the backwash holding tank should be designed similarly to a treated water reservoir to minimize the risk for any pathogens entering the recycle stream.

- **Determining the settling time needed** – Settling time is a key design parameter in sizing any backwash holding tank. Settling columns can be used to estimate the time required to separate the supernatant from the solids.

- **Reintroduction of the backwash supernatant** – The supernatant should be reintroduced to the treatment process prior to the any chemical addition and the supernatant recycle flow rate should be limited to no more than 10 percent of the source water flow rate to minimize the risk for process upset. For surface water treatment plants, the recycle flow **must** reintroduced prior to where the primary coagulant into the main flow stream and take other special precautions to avoid hydraulic overloads (WAC 246-290-660(4)).

- **Monitoring of the backwash supernatant** – In addition to monitoring and controlling the flow rate on the supernatant, the supernatant stream should also have continuous turbidity monitoring. While turbidity is not a regulated parameter for groundwater sources, it is a useful parameter for measuring the quantity of solids in the supernatant and minimizing the risk of overloading the filters with recycled solids for both surface water and groundwater treatment plants.

- **Disposal of solids from the backwash recycle process** - In some cases, such as for arsenic, iron, and manganese removal, the settled material may concentrate these naturally occurring metals making it more challenging to dispose of the concentrate in a sanitary sewer. Other means of disposal such as drying the solids for landfill disposal or the use or geotextile tubes for capturing the solids should be considered.

- **Emergency disposal of backwash water** – The engineer should include in the design a way to dispose of backwash water in the event of a treatment process upset that overwhelms the recycle process. Emergency disposal could include disposal to land, a drainage ditch or a sanitary sewer so that the treatment process can function properly and public health is protected.

Water treatment plants (WTPs) that discharge wastewater are considered industrial dischargers, no matter where they discharge their wastewater (to the land, surface water, or local public treatment works). The Department of Ecology (Ecology) is the lead agency in permitting discharges of wastewater from WTPs through either a general or individual permit.

Ecology permits discharges of wastewater produced from a water treatment filtration process (filter backwash, sedimentation or presedimentation basin washdown, sedimentation/clarification, or filter-to-waste) under its combined National Pollutant Discharge Elimination System (NPDES) and State Waste Discharge General Permit ("General Permit"). All eligible facilities **must** apply for coverage. See Table 10-4.

Wastewater discharge from a WTP is covered under the General Permit if <u>all</u> the following criteria are met:

- The WTP is not covered by an NPDES waste discharge <u>individual</u> permit.
- The WTP produces water for potable or industrial use as its primary function.
- The WTP produces an average of 35,000 gallons per day or more of finished water, as determined on an average monthly basis.
- The WTP discharges its wastewater directly to surface water or to a settling pond or basin if an overflow from the pond or basin can flow to surface water. Surface waters include: lakes, rivers, ponds, streams, inland waters, wetlands, marine waters, estuaries, and all other fresh or brackish waters and water courses, plus drainages to those waterbodies.
- The discharged wastewater is produced from a water treatment filtration process (filter backwash, sedimentation/presedimentation basin washdown, sedimentation/clarification, or filter-to-waste).
- The discharged wastewater is <u>not</u> produced from ion exchange, reverse osmosis, or slow sand filtration.

Ecology considers WTPs producing an average of 35,000 gallons per day or more of finished water to be "conditionally exempt" from operating under the General Permit requirements for discharges of filter backwash wastewater if they meet <u>all</u> of the following conditions. This exemption is subject to periodic review by Ecology of WTP processes and discharge characteristics. Part of Ecology's review includes a determination of whether a "reasonable potential to pollute" exists, based on defined USEPA methods. See Table 10-4.

- The WTP discharges its filter backwash wastewater to the ground so that the majority of the liquid either evaporates or infiltrates to the subsurface, provided that the area receiving the discharge does not contain highly permeable soils; and does not lie directly above a shallow aquifer, above an aquifer with limited recharge, or in a location where groundwater quality appears to be threatened.
- Discharge to a drain field, infiltration pond, or trench should be utilized only when discharge via land application (irrigation) or into a grass-lined swale is not possible. Note: Discharge to a "dry well" is prohibited under the State Underground Injection Control Act.
- Infiltration ponds and trenches must have sufficient freeboard to prevent over-topping and must be managed so that no reasonable potential exists for discharge to surface water.
- The wastewater must be free of additives and any amounts greater than *de minimis* of toxic materials that have the potential to reach waters of the State.
- The volume of the discharge and the concentration of dissolved solids do not demonstrate a reasonable potential to contaminate groundwater.
- Discharge must not cause soil erosion or deterioration of land features.
- Residual solids that accumulate in infiltration ponds and trenches must be disposed of as necessary to avoid a build-up and concentration of these materials.
- Disposal of solids must be consistent with requirements of the local health jurisdiction.

WTPs discharging wastewater produced from a water treatment filtration process (filter backwash, sedimentation or presedimentation basin washdown, sedimentation/clarification, or filter-to-waste) that have an actual average production rate of less than 35,000 gpd of finished water generally do not require a permit to discharge filter backwash wastewater. Generally, such WTPs are assumed to have no reasonable potential to pollute. See Table 10-5.

Ecology excludes from coverage under its General Permit wastewater discharges from WTPs that employ ion exchange, reverse osmosis, or slow sand filtration. Depending on site-specific circumstances, Ecology may require such WTPs to obtain coverage under an individual permit. Design engineers employing ion exchange or reverse osmosis should evaluate waste generation issues early and consult with Ecology because waste discharge permit requirements could significantly affect the cost or feasibility of the proposed treatment. See Table 10-6.

Ecology's web page on general permits for water treatment plants provides a link to the current general permit, and a link to the general permit fact sheet. The fact sheet explains how the general permit conditions were developed, presents the legal basis for permit conditions, and provides background information on water treatment facilities.

Disposal of Analyzer Reagent Waste

Due to the small volumes and low toxicity, Ecology typically does not require a discharge permit for disposal to ground of analyzer reagent waste streams, but Ecology does require implementation of appropriate best management practices and all known, available, and reasonable methods of prevention, control, and treatment (AKART). Ecology's position is:

- Water systems must not discharge this wastewater to surface water.
- Water systems should discharge this wastewater to the sanitary sewer if at all possible.
- If discharge to the sanitary sewer is not possible, then water systems should discharge the wastewater to the ground in such a way as to maximize evaporation of the reagent. The discharge site must be owned by the water system and should be located as far as possible from any drinking water supply. Runoff from the discharge site may not flow off the water system property.

10.9 Placing a Water Treatment Plant into Service

Before a water treatment plant can be placed into service, it **must** be properly tested, inspected, and disinfected (WAC 246-290-120(4)). A licensed engineer **must** complete a *Construction Completion Report Form* (DOH 331-121) and submit it to ODW before a water systems uses treatment facilities to serve water to the public (WAC 246-290-120(5)).

It is often useful to have one or more meeting(s) with all parties involved in the design, construction and operation of a treatment facility in the weeks or months before start-up. These parties include representatives of the:

- Construction manager
- Design engineer
- Prime contractor
- Operators
- Owner
- Regulatory staff, usually the ODW Regional Engineer.

Such a meeting can help facilitate a smooth start-up and be useful in addressing issues if they arise during the start-up process.

Water treatment plants often consist of a collection of pipes, pumps, reservoirs and valves. As such, the following sections in this manual should be reviewed for relevant information about placing water treatment plant components into service:

- Section 6.6 - Placing a Water Main Into Service
- Section 7.7 - Placing a Reservoir into Service
- Section 8.5 - Placing a Booster Pump Station into Service

There can be water quality issues associated with the start-up of any treatment facility that potentially compromises the safety of the water supply. At the very least, samples of regulated water quality parameters should be collected to demonstrate that the process is effective.

Process reliability is another concern associated with start-up of a treatment plant. For this reason, the water treatment plant should initially send treated water to waste long enough to demonstrate the process is working reliably. The duration of sending treated water to waste can range from a few hours to a couple days depending upon the treatment objective and treatment process(es) used. For biological processes, such as slow sand filtration, treated water may need to be sent to waste for several weeks. Provisions for waste disposal should be identified when large volumes of water may be produced as part of start-up.

Where on-line instrumentation is provided, the critical alarms should be tested by adjusting the water quality to the instrument or the alarm set-points to make sure that the alarms are functional and all communication systems are working. In addition, the readings from the local instrument controller should be checked against the information recorded in the Supervisory Control and Data Acquisition (SCADA) system to make sure that the two match. See Section 10.4.2 for additional information on process control including monitoring, instrumentation and alarms.

In some cases, there may be distribution system impacts as a result of bringing a new treatment plant or significant process on-line. These distribution system impacts include hydraulic as well as water quality changes. Hydraulic changes, such as flow reversals and changes in velocity and pressure in distribution system piping, can lead to suspension of sediments deposited in mains and cause the loss of chlorine residual, elevated turbidity, and customer complaints. Changes in water quality can lead to the release of sediments and metals from pipe surfaces. As a result, additional distribution system monitoring should be planned as part of placing a water treatment plant into service. This additional monitoring may include additional rounds of tap sampling as required under the Lead and Copper Rule (See Section 10.1.3 and 40 CFR 141.86(d)(4)(vii)).

Most water treatment facilities **must** complete additional start-up and testing requirements prior to certification of construction completion (WAC 246-290-120(4)). See Section 10.4.3 for more information on start-up and testing for water treatment facilities.

See Section 11.6 for additional information on placing a surface water treatment plant into service.

Table 10-4

Waste Discharge Permitting Guidance for Water Treatment Plants Greater than or Equal to 35,000 gpd Average Daily Finished Water Production

Treatment is not IX, RO, or Slow Sand Filtration

Waste Stream Characteristics (daily volume, content, etc.)	Disposal Method	Agency with Regulatory Oversight Authority
Wastewater (not the settled sludge) generated by filter backwash (including from microfiltration and ultrafiltration), sedimentation/presedimentation basin washdown, sedimentation/clarification, and filter-to-waste processes	Discharge to surface water	**Department of Ecology** WTP General Permit
Wastewater (not the settled sludge) generated by filter backwash (including from microfiltration and ultrafiltration), sedimentation/presedimentation basin washdown, sedimentation/clarification, and filter-to-waste processes	Discharge to ground	**Department of Ecology** Site-specific: May need a state waste discharge permit. **Department of Health** Wellhead protection requirements
Wastewater (not the settled sludge) generated by filter backwash (including from microfiltration and ultrafiltration), sedimentation/presedimentation basin washdown, sedimentation/clarification, and filter-to-waste processes	Discharge to POTW	**Local municipality** or **Department of Ecology**
Settled sludge (from wastewater) generated by filter backwash (including from microfiltration and ultrafiltration), sedimentation/presedimentation basin washdown, sedimentation/clarification, and filter-to-waste processes	Agronomic or silvicultural use	Land application: **Local health jurisdiction** Statewide Beneficial Use Determination: **Department of Ecology**
Settled sludge (from wastewater) generated by filter backwash (including from microfiltration and ultrafiltration), sedimentation/presedimentation basin washdown, sedimentation/clarification, and filter-to-waste processes	Landfill	**Local health jurisdiction**

Table 10-5

Waste Discharge Permitting Guidance for Water Treatment Plants Less than 35,000 gpd Average Daily Finished Water Production

Treatment is not IX, RO, or Slow Sand Filtration

Waste Stream Characteristics (daily volume, content, etc.)	Disposal Method	Agency with Regulatory Oversight Authority
Wastewater (not the settled sludge) generated by filter backwash (including from microfiltration and ultrafiltration), sedimentation/presedimentation basin washdown, sedimentation/clarification, and filter-to-waste processes	Discharge to surface water	**Department of Ecology** No reasonable potential to pollute.
Wastewater (not the settled sludge) generated by filter backwash (including from microfiltration and ultrafiltration), sedimentation/presedimentation basin washdown, sedimentation/clarification, and filter-to-waste processes	Discharge to ground	**Department of Ecology** No reasonable potential to pollute. **Department of Health** Wellhead protection policy.
Wastewater (not the settled sludge) generated by filter backwash (including from microfiltration and ultrafiltration), sedimentation/presedimentation basin washdown, sedimentation/clarification, and filter-to-waste processes	Discharge to POTW	**Local municipality** or **Department of Ecology**
Settled sludge (from wastewater) generated by filter backwash (including from microfiltration and ultrafiltration), sedimentation/presedimentation basin washdown, sedimentation/clarification, and filter-to-waste processes	Agronomic or silvicultural use	Land application: **Local health jurisdiction** Statewide Beneficial Use Determination: **Department of Ecology**
Settled sludge (from wastewater) generated by filter backwash (including from microfiltration and ultrafiltration), sedimentation/presedimentation basin washdown, sedimentation/clarification, and filter-to-waste processes	Landfill	**Local health jurisdiction**

Table 10-6

Waste Discharge Permitting Guidance for Water Treatment Plants for Any Treatment Plant using IX, RO, Microfiltration, Ultrafiltration, or Nanofiltration

Waste Stream Characteristics (daily volume, content, etc.)	Disposal Method	Agency with Regulatory Oversight Authority
IX or RO brine, or filter backwash that contains dissolved solids removed from the source water (consisting of regeneration liquid, ionic pollutants, and rinse water)	Discharge to surface water	**Department of Ecology** Individual NPDES permit, except for discharges from desalinization processes of up to 5,000 gpd to salt waters.
IX or RO brine, or filter backwash that contains dissolved solids removed from the source water (consisting of regeneration liquid, ionic pollutants, and rinse water)	Discharge to ground	**Department of Ecology** Site-specific: May need an NPDES individual permit or a state waste discharge permit.
IX or RO brine, or filter backwash that contains dissolved solids removed from the source water (consisting of regeneration liquid, ionic pollutants, and rinse water)	Discharge to POTW	**Local municipality** or **Department of Ecology** Site-specific: May need a state waste discharge permit.
IX or RO brine, or filter backwash that contains dissolved solids removed from the source water (consisting of regeneration liquid, ionic pollutants, and rinse water)	Agronomic or silvicultural use	**Department of Ecology** Site-specific: May need a state waste discharge permit.
Settled sludge (from wastewater) generated by filter backwash, sedimentation/presedimentation basin washdown, sedimentation/clarification, and filter-to-waste processes	Landfill or recycling	**Local health jurisdiction**

IX = Ion exchange.

RO = Reverse osmosis

The main assumption for this table is that wastes and discharges are "typical," i.e., they do not contain unusually large amounts of pollutants.

Single domestic or point-of-use IX or RO systems do not require a state waste discharge permit because they are considered to have no reasonable potential to pollute.

Table 10-7
Disinfection Treatment

Established Technologies	Typical Application	Notes
Chlorine Gas	Primary/ Secondary	Consumes alkalinity and may reduce pH. Requires a risk management plan if >2,500 lbs. stored on site (see 40 CFR 68). Use of gaseous chlorine may trigger an International Fire Code requirement for spill mitigation measures such as containment or scrubbers. Proponents of new installation should coordinate this with the local fire prevention authority. Evaluate total trihalomethane (TTHM) and haloacetic acids five (HAA5) formation.
Hypochlorination	Primary/ Secondary	Design should evaluate expected storage time and effect on solution strength, potential for strength dilution to minimize these problems; evaluate TTHM and HAA5 formation.
Chlorine Dioxide	Primary/ Secondary	On site generation. Maximum allowable ClO_2 concentration at entry = 0.8 mg/L (as ClO_2), MCL for chlorite = 1.0 mg/L Use triggers additional monitoring for chlorine dioxide and chlorite.
Ozone	Primary	Pilot work required to determine decay and demand characteristics. May significantly increase biodegradable organic matter in treated water, which may require secondary disinfection. MCL for bromate = 0.010 mg/L.
Chloramines	Secondary	Background ammonia levels must be considered and requires close operator attention to ensure proper ammonia-chlorine ratio. Design must provide overfeed protection. Water systems proposing changeover from free chlorine should evaluate the potential for elastomer degradation (Reiber 1993).
Irradiation (UV light) See Appendix I	Primary	Minimum applied UV dose for groundwater applications is 186 mJ/cm^2 for 4-log virus inactivation. Reactor validation uncertainties will require the applied reduction equivalent dose (RED) to be even greater than this threshold. If UV is the primary surface water disinfectant, a RED of at least 40 mJ/cm^2 is required for Giardia inactivation with virus inactivation provided by chlorine. Additional information and guidance is available from EPA (USEPA 2006a) and ODW.
On-Site Hypochlorite Generation	Primary/ Secondary	See notes for hypochlorination above. ANSI/NSF Standard 60 certified sodium chloride (salt) must be used to generate the hypochlorite solution. The design should address ventilation for hydrogen gas to minimize the risk of explosions.
Tablet Chlorinators	Primary/ Secondary	See notes for hypochlorination above. Design should consider potential for variations in chlorine dosage.

Notes:

1. Primary disinfection used to inactivate pathogenic organisms from source water.
2. Secondary disinfection used to maintain a distribution system residual.
3. Disinfection performance requirements are detailed in chapter 246-290 WAC, Parts 5 and 6.

Table 10-8
Corrosion Control Treatment

Established Technologies	Notes
pH/alkalinity adjustment	
--Chemical Addition	Caustic soda (NaOH), lime ($Ca(OH)_2$), soda ash (Na_2CO_3), sodium bicarbonate ($NaHCO_3$), calcium carbonate ($CaCO_3$), potassium hydroxide (KOH) and carbon dioxide (CO_2) (USEPA, 1992; Economic and Engineering Services, 1990).
--Calcite Contactor	Applies to small water systems (generally less than 500 people). No danger of chemical over-feed and is usually not operator intensive. Generally applies when Ca^{2+} < 30 mg/L, alkalinity < 60 mg/L (both as $CaCO_3$) and pH low (<7.2). Potential clogging due to Fe/Mn and other particulate matter. Waters with significant natural organic matter (>2 mg/L total organic carbon (TOC)) should be evaluated to ensure that organic deposits will not interfere with the dissolution of media over time.
--Aeration/Air Stripping	Suitable for groundwater high in CO_2, effectiveness controlled by alkalinity and aeration system design, capital costs usually high, pre- or post-aeration disinfection should be provided. Pilot work to verify design parameters (for example, height, packing, air and water ratio) must be completed.
Calcium Carbonate Precipitation	Calcium carbonate precipitation is not a viable approach for corrosion control in the Pacific Northwest due to the region's relatively soft waters.
Inhibitors	
--Ortho - / Poly - / Blended Phosphates	Phosphate based inhibitors are pH sensitive, so the pH range should be maintained with the range of 7.2 to 7.8. Since phosphate can increase biological activity in the distribution system, disinfection may be required along with the addition of phosphates.
--Silicates	Sodium silicate inhibitors are not well understood (USEPA 1992; Reiber 1990). Silicate effectiveness thought to be a combination of concurrent pH increase and protective film on piping walls.

Table 10-9
Treatment Technologies for Selected IOCs

Established Technologies	Contaminant	Notes	References
Oxidation/Filtration	As, Fe, Mn	Oxidation kinetics are pH sensitive (principally Mn), organic matter will increase oxidant demand. Fe addition may be required to remove As. Filtration rates dependent upon technology and water quality.	Hoffman et al. 2006; HDR 2001
Cation Exchange	Fe, Mn	Should not be used if the concentration of Fe and Mn is greater than 0.3 mg/L. Prevent oxidation of Fe and Mn prior to ion exchange or resin will foul. Waste disposal of brine may be an issue.	Ten State Standards 2012
Anion Exchange	As, NO_3	Use nitrate selective resin (for NO_3), As: Oxidize As(III) to As(V), Competition with sulfate and other ions must be evaluated. Total dissolved solids should be <500 mg/L. Post-column pH adjustment required. Evaluate waste disposal issues.	Clifford 1999; USEPA 2003; WSDOH 2005
Activated Alumina	F	pH adjustment required to maximize adsorption, pH adjustment not recommended for small water systems due to operational complexity and safety issues.	Clifford 1999
Iron Based and Other Specialized Adsorbents	As	Performance of adsorbents varies with vendor and water quality. Some adsorbents do not remove As(III). If As(III) is present, pre-oxidation may be required.	USEPA 2003
Reverse Osmosis (RO)	As, F, Fe, Mn, NO_3	Post treatment corrosion control may be required, high operation cost, sizing strongly temperature sensitive, concentrate disposal issues must be evaluated. As (III) should be oxidized to As (V). Side stream blending may be appropriate.	USEPA 2005; USEPA 2003
Sequestration	Fe, Mn	For source water with a combined Fe/Mn concentration of less than 1.0 mg/l (Mn < 0.1 mg/l). May be applicable at higher concentrations, however, these applications should conduct bench scale studies and will be allowed only on existing sources. Disinfection required.	Robinson et al. 1990; HDR 2001; Ten State Standards 2012.
Alternative Technologies	**Contaminant**	**Notes**	**References**
Biological Removal	NO_3, Fe, Mn	Not in widespread use in United States. Substantial pilot work (1 year continuous operation at a minimum) would be required to establish biological process, and post-treatment disinfection must be provided. Taste and odor control issues.	HDR 2001; WSDOH 2005

Notes:
1. Pilot testing is expected for all technologies listed above. See Section 10.3 for additional pilot testing information.
2. The listed technologies may be capable of removing other inorganic chemicals. Contaminants are listed in this column if typical removal rates for the specific technology are expected to be greater than 70 percent in most applications as indicated in selected references.
3. Manufactured media and equipment must meet the requirements of WAC 246-290-220.
4. Processes listed above are expected to require a minimum of 6-8 hours per week of operator involvement, although some may require more. Water systems proposing to install a treatment system should contact existing facilities and participate fully in pilot work to better assess long-term operator needs.
5. Instrumentation/control that may be appropriate includes: Automatic plant shut down for process equipment and pump failure, auto-dialers or similar equipment to alert 24-hour on-call personnel of plant failures, on-line filtered or finished water monitoring equipment and automatic filter-to-waste capability.

Table 10-10
Treatment Technologies for VOCs and SOCs

Technologies	Notes
Granular Activated Carbon (GAC)	A best available technology for removal of VOCs and SOCs. May require pre-filtration to remove particulate matter. Competition for GAC sorption sites with natural organic matter may occur. Seasonal increases in competing species may cause desorption of contaminant and must be fully evaluated. Requires reactivation of carbon on a regular basis (site and contaminant specific).
Powdered Activated Carbon (PAC)	May be effective for VOC and SOC removal, adequate mixing and contact time must be provided, existing settling and filtration must effectively remove added PAC. May be used seasonally if problem is not continuous. EPA considers PAC an “emerging” technology for VOC removal (USEPA 1998).
Aeration	A best available technology for removal of VOCs and some of the more volatile SOCs. Established technologies include packed tower, diffused, and multiple tray aeration. Some alternative configurations require evaluation through pilot studies (see WAC 246-290-250). Design goals and operational parameters control performance. Aerated water should be disinfected to prevent significant growth of heterotrophic plate count bacteria (Umphres et al. 1989).
Chlorine/Ozone oxidation	Applies to glyphosate only. See Disinfection Section for specific issues related to these technologies.

Notes:

1. Pilot testing is required for all technologies listed above, and may be required over periods of varying water temperature, and varying contaminant concentrations, if applicable. See Section 10.3 below for additional pilot testing information.
2. Manufactured media and equipment must meet the requirements of WAC 246-290-220.

Table 10-11
Treatment Technologies for Reduction of DBPs

Precursor Removal	Notes
Enhanced Coagulation	Suitable only for conventional surface water plants. Nature of source water organic material, treatment conditions (coagulation pH) and background alkalinity control effectiveness. Requires significant coagulant doses. Required Treatment Technique according to the Stage 1 D/DBP Rule for surface water treatment plants that use conventional rapid rate filtration.
Granular Activated Carbon	GAC10 (empty bed contact time of 10 minutes) and reactivation period of carbon of no more than every six months. This is a best available technology for removal of DBP precursors, although performance is dependent on the selected GAC and the nature of the organic matter to be removed.
Powdered Activated Carbon (PAC)	Suitable only for conventional surface water plants or potentially membrane applications. Effectiveness dependent on the nature of the organic matter present, the must be demonstrated through long-term pilot (at least 1 year of operation).
Biologically Active Filtration	Use of preozonation followed by a rapid rate filtration process. Filter media may be GAC, anthracite, sand, or some combination. TOC removals in the 20-70 percent range possible, dependent on the nature of the organics present, ozone: TOC dose, and filter contact time (Carlson and Amy 1998).
Slow Sand Filtration	Standard slow sand filtration expected to remove 5-25 percent of source water organic matter (as TOC) Use of preozonation will increase removal, however long term piloting (at least 1 year of operation) is required to determine effectiveness and effect on filter cleaning requirements (Eighmy et al. 1993).
Membranes	Nanofiltration can effectively remove DBP precursors. Unamended ultra- or microfiltration will not generally remove precursors. Use of PAC in ultrafiltration water systems has been effectively demonstrated (AwwaRF et al. 1996).
DBP Removal or Mitigation	
Aeration	Some volatile DBP (such as chloroform) can be significantly removed through appropriately designed aeration processes (Billeo et al. 1986; Walfoort et al. 2008). Temperature and air-water ratio are significant design factors.
Alternative Disinfection/Application	Use of chloramines in distribution systems with long detention times or ozone or chlorine dioxide as a primary disinfectant may mitigate the formation of regulated DBP sufficiently. See Table 10-4 for issues specific to these approaches.

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——, (d) Chapter 19. Pilot Plant Design and Construction

——, (e) Chapter 20. Chemical Systems

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Chapter 11: Surface Water Treatment

11.0 Introduction

This chapter covers important concepts that are unique to the design of treatment facilities for surface water sources and sources designated as groundwater under the direct influence of surface water (GWI). Surface water and confirmed GWI sources must meet the same regulatory requirements and are synonymous throughout this section. These sources carry a high microbial risk. As a result, the basic treatment framework requires the following minimum level of treatment always be achieved:

- 2-log removal or inactivation of Cryptosporidium oocysts,
- 3-log removal or inactivation of *Giardia lamblia* cysts, and
- 4-log removal or inactivation of viruses.

For poor quality surface water sources with the highest risk of contamination, and for purveyors seeking approval for a *Limited Alternative to Filtration* (see Policy F.10), Washington state rules may require a greater level of treatment than the basic framework of federal rules outlined above.

Chapter 246-290 WAC Part 6 describes the basic regulatory requirements for surface water treatment facilities, which stem from multiple federal rules developed over the past three decades including the Surface Water Treatment Rule (SWTR), Interim Enhanced Surface Water Treatment Rule (IESWTR), Long Term 1 Enhanced Surface Water Treatment Rule (LT1ESWTR), Filter Backwash Recycle Rule (FBRR), and Long Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR). Effective implementation of these rules and protection of consumers depends upon a multiple barrier framework including effective source water protection, filtration, and disinfection, and maintenance of treated water quality within the distribution system overseen by trained and properly-certified operators.

This chapter is organized similarly to Chapter 10, with the following sections narrowly tailored to surface water treatment issues:

- Alternatives Analysis (Section 11.1)
- Treatment Technologies (Section 11.2) including:
  - Clarification and Sedimentation
  - Filtration
  - Disinfection
- Predesign Studies (Section 11.3) including pilot studies
- Project Reports (Section 11.4) including identifying
  - Design Criteria
  - Process Control – Monitoring, Instrumentation, and Alarms
  - Start-up and Testing Procedures
  - Operations and Staffing
  - Process Reliability
- Operations Program (Section 11.5)
- Placing a Surface Water Treatment Plant into Service (Section 11.6)

Design engineers should familiarize themselves with the content of Chapter 10, since some design topics are only covered in Chapter 10.

- Construction Documents (Section 10.5)
- Treatment Chemicals (Section 10.6)
- Cross Connection for Water Treatment Facilities (Section 10.7)
- Waste Residuals Management (Section 10.8)

Design engineers need to consider many factors in developing surface water treatment projects since source water quality and quantity can be expected to change seasonally and from year to year. In addition, surface waters have a greater microbial risk than groundwater sources. Therefore, the design process may be iterative and require a more thorough characterization of the source water, longer and more detailed pilot studies, and a well-planned commissioning process. As such, the time to develop a surface water treatment project is usually much longer than the time required to develop other types of projects. Figure 2-3 outlines the general design and review process for water treatment projects.

11.1 Alternatives Analysis

A surface water treatment facility is a major capital investment with high life-cycle costs and a potential high risk to public health if the treatment processes fail to operate as intended. Therefore, the engineer **must** evaluate all appropriate and applicable alternatives before selecting a particular treatment technology and design approach.

The engineer should cover the following items in the alternatives analysis applying detailed guidance provided in professional references:

- Source capacity and projected demands
- Source water protection
- Source water quality
- Operational complexity, reliability, and staffing
- Secondary impacts of water treatment (see Chapter 10)
- Waste disposal and management (see Chapter 10)
- Life cycle costs (see Chapter 10)

11.1.1 Source Capacity and Projected Demands

Surface water sources are generally more prone than groundwater sources to changes in flow from year to year. In addition, some watersheds are experiencing prolonged hydrologic shifts as a result of global warming (Mote et al. 2005). These hydrologic shifts include reduced snowpack and lower summer stream flows. Logging, wildfires, and changes in land use can also cause adverse hydrologic shifts. In some cases, it may be necessary to evaluate these hydrologic shifts in assessing the safe yield of the water supply. In general, the safe yield of a surface water supply is the reliable withdrawal rate of water that can be provided by a watershed through the critical drought period. See Section 5.8.1 for additional information on assessing the safe yield of surface water supplies.

Periodically, filters need to be taken out of service for backwashing (rapid rate filtration), filter scraping and ripening (slow sand filtration), recovery cleaning (membrane filtration), and general maintenance (all types of treatment). Since these activities cannot always be scheduled for periods of low demand, the design engineer should also consider the production capacity of the treatment process and the ability to satisfy consumer demand with one filter out of service. In addition, water consumed during backwash and filter to waste should be considered when analyzing the overall process capacity, both on a daily and annual basis. See Chapter 4 for further details on estimating future water system capacity needs.

11.1.2 Source Water Protection

Source water protection is an important barrier in the overall multiple barrier framework to protect consumers from risks associated with surface water. As such, a watershed control program **must** be developed for all surface water and GWI sources and included in the planning document for the water system (WAC 246-290-135(4)). The overall purpose of the watershed control program is to assess and reduce microbial and chemical contaminant risk in the source(s) supplying a treatment plant. Poor source water protection can lead to increased risks from pathogens, algae, and chemicals such as pesticides and herbicides, as well as present other risks and treatment challenges.

11.1.3 Source Water Quality

Source water and finished water quality objectives form the primary basis for selecting treatment process alternatives. Higher quality sources present lower risks to public health and are usually less expensive to treat, both initially and over the long term. Therefore, in general, the source of supply **must** be obtained from the highest quality source feasible (WAC 246-290-130(1)). Logging, forest fires, and changes in land use can also lead to changes in water quality such as higher turbidity and microbiological risks; risks that should be considered if applicable.

The extent and availability of raw water data may affect preliminary screening of alternatives and the duration of the pilot study. Surface water sources and GWI sources can experience rapid and seasonal changes in water quality. Source water characterization should account for source water quality variability. For most surface water sources the design engineer should have at least one year of water quality information before making a preliminary determination of a treatment method.

The scope of source water sampling will vary depending upon the type of source water, location of the intake, seasonal changes, and other issues. For example, rivers and streams can experience rapid changes in turbidity and inorganic parameters associated with changes in precipitation and runoff. Lakes and reservoirs often experience seasonal water quality changes associated with algae growth, lake stratification, and turnover. Basic water quality information needed to plan appropriately for a water treatment facility includes:

- **Turbidity –** High turbidity can rapidly clog filters triggering frequent backwashes and other operational issues such as coagulant demand and control. For these reasons, both daily maximum and average monthly turbidity data should be compiled for the source water.

- **Temperature –** Low temperature makes water more viscous, affecting headloss through filters (especially membrane filters) and the ripening time for slow sand filters. Temperatures of less than 8°C can be challenging for rapid rate filtration (Kawamura 1999). Low temperature affects chemical disinfection processes, requiring more contact time or higher disinfectant residual concentrations to meet pathogen inactivation requirements.

- **Microbiological Risk –** Under the Long Term 2 Enhanced Surface Water Treatment Rule, all surface water sources **must** undergo source water quality monitoring to assess microbial risk. For a new source, the water system must submit to ODW a schedule for monitoring planned under the LT2ESWTR (WAC 246-290-630(16)). The results of this monitoring should be included in the project report. The concentration of coliform in the source water can also be useful in determining the suitability of various filtration technologies.

- **Inorganic Parameters –** The seasonal variability of pH should be assessed along with regulated inorganic contaminants. Some surface water treatment processes are sensitive to the pH and alkalinity of the source water. Surface waters in parts of Washington State may require the addition of alkalinity for effective and stable treatment. Conversely, the efficacy of free chlorine as a disinfectant decreases as the pH increases. For lakes and reservoirs, seasonal turnover can result in sudden increases in iron and manganese. Significant changes in pH can also be indicative of algal issues with a source water.

- **Volatile and Synthetic Organic Contaminants (VOCs and SOCs) –** A complete set of samples for VOCs and SOCs is required as part of the approval process for any new surface water source. Detection of a regulated VOC or SOC in source water should be considered in watershed risk assessment and their control reflected in treatment objectives.

- **Total Organic Carbon (TOC) –** Natural organic matter can lead to the formation of disinfection byproducts. Natural organic matter measured in milligrams per liter (mg/L) of TOC or dissolved organic carbon (DOC) often imparts a pale tea color to the water. UV absorbance at 254nm (UV_{254}) is good surrogate indicator of TOC and DOC once the relationship between UV254 and TOC/DOC is established for a given source water. Analyzing for UV254 is less expensive, less complicated, and can easily be done on-site.

- **Algae and Chlorophyll-a** – Algae can cause filter clogging, changes in source water pH, and taste and odor issues. Algae can be a health concern as some species of cyanobacteria – commonly called blue-green algae – can produce toxins. It may be appropriate to identify and enumerate phytoplankton, including algae, seasonally and as frequently as weekly, especially in lakes prone to stratification and turnover. Chlorophyll-a is an inexpensive surrogate for assessing the abundance of algae in a surface water supply.

Phycocyanin is another possible surrogate, though it is specific to cyanobacteria and currently can only be monitored using in-situ probes and satellites.

Once the source water quality is well characterized, the design engineer can assess which surface water filtration technologies are most appropriate. See Table 11-1 for design guidance on source water quality limitations for various filtration technologies. Pretreatment processes can allow for greater levels of coliform, color, or turbidity than listed below. See Section 11.2.1 for additional information about pre-treatment technologies.

Table 11-1
General Source Water Limitations for Filtration Technologies

Filtration Technology	Turbidity (NTU)[1]	Total Coliform (#/100 mL)[1]	Color (CU) [1]
Conventional Filtration	<3000	<5,000 – 20,000	< 75
Direct Filtration	< 15	<500	< 40
In-line Filtration	<15	<500	<10
Slow Sand Filtration[2]	< 10	<800	< 5
Diatomaceous Earth Filtration	< 5	<50	< 5
Pressure Filtration[3]	**DO NOT USE**		
Bag and Cartridge Filtration[2]	< 5	See Note 4	See Note 4
Membrane Filtration	See Note 4	See Note 4	See Note 4

Notes:

1. The water quality limitations derived from a combination of sources including Letterman,1986, USEPA 1975, Cleasby et al. 1984, AWWA/ASCE 2012b.
2. These limits are for applied filter turbidity. The treatment process can handle higher source water turbidities if additional pretreatment is provided.
3. According to Section 4.2.2 *Recommended Standards for Water Works* (Ten State Standards 2012), pressure filters are for iron and manganese removal, and **must not** be used for filtration of surface water.
4. Special studies are required to determine limitations, which are equipment specific.

11.1.4 Operational Complexity and Staffing

Surface water treatment technologies vary in their operational complexity and staffing needs. For example, rapid rate filtration is much more mechanically and operationally complex than slow sand filtration. Rapid filtration process upsets or failures are more likely and can cause significant risks to public health. Such complex treatment processes usually require closer operational oversight and more operator time than simpler processes. More complex treatment processes also require a higher level of operator certification.

Finding appropriately trained and certified staff can be difficult for some water systems. Operational and staffing considerations should be addressed as part of the alternatives analysis. Most small communities struggle to find and retain operators with the skills and qualifications needed to operate complex filtration technologies. For this reason, we generally encourage small systems to choose the simplest available technology that can effectively treat the source water quality.

See Section 11.4.4 for additional information on operational and staffing considerations for various surface water treatment technologies.

11.2 Treatment Technologies

As noted above, the evaluation of treatment technologies should consider source water quality, as well as other factors including space availability, treatment complexity, and cost. Multiple barriers are used to meet surface water treatment requirements including:

- Screening and prefiltration (Section 11.2.1)
- Chemical addition (Section 11.2.2)
- Clarification and sedimentation (Section 11.2.3)
- Filtration (see Section 11.2.4)
- Disinfection (Section 11.2.5)

These are the typical process involved in protecting public health through the removal and inactivation of pathogens. However, surface water treatment can provide other benefits such as the removal of disinfection byproduct (DBP) precursors, organic chemicals, inorganic chemicals, and taste and odor causing compounds. More information about removing these compounds is in Section 10.2.

11.2.1 Screening and Prefiltration

Screening and prefiltration are often required for bag, cartridge and membrane filters to prevent rapid fouling of or damage to the filters. Screens are also used with other treatment technologies to minimize the potential for damage to equipment such as pumps, valves, and the filters themselves. The size and type of screening or prefiltration may be identified by the manufacturer of the downstream filtration equipment. For membrane filtration, self-cleaning screens in the 200 to 500 micron (μm) range are often used. For bag filtration, a series of prefilters with nominal pore sizes of 25 to 2 um are commonly used. Other types of prefiltration such as roughing filters can be used to extend the run times for downstream filtration processes. All types of screening and prefiltration equipment should be equipped with a means of measuring the differential pressure or headloss across the units.

11.2.2 Chemical Addition (Initial)

Many clarification, filtration, and disinfection processes can be affected by upstream chemical addition. For some process, such as rapid rate filtration, chemical addition is essential. Without closely controlled chemical addition, rapid rate filtration does not provide effective pathogen reduction.

There are a wide variety of chemicals that are used to make surface water treatment processes function effectively including oxidants, coagulants and other specialized chemicals such as powdered activated carbon (PAC) used for taste and odor control .

One key aspect of the predesign process is to identify the type and dose range of oxidants, coagulants, and other chemicals needed to make downstream treatment processes function most effectively. Any chemical used in a surface water treatment facility **must** be ANSI/NSF Standard 60 approved and applied within its maximum application dosage (WAC 246-290- 220(3)). More information on chemical addition for specific unit processes is included below. Also see Section 10.6 for more general design information on chemical feed systems.

11.2.3 Clarification and Sedimentation

Clarification and sedimentation processes are used upstream of filtration processes to remove pathogens, turbidity, and other materials that can clog filters. Common clarification and sedimentation processes include:

- Ballasted sedimentation
- Contact adsorption clarification
- Dissolved air flotation
- Gravity sedimentation, usually with tube or plate settlers

In general, sedimentation processes are well suited to source waters from rivers and other very turbid surface waters, while dissolved air flotation is well suited for lakes, especially ones with high concentrations of algae or organic carbon (Figure 11-1). Turbidimeters, level sensors, and flow monitoring should be provided to allow the operators to observe and control sedimentation and clarification processes.

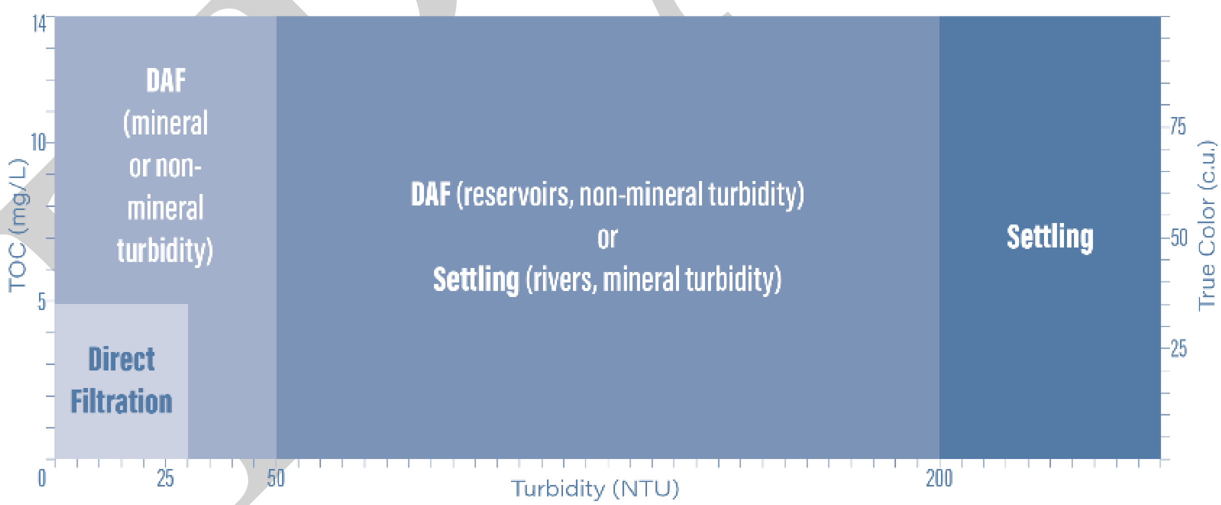

Figure 11-1: Surface Water Treatment Process Selection using Maximum Source Turbidity and TOC (Valade et al. 2009)

Clarification and sedimentation processes may be granted pathogen removal credit of 0.5-log removal for *Giardia lamblia* cysts and 1.0-log removal for viruses (WAC 246-290-660(2) if they demonstrate consistent and effective pathogen removal and are in conformance with accepted professional standards and guidelines such as those noted in this section. If such pathogen removal credit is granted, the water system **must** meet the disinfection byproduct precursor removal requirements under 40 CFR 141.135 (WAC 246-290-660(3)).

Ballasted Sedimentation – is a high rate sedimentation process that utilizes polymers and very fine sand to accelerate the sedimentation process. As a result, the detention time in the flocculation and sedimentation process is short, and surface loading rates are high, usually between 8 and 20 gpm/ft^2. The process is very effective at removing high turbidity loads usually associated with river sources. In addition, the process has been shown to consistently and effectively remove *Giardia lamblia* cysts, so can be granted 0.5 log Giardia removal credit provided the surface loading rate does not exceed 20 gpm/ft^2 (AWWA/ASCE 2012a, Alvarez et al. 1999, Kawamura 2000). With the short detention times, the process needs to be monitored closely to minimize the risk of process upsets that can affect downstream filtration and disinfection processes. Ballasted sedimentation should not be used upstream of membrane filters since the polymers used in the sedimentation process may adversely affect performance of the membrane filters.

Contact Adsorption Clarification – is commonly used in small package plants on low to moderate turbidity sources. The process itself functions more like a roughing filter than a gravity sedimentation or flotation process. Coagulated water flows through coarse media about the size of pea gravel. As coagulated water passes through the media, larger flocculated particles develop and attach to the clarifier media. The media is then periodically cleaned using an air and water flushing process to remove the solids. For non-buoyant media, prescreening is important to prevent plant debris and other materials from clogging the clarifier. In addition, flow-to-waste piping and controls should be provided to minimize start-up spikes from being passed on to downstream filters.

High turbidity levels and high organic loads requiring significant coagulant addition - such as turbidity greater than 30 NTU and total organic carbon (TOC) greater than 5 mg/L - can quickly clog the clarifier media or cause high levels of DBPs. Where suitable, typical surface loading rates are 8 to 10 gpm/ft^2 (AWWA/ASCE 2012a). Given the cyclical nature of CAC performance and limited research demonstrating the effectiveness of contact adsorption clarifiers for removal of pathogenic protozoa and DBP precursors, currently no pathogen removal credit can be granted for this clarification process.

Dissolved Air Flotation (DAF) – is typically used on lakes and reservoirs, especially those that experience algal blooms or have high concentrations of natural organic matter. In the DAF process, coagulated and flocculated water is introduced into the clarifier where very fine bubbles float flocculated particles to the surface where they are removed. The clarified water is then removed near the bottom of the basin.

DAF can be classified as conventional or high-rate depending upon how the basin is designed. For conventional DAF, the clarifiers are typically less than 8 feet deep, have length-to-width ratios of at least 1.0, and have surface loading rates of less than 6 gpm/ft^2. Conventional DAF has demonstrated significant removal of *Giardia lamblia* cysts, so can be granted 0.5 log Giardia removal credit provided the surface loading rate does not exceed 6 gpm/ft^2 (Alvarez et al. 1999, Edzwald et al, 2001; Plummer et al 1995). High-rate DAF is a more recent development. In this process, the clarifiers are deep, up to 16 feet, the length-to-width ratio can be less than 1.0, and loading rates may be as high as 16 gpm/ft^2. Since there is limited research on the effectiveness of high rate DAF for removal of pathogenic protozoa, currently no pathogen removal credit can be granted for this clarification process.

Gravity Sedimentation – commonly employs tube or plate settlers to allow for greater surface loading rate and improved process performance than open basins. Gravity sedimentation has been used for a variety of source waters, though is generally better suited for those that experience high turbidity loads. Flocculation is necessary prior to gravity sedimentation to develop a floc that will settle. The surface loading rate above the portion of the basin utilizing tube settlers is generally limited to less than 2.0 gpm/ft^2; the loading rate for plate settlers is generally limited to 0.5 gpm/ft^2 based on 80 percent of the projected horizontal plate area.

Recommended loading rates are even lower if color or algae removal are treatment objectives. The process has been shown to provide effective removal of pathogenic protozoa (Logsdon et al. 1985, Haas et al. 2001). Therefore, the process can be granted 0.5-log Giardia removal credit provided that the design follows industry guidelines such as those in the *Recommended Standards for Water Works*.

11.2.4 Filtration

All surface water sources **must** be filtered unless they meet very stringent source water protection, control, monitoring, disinfection, and operational criteria (WAC 246-290-630). There are a variety of filtration technologies that provide effective pathogen removal and can be used to meet surface water treatment requirements including:

- Rapid rate filtration
- Diatomaceous earth filtration
- Slow sand filtration
- Membrane filtration
- Bag and cartridge filtration

The most appropriate filtration technology will vary depending upon a variety of factors including source water quality, the availability of skilled operators, land and energy requirements, costs, availability of replacement part for proprietary technology, and other resource considerations. For example, rapid rate filtration may be well suited a large system with a high turbidity river source, and poorly suited to a cold, clear lake in a remote, mountainous region (where a bag filter would be more appropriate). The best choice for a small water system is usually the simplest available technology that can effectively treat the source water quality. Basic information for these filtration technologies is included in Table 11-2 and the following summaries.

A well designed, operated, and maintained filter is very effective at removing pathogens and many other contaminants. However, at the start of a filtration cycle the filtered water quality may be less than optimal. For this reason, filter-to-waste capability **must** be provided for all surface water filtration facilities (WAC 246-290-676(4)(b)(iii)).

Table 11-2: Filtration Technologies

Filtration Technology	Prefiltration Chemicals Used	Maximum Filtration Rate (gpm/ft²)	Pathogen Removal Credit: Giardia Crypto	Pathogen Removal Credit: Viruses	Operational Complexity
Rapid Rate Filtration	Oxidant, coagulant, polymers	6.0[1]	2.0-log	1.0-log	High
Diatomaceous Earth Filtration	Diatomaceous Earth	1.0[2]	2.0-log	1.0-log	Moderate
Slow Sand Filtration	Usually none	0.1	2.0-log	2.0-log	Low
Bag and Cartridge Filtration	Usually none	See Note 3	Mfr. specific	None	Low
Membrane Filtration	Varies	See Note 3	Mfr. specific	None	Moderate

Notes:

1. Maximum filtration rates are lower if single media filter beds are used, instead of deep bed, dual or mixed media filters (see WAC 246-290-654).
2. DE filtration may be designed for up to 2.0 gpm/ft^2 provided the design engineer demonstrates that filtration at the higher rate consistency achieves at least 99 percent removal of Giardia and 99 percent removal of Cryptosporidium, and meets the performance requirements of WAC 246-290-660. To maintain the integrity of the DE filter cake, filters should be designed to prevent the filtration rate from dropping below 0.25 gpm/ft^2.
3. Maximum loading load rates are manufacturer and equipment specific.

Rapid Rate Filtration – This filtration technology involves the addition of multiple chemicals upstream of granular media filters. Commonly used chemicals include oxidants, coagulants, and various polymers. Typical filter media include anthracite, sand, granular activated carbon, garnet, and ilmenite. Periodically, the filters are backwashed to remove the particles and pathogens that attach to the filter media. Rapid rate filtration can be further classified by the pretreatment process prior to filtration and include:

- Conventional filtration – This process includes coagulation, flocculation and clarification prior to filtration. The clarification process may be awarded pathogen removal credit.

- Direct filtration – In this process, coagulation and flocculation are provided prior to filtration. It does not include sedimentation or clarification. Some direct filtration plants employ a contact adsorption clarifier (see Section 11.2.3).

- In-line filtration - In this process, only coagulation and rapid mixing is provided prior to filtration. With such limited pretreatment, in-line filtration should only be used to treat sources with low turbidity and low concentrations of natural organic matter.

The design of rapid rate filtration is covered in detail in numerous other texts and standards including:

- AWWA Standard B100: Standard for Granular Filter Media (AWWA 2016)

- Water Treatment Plant Design: Ch. 9. – High-Rate Granular Media Filtration (AWWA/ASCE 2012b)
- Recommended Standards for Water Works – Section 4.3.1 Rapid Rate Gravity Filters (Ten State Standards 2012)
- Integrated Design and Operation of Water Treatment Facilities (Kawamura 2000)

These references and other resources should be reviewed as part of the evaluation of rapid rate filtration for any source.

Diatomaceous Earth (DE) Filtration – In this filtration process, a pre-coat of DE is established on a mesh screen or fabric, called a septum, prior to initiating the filtration process. Once the filter septa are coated with about 1/8-inch of DE, water can be treated. During each filter run, a small amount of DE is continuously fed in order to maintain the porosity of the filter cake that accumulates on the septa. DE filtration is affected very little by temperature though accumulates headloss rapidly if there is a significant particle load. For these reasons, DE filtration may be a good choice for clear, cold source waters. However, DE is more mechanically challenging and requires greater operator oversight than some other filtration technologies.

The design of DE filtration is covered in detail in numerous other texts and standards including:

- AWWA Manual M30 Precoat Filtration (AWWA 1999)
- AWWA B101: Standard for Precoat Filter Media (AWWA 2016)
- Water Treatment Plant Design: Ch. 10. – Slow Sand and Diatomaceous Earth Filtration (AWWA/ASCE 2012c)
- Recommended Standards for Water Works – Section 4.3.3 Diatomaceous Earth Filtration, (Ten State Standards 2012)

Slow Sand Filtration – This process, as the name implies, involves the filtering of water through biologically active sand at a maximum rate of 0.1 gpm/ft^2, which is 10 to 60 times slower than the filtration rate for most rapid rate filters. As a result, slow sand filters require more surface area and space. However, the process is much simpler. Most installations utilize no chemical addition beyond those required for disinfection.

The filter media size used in slow sand filters is smaller than that used in rapid rate filters, and the filtration process can take a long time to ripen when treating cold, low nutrient source waters. Assessing proper ripening of the filter, or development of an effective s*chmutzdecke*, on top of the filter is a key part of the design process. High turbidity, nutrient-rich source waters can cause headloss to rapidly develop leading to short filter runs. Extensive pilot testing is necessary to determine the suitability of available sand, optimal filtration rate, filter run length, necessary ripening duration, and prefiltration requirements.

Appendix H includes additional information about the design of slow sand filtration. The design of slow sand filtration is covered in detail in numerous texts and standards, including:

- Manual of Design for Slow Sand Filtration (Hendricks et al. 1991)
- Slow Sand Filtration (Huisman and Wood 1974)

- Slow Sand Filtration for Community Water Supply: Planning, Design, Construction, Operation and Maintenance (Visscher et al. 1987)
- Water Treatment Plant Design: Ch. 10. – Slow Sand and Diatomaceous Earth Filtration (AWWA/ASCE 2012)
- Recommended Standards for Water Works – Section 4.3.4 Slow Sand Filters (Ten State Standards, 2012)

Membrane Filtration – Low pressure membrane filtration, commonly referred to as microfiltration and ultrafiltration, is very effective at removing pathogenic protozoa such as *Giardia* and *Cryptosporidium* from source waters. Often, a coagulant is used to decrease fouling of the membrane, extend the periods between cleaning processes, and provide some removal of color due to natural organic matter. Natural organic matter can lead to the undesirable formation of disinfection byproducts and exerts a disinfection demand.

Membrane filtration systems are proprietary, with fiber and module designs differing significantly between manufacturers. For these reasons, pilot testing conducted on one type of membrane is not transferrable to another. As proprietary systems, membranes are granted pathogen removal credit on a case-by-case basis (WAC 2465-290-660(e)). Also, as proprietary equipment, there is some financial risk to the water system if the membrane manufacturer goes out of business or stops manufacturing membrane system components. See Publication 331-XXX for a list of currently approved membrane filtration systems.

The design of membrane filtration is covered in detail in numerous other texts and standards including:

- Membrane Filtration Guidance Manual (USEPA 2005)
- AWWA B110: Standard for Membrane Systems (AWWA 2009)
- AWWA Manual M53 Microfiltration and Ultrafiltration Membranes for Drinking Water (AWWA 2016)

Bag and Cartridge Filtration – These filtration technologies are usually used to treat sources with very low turbidity, and low color since the filters cannot be backwashed and do not remove any organic matter. To decrease the frequency that filters need to be replaced, the final filter unit is usually preceded by one or more pre-filtration steps. A common approach is to use multiple pre-filters in series, such as 50 *u*m, 10 *u*m, and 5 *u*m nominal size cartridge filters, upstream of the compliance filter.

Bag and cartridge filters used for pathogen removal credit are proprietary equipment, similar to membrane filtration systems. As such, they are granted pathogen removal credit on a case-by-case basis (WAC 246-290-660(e)). The pathogen removal credit is specific to a bag and housing combination. Also, as proprietary equipment, there is some financial risk to the water system if the manufacturer of the filter system goes out of business, stops manufacturing the filters, or otherwise no longer supports the technology. See Publication 331-XXX for a list of currently approved bag and cartridge filtration systems.

11.2.5 Disinfection

Disinfection treatment approaches differ depending on the intended purpose of the application. Water systems **must** treat surface water and GWI sources to inactivate protozoa and viruses (WAC 246-290-601(1)). See Section 10.2.1 for general information on preliminary design considerations for disinfection processes.

For surface water and GWI sources, disinfection combined with filtration **must** provide at least 3-log (99.9 percent) removal or inactivation of *Giardia lamblia*, and at least 4-log (99.99 percent) removal or inactivation of viruses (WAC 246-290-662(1)). Filtration credit for removal of *Giardia lamblia* cysts and viruses establishes the minimum disinfection inactivation requirement. Regardless of the removal credit granted for filtration, water systems **must** provide at least 0.5-log (68 percent) inactivation of *Giardia lamblia* cysts and 2-log (99 percent) inactivation of viruses (WAC 246-290-662(1)(c)) through continuous disinfection. This disinfection process is often referred to as primary disinfection.

Disinfection is also required to maintain a distribution residual throughout the distribution system (WAC 246-290-662(6). This maintenance of a distribution system residual is often called secondary disinfection.

If ultraviolet disinfection is being considered as a primary disinfectant, the design engineer should review the information in [Policy F.13] and Appendix I.

11.2.5.1 Determination of Disinfection Efficacy

The effectiveness of disinfection is dependent upon multiple factors. The two main factors under control of the operator are the concentration of the disinfectant (C) and contact time (T), which is proportional to flow. The required CT is, in turn, dependent primarily upon the water temperature and pH. The ratio of $CT_{credited}$ to $CT_{required}$ is called the inactivation ratio (IR) and should always be more than 1.0, and preferably greater. An IR less than 1.0 on more than one day in a month is a treatment technique violation. To account for unexpected conditions and provide a factor of safety we recommend designing the disinfection process, including process control elements, to provide a minimum IR of 1.2 to 1.5.

The design engineer should clearly identify how daily $CT_{credited}$ values will be determined. At a basic level, $T_{credited}$ through a structure - clearwell, reservoir, or pipe – can be defined by Equation 11-1. Where there are multiple different segments, the $CT_{credited}$ for each segment needs to be monitored, calculated separately, and added together.

Equation 11-1:

$$T_{credited} = BF\left(\frac{V}{Q}\right)$$

Where:

BF = the baffling factor (BF) of the structure, sometimes referred to as T_{10}/T.
V = the volume of water in the structure.
Q = the flowrate of water through the structure.

The baffling factor is the ratio between the time for 10 percent of the water to flow through a structure divided by the mean residence time in the structure. The greater degree of baffling the higher the baffling factor. A baffling factor of no more than 0.1 should be applied to structures with separate inlets and outlets and no internal baffling.

For other structures, a conservative estimate based upon guidance documents can be used (Crozes et al. 1999). However, do not use the empirical or estimated baffling factors outlined in Surface Water Treatment Rule related guidance as the baffling factors of poor, average and superior are poorly defined and not reflective of typical designs (USEPA 1990; WSDOH 1995; AWWA/ASCE 2012d).

Use caution when evaluating contact basins where the depth is significantly greater than the width. In these cases, significant short circuiting can occur without proper design of the inlet, outlet and baffling structures.

For pipes, a length-to-diameter ratio of at least 160 is needed to achieve a baffling factor of 1.0 provided that the flow through the pipe is turbulent and not laminar; and pipe segments that have a length-to-diameter ratio at least 40, a baffling factor of 0.7 can be used (CDPHE 2014). For shorter pipe segments, the baffling factor can be estimated using the length-to-width ratio and information in *Improving Clearwell Design for CT Compliance* (Crozes et al. 1999).

Some other important design considerations include:

- **Flow monitoring –** To provide accurate calculations of contact time, flow monitoring is necessary leaving each contact chamber. When parallel clearwells are used, each side of the clearwell should have flow monitoring. Unequal flow splits occur with parallel clearwells, which leads to short-circuiting even when valves and gates are provided to try to develop an even flow split.

- **Level monitoring –** The most robust design is where the volume does not fluctuate. This could be accomplished by installing a fixed weir at the outlet to the contact basin and by separating distribution storage from the disinfection contact basin. Where such separation and installation of a weir are not possible, the design engineer should specify a minimum level in the contact basin and provide controls which will permanently maintain this minimum level setting.

- **Water quality parameter monitoring –** Disinfectant residual pH, and temperature, **must** be provided at the outlet for any CT segment (WAC 246-290-664(4)). In addition, disinfectant residual monitoring should be provide at the inlet to any CT structure to detect abnormally low or high chlorine residual concentrations that could raise public health or consumer concerns.

Once construction is complete, a tracer study may be required to confirm an empirically estimated baffling factor. While a tracer study is simple in theory, there are many complexities and nuances to a well conducted tracer study. For this reason, a tracer study plan **must** be submitted for review and approval prior to starting the tracer study itself (WAC 246-290-636(5)). Additional guidance on conducting tracer studies is in Appendix B.4.

11.2.5.2 Disinfection Profiling and Benchmarking

A disinfection profile is a graphical plot of a system's level of *Giardia lamblia* or virus inactivation measured over a one to three year period. Disinfection profiles are useful tools that can identify erratic operation of the disinfection process. They can also show when a system is adding more disinfectant than needed, either seasonally or year-round. Adding more disinfectant may increase disinfection byproduct (DBP) formation. A disinfection benchmark is the lowest monthly average pathogen inactivation ratio during the disinfection profile time period. The benchmark is used to evaluate the impact of a change in treatment.

When treatment plants are modified, there can be changes to the disinfection practices and DBP formation. Therefore, water systems with surface water or GWI sources **must** develop a disinfection profile if they are classified as a community or non-transient non-community system and have elevated levels of DBPs; however, we recommend that all surface water systems develop one. In this context, “elevated” means that either 1) the annual average TTHM level is 0.064 mg/L or greater, or 2) the annual average HAA5 level is 0.048 or greater (40 CFR 141.530 and 141.172).

When water systems required to develop a profile propose any change to their disinfection process, they **must** include the profile and calculated disinfection benchmark in the project report and include an analysis of how the proposed change will affect the current level of disinfection (WAC 246-290-630(4) and (12)). Examples of disinfection changes that trigger this requirement include: moving the point of disinfection; changing the disinfectant(s) used; increasing pH; and altering contact basin geometry, inlet/outlet piping, or baffling conditions.

11.3 Predesign Studies

Predesign studies, including pilot studies as appropriate, are **required** for proposed treatment projects (WAC 246-290-250). Refer to Chapter 10 and Figure 2-3. The goal of the predesign study is to establish the most effective treatment approach, considering life-cycle costs, to produce treated water that meets all regulatory requirements. A predesign study should precede the project report. As such, the information from the predesign study **must** be included in the project report (WAC 246-290-250). Pre-design study approaches include desktop, bench-scale, and pilot studies, which are described in more detail in Section 10.3. For surface water projects, desktop and bench-scale studies are often used to assess various treatment options before conducting a more thorough and detailed analysis of a treatment approach through a pilot study.

11.3.1 Pilot Studies

Pilot studies attempt to replicate as closely as possible the operating conditions and treatment results expected at full scale. Pilot plants are scaled-down versions of a proposed process, and may be skid or trailer mounted. Engineers use pilot plant testing to ensure treatment is effective, determine final design parameters, and estimate construction and operation costs.

Pilot studies are generally **required** for proposed treatment projects (WAC 246-290-250 and WAC 246-290-676(3)). For surface water treatment, the limited situations where a pilot study may not be needed include:

- Construction of a new water treatment plant that replaces an existing one using essentially identical treatment processes and design criteria.
- Identical treatment processes applied to nearly identical source waters, such as using the same membrane filtration equipment, including pretreatment on withdrawals with practically identical water quality.

Since equipment for proprietary processes is usually so specialized that pilot testing results are unique to a specific equipment design, it is usually impractical to transfer pilot results from one proprietary design to another. For example, low-pressure membrane filtration systems have different fiber sizes, materials, packing densities and other factors that affect system performance and limit transferability of design criteria to another membrane system.

11.3.2 Pilot Study Duration

Pilot studies should be of long enough duration to demonstrate the effectiveness, stability, and reliability of the proposed treatment system. Pilot testing of surface water treatment **must** capture seasonal changes in water quality, such as fluctuations in source water alkalinity, temperature, pH, color, turbidity, tastes, odors, and organic matter (WAC 246-290-676(3)). The testing should include the period of most challenging water quality for the piloted treatment technology.

The number of samples collected and study duration can vary widely depending on the type of source, amount of historical data, water quality, and the proposed treatment technology (Logsdon et al. 1996; Ford et al. 2001; AWWA 2012c). In some cases, design engineers can use bench-scale testing to determine the initial operational parameters for pilot testing and possibly decrease the duration of the pilot study. See Table 11-3 for guidance on the duration and objectives of pilot studies for a variety of surface water treatment processes.

11.3.3 Pilot Study Plan and Report

Review the section on pilot studies in Chapter 10. Sections 10.3.3 and 10.3.4 discuss recommended scope and content of a pilot study plan and final pilot study report.

11.4 Project Reports

Sometimes referred to as a basis of design report, a project report should define the size, scope, and design parameters for a proposed treatment project. The design engineer should seek project report approval before submitting construction documents for all surface water projects. The purveyor **must** obtain ODW approval of a project report before modifying or expanding existing treatment facilities and before beginning construction of new treatment facilities (WAC 246-290-110).

Table 11-3
Surface Water Filtration Pilot Study Duration and Objectives

Treatment	Minimum Recommended Duration[1]	Typical Objectives	References
Rapid Rate Filtration	6-12 months[2]	Coagulant dose(s), polymer dose(s), sufficient alkalinity, sedimentation rate, hydraulic loading rate, backwash parameters, disinfection byproduct (DBP) precursor removal, finished water quality.	Kawamura 2000a; Logsdon et al. 1996
Slow Sand Filtration	12 months	Pretreatment requirements, ripening period, run length, filter loading rate, sand type, finished water quality.	Hendricks et al. 1991
Diatomaceous Earth (DE) Filtration	1-4 months	Pretreatment requirements, pre-coat rate, filter media grade, screen size, body feed rate, run length, finished water quality.	AWWA 1999
Bag and Cartridge Filtration	2-6 weeks	Pretreatment requirements, replacement frequency, finished water quality.	USEPA 2003
Membrane Filtration	4-7 months	Pretreatment requirements, flux rate and stability, back flush parameters, chemical dose(s), cleaning frequency, fiber breakage, DBP precursor removal, finished water quality. To determine which membrane manufacturer is best suited for source water	Freeman et al. 2006; USEPA 2005

Notes:

3. Where a range of duration is provided, the design engineer should justify anything less than the maximum duration listed.
4. Engineers can consider a series of multiple week pilot studies to cover the expected seasonal variation in water quality instead of operating a full-time pilot plant.

The design engineer should await approval of the project report before submitting 100 percent complete construction documents. Project reports for treatment facilities should reference all planning, design, and applicable pilot study reports for the proposed facility. They **must** include:

- Detailed design criteria and calculations
- Process control information
- Proposed methods and schedules for start-up, testing, and operating the completed treatment facility
- Operational complexity and staffing
- Operator training and certification requirements
- Reliability

Since surface water treatment projects are often more complex than other types of projects, multiple ODW staff participate in review. The design engineer should submit to ODW at least three copies of the project report. See Chapter 2 for additional guidance on preparing project reports.

11.4.1 Design Criteria

Design criteria are a key element of the project report. They define the specific treatment objectives, basis for sizing equipment, and operational requirements. As such, project reports **must** include design criteria for all major treatment-facility project elements (WAC 246-290-110(4)(h)). See Section 10.4.1 for additional guidance on project design criteria and calculations and Section 11.2 for basic information on pretreatment, clarification, filtration and disinfection processes.

11.4.2 Process Control - Monitoring, Instrumentation and Alarms

Process control tools - including monitoring, instrumentation, and alarms - help ensure that treatment processes are safe and reliable. In addition to meeting regulatory monitoring requirements, these tools allow operators to adjust the treatment process and alert staff when a process may not be functioning properly. See Section 10.4.2 in the previous chapter for basic design guidance on monitoring, instrumentation and alarms. For surface water treatment facilities, the water quality monitoring needed varies depending upon the type of surface water treatment process. These needs are summarized in Table 11-4.

Some special aspects of process control for surface water treatment include:

- **Streaming Current Monitors or Zeta Potential Meters –** These types of instruments are used for coagulation control. For on-line instruments, there should be about a 1- to 3-minute lag time between coagulant addition and when the sample reaches the sensor (AWWA 2011).

Table 11-4
Surface Water Treatment: Water Quality Instrumentation

	Water Quality Instrumentation for Specific Treatment Processes						
Treatment Process	**Streaming Current/Zeta Potential**	**Turbidity**	**Particle Counts**	**pH Analyzers**	**Temperature**	**Disinfectant Residual**	**UV Intensity/ Absorbance**
Clarification - General		R					
Disinfection - Chemical				E	E	E	
Disinfection - Ultraviolet		R					E
Filtration - Bag and Cartridge		E					
Filtration - DE		E					
Filtration - Membrane	O	E	E[1]				O
Filtration - Rapid Rate	E	E	E/R[2]				O
Filtration - Slow Sand		E					

E – Essential, either a regulatory requirement or necessary for process control.
R – Recommended as part of good design practice.
O – Optional. Not typically used, but could be considered for process control.
Notes:

1. Either a particle counter or laser turbidimeter is required to indirectly assess membrane integrity.
2. In most cases particle counters are a useful tool in assessing particle breakthrough, and in some cases, particle counters are required as part of the design approval.

- **Jar Testing –** Jar testing can be a useful tool for operators to evaluate modifications to treatment processes and chemical dosages. The design engineer should provide the initial jar test settings that reflect plant operations as part of the design process. This should include the mixing speed, mixing times, and chemical injection sequences.

- **Turbidimeters and Particle Counters –** Sample lines to these online instruments should be kept short to keep the delay between sample collection and the instrument itself to one minute or less. In addition, for turbidimeters, bench-top equipment **must** be provided so that operators can perform weekly verification checks (WAC 246-290-638(4)). You can find useful information about setting up these types of instruments in *Turbidity Monitoring and Meter Setup* (DOH 331-538) and *Plumbing Sample to a Process Analyzer* in Appendix F.9.

- **Turbidity Data Recording and SCADA for rapid rate filtration plants -** Turbidity data recording is usually done on the SCADA computer, and the file is referred to as the data log. The data logs in a SCADA system must have the capability of handling long-term turbidity data storage needs. Turbidity from each individual filter effluent (IFE) must be monitored continuously and recorded at least every 15 minutes. Turbidity from the combined filter effluent (CFE), must be recorded every 4 hours and the maximum value of the continuous measurements must also be recorded. To support plant operators in achieving turbidity optimization goals, maximum IFE and CFE values within a 15 minute period, or data capture at intervals of 1 minute or less should be recorded. The recorded IFE data must be stored for at least 3 years and the CFE data must be stored for at least 5 years.

B.

The data log should be created daily in an easily accessible format (e.g. csv, xlsx), including date, time and turbidity value for each continuous reading turbidimeter. Logged turbidity data should be tagged to identify the plant operating conditions (i.e. filter-to-clearwell, filter-to-waste, backwash, out of service). The data log files should be located in a directory easily accessible by the plant operators. The system should include automatic routine backups both onsite and offsite. Data log storage devices should have surge protection to mitigate power failures. The SCADA monitoring system should be programmed to allow operators to create their own trend lines using a flexible turbidity scale and a flexible time scale. It should also allow operators flexibility to create plant-specific control screens showing selected trend lines (e.g. selected filter IFE turbidity, filter flow rate and valve open-closed positions for the selected filter in the same view).

11.4.3 Start-up and Testing

A well-planned start-up of any treatment process, including initial testing, is important to ensure the treated water is protective of public health and the process is safe and reliable. Treatment facility submittals **must** include proposed methods and schedules for start-up and testing (WAC 246-290-110(4)(h) and 100(4)). See Section 10.4.3 for additional information on start-up and testing and Section 11.6 for guidance on placing a surface water treatment plant into service

11.4.4 Treatment Plant Operations and Staffing

Project reports for treatment facilities **must** also address operation of the completed project including staffing needs (WAC 246-290-110(4)(h)). The design engineer should identify the organization or people responsible for operating the proposed facility and their required qualifications.

The level of operator certification is dependent upon the population served, treatment capacity, source water quality, and complexity of the treatment process. The design engineer should complete a preliminary "Purification Plant Rating Criteria Worksheet" as part of the project report to help identify the correct treatment plant classification. A list of certified operators available to provide required services is available on our operator certification web page.

Staffing needs for a surface water treatment plant are dependent upon the treatment technology, location, degree of automation, water quality, and capacity of the treatment facility.

- **Treatment Technology** – Rapid rate filtration and to a lesser extent diatomaceous earth (DE) filtration are sensitive to changes in raw water quality and are dependent upon precise chemical feed processes to function effectively. A significant shift in water quality or even a short interruption in chemical feed can cause the filtration process to suddenly perform poorly and place public health at risk. For this reason, rapid rate and DE treatment plants need to be staffed continuously if there is limited or no automation of the treatment process. These filtration processes are also more staff intensive, usually requiring significantly more operational oversight than slow sand filtration, bag filters, or membrane filtration. Basic report forms for these filtration technologies are available on the forms page of our website, which may be useful to review in assessing staffing needs.

- **Location –** For small plants, especially those located in remote locations, it may be difficult to attract and retain highly certified operators. For these locations, simpler technologies less prone to failure are more appropriate than complex treatment processes requiring a higher level of operator certification.

- **Automation** - A well-trained operator is essential to ensure that the health of consumers is protected. Automation can be useful in improving process control and ensuring smooth operations when treatment plants are unattended, but cannot replace a well-trained professional to oversee operations and take action in the event of an equipment failure or process upset.

- **Water Quality –** Some sources, such as rivers or lakes prone to algae blooms, may experience rapid changes in water quality that can affect treatment processes. The vulnerability of the source to such changes should be considered in assessing staffing needs.

- **Treatment Capacity –** Larger treatment facilities have more equipment and instrumentation than smaller treatment facilities, so more staffing time is necessary just to keep the facility in good working condition. In addition, the consequences of failure for larger treatment facilities are more significant. For these reasons, most treatment plants greater than 10 MGD provide continuous operational oversight (USEPA 2002).

The design engineer should consult with ODW early in the design process to determine the conditions under which remote monitoring may be appropriate. Table 11-5 was developed from reviewing staffing of existing WTPs and other references, and can be used as a general guide in developing staffing plans for proposed facilities (USEPA 2002; Ohio EPA 2016; Florida DOH 2013).

Table 11-5
Surface Water Filtration: Minimum Recommended Staffing

Filtration Process	Recommended Minimum Staffing per Day		
	Maximum Design Rate		
	<2 MGD	2-12 MGD	>12 MGD
Bag and Cartridge Filtration	1 to 2 hours		
DE Filtration	1 to 2 hours	2 to 4 hours	
Membrane Filtration	1 to 2 hours	2 to 6 hours	6 to 10 hours

Rapid Rate Filtration	4 to 8 hours[1]	8 to 12 hours[1]	At all times
Slow Sand Filtration	1 to 2 hours	2 to 4 hour	3 to 6 hours

Notes:

1. If the recommended process control noted in Table 11-6 and automatic shutdown are not provided, rapid rate WTPs should be staffed at all time they are in operation.

Given the high public health risk associated with inadequately treated surface water, all surface water treatment plants should be visited every day they are in operation including weekends and holidays. Except for rapid rate filtration facilities, this daily visit can be brief, simply to make sure that all the equipment is functioning properly, and collect any required water quality samples. In limited cases, water systems using slow sand, membrane, diatomaceous earth, or bag or cartridge filtration employing comprehensive automation and surveillance capability may substitute some of the on-site visits with remote monitoring.

For even the smallest and simplest surface water treatment plant, there should be at least two trained and appropriately certified operators so that public health is not placed at risk should an operator become ill, need to attend to family matters, take scheduled leave, or participate in required training and professional development opportunities. The design engineer and water system should develop a staffing plan based upon these and other criteria relevant to the water system, and include it in the project report.

11.4.5 Process Reliability

A high degree of reliability is especially important for surface water treatment facilities since a treatment process failure can present an acute health risk. Most surface water treatment plants include some automation, which can improve the process reliability. The extent of automation varies depending upon the type of treatment process employed.

Analysis of automation should identify the response to these reliability and process control issues including:

- Variations in source water quality – high or sudden increases in turbidity or color, or decreases in alkalinity, associated with extreme precipitation events, algal blooms, chemical spills in source water, and other factors that could result in risks to public health.

- Chemical feed issues – Loss of chemical feed or overfeed.

- Equipment failures – Malfunctioning valves, pumps and process control equipment.

- Finished water quality – high turbidity, inadequate disinfection and other changes in water quality that are indicative of process upsets.

- Waste handling and disposal – Provisions for waste handling should be considered so that waste disposal does not impose restrictions on the main treatment process. See Section 10.8 for special considerations if the design includes recycling any of the backwash waste stream.

A basic overview of recommended process control by treatment process is provided in Table 11-6, with more extensive guidance provided in standard professional references.

Table 11-6
Surface Water Treatment: Recommended Process Control

Treatment Process	Recommended Process Control
Disinfection - Chemical	Continuous flow, pH and disinfection residual monitoring with alarms and shutdown conditions clearly identified. If a minimum water level in the clearwell or reservoir is not maintained via a weir, there should be alarms for critical water levels.
Disinfection - Ultraviolet	Continuous monitoring of the parameters used to calculate UV efficacy (flow, UV intensity, and UV absorbance) with alarms and shutdown conditions clearly identified. See Policy F.13.
Sedimentation/Clarification	Continuous monitoring of turbidity from basin or clarifier.
Filtration - Bag and Cartridge[1]	Continuous monitoring of turbidity from each filter and the combined filter effluent. Continuous flow and differential pressure measurements.
Filtration – DE[2]	Continuous monitoring of turbidity from each filter and the combined filter effluent. Continuous flow and differential pressure measurements. Uninterruptible power supply for recirculation pumps.
Filtration – Membrane[2]	Continuous monitoring of turbidity from the combined filter effluent. Daily direct integrity monitoring. Continuous indirect integrity monitoring of each membrane filtration unit. Continuous flow and differential pressure measurements for membrane filtration unit. Flow-paced chemical feed systems if coagulant control is provided.
Filtration - Rapid Rate[2]	Continuous monitoring of turbidity from each filter and the combined filter effluent. Flow-paced chemical feed systems and streaming current-adjusted coagulant feed control. Continuous flow and headloss measurements for filter. Filters should automatically backwash if headloss or turbidity set-points are reached. Continuous monitoring of clearwell or reservoir levels if used for backwash supply.
Filtration - Slow Sand[1]	Continuous monitoring of turbidity from each filter and combined filter effluent. Continuous monitoring of flow and headloss for each filter.
Backwash Recycle	Continuous monitoring of flow and turbidity for the settled water that is recycled. Continuous monitoring of water level in the settling process with alarms if the tank level gets too low.

Notes:

1. Recommended monitoring parameters may not be warranted or feasible for all installations.
2. If the recommended automation is not provided for these processes, the treatment needs to be actively monitored by an appropriately certified operator at all times.

There are certain process reliability features that **must** be included in the design of any new filtration facility (WAC 246-290-678). In general, these reliability features include:

- **Alarms –** Alarms play a critical role in process control, especially when surface water treatment facilities operate without staff present. Therefore, project reports **must** describe proposed alarms and their settings (WAC 246-290-110(h)). Critical alarms include those for coagulation, filtration, and disinfection. See Section 10.4.2 for additional guidance.
- **Standby equipment –** such as pumps, chemical feed equipment, and mixers to assure continuous operation and control of coagulation, clarification, filtration and disinfection processes.

- **Redundant treatment units –** Redundant units include those for filtration so that treatment can continue to be provided when filters are out of service for operational reasons such as ripening, backwash, or maintenance. In addition, redundant disinfection units are required for sources that meet the criteria to remain unfiltered. See Policy F.10.

Another important aspect of process reliability is a reliable power supply. For most treatment plants, standby generators are recommended to maintain operations and life-safety equipment during power outages. In addition, it may be necessary to install uninterruptible power supplies on some critical process components such as key chemical feed pumps, instrumentation, SCADA systems, and recirculation pumps for DE filtration to avoid significant process upsets.

Reliability guidelines for surface water facilities include those in the "Policy Statement on Automated/Unattended Operation of Surface Water Treatment Plants" in the *Recommended Standards for Water Works* (Ten State Standards 2012). According to this policy statement, in their project reports, engineers should:

- Identify all critical features in the treatment facility that will be monitored electronically. Describe automatic plant shutdown controls with alarms and conditions that would trigger shutdowns. Dual or secondary alarms may be necessary for certain critical functions.
- Provide automated monitoring of all critical functions with major and minor alarm features. Automated plant shutdown for all major alarms. Inability to automatically startup the plants following a major alarm. Built-in control test capability to verify the status of all major and minor alarms.
- Discuss the ability to operate all treatment plant equipment and process functions manually through the control system.
- Outline plans to challenge test each critical component.

Additional information on treatment-process reliability is in other published design references (AWWA/ASCE 2012f; Ten State Standards 2012).

11.5 Operations Program

Surface water treatment involves multiple treatment processes that if not properly operated and maintained can result in delivery of unsafe water. An Operations Program is an indispensable tool supporting plant operations staff in achieving optimized treatment.

Creating an Operations Program will inform the design and initial plant start-up, establish routine and contingent activities operators must perform, and create the framework upon which facility performance will be judged. New operators will rely on the treatment plant Operations Program document for support in quickly gaining an understanding of the procedures and decisions necessary to achieve optimized treatment plant performance. The design engineer **must** prepare a detailed Operations Program for a water treatment facility treating a surface water or GWI source (WAC 246-290-654(5)).

The purpose of the Operations Program is to help water system personnel reliably produce optimally filtered water quality. As such, it should identify specific, quantifiable optimization goals. Engineers can use the following to develop treatment optimization goals:

- ODW Treatment Optimization Program
- EPA Composite Correction Program
- AWWA-EPA Partnership for Safe Water
- AWWA Standard G100: Water Treatment Plant Operation and Management

At a minimum, the Operations Program **must** describe:

- Coagulation control procedures (when a coagulant is used)
- Procedures used to determine chemical dosages and feed rates
- Operations and maintenance for each unit process, including overall goals and specific water quality targets
- Treatment plant performance monitoring. Monthly operational report forms for each type of filtration technology are available on our website under surface water forms.
- Laboratory procedures
- Reliability features
- Data validation procedures
- Emergency response plans, especially for treatment process failures and watershed emergencies

The following items should also be included:

- An overall schematic of the treatment process
- Process and instrumentation diagrams for the treatment facility
- Control loop descriptions

- Processes for verifying and calibrating instrumentation such as turbidimeters, particle counters, pH meters, and chlorine residual analyzers
- Procedures for testing alarms and confirming communications with on-call operators and utility staff
- Procedures for completing required monthly operations reports
- Documenting circumstances under which a health advisory should be issued, who at the water system will decide to issue the health advisory, and how communication to customers will be provided
- Procedures for other routine or ongoing monitoring that are not already included in the regulatory monitoring but may be used for process control

The engineer or water system **must** submit the draft Operations Program for new, expanded, or modified treatment facilities (WAC 246-290-654(5)). Often, not all of the information needed to complete the Operations Program for these facilities is available until after construction starts. Usually a draft Operations Program is submitted for review and comment during construction. This process allows a revised final draft to be available prior to startup along with the construction completion report

When developing the Operations Program, the design engineer should remember that water system staff will need to periodically update and modify it to reflect their current water treatment practices. As such, the water treatment plant operators and managers should be heavily involved in developing the initial Operations Program, and be provided the capacity to readily update it.

The project report should initially specify applicable operator training requirements, specific training the equipment supplier(s) will provide, and related schedules. The final Operations Program and equipment-specific operations and maintenance manuals should be available during the operator training sessions.

11.6 Placing a Surface Water Treatment Plant into Service

Given the importance of reliable and effective treatment to the protection of public health, a thorough commissioning process is warranted. Before a surface water treatment plant can be placed into service, it **must** be properly tested, inspected, and disinfected (WAC 246-290-120(4)). A licensed engineer **must** complete a *Construction Completion Report Form* (DOH 331-121) and submit it to ODW before a water system uses treatment facilities to serve water to the public (WAC 246-290-120(5)).

This section complements the more general information about placing a water treatment plant into service in Section 10.9. These more general topics include:

- Prestart-up meetings
- Start-up of other plant components
- Testing of process instrumentation

- Potential for impacts on the distribution system (sediment release and corrosion control).

Monthly operational report forms for each type of filtration technology are available on our website under surface water forms. Water system operating staff should review these forms before final commissioning. This will allow operating staff to work with their DOH Regional Engineer on adapting the forms specific to the treatment facility, integrating them with process control, SCADA, and operational practices. The operator in responsible charge should plan to submit a completed set of operational reporting forms to ODW, addressing any reporting issues raised by ODW staff, before the facility begins to serve water to consumers.

Each set of operational reporting forms for a surface water treatment plant includes one form for disinfection. The process for calculating the $CT_{credited}$, including determination of volume(s), flow(s), and baffling factor(s), should be established in the project report as outlined in Section 11.2.4. The design engineer should confirm the process for calculating the $CT_{credited}$ prior to commissioning, as the initial design assumptions could have changed in the design process. The operator should plan to submit sample or example disinfection monitoring results to ODW prior to serving water to the public.

Planning for disposal of treated water is an important part of the start-up due to the large volume of water that may need to be disposed of prior to serving water to customers. To address this issue, it may be necessary to operate the water treatment plant intermittently.

Table 11-7 summarizes the final commissioning tasks that should be completed prior to serving water to customers. These comprehensive commissioning tasks usually follow thorough testing of individual treatment plant components, as well as disinfection and bacteriological testing. This type of testing is covered in Section 10.9, and in other chapters on individual plant components such as testing of pumps and treated water storage facilities.

Table 11-7
Surface Water Treatment Technologies: Start-up and Testing

Treatment	Minimum Recommended Duration[1]	Final Commissioning Tasks
All Types of Treatment	NA	Confirm instrumentation and process control working correctly; test alarms. Compare instrumentation output with readings in SCADA. Complete applicable portions of monthly operational reports and submit. Check finished water quality.
Rapid Rate Filtration	5 days	Assess backwash process, settings and filter-to-waste. Complete at least two filter runs including backwash and filter-to-waste cycles.
Slow Sand Filtration	3 months	Allow filters to fully ripen. Coliform or other biological testing.
Diatomaceous Earth (DE) Filtration	2 days	Complete at least two filter cycles (pre-coat, body feed, DE removal) to ensure all systems work.

Bag and Cartridge Filtration	8 hours	Confirm instrumentation working correctly, test alarms (if applicable).
Membrane Filtration	3 days[2]	Complete at least 16 hours of operational multiple filtration cycles, test maintenance cleaning process.
UV Disinfection	2 days	See Appendix I

Notes:
1. The design engineer and water system should justify anything less than the duration listed.
2. For membrane systems, there may be initially high particle counts or filtered water turbidity. If these conditions persist, direct integrity testing may be required every 4 hours until the indirect integrity monitoring provides reliable results.

For most surface water treatment facilities, there are a few tasks that are usually completed after the plant is fully operational and serving water to customers. These tasks include:

- **Field Data Sheet –** This document provides a summary of the processes and operational settings for the treatment plant. It should be developed with the DOH Regional Engineer who can provide an appropriate template for the treatment technology being installed.
- **Tracer Study** - It is usually necessary to make some assumptions about the operational levels and baffling efficiency for the disinfection process to initially calculate the CT_{credit} of the disinfection process. Often, it is necessary to conduct a tracer study after the disinfection facilities are constructed to confirm or revise the initial hydraulic assumptions. See Appendix B.4.

- **Disinfection Summary –** This document, sometimes referred to as a CT Summary, provides the basics of how disinfection effectiveness is determined for a treatment facility including any underlying assumptions.

- **Final Operations Program –** As noted in Section 11.5, the engineer or water system **must** submit an Operations Program for new, expanded, or modified treatment facilities (WAC 246-290-654(5)). A draft Operations Program should be submitted during construction of the facility and a final version submitted during or shortly after the commissioning process.

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——, (d) Chapter 11. Oxidation and Disinfection

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——, (f) Chapter 29. Design Reliability Features

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Appendix A: Forms, Policies, and Checklists

Appendix A.1 Forms

You can obtain ODW forms for drinking water projects by contacting the ODW regional offices listed in Table 1-1. The forms referenced in this manual are online at http://www.doh.wa.gov/CommunityandEnvironment/DrinkingWater/PublicationsandForms/Forms.aspx. For persons with disabilities, forms are available on request in other formats. To submit a request, call (800) 525-0127 (TTY 1-800-833-6388).

- ✓ **Project Approval Application Form**. The purpose of the *Project Approval Application* is to identify the project applicant, design engineer, water system, and type of project. ODW uses this information to determine the review and approval fees.

- ✓ **Water Right Self-Assessment Form**. The purpose of the *Water Rights Self-Assessment Form* (publication 331-370 or 331-372) is to identify the water right information necessary for ODW to review and approve construction documents and project reports. The *Water Right Self-Assessment Form* will be forwarded to the Department of Ecology for review.

  Water system plans, small water system management programs, and project reports for projects involving a new or expanding source or increased water system capacity must include a completed *Water Right Self-Assessment Form*.

- ✓ **Construction Completion Report Form**. The purpose of the Construction Completion Report (publication 331-121, 331-146, or 331-147, depending on the project type) is to document that the project has been constructed according to ODW-approved plans and specifications. This form must be completed and submitted to ODW within 60 days of completion and before use of any new or modified water system facility approved for construction by ODW. Distribution mains and other distribution-related facilities designed by a professional engineer, but not required to be submitted to ODW for approval under WAC 246-290-125, must all have a construction completion report on file with the water system.

Appendix A.2 Policies

ODW policies are available online at http://www.doh.wa.gov/CommunityandEnvironment/DrinkingWater/RegulationandCompliance/Policies.aspx.

Appendix A.3 Project Checklists

Checklists can help the design engineer determine if minimum design requirements are met. On the following pages, you will find various project submittal checklists that can be used to prepare submittals for review by ODW staff or third parties.

The purpose of the project checklist is to ensure a complete and properly organized project submittal to ODW. Incomplete submittals may be returned and will result in a delayed project review due to the time required to receive the missing information.

ODW has developed a project checklist covering each of the project types listed below:

1. General
2. Groundwater Source of Supply
3. Transmission and Distribution Mains
4. Hydraulic Analysis
5. Reservoirs
6. Booster Pump Stations
7. Pressure Tanks
8. Water Treatment Facilities
9. UV (see Appendix I for project checklist)

Appendix A.3.1 General Project Report Checklist

Include the following information in the project report, as applicable to the project and water system's planning status. Refer to Chapter 2, including the project development flowcharts therein, and WAC 246-290-110 and -120 for further design guidance and requirements.

☐ The signed and dated stamp of a Washington State-licensed Professional Engineer. Federal facilities can have a PE from any state, but still must have a PE stamp.

☐ Narrative discussion that establishes the need for the project. It should include a construction schedule for the recommended alternative, project cost, and method of financing. Also, indicate the relationship of the project to the water system plan either currently approved or in the process of being prepared or updated.

☐ Alternative analysis and rationale for selecting the proposed project. It should include an evaluation of life cycle costs, including initial capital costs and on-going operations and maintenance costs.

☐ Appropriate planning elements: either cite appropriate reference in an approved water system plan, prepare an amended water system plan, or included as part of the project report.

☐ Capacity analysis if seeking a change in the number of approved service connections. Include rationale and calculations to justify total number of service connections and equivalent residential units (ERUs). The analysis should identify the number of residential, industrial, commercial, and municipal connections presently served by the water system. If the water system seeks to increase its approved number of connections through construction of new facilities, document water system plan approval status.

☐ *Water Right Self-Assessment Form* must be completed for new sources and all projects that increase the approved number of connections.

☐ Hydraulic analysis that demonstrates the ability of the project to supply minimum pressure requirements during peak flows and fire events. The analysis should include a narrative discussion that describes the hydraulic analysis method, explains critical assumptions, and summarizes the effect of the proposed expansion on the existing water system.

☐ Measures to protect against vandalism.

☐ Disinfection procedures according to AWWA or APWA/WSDOT standards and a narrative discussion on how the project will be disinfected and tested prior to use.

☐ Provisions to discharge water to waste including description of how wastewater is disposed, and documentation that procedures are acceptable to the Department of Ecology and local authorities.

☐ Routine and preventive operations and maintenance tasks and their frequency, and the role of a certified operator in completing them.

Appendix A.3.2 Groundwater Source of Supply Checklist

Address these design elements in source of supply project report and construction document submittals. Refer to Chapter 5, WAC 246-290-130 and -135, and Appendix E for further design guidance and requirements. If the new groundwater source requires treatment, refer to Appendix A.3.8.

The following guidance also applies to sources serving existing, unapproved water systems. If the source is an existing well proposed to be converted into an "approved" water source, information requested by ODW that is not available (such as a missing well log) should be brought to the attention of the reviewing engineer.

For new surface water supplies, contact your regional engineer for further guidance. Applicable design references include Chapters 5, 10, and 11, and Appendix A.3.8.

Project Report

☐ Source of supply analysis that justifies the need for a new or expanded source of supply and the alternative source options evaluated.

☐ Water right permit or certificate issued by the Department of Ecology plus a completed *Water Right Self-Assessment Form*.

☐ Copies of legal documents (easements or covenants) for the sanitary control area (WAC 246-290-135). See DOH 331-453 and DOH 331-048.

☐ Water quality test results for each source, including:

- Bacteriological/coliform test (bacti/coli)
- Inorganic chemical and physical analysis (IOC)
- Volatile organic chemical (VOC) test
- Radionuclide test (only required for community water systems)
- Synthetic organic chemical (SOC) tests, unless demonstrated that source can meet ODW 's requirements for a monitoring waiver
- Results of any other tests required due to site-specific concerns

☐ Assess potential impacts of the new source of supply on water quality in the distribution system, especially with respect to corrosion and compliance with the Lead and Copper Rule (WAC 246-290-110(4)(d)).

☐ Assess adequacy of each reservoir overflow capacity to safely discharge total supply capacity to the reservoir, including sources, booster pump stations, and PRVs, to ensure the structural integrity of each reservoir in the event of control system failure.

☐ Well site inspection made by the ODW or local health jurisdiction.

☐ Susceptibility assessment, wellhead protection area (WHPA) delineation, and contaminant inventory within the WHPA per WAC 246-290-130 and -135. See DOH 331-274-F.

☐ Update the Wellhead Protection Plan (WHPP). See DOH 331-018 and 331-106.

☐ Well log including unique well identification tag number, surface seal, depth to open interval or top of screened interval, overall depth from well the top of the casing, and elevation of top of casing.

☐ ODW well pumping test results following procedures in Appendix E.

☐ Source pump control logic and pump cycle protection. Refer to Chapter 9 for pressure tank sizing requirements and Chapter 7 for appropriate pump control levels for reservoirs.

☐ Alarm conditions.

☐ Given the triggers for mandatory CT6 and 4-log virus inactivation treatment of groundwater sources (see WAC 246-290-451 and -453, respectively) after being placed into operation, we believe it is advantageous to assess and, given construction cost and other constraints, design and install facilities capable of providing 4-log virus inactivation treatment prior to the first connection at each new groundwater source. Submit such assessment. See Appendix F.1 for hypochlorination submittal outline.

☐ Report on the evaluation of a potential groundwater under the direct influence of surface water source. See WAC 246-290-135.

☐ Natural and geotechnical hazards analysis of the well site and well house building.

Construction Documents

☐ Site piping plans including:

- Source meter set according to manufacturer's minimum required upstream and downstream pipe configuration
- Valves (i.e., isolation, check, well pump control, air/vacuum, pressure relief). Note: show screen secured on each valve discharge outlet.
- Sample taps for raw and finished water
- Location, size, type and class of pipe

☐ Pumping equipment specifications including:

- Horsepower, GPM, head, pump controls, and alarm system.
- Specific pump curve being used and operation range of head and flow conditions clearly indicated on pump curve.
- Narrative discussion of ability of the source and pumping system to supply peak daily water volumes.

☐ Well construction details including general design and construction standards, casing specifications, general sealing requirements and material specifications, adequately sized and screened inverted well casing vent constructed to prevent entry of contaminants, and access port for measuring water level. See Policy M.01 for information on pitless units and well caps.

☐ Map of the site and vicinity drawn to scale, including well location (both township/range and latitude/longitude), pump house, water lines, site topography, sanitary protection area, and location of potential sources of contamination including septic systems, sanitary sewers, buildings, roads, and driveways.

☐ Well house details including security measures, casing and pump house slab elevations, ventilation, room for future disinfection equipment if and when it's needed (if not currently being designed), and electrical connections allowing the use of emergency power.

☐ Building equipment and instrument layout demonstrating adequate clearance to safely enter, operate, and maintain all well house components.

Appendix A.3.3 Transmission and Distribution Main Checklist

Address these design elements in transmission and distribution main project report and construction document submittals or reference this information in an approved water system plan. Refer to Chapter 6 and WAC 246-290-230 for further design guidance and requirements.

Project Report

☐ Water system sizing analysis documenting availability of adequate source and storage to serve the proposed service area.

☐ Hydraulic analysis used to size mains and to confirm required pressures can be maintained (see Checklist A.3.4 Hydraulic Analysis for additional details).

☐ Hydraulic transient analysis for transmission mains and distribution mains where warranted by high pressures or high velocities (see Checklist A.3.4 Hydraulic Analysis for additional details).

☐ Identification and description of proposed land use within area to be served by the new distribution system.

☐ Service area map identifying the properties to be served by the new distribution system.

☐ Review of soils in the proposed construction area. Assess corrosivity of soils along the proposed pipe alignment. A more detailed geotechnical analysis by a qualified professional may be needed in areas prone to landslides, liquefaction in an earthquake, or other geologic hazards.

☐ Assess water quality impacts associated with physically disturbing existing pipe tuberculation resulting from increased flow velocity and/or reversal of traditional flow direction (see Section 6.2.9).

☐ Distribution system map showing location of proposed water lines, pipe sizes, pipe material, pressure zone boundaries and hydraulic grade line elevation, easements, and location of control valves, hydrants, meters, and blow-off valves.

☐ Identification of cross connections and measures designed to eliminate or control such connections.

Construction Documents

☐ Specifications for:

- Pipe materials,
- Disinfection,
- Bacteriological testing,
- Pressure and leakage testing.

☐ Adequate separation from sewer mains, nonpotable conveyance systems, and other buried utilities.

☐ Construction drawings including:

- Plan views with a scale of no more than 100-feet to the inch.
- Easement locations and dimensions, if applicable.
- Profiles or crossing details with a vertical scale of no more than 10-feet to the inch for:
  - Areas where pipeline projects encounter utilities that cannot be easily located or that could conflict with the proposed pipeline, such as storm and sanitary sewers, gas mains, and telecommunications lines.
  - Pipelines proposed across a streambed.
- Location, size, and construction materials of all proposed pipelines in the project area. Show all hydrants, valves, vaults, sample stations, meters, blow-off valves, and other distribution system features.
- Typical construction details of:
  - All new pipeline tie-ins to existing pipelines.
  - Pipeline trench cross-section indicating bedding, backfill, and compaction requirements.
  - Installations of air and vacuum relief valves and vaults, pressure-reducing valves and vaults, backflow assemblies, fire hydrants, blow-offs, sampling stations, and other system appurtenances.
  - Thrust blocking or restraints.
  - Service connection details, where appropriate.
  - Corrosion mitigation measures, where appropriate.
- All other buried utilities, including storm and sanitary sewers, dry wells, telephone, natural gas, power and TV cable lines in the project area (existing or proposed concurrent with pipeline construction) to the extent possible, given existing available records.
- Construction drawings should note that all buried utilities are to be field located prior to construction. Notification of the One Call Center/811 is **required** at least two days before any excavation (RCW 19.122).

Appendix A.3.4 Hydraulic Analysis Checklist

A hydraulic analysis **must** be used to size and evaluate new, or expanding to existing, distribution systems. Refer to Chapter 6 and WAC 246-290-230 for additional design considerations. Address these design elements in a hydraulic analysis:

☐ Description of model whether steady state or extended period simulation.

☐ Assumptions are described including:

- Allocation of demands
- Friction coefficients, which will vary with pipe materials and age
- Pipe network skeletonization, as appropriate
- Operating conditions (source, storage booster pumps, valves)

☐ Minimum design criteria are met, including:

- Peak hourly demand: 30 psi or greater when equalizing storage has been depleted (Section 6.2.5).
- Maximum day demand plus fire flow: 20 psi or greater when equalizing storage and fire flow storage have been depleted (Section 6.2.5).
- Transmission main pressure 5 psi or more, except adjacent to storage reservoirs (Section 6.2.2).
- Maximum pipe velocity: 10 ft/sec or less in transmission mains and 8 ft/sec or less in distribution mains (Section 6.2.6). If not, include hydraulic transient analysis.

☐ Demand scenarios are described, including:

- Current demand
- Projected 6-10 year demand
- Projected build-out demand (for small water systems)

☐ Water age analysis and, where applicable, updating disinfection byproducts monitoring plan and coliform monitoring plan.

☐ Provide copies of input and output, including:

- Input data, (demands, elevations, friction losses, and pump curves)
- Hydraulic profile
- Node diagram

☐ Model calibration results satisfy one of the industry criteria for hydraulic models (see Section 6.1.3 and Table 6-1)

☐ Summary of results, deficiencies and conclusions including:

- Identification of deficiencies addressed in a capital improvement plan.
- Locations in distribution system where pressures exceed 80 psi (Section 6.2.7).
- Hydrant flow and placement on undersized mains.

- Fire flow reliability, if applicable. The Water System Coordination Act (chapter 70.116 RCW) requires water systems that serve more than 1,000 connections or that are located in a critical water supply service area to meet certain reliability standards when fire flow is provided by pumping (see WAC 246-293-660).

Appendix A.3.5 Reservoir Checklist

Address these design elements in reservoir project report and construction document submittals. Refer to Chapter 7 and WAC 246-290-235 for further guidance and requirements.

Project Report

☐ Reservoir purpose and sizing analysis including, at a minimum, operating, equalizing, standby, and fire suppression storage requirements, including documentation of nesting standby and fire suppression storage, if applicable. Adequate tank freeboard must also be provided.

☐ Reference to applicable industry design standards (e.g., AWWA D-100).

☐ Reference to and confirmation that applicable OSHA and WISHA safety requirements are satisfied.

☐ Site feasibility considerations:

- Location and site considerations (see Section 7.3)
- Natural hazards analysis (see Section 7.3.1)

☐ Basis for overflow, bottom of equalizing storage, and bottom of standby/fire suppression storage elevations including justification by hydraulic analysis.

☐ Assess capacity of the reservoir overflow to safely discharge total supply capacity to the reservoir, including sources, booster pump stations, and PRVs, to ensure the structural integrity of the proposed reservoir in the event of control system failure. Basis for overflow and reservoir vent design capacity.

☐ A description of the level control system with specific control levels identified; SCADA interface.

☐ If the new reservoir is intended to support aeration or disinfection contact time refer to Chapter 10 (Treatment).

☐ Mixing, water circulation, and water age analysis if nominal residence time exceeds 3 to 5 days during the summer.

- Measures needed to maintain water circulation and prevent water stagnation (dead zones)

Construction Documents (or shop drawings if the contract is so structured)

☐ Construction details including but not limited to:

- Reservoir isolation valve(s), which permit isolating the tank from the water system (WAC 246-290-235).
- Altitude valve and valve vault, if applicable.
- Air release/vacuum release valve on the distribution system side of the isolation valve if there is no other atmospheric reservoir in the pressure zone, to prevent vacuum conditions in the distribution system.

- Smooth-nosed sample tap on the tank outlet pipe, located between the tank and outlet pipe isolation valve (WAC 246-290-235).
- The capacity to measure water quality at various depths within the reservoir (see Section 7.6)
- Drain pipe outlet (WAC 246-290-235) (see Section 7.4.3).
- Overflow pipe inlet and outlet details (including outlet screen, flapper, or duckbill valve protection) (WAC 246-290-235) (see Section 7.4.4).
- Overflow and drain discharge disposal/drainage pathway/easement.
- Inlet, outlet, overflow, and drain piping size, material, location, invert elevations, pipe joints, and couplings below the reservoir and within 20 feet of the reservoir foundation.
- Tank atmospheric vent, vacuum relief, and screening material (WAC 246-290-235) (see Section 7.4.5).
- Weather proof, raised, lockable access roof hatch (WAC 246-290-235) (see Section 7.4.6).
- Access ways and ladders providing ready, safe access for maintenance (WAC 246-290-235).
- Security features to protect stored water from contamination due to unauthorized entry or vandalism (see Section 7.4.8).
- High- and low-level alarm system that directly notifies operations personnel.
- Location of level control system and details of any wall or roof penetration.
- Lightning arresters and electrical grounding, as applicable.
- Silt-stop on the outlet pipe to keep sediment from entering the distribution system.
- Construction joint details (for concrete reservoirs).
- Cathodic protection details (for steel reservoirs).
- Slope of reservoir roof at least 2 percent (¼ inch per foot) (see Section 7.4.7).
- Natural hazard design elements such as seismic design.

☐ ANSI/NSF Standard 61 certification of coatings, concrete form release and curing agents, liners, or other materials, if any, that would be in substantial contact with potable water. Application procedures for coatings should be specified in plans and specifications (WAC 246-290-220). See Appendix G for more information.

☐ Leakage testing procedures per AWWA and a narrative discussion of how the tank will be tested for leaks.

☐ Disinfection procedures per AWWA and related bacteriological sampling conducted prior to use (WAC 246-290-451).

☐ Procedures, if any, to test for taste and odor compounds prior to use. To confirm the applied coating system has cured properly, a VOC sample taken from the filled but not yet commissioned reservoir may be specified.

In addition to applicable elements from the list above, address these design elements in plastic reservoir construction documents (or shop drawings if the contract is so structured):

☐ Install the tank inside a secured (locked) building due of the vulnerability of these tanks to vandalism, damage from tree fall, freezing, and UV exposure when located outdoors. If installed outside a building address all physical security risks to the tank.

☐ Provide head space and structural support above or beside the tank to enable inspection and safe entry to the tank.

☐ Cover the screw-in disc hatch with a secondary cover to keep the hatch cover and screw-in frame clean of dust, rodent droppings, and other potential contaminants. If installed outside a building provide a weather proof, raised, lockable access roof hatch (WAC 246-290-235) (see Section 7.4.6).

☐ A reservoir roof vent installed independent of the access hatch.

☐ Identify all intended modifications (post-delivery) of a plastic tank and provide written approval from the tank manufacturer of the intended modification(s).

Appendix A.3.6 Booster Pump Station Checklist

Address these design elements in booster pump station project report and construction document submittals. Refer to Chapter 8 and WAC 246-290-230 for further guidance and requirements.

Project Report

☐ Sizing analysis, including pumping system discharge capacity requirements, as well as fire flow requirements, if any.

☐ Flow and pressure control.

☐ Alarm conditions.

☐ Hydraulic analysis that demonstrates the ability of the project to meet minimum pressure requirements during peak hourly demands and maximum day demands plus fire flow. The analysis should include a narrative discussion that describes the hydraulic analysis method, explains critical assumptions, and summarizes the effect of the proposed demands on the existing system (see Checklist A. 3.4 Hydraulic Analysis for details).

☐ Service area map for the zone(s) to be served.

☐ Site feasibility considerations:

- Location and site considerations (see Section 8.2)
- Natural hazards analysis (see Section 8.2.1)
- Noise from the pumps and equipment, and any need for noise mitigation

☐ Assess capacity of each reservoir overflow to safely discharge total supply capacity to the reservoir, including sources, booster pump stations, and PRVs, to ensure the structural integrity of each reservoir in the event of control system failure.

☐ Assess potential for damaging transient pressure wave during pump start up and abrupt pump station shutdown.

☐ Electrical power issues including:

- Supply – voltage, quality, and desired phase configuration
- Reliability – frequency of power outages
- Assessing the need for backup power.

Construction Documents

☐ Map of the site and vicinity drawn to scale, including the pump station structure, water lines, site topography, roadways, and all above and underground utilities.

☐ Pump station details including security measures, slab elevation, ventilation, and electrical connections allowing the use of emergency power.

☐ Building equipment and instrument layout demonstrating adequate clearance to safely enter, operate, and maintain all pump station components.

☐ Pumping equipment specifications including:

- Horsepower, flow rate (gpm), head, pump controls, and alarm system.
- The specific pump curve used and operation range of head and flow conditions.

☐ Flow and pressure control and instrumentation specifications

☐ Site piping plans including:

- Sample tap(s)
- Isolation valves on the suction and discharge sides
- Flexible couplings
- Check valves on the discharge side
- Surge anticipation valves, as needed
- Suction side pressure gauge(s)

☐ Pump station start-up task including:

- Field testing of pumps for output, efficiency and vibration
- Disinfection of piping
- Pressure, leakage, and bacteriological testing.

☐ General facility considerations including:

- Security measures
- Special anchoring or support requirements for equipment and piping
- Heating, cooling and humidity control for equipment protection and operator comfort

Appendix A.3.7 Pressure Tank Checklist

Address these design elements in pressure tank project report and construction document submittals. Refer to Chapter 9 (Pressure Tanks) and Appendix B.2 (Cycle Control Valves), and Appendix B.3 (Variable Frequency Drives) for further design guidance.

Project Report

☐ Sizing analysis, pump protection, and pump discharge control.

☐ Pressure settings including a narrative justification regarding water system hydraulics and operating pressure range.

Construction Documents

☐ Pressure relief valves:

- Specify an ASME Section VIII pressure-relief valve installed between a pressure tank greater than 37.5 gallons gross volume and the tank isolation valve.
- Specify a properly sized pressure relief valve manufactured in accordance with a recognized national standard installed between a pressure tank equal to or smaller than 37.5 gallons gross volume and the tank isolation valve.
- Pressure relief valve capacity.
- See DOH publication 331-429

☐ Isolation valve for each pressure tank.

☐ Site piping plans including location, size, type, and class of pipe.

☐ Clearance provided around each tank adequate for operations and maintenance.

☐ Bladder tanks only:

- Pre-charged pressure

☐ Hydropneumatic tanks only:

- Confirmation of oil-less or food-grade oil lubricated air compressor.
- Air filter
- Access hatch with minimum 5-foot clearance
- Level control
- Sight glass
- Structural support and earthquake resiliency/bracing

Appendix A.3.8 Water Treatment Facilities Checklist

Address these design elements in water treatment project report and construction document submittals. Refer to Chapter 2, including the project development flowcharts therein; Chapters 10 and 11; chapter 246-290 WAC Parts 3 and 6; DOH subject-specific publications; and the project-specific guidance located in these appendices for further guidance and applicable requirements

- Hypochlorination in Appendix F.1
- Sodium fluoride saturators in Appendix F.2
- Arsenic removal in Appendix F.3 and F.4
- Ozone in Appendix F.5
- Desalination of seawater or brackish water in Appendix F.6
- Rainfall catchment in Appendix F.7 and F.8
- Plumbing sampling line to a process analyzer in Appendix F.9
- Iron and Manganese treatment in Appendix F.10 and F.11
- Nitrate removal by Ion Exchange in Appendix F.12
- Slow sand filtration in Appendix H
- Tracer studies in Appendix B.4

Before any significant design work begins, the engineer should contact the appropriate ODW regional engineer (see Table 1-1). Pilot studies are generally required.

Project Report

☐ Narrative discussion describing water quality problem and type of treatment proposed.

☐ Analysis of alternatives

- Raw water quantity and quality
- Secondary impacts of water treatment: Assess how the proposed project could affect water quality in the distribution system (WAC 246-290-110(4)(d)), and specifically address how a change in treatment will affect compliance with the Lead and Copper Rule (40 CFR 141.86(d)(4)(vii)).
- Distribution system water quality impacts
- Operations and maintenance plan, including operator certification requirements, monitoring and reporting requirements, alarm conditions, and daily/weekly/monthly tasks necessary for reliable operation and achievement of treatment performance goals.
- Waste disposal
- Life cycle cost analysis
- Plant location and site considerations
- Natural hazards analysis

☐ Pre-design study

- Pilot study plan

- Pilot study report

☐ Assessment of treatment process reliability

☐ Detailed design criteria and calculations for
- Proposed treatment process
- Process control

☐ Power reliability

☐ Operator and plant safety considerations

☐ Performance standards for water treatment facility based on desired water quality at the defined point of compliance.

Construction Documents

☐ Map of the site and vicinity drawn to scale, including the treatment plant building(s), water lines, site topography, roadways, and all above and underground utilities.

☐ Treatment plant details including security measures, slab elevation, heating, cooling and ventilation, and provision allowing the use of emergency power.

☐ Building equipment and instrument layout demonstrating adequate clearance to safely enter, operate, and maintain all treatment plant components.

☐ Equipment specifications

☐ Flow and pressure control and instrumentation specifications

☐ Site piping and instrumentation plans including:

- Sample tap(s)
- Isolation, check, flow control, backflow, and pressure valves
- Pressure, flow, level, temperature, and on-line water quality (e.g., pH, turbidity, conductivity, chlorine residual)
- Alarm conditions

☐ Start-up task including:

- Field testing of equipment to satisfy specifications, including sequence and duration
- Disinfection
- Pressure, leakage, and bacteriological testing

☐ General facility considerations including:

- Security measures
- Special anchoring or support requirements for equipment and piping
- Heating, cooling and humidity control for equipment protection and operator comfort

☐ Specifications for materials and equipment for the treatment facility.

☐ ANSI/NSF Standard 60 certification within approved application dosage of any additives used in the treatment process.

☐ ANSI/NSF Standard 61 certification of coatings, liners or other materials, if any, that would be in substantial contact with potable water. Application procedures should be specified in the plans and specifications.

☐ Methods and schedules for start-up and performance/acceptance testing of the completed treatment facility.

☐ Provisions to dispose of solid waste material from treatment process, including description of how waste is to be properly disposed, and documentation that procedures are acceptable to the Department of Ecology and local authorities.

When the source is surface water, or confirmed to be groundwater under the direct influence of surface water, submittals **must** meet the following additional requirements:

☐ Disinfection analysis such as a tracer study (see Appendix B.4) to determine that adequate disinfection can be provided.

☐ Filter design details including the filter-loading rate and backwash design.

☐ Turbidimeter locations including those for individual filter turbidimeters and a combined filter effluent turbidimeter prior to the clearwell.

☐ Filter-to-waste design including an adequate air gap and properly sized waste pipe.

☐ Alarms for critical process control elements such as water levels, coagulation, filtration, and disinfection. Alarms must be set to provide sufficient warning to allow operators to take action or shut the plant down as appropriate.

☐ Standby equipment for critical processes such as coagulation, filtration, and disinfection to ensure that the plant can operate continuously.

☐ Multiple filtration units to allow for major maintenance and repairs on the filtration units. Complete redundancy for peak design flows does not need to be provided.

☐ Detailed Operations Program (O&M manual) detailing how the treatment facility will be operated. The document must describe:

- Coagulation control methods
- Chemical dosing procedure
- Each unit process and how it will be operated
- Maintenance programs for each unit process
- Treatment plant performance monitoring
- Laboratory procedures
- Recordkeeping
- Reliability features
- Emergency response plans, including ones for treatment process failures and watershed emergencies

Appendix B: Selected Guidelines

Appendix B.1 **Well Field Designation and Source Sampling Guidelines**

Appendix B.2 **Cycle Control Valve Guidelines**

Appendix B.3 **Variable Frequency Drive Pumps and Motors**

Appendix B.4 **Tracer Study Guidelines**

Appendix B.1 Well Field Designation and Source Sampling Guidelines

The Office of Drinking Water (ODW) supports the concept of designating nearby wells that draw from the same aquifer as a “well field”. A well field designation allows the wells of a well field to be monitored as a single source. This designation reduces the number of samples that must be collected to meet water quality monitoring requirements, which reduces monitoring costs for water systems while still being protective of public health.

This guidance is meant to ensure consistency when dealing with well field designations for Washington State water systems. A design engineer seeking designation of two or more individual wells as a well field should demonstrate all of the following criteria are met:

1. The depth to first open interval of all individual wells must be within 20 percent of each other after taking wellhead elevation differences into account.

2. All individual wells must draw from the same aquifer(s) as determined by:
   a. An analysis of water chemistry of all sources that demonstrates similar water chemistry for all analytes; and
   b. An evaluation of geology as documented in well logs or water well reports for all of the wells being considered; and
   c. A demonstration that the cones of depression of the wells in the well field overlap under normal operating conditions (e.g. that the water level(s) in the adjacent well(s) in the well field drop when the proposed well of the well field is pumping); or
   A report submitted by a hydrogeologist licensed in the State of Washington that demonstrates a well field designation is appropriate. If ODW staff disagrees with the conclusions of this report, the well field designation may be denied.

3. All individual wells must:
   a. Have a sample tap, installed upstream of treatment, that is not influenced by any other well in the well field.
   b. Discharge to a common pipe, treatment system or storage facility that has a sampling tap located downstream on the common pipe or after the treatment or storage facility but prior to any service connections; and
   c. Be under the control of the same purveyor.

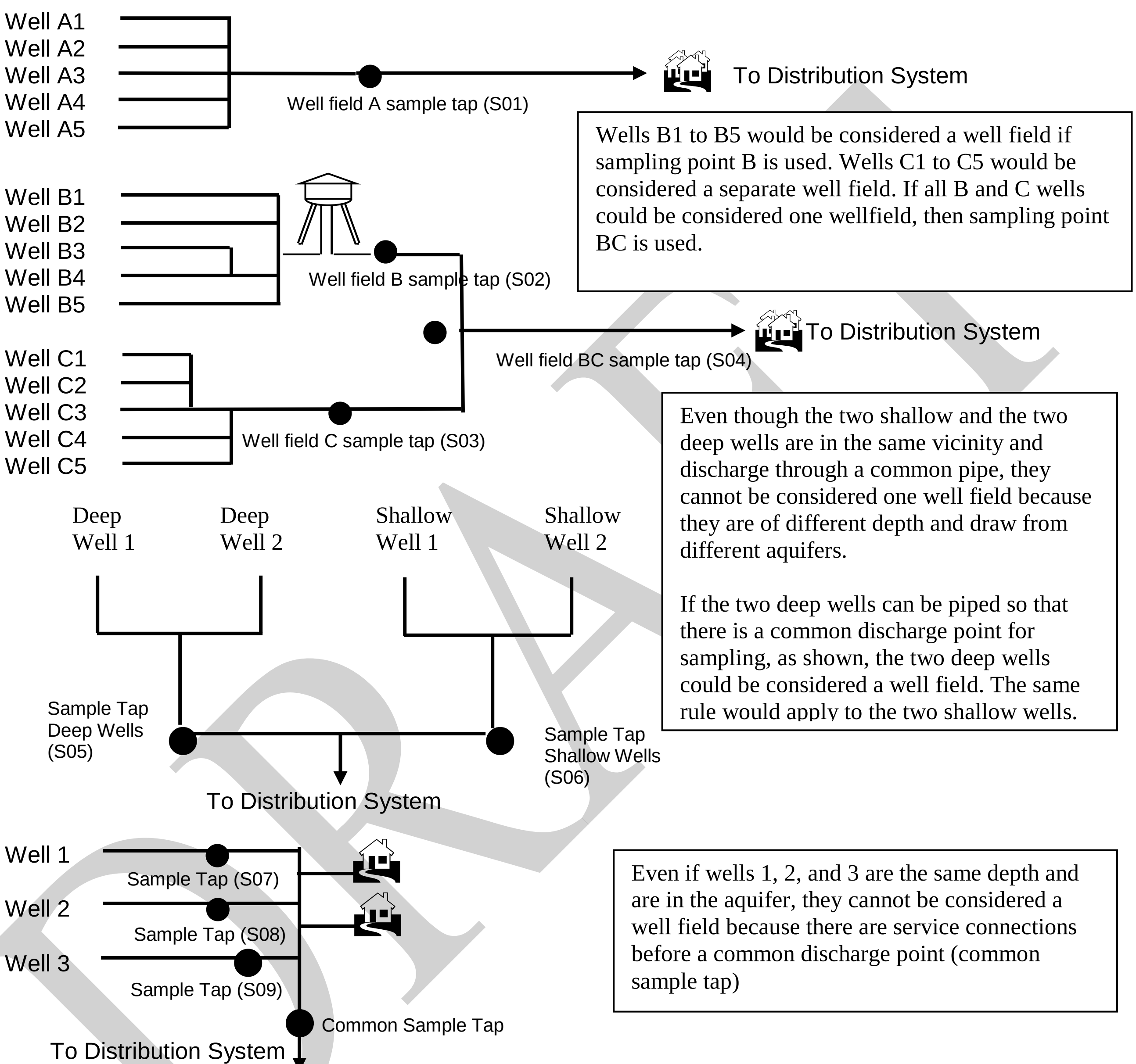
Notes on monitoring for well fields

Samples required for compliance with source water quality monitoring requirements from a well field:

1. Should be collected from the sample tap mentioned in requirement 3 above (prior to the entry point to the distribution system); and
2. Should represent normal operating conditions of the well field at the time the sample is collected (that is, every well in a well field need not necessarily be pumping at the time of sample collection); and

3. May ***not*** be composited with other samples.

Well Field Examples



Composite Sampling

DOH Source Code	PWS Source Number	Composite Sampling Allowed?
S01	**Well Field A**	**No – cannot be composited with other sources**
S02 and S03	**Well Field B and Well Field C**	
S04	**Well Field BC**	
S05	**Well Field Deep Wells**	
S06	**Well Field Shallow Wells**	
S07	**Well 1**	

S08	Well 2	Yes – can be composited together for VOC and SOC monitoring compliance
S09	Well 3	

Appendix B.2 Pump Cycle Control Valve Guidelines

A pump cycle control valve (CCV) may be used to control the pressure in a distribution system. The CCV is intended to extend run time with minimal pressurized storage. It will maintain constant downstream pressure (i.e., the valve's set point) until demand downstream of the valve falls below the valve's prescribed low flow level, at which point the pressure will rise to the pressure switch pump-off set point. The valve is mechanically prevented from restricting flow past its preset minimum.

Pressurized storage is needed with the CCV to supply the distribution system when demand falls below the valve's minimum flow setting and pump operation gets shut down. The size of the pressure tank(s) will depend on several factors as described below, but the size and number always will be less than that required if a cycle control valve had not been installed. Designers should review manufacturer's recommendations to ensure all valve application requirements are met.

Advantages of using a CCV include:

1. Limiting well pump on-off cycling and the associated wear on water system components.
2. Reducing the size or number of pressure tanks required for any given installation.
3. Reducing the potential for damaging transient pressure waves ("water hammer") resulting from hard pump-start and pump-stop conditions.

Design considerations and challenges of using a CCV include:

- The control valve itself can impose significant energy loss ("head loss") at the high end of its flow range when fully open (a 1¼-inch control valve causes the loss of about 10 psi at 50 gpm). The well pump design must account for the head loss imposed by the control valve.
- It is difficult to predict whether the savings through limiting the number of "pump-start" events and reduced initial capital cost associated with fewer bladder tanks will offset the cost of the additional energy used in prolonging the pump-on portion of the cycle.
- Water quality may affect control valve performance. Particulate matter (sand) may adversely affect the performance of the control valve.
- At low flow conditions, the pressure on the upstream side of the control valve will be near the pump's shut-off head. You should pay attention to the design, material specifications, and construction of the pump to ensure it can operate near its shut-off head for extended periods, and to the pressure rating of the piping and valves on the upstream side of the control valve.

- CCV consumes greater amounts of energy per gallon pumped due to prolonged operation at low pump efficiency.

The CCV is usually installed between the pump(s) and the pressure tank(s). The valve's downstream pressure setting should fall between the pressure switch on and off pressure settings. As the demand in the water system varies, the cycle control valve adjusts the pressure generated by the pump by modulating the size of the valve opening. The pump-on phase of the pump cycle will continue until the water system demand drops below the valve's minimum flow setting. At this point, pump supply in excess of system demand goes into pressurized storage until the pressure tank reaches the pressure switch "pump-off" setting. If demand (including leaks) never drops below the valve's minimum flow setting, the pump will never shut off.

While the pump is off, all water demand is satisfied by water released from the pressure tank(s). The length of the "pump-off" period depends on water system demand and the available withdrawal volume of the pressure tank(s).

The number of pump starts per hour is important since pump motors may be damaged by excessive heat build-up from too-frequent starts. In the absence of the pump motor manufacturer's specification, pump starts should be limited to no more than six (6) per hour.

In order to design the pressure tank system so pump starts are limited to no more than six starts per hour (or per the manufacturer's specification), designers should consider:

- The valve's minimum flow setting and pre-set downstream pressure setting;
- Pump-on and pump-off pressure setting; and
- Where the valve pressure set-point falls within the pump-on-off pressure range.

Example

Given:

- Bladder tank system
- Pump on pressure = 40 psi
- Pump off pressure = 60 psi
- Cycling control valve pressure setting = 50 psi
- Pump control valve low-flow setting = 5 gpm

Find:

- Volume ("V") of pressurized storage between 60 and 40 psi available to the distribution system while the pump is off to provide for a minimum pump cycle time of 10 minutes (equal to 6 cycles per hour)

Solution:

- The shortest pressure tank fill time + tank draw time occurs when distribution system demand is approximately equal to one-half the low-flow valve setting (2.5 gpm in this example). System demand is "Y" and valve flow setting is "X".

- To simplify and remain conservative, assume the time to fill the pressure tank from the low-pressure pump-on setting (i.e., 40 psi in this example) to valve pre-set pressure setting (i.e., 50 psi) is instantaneous. Also, assume pressurized volume from 40 psi to 50 psi is equal to the pressurized volume from 50 psi to 60 psi.

- Time to fill pressure tank from 50 psi to 60 psi:
$$\frac{0.5V}{\mathrm{X - Y}}$$
- Time to draw down pressure tank from 60 psi to 40 psi while pump is off:
$$\frac{V}{\mathrm{Y}}$$

- Solve this equation:
$$\frac{0.5V}{\mathrm{X - Y}} + \frac{V}{\mathrm{Y}} = 10 \text{ minutes} = 6 \text{ cycles per hour}$$

  If X = 5 gpm and Y = 2.5 gpm, then V = 16.7 gallons

In the above example, the bladder tank system must provide at least 16.7 gallons of storage between 40 psi and 60 psi. Based on the following pressure tank manufacturer's information, the drawdown for a nominal 34-gallon pressure tank is 9.1 gallons from 40-60 psi. In order to provide 16.7 gallons, two 34-gallon pressure tanks are needed. Alternately, one 62-gallon pressure tank will satisfy the pressurized storage requirement.

IN-LINE MODELS

MODEL NUMBER	CAPACITY GALLONS	DRAWDOWN/GALLONS			HEIGHT INCH	DIAMETER INCH	SYSTEM CONNECTION	ASSEMBLY WEIGHT LBS.
		20-40 PSI	30-50 PSI	40-60 PSI				
WX-101	2.0	0.7	0.6	0.5	12-5/8	8	3/4" NPTM	5
WX-102	4.4	1.6	1.4	1.2	15	11	3/4" NPTM	9
WX-103	8.6	3.1	2.7	2.2	25	11	3/4" NPTM	15

STAND MODELS

MODEL NUMBER	CAPACITY GALLONS	DRAWDOWN/GALLONS			HEIGHT INCH	DIAMETER INCH	SYSTEM CONNECTION	ASSEMBLY WEIGHT LBS.
		20-40 PSI	30-50 PSI	40-60 PSI				
WW-20	20.0	7.3	6.2	5.3	31-5/8	15 3/8	1" NPTF	35
WW-32	32.0	-	9.9	8.5	46-3/8	15 3/8	1" NPTF	43
WX-201	14.0	5.1	4.3	3.7	23-7/8	15 3/8	1" NPTF	27
WX-202	20.0	7.3	6.2	5.4	31-5/8	15 3/8	1" NPTF	35
WX-203	32.0	-	9.9	8.6	46-3/8	15 3/8	1" NPTF	43
[illegible]	34.0	12.4	10.5	9.1	29-1/2	22	1-1/4" NPTF	61
WX-250	44.0	16.3	13.6	11.9	35-5/8	22	1-1/4" NPTF	69
[illegible]	62.0	22.9	19.2	16.7	46-3/4	22	1-1/4" NPTF	92
WX-252	86.0	34.6	29.2	24.9	62-1/4	22	1-1/4" NPTF	114
WX-255	81.0	34.6	29.2	24.9	56-13/16	22	1-1/4" NPTF	114
WX-302	86.0	34.6	29.2	24.9	46-13/16	26	1-1/4" NPTF	123
WX-350	119.0	44.0	36.9	32.1	61-7/8	26	1-1/4" NPTF	166

UNDERGROUND MODELS

MODEL NUMBER	CAPACITY GALLONS	DRAWDOWN/GALLONS			HEIGHT INCH	DIAMETER INCH	SYSTEM CONNECTION	ASSEMBLY WEIGHT LBS.
		20-40 PSI	30-50 PSI	40-60 PSI				
WX-202-UG	20.0	7.4	6.2	5.4	29-3/4	15 3/8	1" NPTF	33
WX-250-UG	44.0	16.3	13.6	11.9	33-3/8	22	1-1/4" NPTF	63

Appendix B.3 Variable Frequency Drive Pumps and Motors

A variable-frequency drive (VFD) is an electronic controller that adjusts the speed of an electric motor by modulating frequency and voltage. VFDs provide continuous control by matching motor speed to the specific demands of the work being performed. VFDs allow operators to fine-tune pumping systems while reducing costs for energy and equipment maintenance.

Use in potable water systems

VFDs are becoming more popular at water facilities, where the greatest energy demand most often comes from pump motors – an application particularly suited to variable-frequency drives. VFDs enable pumps to accommodate fluctuating demand, running pumps at lower speeds and drawing less energy while still meeting water system needs.

Benefits

Single-speed drives start motors abruptly, subjecting the motor to high torque and current surges up to 10 times the full-load current. In contrast, variable-frequency drives offer a “soft start” capability, gradually ramping up a motor to operating speed. This lessens mechanical and electrical stress on the motor system, can reduce maintenance and repair costs, and extend motor life.

VFDs allow more precise control of processes, such as water production and distribution. They can also maintain pressure in water distribution systems to closer tolerances. Energy savings from VFDs can be significant. Affinity laws for centrifugal pumps suggest that a reduction in motor speed will generate energy savings. While motor speed and flow are proportional (e.g., 75% speed = 75% flow), motor speed and horsepower have a cubed relationship (e.g., 75% speed = 40% power consumption). Despite some of the VFD controller’s additional energy requirements VFDs can reduce a pump’s energy use over many single speed pumping applications.

Pumps may be designed and installed for the built-out condition, and operate economically and efficiently for the many years it will take to reach the full demand design condition.

Disadvantages and Design Challenges

- Outdoor installations can be a problem, since VFDs can’t tolerate extremely cold weather. Check the manufacturer’s specifications for ambient air temperature limitations.
- VFD controllers are sensitive to high temperature, humidity, and particulates. The manufacturer should be consulted on the need for air conditioning and air filtering.
- Placing the controller more than 100 feet from the motor can be a problem without taking special provisions. Check with the VFD manufacturer for specific requirements.
- Power and control wires must be in separate conduits.
- VFDs only work on three-phase motors, except in very small pump applications.

- Pumps controlled by a VFD may not meet the minimum water flow required to keep the motor winding cool. Care should be taken to ensure that the pump is not operating below this speed. Sleeving may also be an option to protect the pump motor. Confirm with the submersible pump manufacturer the minimum flow rate across the motor needed for motor cooling.

- The quality of the power coming into the VFD controller can have a significant impact on controller performance. Voltage fluctuations should be monitored prior to installing a VFD controller.

- The resonant frequency of the pump and motor should be checked and accommodations made if the resonant frequency is within the range of expected pump speeds.

- Experienced electronics personnel will be required for maintenance and repair.

When designing a VFD pumping system

Certain rotational speeds may induce resonance and excessive vibration. Designers should check with the manufacturer the resonant frequency of the pump/motor, and whether that frequency could be induced by a speed within the predicted operating range of the pump.

The designer should reference the minimum flow requirements of the pump when establishing the operating range of the pumping system. Each manufacturer will have its own specific requirements for pressurized storage volume to ensure compatibility with the specific low-flow pump off discharge rate, ramping speed, and the system control pressure range.

Appendix B.4 Tracer Study Checklist

Tracer studies are simple in concept; a tracer is added prior to the inlet of a structure and the concentration is measured at the outlet. However, conducting a tracer study that gives you meaningful results can be very challenging. To maximize the potential for success of a tracer study, you must submit a tracer study plan to the department before conducting a tracer study (WAC 246-290-636(5)). This checklist provides general expectations for the information that should be included in a tracer study plan.

General Information

The purpose of the study and basic information about the facility should be provided. In addition, please provide the following information:

- ☐ The names, titles, and qualifications of the persons conducting the study.
- ☐ A description of the plant and how it is operated, including range of flows, operating levels, and operational controls. Note if laminar flows due to low velocity occur.
- ☐ An overview of how the study will be conducted.
- ☐ A schematic of the part(s) of the facility through which the tracer will pass showing:
    - Point of tracer addition.
    - Other chemical additions (such as disinfectants, fluoride, etc) and sampling locations used for normal process monitoring.
    - Tracer sampling locations (after tracer addition, after flow passes through the structure(s), other locations as needed).
    - Flow meters.
- ☐ Dimensions, depth(s), and volume(s) of the part(s) of the facility assessed, including:
    - Length to width (or depth) ratio
    - special conditions such as parallel, sequential or segmented basins

Tracer Study Plan

The following information should be included in the tracer study plan.

- ☐ Tracer
- ☐ Type of test
- ☐ Sampling schedule
- ☐ Flow rates
- ☐ Data and instrumentation
- ☐ Procedures to confirm that water level/flow rate instrumentation or measurement is accurate.
- ☐ Field verification of basin dimensions and configuration details.

Tracer

Various tracers can be used, though there is no one ideal tracer. For the best results, the tracer must be conservative, i.e. it must not react over the period of the tracer study. The most commonly used and reviewed ones are fluoride, lithium, sodium, chloride, and calcium. Sometimes conductivity is used as a surrogate when sodium chloride is used as a tracer. Reactive tracers, such as hypochlorite, can be used in limited cases where the tracer study time is short, and preliminary tests have been conducted to demonstrate that there is little or no decay of the tracer under the conditions of the proposed test.

- ☐ Information about the proposed tracer should include:
  - Rationale for selected tracer
  - Sampling locations (after tracer addition, after flow passes through the structure(s), others as needed).
  - ANSI/NSF Standard 60 approved dose or acceptable alternative
  - How solution will be prepared
- ☐ Target dose
- ☐ Background/baseline level of the tracer.
- ☐ Injection method (saturators should not be used. A premixed solution is best).
  - Identify how tracer will be dispersed into the flow stream.
- ☐ Identification of lag times between the flow path and point of sample collection.
- ☐ Analytical method (range, sensitivity, accuracy, field or laboratory analysis).

Type of Test

The two main types of tracer studies are step dose tests and slug dose tests. Slug dose tests can result in density current effects because very concentrated solutions are needed to conduct the study. We generally recommend using a step dose approach because of this issue and other disadvantages of the slug dose approach. We will only accept the use of a slug dose approach with a thorough and adequate justification acceptable to the department.

Sampling Frequency and Duration

It is important to sample for the tracer often enough, especially shortly after the start of the test, so that the baffling efficiency (T_{10}/T) can be clearly identified. T_{10} is the time it takes for 10 percent of the tracer to break through and T is the mean hydraulic residence time. The following general framework from Teefy (1999) can be used for step dose tests.

Period	Time between samples
Start to 0.25T	0.025T
0.25T to 1.5T	0.042T
1.5T to 2T	0.050T
2T to 3T	0.100T
3T to 4T	0.200T

You should also collect samples shortly after the point of tracer injection (at or near the inlet of the basin) at a frequency of 0.10T to confirm constant tracer addition and accurately estimate tracer recovery. In addition, flow and clearwell level measurements should be collected continuously. The duration should be sufficient to achieve at least 95 percent recovery of the tracer.

In some cases, it may be difficult to maintain a constant level in a clearwell and flow for four hydraulic residences times. In these cases, a shorter step dose test can be used, but it should not be shorter than the calculated mean hydraulic residence time, especially in well baffled structures.

Flow Rates

Tracer studies should be conducted at three and preferably four different flow rates, spaced out over the range of flows normally experienced (low, average, and peak). The basic premise is that the baffling efficiency (T_{10}/T) should be fairly constant over the range of flows tested. Studies have shown that the baffling efficiency is slightly lower at lower flow rates. However, a significant variation in baffling efficiency may reveal issues with the ways that the tracer study was conducted or even issues with the facility itself, such as construction anomalies. The highest flow rate tested should be at least 91 percent of the peak hourly flow expected through the portion of the facility evaluated (USEPA 1990).

- Provide an explanation for the determination of peak hourly flow (including historical flow data and any supporting calculations).

Computational fluid dynamic (CFD) modeling conducted over multiple flow conditions can be used to minimize the number of flow conditions tested in the field. At least one field tracer test, conducted after the CFD modeling results are submitted, is necessary to validate the CFD modeling results.

Changes in flow due to backwashing, finished water pump operation or other routine plant operation should be anticipated, and minimized or addressed during the study. For higher flow rates, practical considerations such as where to put the excess water must be addressed. At higher flow rates, it may be necessary to conduct tracer studies in the summer to take advantage of the increased demand.

Data and Instrumentation

The instrumentation used in the tracer study test is critical in the evaluation of the tracer study data. The instruments of interest include the chemical test equipment as well as flow meters. Please provide the following information on tracer study data and instrumentation.

- ☐ Analytical method (range, sensitivity, accuracy, field or laboratory analysis).
- ☐ Identify the field/facility instruments used in the tracer study.
    - Make / model of instrument(s).
    - Calibration date of instrument(s).
    - Reagents used in test and associated expiration dates.
    - Sensitivity range of equipment.
- ☐ Level measurements (if the water level can vary during the test)
    - Method for maintaining a near-constant water level.
- ☐ Provide a copy of the proposed field data sheets. Each data sheet should include the following.
    - Flow, level, temperature, and tracer measurements.

- The instrument from which the data was obtained.

Presentation of results

- ☐ Templates (forms, tables or graphs) that will be used to present the results
- ☐ Instructions for the utility on how to use the results to complete daily CT monitoring, including method for determining peak flow (inflow, outflow).
- ☐ CFD modeling results, if applicable.

References

Teefy, S. (1996) Tracer Studies in Water Treatment Facilities: A Protocol and Case Studies. AWWA Research Foundation. Denver, CO.

USEPA (1990) Guidance Manual for Compliance with the Filtration and Disinfection Requirements for Public Water Systems Using Surface Water Sources – Appendix C: Determination of Disinfectant Contact Time. United States Environmental Protection Agency, Washington, DC.

Appendix C: List of Agencies and Publications

Note: *This list contains the addresses and phone numbers for each agency's main office or location. Many of the agencies also have local or regional offices that offer services. This list of agencies and the information they provide is not intended to be all-inclusive.*

Organization Name	Organization Type	Telephone and Web Site	Information or Publications Available
U.S. Environmental Protection Agency (EPA) Region 10	Federal	(206) 553-1200 or (800) 424-4372 (general) http://www.epa.gov/r10earth/ https://www.epa.gov/ground-water-and-drinking-water	-All topics related to the Safe Drinking Water Act
Center for Disease Control and Prevention	Federal	800-CDC-INFO (800-232-4636) TTY: 888-232-6348 https://www.cdc.gov/fluoridation/index.html	-Fluoridation information
National Oceanic and Atmospheric Administration	Federal	(206) 526-6087 (Weather Service - Seattle) http://www.weather.gov/sew/ Climatic Data Center https://www.ncdc.noaa.gov/	-Climate information

Organization Name	Organization Type	Telephone and Web Site	Information or Publications Available
Occupational Safety and Health Administration (OSHA)	Federal	https://www.osha.gov/	-Employee and construction safety
Department of Ecology	State	Water Resources Program: http://www.ecy.wa.gov/programs/wr/wrhome.html Well Report Viewer: https://fortress.wa.gov/ecy/waterresources/map/WCLSWebMap/default.aspx River and Stream Flow Data: http://www.ecy.wa.gov/programs/eap/flow/index.html Water Quality Program: http://www.ecy.wa.gov/programs/wq/wqhome.html Hazard Waste and Toxics Reduction Program: http://www.ecy.wa.gov/programs/hwtr/index.html Phone: 360-407-6000 (voice) 800-833-6388 (TTY) Publications: https://fortress.wa.gov/ecy/publications/UIPages/Home.aspx	-Water rights -Well logs River and stream flow data -Criteria for sewage works design -Disposal of chlorinated water -Dam safety -Well construction standards -Disposal of WTP backwash -Hazardous waste disposal
Department of Health Office of Drinking Water	State	(360) 236-3100 (800) 525-0127 TTY users dial 711 http://www.doh.wa.gov/ehp/dw Publications:	-Water System Planning Handbook -Small Water System Management Program Guide -Water Use Efficiency information -Group B Design Guidelines

Organization Name	Organization Type	Telephone and Web Site	Information or Publications Available
		http://www.doh.wa.gov/CommunityandEnvironment/DrinkingWater/PublicationsandForms	-Fact sheets -Approved Backflow Assemblies List - Training Opportunities -Drinking water rules, policies and other guidelines -Subject matter experts and technical assistance contacts
Department of Labor and Industries (L&I)	State	(360) 902-5800 (Main) TTY: **1-800-833-6388** (360) 902-5500 (WISHA) (360) 902-5226 (Plumbing and Contractor Registration) (360) 902-5270 (Boiler and Pressure Vessels) http://www.lni.wa.gov/	-Safety rules -Work in confined spaces -Working with asbestos-cement pipe -Statutes and rules on boilers and pressure vessels -Plumber certification and contractor registration
Department of Natural Resources	State	(360) 902-1000 (Main) (360) 902 1450 (Geology and Earth Sciences) http://www.dnr.wa.gov/	-Liquefaction susceptibility maps

Organization Name	Organization Type	Telephone and Web Site	Information or Publications Available
Department of Transportation Standard Construction Specifications	State	(360) 705-7430 http://www.wsdot.wa.gov/Business/Construction/SpecificationsAmendmentsGSPs.htm	-Technical and construction manuals -Standard specifications
State Building Code Council (SBCC)	State	(360) 407-9277 https://fortress.wa.gov/es/apps/SBCC//	-International Building Code -Uniform Plumbing Code -International Fire Code
Office of Washington State Climatologist (OWSC)	State	(206) 543-3145 http://www.climate.washington.edu	-Historical climate information
Office of the State Fire Marshal	State	(360) 596-3900 http://www.wsp.wa.gov/fire/firemars.htm	-Fire prevention information -Fire sprinkler information
Utilities and Transportation Commission (UTC)	State	(360) 664-1300 https://www.utc.wa.gov/regulatedIndustries/utilities/water/Pages/default.aspx	-Requirements related to inventory owned water systems (water companies)
American Water Works Association (AWWA)	Professional	(303) 926-7337 https://www.awwa.org/	-Standards -Water Research Foundation reports -Manuals -Standard methods

Organization Name	Organization Type	Telephone and Web Site	Information or Publications Available
			-Various journals and periodicals
Pacific Northwest Section – AWWA (PNWS-AWWA)	Professional	(503) 760-6460 http://www.pnws-awwa.org/	-Brochures -Bill stuffers -Cross-Connection Control Manual -Training Opportunities
Health Research Inc., Health Education Services Division	Regional	(518) 439-7286 http://10statesstandards.com/	-Ten States Standards
NSF International (formerly the National Sanitation Foundation)	Audit and Certification	800-673-6275 http://www.nsf.org/	-List of NSF-approved products -NSF Standards
University of Southern California (USC) Foundation for Cross-Connection Control and Hydraulic Research	Academic	866-545-6340 http://www.usc.edu/dept/fccchr	-Approved Backflow Assemblies List -Manual of Cross-Connection Control
Washington Surveying and Rating Bureau	Professional	(206) 217-9772 http://www1.wsrb.com/wsrbweb/	-Insurance ratings
Western Regional Climate Center	Academic	(775) 674-7010 https://wrcc.dri.edu/	-Climate data -Rainfall data

Appendix D: Estimating Water Demands

Appendix D.1 Background and Development of Residential Water Demand vs. Precipitation

Appendix D.2 Estimating Nonresidential Demand

Appendix D.3 Deriving Maximum Daily Demand to Maximum Month Average Daily Demand Ratio

Appendix D.1 Background and Development of Residential Water Demand vs. Precipitation

Editor's Note: Appendix D.1 is a study originally published in 1999. D.1 is presented to support the design of new water residential-only systems without metered data or analogous water system data to estimate the ERU_{ADD} *and* ERU_{MDD}.

Data Collection

A basic tenet for the revised design standards was to provide a conservative basis for designing new water system development, or extensions to existing development, whenever more reliable historical data was unavailable. It was recognized that a basic design parameter such as residential water demand may be better estimated if it could be based on available information throughout the state that could be both accessible and reliable. Information gained from water system records throughout the state, or from other locales with residential developments similar to this state, would be useful and generally more applicable to the establishment of a general design standard.

In attempts to secure accurate water use data from all parts of the state, three sources of information were used - two from surveys conducted by the department and one from reviews of documented information contained in various utility Water System Plans (WSPs) which had been submitted to, and approved by, the state DOH. An additional source of information was a report prepared by the California Department of Water Resources.

An initial (1993-94) survey questionnaire was sent to 30 selected utilities representing a uniform geographical distribution throughout the state. This survey was intended to determine a complete accounting of all water uses experienced by the utility from which specific data regarding residential uses could be derived. Questions were asked for number of metered accounts, total annual water demand, total population served, recorded average annual demands for all types of accounts (residential, industrial, commercial), multi-family uses (if possible to discern), recorded maximum day demands, and estimates of unaccounted water uses. The information was requested for the three year period, 1990-92. Also, the average annual rainfall for the utility service area was requested, if it were known by the utility. Where rainfall data was not provided by the responding systems, rainfall levels were determined from Meteorological Service records for the gauging station within, or nearest to, the utility.

For the 19 survey responses received from the initial questionnaire, the information was analyzed in an attempt to identify the water demands associated only with residential uses. This data was in turn correlated with rainfall records for the area. Of the survey responses returned, nine were of sufficient detail that residential demand estimates could be made with a relatively high degree of confidence. Information regarding maximum daily demands was generally not readily available, although in some instances water systems did present estimates of maximum day uses based on ratios to their peak monthly demands. The ratios of maximum day use to peak month use ranged from 1.4 to 2.7 for those utilities providing an estimate.

Because the results of the initial survey were insufficient to develop generalized relationships useful for design standards, a second survey was conducted in early 1995. Thirty-eight (geographically distributed statewide) water utilities were asked for more directed information. Under the logical premise that irrigation demands were strongly associated with residential lot size, questions were asked regarding specific metered residential accounts in service locations where residential densities could be determined (i.e. utilities were asked to provide actual meter data from 20 to 30 accounts located in portions of their service area which ranged from a low density of one or less services per acre to a high density of five services per acre). The year for which utility meter records were to be reviewed and assessed was 1993.

Twenty-six systems responded to the survey and presented information based on actual 1993 meter readings for residential accounts, and where possible, an estimate of the residential density (ranges requested were for one or fewer units per acre, two units per acre, three units per acre, four units per acre, and in some cases, five units per acre) for those locations in their systems from which the meter records were taken.

The analysis of this information provided somewhat more direct, and presumably more accurate, estimates of annual residential water demands. Since 1993 was unusual in that the summer period experienced higher than normal rainfall, the demand data were related to the rainfall records for that year rather than using average annual rainfalls.

Analysis of the relationship of water demand to lot size, although generally showing that higher demands were related to larger lot sizes, and that this aspect was especially pronounced for lots in excess of an acre as compared to higher density developments (especially in eastern Washington), was not supported by sufficient unequivocal data to allow formulation of quantifiable design relationships.

However, the design engineer is to be cautioned that the size of residential lots, especially in eastern Washington, is clearly influential on the expected water demands, particularly for lots larger than an acre in size. As much as 60 percent more water may be used by a residence on an acre-plus lot than on lots which are less than an acre. The engineer must be cognizant of this aspect, and will generally be held accountable for proper consideration of this factor, when estimating water demands for tracts with large lot sizes.

Additional (and considered reliable), information on residential water demands was also found through reviews of 28 Water System Plans (WSPs) which had received DOH approval in 1995 to early 1996. The information from these WSPs was specific to residential water demands associated with meter readings or from professional engineer estimates. These data were then related to the rainfall records information documented in the WSP or from data on file with the Meteorological Service for gauging locations proximal to the utility.

Additional information was also collected from a 1994 report prepared by the California Department of Water Resources (Bulletin 166-4, "Urban Water Use in CA," August 1994). Included in this report (which provided a wide array of recorded water use patterns specific to utilities or geographic areas in California) was some summary data for twenty selected utilities which associated a ten-year average annual demand (on a per capita basis) to average annual rainfall. Using a factor of 2.7 persons per Equivalent Residential Unit, estimates of the average annual demands for 19 of these utilities (in terms of gallons per day per ERU) were made and incorporated into the data set used for this demand analysis. (One utility, Palm Springs, had demands that were influenced so greatly by an abnormally large transient population that it could not be considered reflective of a true residential community, and was therefore not included in the data set).

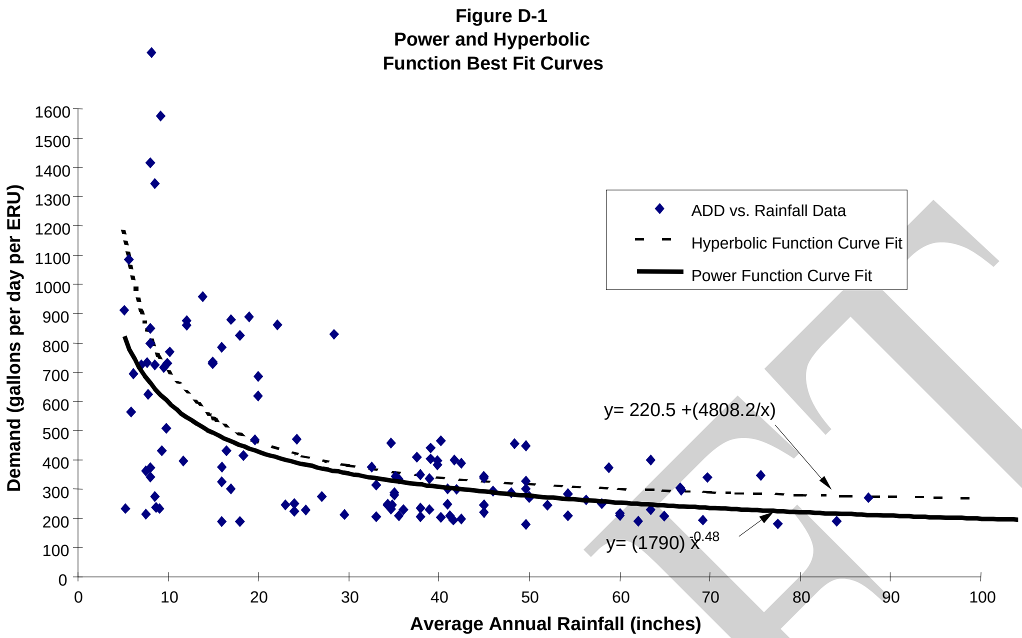
Data Analysis

The data (a total of 122 data points) were evaluated in an attempt to identify and characterize any discernable relationships. Although it was recognized that many factors exist which could influence residential water demands, with the exception of average annual rainfall there appeared to be insufficient information to draw relationships with any other factor which could be used as a numerical and rational basis for specifying design parameters. In earlier drafts of the revised design standards, factors were developed and proposed to account for water demand influences associated with residential density. From the data available there was clear evidence that lot size was related to water use in both Western and Eastern Washington. However, the data were limited and could not be reasonably applied to specific relationships descriptive of statewide observations. The impact of lot size was, therefore, not accounted by some design relationship, but was addressed as a qualitative aspect of design, which must be considered and addressed. For other factors (such as economic status, pricing structure, landscaping practices, conservation practices, etc.) which can be of significant influence on water demand, there was insufficient information to draw any relationships or qualitative conclusions. Some water systems may have in the past developed specific relationships between several of these factors and their water demands, but such relationships would be specific to an individual system and would not be applicable on a statewide basis, unless they could be verified through collection and analysis of additional and reliable information.

Development of Rainfall/Residential Demand Relationship

The data were plotted in an x-y scatter plot and visually inspected. From an examination of the plotted data, there seemed to be a generalized relationship between average annual demand for residential developments and average annual rainfall. It was apparent that use of a single value for demand estimates on a per household basis (as has been historically the practice), for the design of residential water systems was not particularly appropriate. A curvilinear function appeared to be more descriptive of average water demands when associated with such a climatic factor as average annual rainfall.

Accepting that the data could be better described by a curvilinear function, several different fitting models were used to develop best-fit curves for the data. Figure D-1 presents two best-fit curves, one based on a hyperbolic function, and a second based on a power function. Both provide similar fits to the data set, with correlation coefficients (R) of 0.49 and 0.61, respectively. Although these correlations are not as strong as one would like to develop basic relational equations, they were considered sufficient to allow acceptance of the general form of a function which could be used for water demand design criteria. The data scatter in the low rainfall areas contribute significantly to the marginal correlations with rainfall which points out the influence of other factors in determining average daily demands for residential populations.



In order to determine confidence intervals (C.I.s) for the mean of the data set, and more usefully in this application, prediction intervals for individual points, the data were transformed to develop a linear relationship. A log x-log y transformation provided a data set with a linear regression line corresponding to the best-fit power function curve. The linear regression line for the plot of y vs. 1/x corresponds to the hyperbolic function. The transformed data and appropriate C.I.s are presented in Figures D-2 and D-3, and were developed using SPSS statistical software.

Mean and Point Prediction Intervals at 60%

The centerline in Figures D-2 and D-3 represent the mean of the data set. The curved lines on each side of the center line are the 60 percent confidence bounds for the mean of the data, and the parallel lines at the outer portions of the data are the 60 percent Prediction Intervals for individual points. That is, based on the data available, and standard assumptions about the validity of that data as representative of the larger population, it can be said with 60 percent certainty that usage, as a function of rainfall, of any new data point will fall between the two outer, parallel lines. It is noted that although 60 percent represents a relatively marginal level of confidence, the notable data scatter in the low rainfall range biases these results.

Figure D-2: Log Transformation of Average Day Demand vs. Rainfall (Power Function)

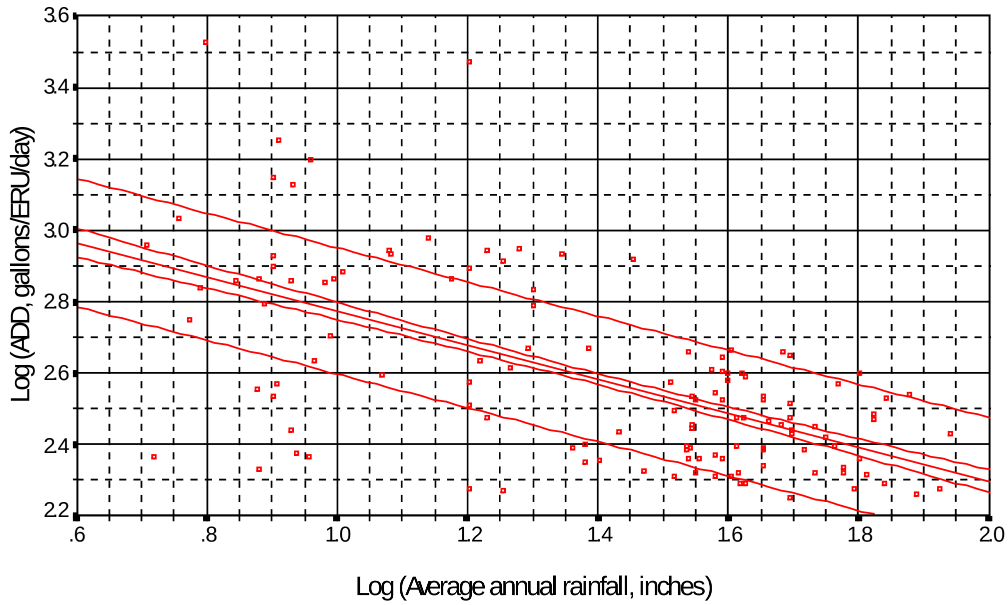
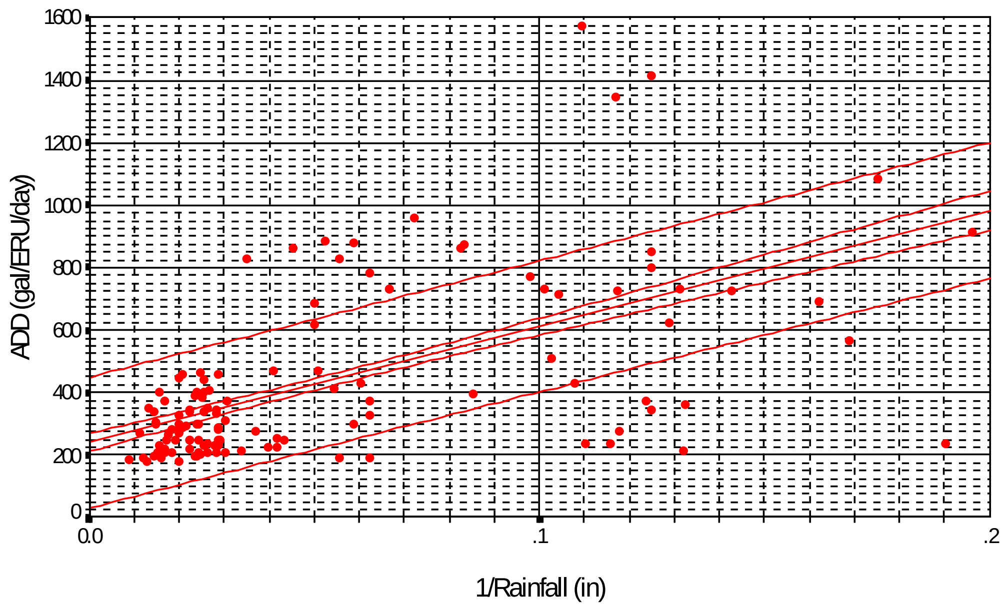


Figure D-3: Transformation of Average Day Demand vs. Reciprocal of Rainfall (Hyperbolic Function)



An upper level curve for the Power Function based on the 60% confidence boundary, when plotted back to arithmetic coordinates, indicates that 85% of all points are below the upper bound. For rainfalls averaging less than 30 inches per year, almost all points are below the upper bound.

Review of Figures D-2 and D-3 indicates that transforming the data based on a hyperbolic function (i.e. y vs. 1/x) provides a slightly poorer linear relationship than the power function. However, the difference was not considered of such significance that use of a hyperbolic relationship could be discounted.

Baseline Residential Water Demand

The data (shown in Figure D-1) shows an interesting aspect which appears to have general application and credence for baseline residential water demands. With only a few exceptions where a few data points can be seen to be lower, all data generally lied above a value of 200 gpd/ERU (i.e., at all rainfall levels, the average annual demands reported were greater than 200 gpd/ERU). This observation may be construed as a threshold level for residential demands which appear to be independent of average annual precipitation levels and may indicate the base level of demand associated with internal household (non-irrigation, etc.) uses. As such, the function which describes the relationship between ADD and average annual rainfall would be more strongly associated with external household uses (irrigation, lawn watering, etc.). Assuming this is the case, design requirements for total demands could be separated into two components - one related to internal uses and the other to external uses. For internal demands, a constant value independent of rainfall could be prescribed and for external demands, a relational function could be established which was dependent upon rainfall levels.

From the data, the single valued level for average annual household demands (internal uses), which would appear to apply statewide independent of rainfall, is about 200 gpd/ERU. Logic dictates that this demand may be consistent on an average annual basis, but cannot be expected to be uniform on a day to day basis. Residential households would be expected to experience peak demand days for internal uses associated with a number of factors. Peak day uses could be expected with increased water demands for showering in the summer, or when visitors or relatives are entertained. The actual levels associated with the peaking demand days would be dependent upon many variables. There were no known relational studies, or anecdotal accounts, that could be found which would assist in development of design parameters for internal household peaking uses. Nonetheless, in order to maintain consistency with stipulations of the state's Group B water system design criteria, and with the Department of Ecology, who in some instances provides estimates of peak day internal uses for water rights issues, a reasonable level for a Maximum Daily Residential Demand for internal uses can be established at 350 gpd/ERU (a value which can be seen is marginally less than double the average annual internal demand of 200 gpd/ERU previously discussed).

For projects that propose to have separate irrigation systems, the design of the potable (internal use) water system can be predicated on the estimate of 350 gpd/ERU. The irrigation portion of the system may be designed based on the respective needs of the customers, or by using the difference between the demand estimated for complete service (Maximum Total Daily Demand) and that for just the internal uses (Maximum Internal Daily Demand).

Selection of Design Functions for Residential Water Demands

In development of a functional design relationship which can be used for estimating the residential water demands in Washington State a number of approaches were examined:

- Based on the statistical features of the data set, a function that described the relationship associated with the upper bound of the 60% confidence interval could be used.
- The current approach that sets demand levels at constant values for Eastern and Western Washington could be retained. However, this "status quo" approach may not be particularly applicable based on a review of the data. There appears to be a trend better described by a continuous function rather than by a single, but separate, value ascribed to water system design simply because of gross climatic differences between East and West Washington.
- Another approach would be to establish a function that gives criteria higher than any recorded data to insure that, at least, the data set available was completely accounted in a highly conservative manner.

The foregoing approaches were all rejected under criteria that were believed appropriate to guide the design function selection process. It was considered reasonable and prudent to establish an approach that would provide for a relationship that was patterned to the "best-fit" curves developed for the data that were sufficiently conservative so that reasonable confidence could be placed on the use of the design relationship (i.e., the function would describe demands that were in excess of at least 80% of the recorded data), that the relationship would be as simple as possible to use and understand, and that the relationship would be asymptotic to a baseline demand of 200 gpd/ERU.

In addition, based on the wide range of reported data in the low rainfall range which showed some, but very few, systems that experienced very high average annual demands (> 1000 gpd/ERU), it was determined appropriate to establish an upper boundary of 1,000 gpd/ERU for any relationship (function) that was developed.

Under these criteria, two functions were developed, one a power function and the other a hyperbolic function, which were asymptotic to the 200 gpd/ERU lower boundary and which were presented in very simplistic terms. Another function was also developed, which does not show an asymptotic boundary associated with the 200-gpd/ERU level, but does parallel the best-fit power function relationship used for the previous data analysis. Each of these functions is conservative in that 80% or more of the data would lie below the curves describing the functions. Presented in Figure D-4 are three graphical relationships with their associative functions. One hyperbolic relationship and two power function relationships are presented, any of which may be used to estimate residential water demands throughout the state when no other better information is available or applied for design.

Although the power function relationship may have somewhat greater statistical strength, the relatively high conservative nature of these functions would allow for any of them to be used for design purposes. Since the hyperbolic function provides more conservative estimates at lower rainfall ranges, and is possibly the simplest to use and understand because of its arithmetic nature, it was selected as the function of choice for estimates of average annual residential demands used for project designs **when more appropriate information is not available.**

Maximum Day Demand

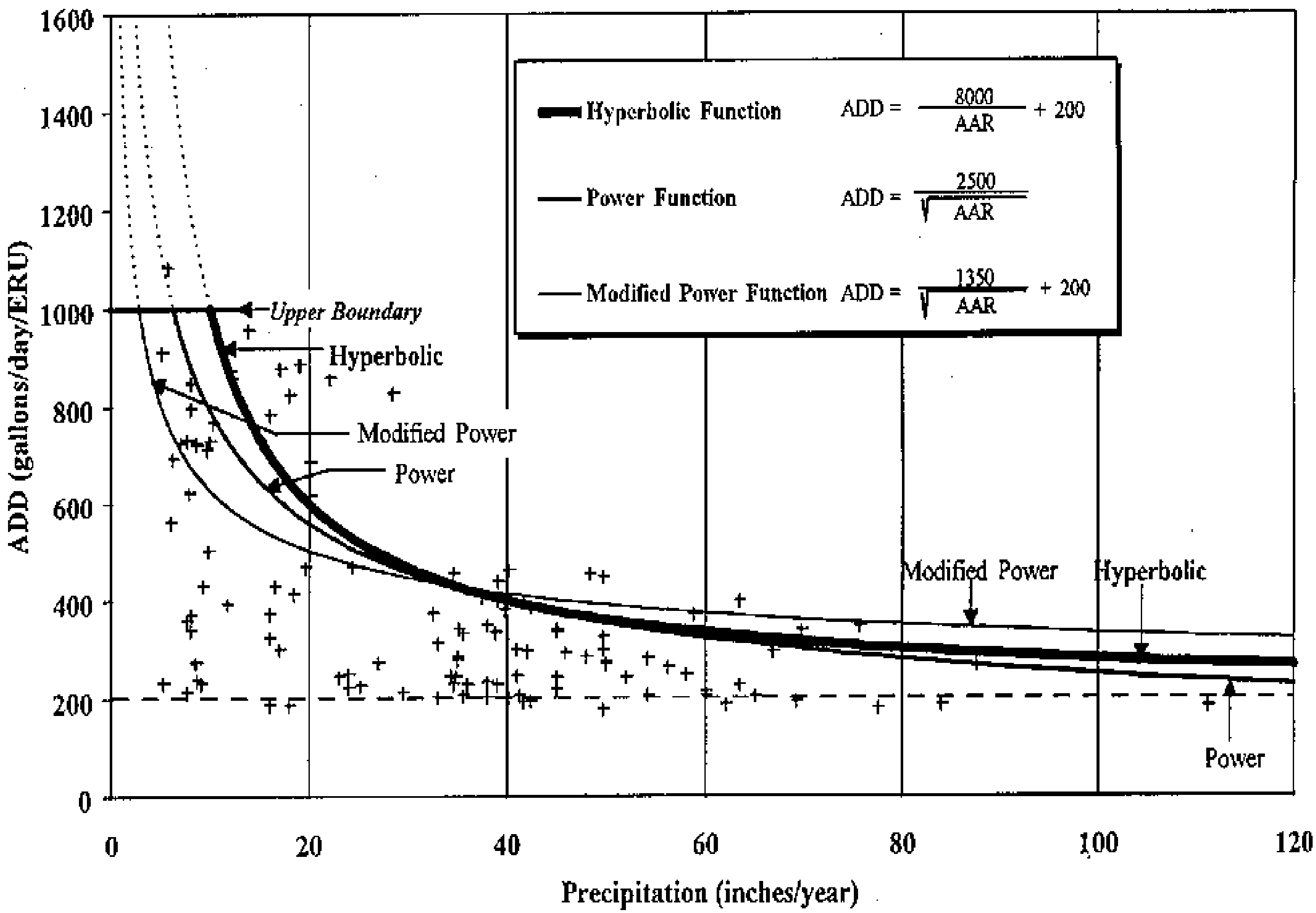
A variety of peaking factors have been reported in the literature and within the data collected for this analysis, but generally the Maximum Daily Demand (MDD) is 1.5 to 3 times average daily demand. By selecting an appropriately conservative approach to estimating ADDs (as was done in this analysis), use of a standard peaking factor of 2.0 was considered to be adequately conservative. MDD can therefore be calculated by multiplying ADD values by a factor of 2.0. Again, an upper maximum level would be been established based on the upper boundary for the average annual demand (1000 gpd/ERU). The MDD value would be 2000 gal/day/ERU as an upper bound. The absolute lower limit MDD values, as previously discussed, are set at 350 gpd/ERU (for developments without irrigation or with restrictions on the external use of water).

Limitations of This Analysis

It is clear from inspection of the graphs presented in this appendix that the data varies widely, and the existence of many other factors that affect both average annual and peak daily water use have been acknowledged. **The intent of this document is to ensure that new systems, or system improvements, are designed based on reasonable and conservative criteria when there is an absence of sufficient production and use data to allow other design parameters to be used.** The approaches presented here reflect this philosophy, and as such, have tried to use relatively sparse data in a reasonable and judicious manner. The water demand design criteria contained in the Design Manual (Chapter 3) represent an improvement over what has historically been used in the state. In the future as more and better information becomes available, even greater refinement of the approaches can be expected.

FIGURE D-4

Functional Relationships Describing Average Residential Demand Relative to Average Annual Rainfall Levels



Appendix D.2 Estimating Nonresidential Demand

Table 3-2 in Chapter 3 of this manual provides guidance on estimating nonresidential maximum daily demand (MDD). Design engineers should apply the following assumptions when using the values presented on Table 3-2:

- Unit nonresidential demand will vary little from day to day.
- MDD is based on a full facility (the campsite or hotel is fully occupied or the school is operating at capacity).

Tables 1 and 2 below provide guidance on establishing nonresidential PHD using the fixture method.

Table 1
Demand Weight in Fixture Units

Fixture Type	Weight in Fixture Units per Fixture Type
Shower	2
Kitchen sink	1.5
Urinal	3
Toilet (flushometer)	5
Toilet (tank flush)	2.5
Bathroom sink (lavatory)	1
Clothes washer	4.0
Drinking fountain	0.5
Dishwasher	1.5
Hose Bibb	2.5

Source: Adapted from the 2009 Uniform Plumbing Code, Appendix A, Table A-2

After determining the total number of fixture units (sum of fixture type times fixture weight), round the total to the next value given in Table 2, and determine the peak hourly demand.

Table 2
Nonresidential Peak Hourly Demand

Total Number of Fixture Units	PHD (gpm)
10	8
15	12
20	15
25	18
30	20
35	22
40	25
50	29
60	32

70	35
80	38
90	41
100	43

Source: Adapted from the 2009 Uniform Plumbing Code, Appendix A

Example

A proposed catering business in Western Washington will employ 20 daytime employees and have no visitors. The proposed building will have its own drinking water system, using a permit-exempt well. The area around the building to be irrigated is 3,000 sq. feet. A fire pond filled by a nonpotable water supply will meet the building's fire-suppression requirements. The fire pond and associated fire-suppression piping have no physical connection with the potable water system.

Step 1: Apply the fixture weight to each fixture type (Table 1), and determine the building's total fixture units.

Fixture Type	Number of Fixtures x Fixture Weight = Fixture Units		
Drinking fountain	2	0.5	1
Toilet (tank flush)	4	2.5	10
Urinal	1	3	3
Lavatory	2	1	2
Kitchen sink	2	1.5	1.5
Dishwasher	2	1.5	3
Total			22

Step 2: Round the total fixture units from 22 up to 25 (the next increment in Table 2).

Step 3: Use Table 2 to establish peak hourly demand for internal use within the building. Nonresidential internal PHD is 18 gallons per minute. If the irrigation system is not operated while the building is occupied, then the estimated design PHD should be 18 gpm. If the irrigation system can be operated while the building is occupied, then the design estimate for PHD should include both the internal PHD (18 gpm in this example) plus the peak flow rate of the irrigation system.

Appendix D.3 Deriving Maximum Daily Demand to Maximum Month Average Daily Demand Ratio

We undertook an analysis of daily water treatment plant data in an attempt to establish a maximum daily demand (MDD) to maximum month average daily demand (MMADD) ratio. For the purpose of this analysis we assumed production was equivalent to total water system demand.

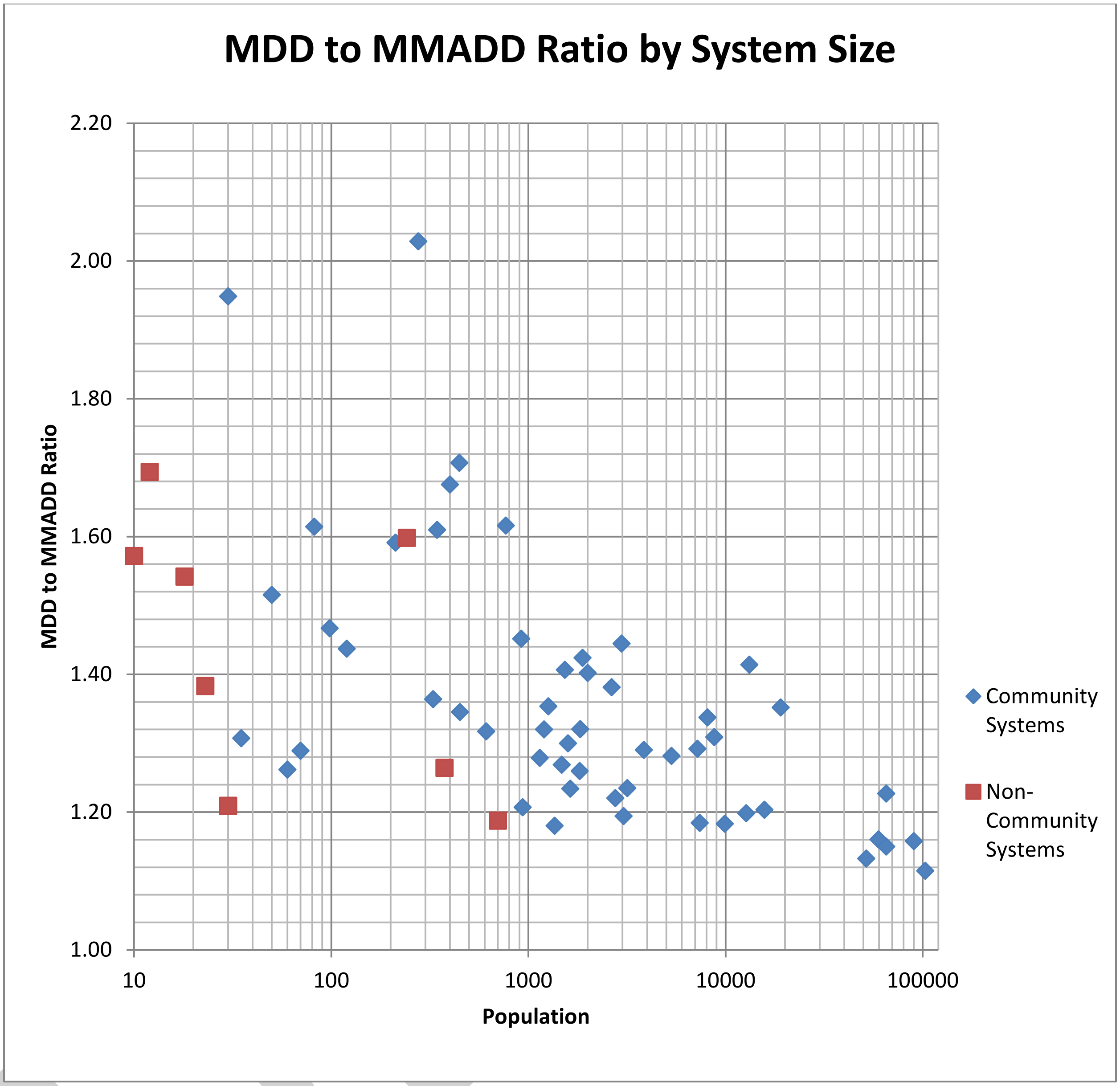
The guidance presented in Section 3.4.1 is based on our analysis of the source production records for 79 water systems using surface water in Washington State for the period 2009 through 2014. These systems operate surface water treatment plants and are required to record and report daily source production. The surface water treatment plants are the only routine supply for these 79 water systems, and we used their net treatment plant production information as a proxy for daily system demand (ignoring changes in storage).

We identified the maximum month of production for each year and calculated the MMADD for that month. We then identified within that month the MDD, and for each year and each system calculated the MDD to MMADD ratio. Significant statistical outliers - those outside two standard deviations - were removed from the data set.

In general, we found strength in correlation between increasing system size and decreasing MDD to MMADD ratio for water systems serving more than **1,000 people**. The larger the system the more likely it is that the system has access to its own daily source production data. But if such primary source production data is unavailable, our analysis of the 80^{th} percentile value indicates a MDD to MMADD peaking factor of 1.35 for systems serving 1,000 to 100,000. We believe applying a 80^{th} percentile threshold for design is appropriate in the absence of daily source meter data. The location of the water system was not seen as a factor.

For the 28 water systems serving fewer than **1,000 people**, we found a weak correlation between population served and the MDD to MMADD ratio. In addition, as with the larger system cohort, system location did not matter. From the primary smaller system production data, we determined the 80^{th} percentile value of 1.65 for the MDD to MMADD ratio.

In the 2009 version of the Water System Design Manual we recommended a MDD to MMADD ratio of 1.7 for water systems in Western Washington and 1.3 for water systems in Eastern Washington. This recommendation was based on review of a single year of data from 35 water systems. As described above, updated analysis is based on 79 systems reporting water production over a five-year period. Based on this expanded data set we observed the MDD to MMADD is more strongly correlated to system size and not to system location.


MDD to MMADD Ratio by System Size
MDD to MMADD Ratio
2.20
2.00
1.80
1.60
1.40
1.20
1.00
10
100
1000
10000
100000
Population
Community Systems
Non-Community Systems

Appendix E: Recommended Pumping Test Procedures

1.0 Introduction

This pumping test guidance is intended to provide Group A water systems with basic information suitable to develop an approach to satisfy source approval requirements in WAC 246-290-130 (3)(c)(iii) and (3)(d). WAC 246-290-130(3)(c)(iii) states that one acceptable approach for demonstrating source reliability is to conduct a pumping test at the maximum design rate and duration.

This guidance provides information on how to establish the physical capacity and reliability of a new or redeveloped well for the proposed use. This pumping test guidance does not address the legal adequacy or availability of water. The Department of Ecology administers a permitting process for appropriating and using the state's water resources. Local government is tasked with determining water adequacy for the purpose of making local land use and building permit decisions. Contact Ecology and the local government for requirements necessary to demonstrate legal adequacy and availability.

The principal objective of a source approval pumping test is to establish a reasonable estimate of the well's sustainable yield and specific capacity. Once established, these values enable the proper selection and positioning of the well pump in the well.

This pumping test guidance includes:

- Basic approach to pumping tests (Section 2.0)
- Pumping test planning elements (Section 3.0)
- Pumping test methods and analysis (Section 4.0)
- Special aquifer settings and considerations (Section 5.0)
- Pumping test results (Section 6.0)
- Reporting (Section 7.0)
- Potable water supply samples (Section 8.0)

The recommendations presented here represent the minimum criteria for obtaining reliable, useful, and verifiable data. Some hydrogeologic settings may require investigative efforts that are beyond the scope of this guidance. The design engineer should consult with a licensed hydrogeologist wherever site conditions warrant expertise in the design and evaluation of the pumping test.

As a rule, the more thorough the pumping test and comprehensive the data collection and analysis the greater the confidence in the assessment of safe yield and specific capacity. It is incumbent upon the design engineer to determine the appropriate scope, duration, and data collection and analysis for each new or rehabilitated source. In so doing, the design engineer should identify and address the degree of uncertainty through application of a suitable factor of safety, water supply contingency planning, and/or the development of redundant water supply facilities.

2.0 Basic Approach to Pumping Tests

For most aquifer conditions source reliability can be adequately defined by conducting step-drawdown and constant-rate pumping tests. If another pumping test approach is considered, contact your DOH regional engineer in advance of submitting your pumping test plan. DOH recommends conducting pumping tests during periods when the water table is at its lowest (e.g., summer and fall months). Bailer tests, air lift tests, and slug tests are not acceptable.

2.1 Step-Drawdown Pumping Test

A step-drawdown test is a single well pumping test designed to assess a wells performance. The test is the most reliable method for determining the pumping rate and pump setting and can provide a maximum design pumping rate to be used for the constant-rate pumping test. The test yields information primarily concerning the performance of the well rather than aquifer properties and does not require observation well data. The step-drawdown test will likely not identify impermeable boundaries, recharge boundaries, interferences from other wells, or conditions of groundwater under the influence of surface water, unless these conditions exist in very close proximity to the well being tested. For the step-drawdown test, the discharge rate in the pumping well is increased from an initially low constant-rate through a sequence of pumping intervals (steps) of progressively higher constant-rates.

The step-drawdown test may not be needed in settings where water quality and water reliability have been established. Such as a situation where a low yield source is located in an aquifer where existing wells have repeatedly shown to produce high-yielding wells with very little drawdown. Proceeding to a constant-rate test without conducting step-drawdown test should be discussed with DOH for approval prior to the start of the pumping test.

An extension of the step-drawdown test in place of the constant-rate test may be suitable for a low yield source approval where the last step is extended for a period longer than the previous steps duration until stabilized drawdown is achieved. Caution should be taken when satisfactory stabilized drawdown is achieved then declines due to poor recharge characteristics of the surrounding material. In such situations the constant-rate tests is required. The ability of the water level to recover should also be observed. If the water level fails to return to 95 percent of static water level as measured prior to conducting the low yield source step-drawdown test after 24-hours, the reliability of the producing zone is open to question and additional constant-rate testing is required. An explanation and justification of this test will need to be provided to and approved by the DOH.

2.2 Constant-rate Pumping Test

The constant-rate pumping test is performed by pumping a well at a constant-rate and measuring drawdown versus time. Typically one or more observation wells are used or installed at an appropriate distance from the pumping well to measure water levels. The constant-rate discharge test would be conducted at the pump setting and maximum design pumping rate determined from the step-drawdown test. Prior to the constant-rate test the aquifer should be allowed to recover to within 95 percent of the static water level as measured prior to conducting the step-drawdown test.

2.2.2 Recovery Phase

Water-level measurements obtained during the recovery phase of the constant-rate test are of equal or greater importance than those collected during the pumping phase because they can confirm disturbances to flow. The recovery phase is not subject to fluctuations such as discharge rate variations or well losses. Section 4.0 presents details concerning these data.

3.0 Planning a Pumping Test

A pumping test plan should be prepared and discussed with DOH for endorsement prior to the start of the test. This planning step supports efficient use of client resources in satisfying the source approval process and in achieving test results supporting conclusions on the safe yield and pump setting depth. Sufficient information about each element, including references and rationale for the planning and design decisions, should be collected, documented and discussed.

The proposed pumping well construction should be finalized prior to the pumping test plan. A pumping test should be designed to reflect the hydrogeology of the site, the hydraulic parameters to be determined, the defined test endpoints, and the degree of confidence. A comprehensive outline for preparing a pumping test plan is provided in the following sections.

3.1 Well Site Description

Existing data should be used to develop a source area description and should include:

- A map and description of the proposed well location, property ownership, topography, land use, known or potential sources of contamination in the vicinity (e.g. hazardous waste sites, sewer lines), and other pertinent related details. Surface water bodies including wetlands, irrigation channels, creeks, streams, rivers, lakes, ponds, estuaries, and coastal waters in the vicinity of the source area should be described. Bedrock outcrops, faults and other potential boundaries should also be described.
- Information about wells located within ½ mile of the source should be summarized in a table (location, date installed, elevation, depth, casing length, screen interval, aquifer where the well is screened). If available, copies of the well logs should be obtained.

- Vicinity, topographic, and other maps depicting the source area features and existing well locations should be included with the source area description.
- Discussion on the aquifer conditions (e.g. confined, semi-confined, unconfined), thickness, lateral continuity, and special aquifer conditions as defined in Section 5.0
- Relevant aquifer/aquitard characteristics including soil/rock type; depth to, thickness, and areal extent of aquifer and aquitard units; known or suspected boundaries, water level data; type of aquifer (confined, leaky, unconfined), and hydraulic properties (hydraulic conductivity, transmissivity).
- Distance from the site to surface water bodies located (within one mile).

3.2 Well Construction and Condition

Include the following in a pumping test plan for a proposed well:

- Drilling method
- Ground surface elevation and method of determination
- Annular (surface) seal material, depth, and thickness
- Intended well diameter, depth, and casing
- Intended well screen type, screen interval(s), and length/depth

In addition to the above items include the following for an existing well:

- Well log
- Well development procedures
- Past pumping test results
- Past seasonal static water level and pumping level
- Documentation of problems with well performance
- History of well rehabilitation, cleaning, and redevelopment
- Present condition of casing, well screen, or perforations

3.3 Data

Provide test data reporting sheets, interval of data collection, means of collection (written or by automated data logger), who will be running the pump test/collecting data, and measuring device specifications, including:

- Clearly define the pumping test objective using definable endpoint(s) (e.g., demonstrate sustainable yield equals or exceeds 500 gpm/0.7 MGD and determine appropriate pump setting depth during crop irrigation season while S01 and S03 are pumping; based on a 24-hour constant-rate pump test).

- Identify and describe data gaps that need to be filled prior to and during the pumping test. (e.g., how will data collected during high groundwater conditions inform the design so that the operational pumping rate and pump setting of the water source are suitable for the dry season?).

- Assess the confidence level obtainable with the proposed pumping test and how the level of uncertainty impacts the overall design of the water system. (e.g., *will data collected provide a single source that will meet the desired demand or will storage capacity be required for the overall design?).*

3.4 Pumping Test Mechanics and Field Procedure

Describe water level and discharge measurement procedures, including measuring device(s), device calibration and accuracy, and the frequency of monitoring. A template of the field data sheets to be used is provided as an attachment to this Appendix. Describe the capacity of the pump to be used in the pumping test and the rationale for its selection. Provided a disposal plan for discharged water and potential impacts to the pumping test.

4.0 Recommended Pumping Test Methods and Procedures

The following components require consideration when designing a pumping test. The information presented is not intended to be a resource for all aspects of pumping tests but the considerations necessary to collect useful, reliable data. The pumping tests should be conducted after full recovery from pumping that occurred prior to the start of the pumping tests. To reproduce the anticipated stress on the aquifer, the pumping test should take place when nearby wells are operating under normal conditions. Likewise, the pumping test plan should reflect multiple wells in a wellfield intended to be pumped simultaneously to meet the design source production.

No pumping should be conducted at or near the test site for at least 24 hours prior to the test. If on-site or nearby pumping cannot be curtailed due to system supply needs or other factors, this should be noted and discussed as it relates to the test accuracy. If an interruption in the pumping test occurs, the interruption needs to be demonstrated to have no significant effect on the data. If the interruption is longer than two hours the test should be terminated, the water level allowed to fully recover, and test restarted.

4.1 Stabilized Drawdown

Stabilized drawdown is when a water level has not fluctuated by more than plus or minus 0.5 foot for each 100 feet of water in the well (i.e., static water level to bottom of well) over the duration of constant flow rate of pumping. See Section 4.3.2 for guidance on the duration of a constant rate pumping test under various aquifer conditions, and guidance on the minimum duration of the stabilized drawdown phase the test. Plotted measurements of drawdown should also not show a trend of decreasing water levels.

Stabilized water level can often be misjudged due to varying hydrogeologic factors and the basis for stabilized water levels provided is the minimum required. Final judgement for stabilized drawdown is ultimately determined by the hydrogeologist or engineer in charge of the pumping test.

4.2 Observation Wells

The use of observation wells provides more representative and accurate data during the pumping test. Wells in the vicinity of the pumping well should be considered for observation well monitoring. Pre-test analysis of well depth and distance can determine the best wells to use as observation wells. Observation wells with an open interval in the same aquifer as the test well are desired. If the aquifer being evaluated is confined it may be useful to select additional observation wells completed within the overlying or underlying aquifer to determine whether there is leakage into or from the confined water system. For saltwater intrusion concerns, observation wells between the pumping well and saltwater body can provide the most useful information. Wells with floating food grade lubricant or other product should not be used for pumping test evaluation, unless there is a shortage of observation wells at the site. The floating lubricant may introduce measurement errors and provide misleading results. If wells with floating food grade lubricant are used, the density of the product should be estimated to calculate the equivalent head in the well.

4.3 Test Duration

The length of a pumping test depends on the purpose of the test and hydraulic properties of the aquifer. This guidance does not support the historical concept of a 4-hour pump test.

4.3.1 Step-Drawdown Test

The step-drawdown test procedure should consist of a minimum of four constant-rate tests each run successively at increasingly higher flow rates until stabilized drawdown occurs. Testing of the well begins at a low constant discharge rate until the drawdown in the well stabilizes then the process is repeated at successively higher pumping rates through at least four steps. The duration of each step is dependent on achieving stabilized drawdown. Each step is typically of equal duration, lasting from approximately 30 minutes to 2 hours (Kruseman and de Ridder 1994). It is important to run the initial step long enough to establish that the effects of well storage have dissipated.

4.3.2 Constant-Rate Test

The length of time required to conduct a constant-rate pumping test depends on the type of aquifer, distance to boundary conditions and the level of accuracy needed to estimate hydraulic characteristics. A confined aquifer should be pumped a minimum of 24 hours while an unconfined aquifer be pumped for a minimum of 72 hours. A 72 hour or longer pumping test is also required for situations where the effects of pumping on aquifers with fracture flow, aquifers of limited areal extent, sea water intrusion, delayed yield or slow drainage effects.

The minimum pumping test durations may need to be extended if a delayed yield or slow drainage effect that will influence the pumping test interpretation is anticipated. While methods are available to analyze fluctuating data, it is good practice to achieve stabilized water levels, especially when accurate information concerning aquifer characteristics is desired. Stabilized drawdown for a constant-rate test should be established for a minimum duration of at least six hour period at the end of the minimum test duration. If stabilized drawdown is not achievable within a reasonable period of time the test should be stopped, the water level allowed to recover to its original static water level, and a new test at a lower pumping rate be conducted.

4.4 Pumping Rate

It is very important that the pumping rate be held constant during each phase of a pumping test. The pumping rate should be monitored to ensure the rate is maintained within 10 percent of its starting value. Fluctuations in pumping rate make the test analysis difficult and raise questions as to whether deviations in the data are actually a result of flow boundaries or other hydrogeologic features. Control of the pumping rate is often best accomplished by accurately measuring and controlling the discharge rate.

4.4.1 Step-Drawdown Test

Pumping rates used in a step-drawdown test should encompass the maximum design pumping rate for the well (*Q* max) and be based on the pumping rate of nearby wells completed in the same aquifer or geologic unit.

The recommended scheme for the pumping rates in a step-drawdown test consisting is shown in Table 1.

Table 1
Step-Drawdown Pumping Rates

Step	Rate
1	0.5 Q max
2	0.75 Q max
3	Q max
4	1.25 Q max

4.4.2 Constant-Rate Test

The constant-rate pumping test should be performed at or above the maximum design pumping rate determined from the step-drawdown test results and for which approval will be sought in the water-supply application.

4.5 Water-Level Measurements

Water-level measurements should be recorded to the nearest 0.01 foot. The well to be tested should be at its static water level prior to the test. Static water levels at the pumping well and observation wells should be measured at least daily for one week prior to the start of the test.

Water level measurements of the pumping test well should also be made at 24, 16, 12, 3, 2, and 1 hours prior to initiating pumping. Within the hour immediately preceding pumping, water level measurements should be taken at 20-minute intervals to establish short-term trends in water level changes that may be occurring. Immediately before starting the pump, water levels should be measured in observation wells and in the pumped well to determine the static water levels upon which drawdowns will be based. These data and the time of measurement should be recorded.

The start of the pumping test should be recorded as time zero. It is important in the early part of the test to record with maximum accuracy the time at which readings are taken. The chosen time schedule should be followed as closely as possible. If the schedule is missed the actual time of a reading should be recorded. Estimating drawdown readings to fit the schedule may lead to erroneous results. Readings in the pumping well and observation wells need not be taken simultaneously as long as the schedule is generally followed and exact time the readings are taken is recorded.

The greatest number of measurements are made within the first 100 minutes when the water levels are changing rapidly. The time intervals given are suggested minimums; more frequent measurements can assist with pumping test analysis and interpretation. Due to the frequency of measurement required during the initial portion of the test, electronic water level indicators marked in tenths and hundredths of a foot or data loggers should be used. Data loggers and pressure transducers provide efficiency and ease and are recommended.

Barometric measurements of atmospheric pressure should be made at the same water level measurement intervals. These measurements will allow appropriate corrections to be applied to the drawdown data. In settings where tidal influences may affect the pumping test results, measurements should be made at a frequency sufficient to correct the pumping test drawdown data for observed tidal influences.

4.5.1 Step-Drawdown Test

Recording water levels during a step-drawdown pumping test should start for each change in rate. A schedule appropriate for recording water levels during a step-drawdown test is provided in the table below.

Table 2

Time Intervals for Measuring Water Level and Recording Data
Step-Drawdown Pumping Test

Time after the start of each step in the drawdown test and after pump shut off for recovery data	Time intervals to measure water levels and record data
0 to 10 minutes	0.5 minutes
10 to 15 minutes	1 minutes
15 to 30 minutes	2 minutes
30 to 60 minutes	5 minutes
60 to end of step*	10 minutes

*End of the step may be extended for a low yield source

4.5.2 Constant-Rate Test

Before the constant-rate test is started, sufficient time should be allowed for water levels to return to static conditions. As a general rule, the aquifer should be allowed to recover to within 95 percent of the static water level measured prior to the test. Adherence to the time schedule should not be at the expense of accuracy in the drawdown measurements. A schedule appropriate for recording water levels during a constant-rate pumping test is provided below.

Table 3
Time Intervals for Measuring Water Levels
Constant-Rate Pumping Test

<table>
<tr><th rowspan="2">Time after pumping started for constant-rate test and after pump Shut off for recovery</th><th colspan="2">Time intervals to measure water levels and record data</th></tr>
<tr><th>Pumping Well</th><th>Observation Well</th></tr>
<tr><td>0 to 10 minutes</td><td>0.5 minutes</td><td rowspan="4">2 minutes</td></tr>
<tr><td>10 to 15 minutes</td><td>1 minutes</td></tr>
<tr><td>15 to 30 minutes</td><td>2 minutes</td></tr>
<tr><td>30 to 60 minutes</td><td>5 minutes</td></tr>
<tr><td>60 to 120 minutes</td><td rowspan="3">30 minutes</td><td>5 minutes</td></tr>
<tr><td>120 to 240 minutes</td><td>10 minutes</td></tr>
<tr><td>240 to 360 minutes</td><td>30 minutes</td></tr>
<tr><td>360 minutes to end of test</td><td>60 minutes</td><td>60 minutes</td></tr>
</table>

4.5.3 Recovery Data

When pumping is stopped water levels rise toward their pre-pumping levels. The rate of recovery provides a means for calculating the coefficients of transmissivity and storage. In addition, the recovery phase is not subject to induced variations and can provide more reliable information. The time-recovery record, therefore, is an important part of an aquifer test.

Recovery measurements should begin immediately upon pump shut down and be collected using the same procedure and time pattern followed at the beginning of the pumping test. Recovery data should be recorded for at least 12 hours or until recovery to the static water level. Water level measurements should be made to the nearest 0.01 foot, and also be collected in all measured monitoring wells. A check valve should be used to prevent backflow of water in the riser pipe into the well, which could result in unreliable data.

4.6 Surface Water

Fluctuations in surface water stages (or stream flow) for all surface waters within 500 feet of the pumping well should be measured. Measurements should be made using, as appropriate: weirs, staff gages, piezometers, etc. Weir flow measurements should be conducted for small streams. The horizontal distance between each observation point and the pumping well should be measured to the nearest foot. The vertical elevation of a fixed reference point on each observation point should be established to the nearest 0.01 foot and reported in NAVD 1988.

Measurements should be read and recorded at least once daily for one week prior to the start of the test and at least twice per log cycle after the first ten minutes for the duration of the test. Measurements should be made more frequently if surface water levels are changing rapidly.

4.7 Conveyance of Pumped Water

The design engineer should carefully review applicable requirements and permits for discharging pumping test water. The water being pumped from the well should be disposed of legally and the discharge practices should follow local, state and federal rules and regulations. Water discharged during the pumping test should be conducted away from the pumping well in a down gradient direction and at sufficient distance to eliminate recharge of this water to the aquifer.

The objective of conveying pumped water as far from the site as possible is to minimize the possibility of artificially recharging the aquifer and producing an erroneous pumping test or affecting the later stages of the test. There is no fixed rule on how far the water produced during the pumping test should be discharged from the vicinity of the well. It is best to pipe the water outside of the area likely be influenced by the pumping test. Water conveyance is particularly important when conducting pumping tests in shallow unconfined aquifer settings. Considerations for determining a suitable distance include:

- If the aquifer is confined: less distance will be necessary.
- The duration of the pumping test: the shorter the test, the less distance necessary.
- Depth to water and nature of geologic materials overlying the water producing materials: the greater the depth to water, the less distance necessary; and, the more transmissive the aquifer materials, the greater distance necessary.
- If possible do not discharge conveyed water between the test well and observation wells or suspected flow boundaries.

5.0 Concerns in Special Aquifer Settings

Several aquifer settings have been identified as having reliability concerns including water quality concerns and water quantity concerns. These aquifer settings do not require a different pumping test approach but require longer tests, more rigorous analysis of the test data and additional pumping test design details. Because of the greater difficulty and complexity in pumping test design and evaluation presented by these settings, consultation with a licensed hydrogeologist is advisable. The design engineer should also contact the DOH regional engineer prior to developing the pumping test plan for special aquifer settings. Elements and concerns unique to these conditions are discussed below.

5.1 Low-Flow Conditions

Low-flow conditions may be encountered in wells completed in aquifers with low transmissivity. In these situations the ability of the aquifer to yield the required volume is of concern. Observation wells are generally not necessary because effects of pumping in low-flow conditions are typically not far reaching.

5.2 Fracture Flow

Typically, sources completed in bedrock or consolidated material may have fractured flow. The continuity of fractures can vary significantly within an aquifer and affect the aquifers ability to provide water in a consistent manner. Restrictive conditions identified by the pumping test could include lack of stabilized drawdown that could signify that recharge does not occur at a rate sufficient to maintain the desired pumping rate or the presence of multiple recharge boundaries.

There is also a possibility that the rate of drawdown decreases with time. This effect would suggest that a recharge boundary was encountered. An extension of the pumping test duration and additional observation wells may be required for determining any impact the fracture flow has on long term well yield. In order to obtain the appropriate data, the goals of a bedrock source pumping test and fracture density should be carefully considered before the test duration and observation well network are planned.

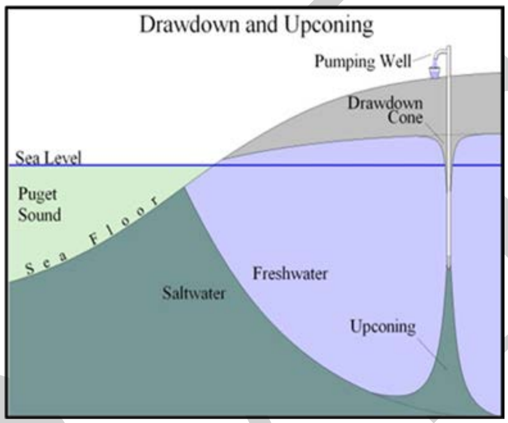
5.3 Aquifer of Limited Areal Extent

Aquifers of limited areal extent present similar concerns as fracture flow. Wells in this setting may provide a reliable source of water initially, but the limited areal extent and recharge capacity of these aquifers result in a reduced volume of water each day over the long term. These aquifers are commonly made up of highly variable material that can impact the ability to transmit water. The pumping test should be designed to identify recharge or impermeable boundaries. Observation wells should be used so that the coefficient of storage can be determined as accurately as possible.

5.4 Seawater Intrusion

When water is pumped from an aquifer located in a coastal area, the gradational boundary between seawater and freshwater will move in response to the pumping (Culhane, 1993). The location of the well, aquifer characteristics, and amount of the groundwater withdrawn impact the rate and extent of seawater movement. In addition to changing the lateral boundary between fresh and seawater, large withdrawals in individual wells can also cause underlying seawater to migrate upward into the well, which is referred to as "upconing" (Island County, 2005) (See Figure 1).

Figure 1
Seawater Upconing



An assessment of inducing seawater into the pumping well or nearby wells is required and the pumping test should be designed to establish a rate at which stabilized drawdown is achieved without an associated increase in chloride levels in the pumping well or observation wells. A minimum of one observation well, positioned between the pumping well and the shoreline, is highly recommended to assess seawater intrusion.

Water quality tests conducted during the pumping test in potential seawater intrusion areas differ from the other aquifer settings primarily in that chloride, total hardness (as $CaCO_3$), and specific conductivity are monitored in both the pumped wells and observation wells. The specific sampling intervals need to be outlined in the pumping test design. Seawater is not the only potential source of chloride in groundwater (Culhane, 1993). Highly mineralized groundwater (hard water) can contain high levels of chloride. Other possible sources of chloride include septic tank effluent, windblown sea spray, agricultural practices, recharge from irrigation, and well disinfection. Testing for hardness will help determine whether elevated chloride concentrations are a result of water hardness or possible seawater intrusion.

The water quality indicators can be monitored in the field using instruments or test kits specific to these parameters. Water quality measurements are used to determine whether concentrations are increasing, potentially signifying that saltwater is being drawn towards the pumping well. Field tests are good screening tools but DOH recommends submitting some samples to an analytical laboratory to verify the field test results. Once the well is put into full time use, continue to monitor for chloride, total hardness, and specific conductivity to identify trends and maintain potability. Tidal influences need to be evaluated prior to conducting the pumping test to determine whether the drawdown data needs to be adjusted (Kruseman and de Ridder, 1991).

5.5 Multiple Wells/Well Fields

This setting refers to two or more wells completed in the same aquifer that will be pumped either alternately or concurrently. DOH's primary concern in this setting is whether the new well interferes with other wells or with aquifer recovery.

In situations where a pumping test has been conducted for an existing well and data was also collected from an observation well(s), the potential for well interference due to adding another well can be determined using a distance-drawdown graph and evaluating additive drawdown for the pumping wells. This is based upon the principle of superposition where the drawdown at any point in the area of interference caused by the discharge of several wells is equal to the sum of the drawdowns caused by each individual well.

For cases in which an applicant is seeking approval for multiple production wells, all wells should be monitored during the test. In addition, the test should be conducted in a way that will obtain information pertinent to the operational needs of the wellfield. Pumping test design should reflect intended operating conditions of all wells. Conducting an additional pumping test exclusively on the new well would provide little new information beyond validating the findings of the initial pumping test. In general, an evaluation of potential well interference for either cyclical or concurrent pumping can be determined using the additive drawdown approach. If an observation well was not used during the pumping test the same approach can be used, however, the results will likely be less accurate in predicting well interference.

DOH considers each source approval in a multiple well setting to be considered on a well by well basis. It is recommended that the purveyor contact DOH to discuss a pumping test approach prior to conducting the test.

5.6 Groundwater Wells Potentially Under the Direct Influence of Surface Water

Wells identified as potentially being under the direct influence of surface water may need a pumping test to determine if a hydraulic connection exists with a nearby surface water. A pumping test may be conducted to supplement water quality data and/or an MPA test or by itself. The pumping test evaluation should be used to delineate the well capture zone and estimate the time of travel under various pumping and water level conditions. The pumping test duration will need to be adjusted to evaluate hydraulic connectivity.

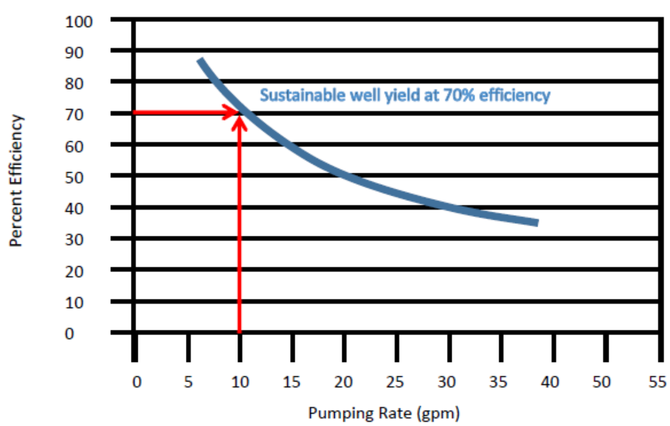
6.0 Pumping Test Results

To accurately analyze pumping test data it is necessary to use methods and formulae appropriate for the hydrogeologic and test conditions encountered at, and specific to, the pumping test site. Knowledge of the hydrogeologic conditions of the area is necessary in order to ensure the use of appropriate techniques of analysis. The analysis and evaluation of pumping test data is covered by numerous texts on the subject (USEPA, 1993; Kruseman and de Ridder, 1991; Walton, 1970; and Ferris, Knowles, Brown, and Stallman, 1962).

6.1 Pumping Rate Determination

The pumping rate for a well can be selected from the step-drawdown test. Calculating the well efficiency then plotting well efficiency versus the corresponding pumping rate to create a sustainable well yield graph. Well efficiency is defined as the ratio of the theoretical drawdown in the formation to the actual drawdown in the well. By selecting a pumping rate that corresponds to the desired well efficiency, a sustainable well yield can be identified. See Figure 2 for an example of the plot. A well efficiency of 70 percent or more is usually desirable for a sustainable pumping rate.

Figure 2
Sustainable Well Yield Graph



A specific capacity plot of drawdown versus yield may also be used to show a point that defines a sustainable well yield. By plotting well yield versus drawdown, a specific capacity curve can be created. On this curve deviations from a linear response can be identified and the approximate point of departure noted that can be selected as a sustainable well yield.

6.2 Pump Setting

The pump setting is determined by pump and well characteristics. Specific capacity plots combined with constant-rate test data can identify correct pump settings by correlating points on the plots where stabilized drawdown occurs. The drawdown depth determined from this correlation, incorporation of a safety factor, and the design engineers chosen distance of pump intake below the pumping level will define the pump setting.

6.3 Safety Factor

There are no rules for a safety margin determination but it is a good idea to incorporate a safety margin for long term operational considerations. Failure to make these provisions may require a change in pump design and/or setting the pump deeper into the well at a future date. A safety factor helps account for inaccuracies in the pumping test, potential impacts from boundary conditions, seasonal water level fluctuations or other special aquifer settings. These inaccuracies usually arise because of lack of complete information about the overall aquifer characteristics or changes that may occur over time.

7.0 Reporting

A hydrogeologic report summarizing current hydrogeologic data and pumping test data is required. The report should contain a summary of the pumping test and be prepared using the elements of the planning and design stage. The report should include the specific capacity, maximum production rate and estimated aquifer properties. These values will need to be substantiated by presenting the worksheets, graphs and calculations that led to the parameters. The report should discuss interpretations made about the aquifer at the site including confining conditions and boundary conditions observed during the course of the test.

Observed drawdown characteristic should be reconciled with the hydrogeologic setting and the narrative should explain why the analysis technique chosen (curve-fitting or numerical modeling) is appropriate. All raw data will need to be submitted in tabular form along with the analysis and computations. All calculations and data analyses should accompany the final report. All calculations should clearly show the data used for input, the equations used and the results achieved. Any assumptions made as part of the analysis should be noted in the calculation section (EPA, 1993). Data manipulations should be clearly described and may include issues or violations of assumptions such as recharge, partial penetration of wells, fluctuating pumping rate, delayed yield, leakage, atmospheric responses, regional water-level trends, tidal influence and drawdown interference from other wells.

The report should document and discuss the results of water quality analyses completed as part of the pump test, including any field measurements (e.g., pH, conductivity and temperature) and water-quality trends that may have occurred during the test. The data and parameters will need to be provided that supports the selection and positioning of the well pump in the well, along with the determination of a sustainable well yield.

7.1 Pumping Test Data Presentation

Presentation of water-level data should include a graph of the arithmetic water-level elevation versus time for the data from each well. From these graphs, long- and short-term trends, recovery of water levels and evidence of aquifer boundaries can be discussed. Incomplete recovery and deviations from the theoretical recovery trends should be addressed if appropriate.

Graphs of drawdown versus time and distance versus drawdown on semi-log and/or log-log paper should be presented. Water level data, graphs, and interpretations should be corrected as appropriate for the effects of ambient water level trends; partially penetrating production well(s); partially penetrating observation wells; delayed yield from well storage and unconfined aquifers; aquifer thickness, recharge and/or impermeable boundaries; barometric pressure changes; changes in stage in nearby surface water bodies; recharge events (rainfall, snow melt) during the week preceding the test, during the test, or during the recovery period; influence from nearby pumping wells; and other hydrogeologic influences should be presented.

Time drawdown graphs should be prepared from the recorded time and the corresponding drawdown. The graphs should be constructed on semi-logarithmic scale with time plotted on the log scale. The plotted points will form a straight line after a certain pumping time and the slope of that line will estimate the transmissivity of the aquifer. Changes in the slope of the line may be indicative of boundary conditions.
Theoretical distance-drawdown graphs should be prepared by plotting the drawdown in each observation well versus the distance of those wells from the pumping well. The graphs should set time equal to the length of the pumping test and groundwater withdrawal equal to the pumping test production rate. Storativity values can be determined from the distance-drawdown graph. Recovery data should be analyzed in a similar manner to drawdown data.

8.0 Potable-Water Supply Samples

Samples should be taken at a time representative of aquifer water quality. Ideally, samples should be taken within the last 15 minutes of the constant-rate pumping test. Water samples must be collected from the source using proper sampling procedures and analyzed by an accredited laboratory (WAC 246-290-130(3)(g)). Source monitoring requirements will be determined by DOH prior to the pumping test. The minimum water quality parameters required for a source approval are listed in Table 4.

Table 4
Minimum Water Quality Sampling for Source Approval

Community	Nontransient Noncommunity	Transient Noncommunity
Coliform Bacteria	Coliform Bacteria	Coliform Bacteria
Inorganic Chemicals	Inorganic Chemicals	Inorganic Chemicals
Volatile Organic Chem	Volatile Organic Chem	Volatile Organic Chem

Synthetic Organic Chem[1]	Synthetic Organic Chem[1]	
Radionuclides		

1. Synthetic organic chemicals (SOCs); unless the source qualifies for a waiver, exempting the source from analysis of all or a partial list of SOCs.

References

Ferris, J. G., D. B. Knowles, R. H. Brown and R. W. Stallman, 1962. Theory of Aquifer Test. U.S. Geological Survey Water Supply Paper 1536-E, pp 69-174

Kruseman, G.P. and N.A. de Ridder, 1994. *Analysis and Evaluation of Pumping Test Data (2nd ed.)*, Publication 47, Intern. Inst. for Land Reclamation and Improvement, Wageningen, The Netherlands, 370p.

USEPA, 1993. Suggested operating procedures for aquifer pumping tests (EPA/540/S-93/503), Robert S. Kerr Environmental Research Laboratory, Ada, Oklahoma, 23p.

Walton, W.C., 1970. *Groundwater Resource Evaluation*, McGraw-Hill, New York, 664p.

Pumping Test Forms

Measurements for elevation and depth to water should be to the nearest ±0.01 foot.

Owner name:____

Mailing address:____________________ City____________ County__________ Zip________

Well Location: Address (cross streets ok):____________ City____________ County__________ Zip________

or **Legal description:** Sec._______ in the____1/4 of____1/4 Twp.__________ Rg.__________ E or W

Description of well location (attach sketch, if nec.):____

NAD 83: Zone:______and **UTM Easting:**______________ft or **Latitude:** deg:______min:______sec:________

(Datum must be set to NAD83) **UTM Northing:**______________ft **Longitude:** deg:______min:______sec:________

Ground elevation:________(ft) amsl Method: ☐ GPS ☐ Level survey ☐ Other (specify):____________________

Class of well: ☐ Group A ☐ Group B Tax Parcel Number:__________________

Pumping Test Summary Information

Type of well pump:

☐ Submersible ☐ Jet (end-suction)

☐ Vertical turbine ☐ Other (specify)______________

Depth of pump setting:__________________ft (btoc)

Type of Pumping Test:

☐ Constant Rate ☐ Step Test ☐ Other (specify)______

Method of water level measurement:

☐ Water level sounder ☐ Data logger ☐ Air line

☐ Wetted tape ☐ Other (specify)____________

Reference datum for water level measurements:

☐ Top of casing ☐ Ground level ☐ Other (specify)______

Final stick-up ____ft ±0.01 foot

Method of flow measurement:

☐ Flow meter ☐ Orifice ☐ 45-gallon drum

☐ 5-gallon pail ☐ Other (specify)______________

Start date of pumping test:__________(MM/DD/YYYY)

Static water level:__________________________ft

Duration of pumping:___hrs

Duration of recovery:___hrs

Well yield estimated from pumping test:________gpm

Available drawdown:_ft

Specific Capacity:________gpm/ft

Method of estimating long-term well yield from pumping test:________________________________

Pumping test data sheet(s) attached: ☐

Person conducting the pumping test (please print):

Name (first, last):____________________________

Company name:____________________________

Phone No. ____________________________

Registration number of person responsible*:__________

Consultant (if applicable; please print):______________

* Fill in the registration of the Qualified Well Driller/Pump Installer. If the test was conducted by a driller/pump installer who is not registered, the Qualified Well Driller/Pump Installer who is directly supervising the work should fill in their registration number.

Declaration:

The pumping test has been done in accordance with the requirements in the Water System Design Manual Appendix E (Pub DOH 331-123) or the Group B water System Design Guidelines Appendix F (Pub DOH 331-467).

PLEASE NOTE: The data recorded in this pumping test report reflect conditions at the time of the test. Water levels, well performance, estimated long-term well yield and water quality are not guaranteed as they are influenced by a number of factors, including natural variability, human activities, and condition of the works, which may change over time.

Signature of Person Responsible:__________________

Return Completed Report, Data Sheets and Data Plots to:
Department of Health Office of Drinking Water Regional Office

Table 1: Definitions of Abbreviations

amsl …… above mean sea level	gpm….......gallons per minute	Rg. ....Range
btoc ....below top of casing	in ....inches	sec. ………seconds
deg ………degrees	no. ……….number	Sec. ....Section
ft ....feet	deg………degrees	Twp. ....Township
hh ………..hour	min ………minutes	UTM ....Universal Transverse Mercator Grid
hrs………..hours	mm……….minute	NAD 83…..North American 1983 datum
Temp……..temperature	Cond..........Conductivity	

Table 2: Recommended Minimum Frequency for Water Level Measurements for Pumping Tests

The recommended minimum frequency for water level measurements during pumping and during recovery is shown below:

Step-Drawdown Test

Time After Pumping Started For Each Step-drawdown Test And After Pump Shut Off For Recovery	Time Intervals To Measure Water Levels And Record Data
0 to 10 minutes	0.5 minute
10 to 15 minutes	1 minutes
15 to 30 minutes	2 minutes
30 to 60 minutes	5 minutes
60 to end of step	10 minutes

Constant-Rate Test

<table>
<tr><th rowspan="2">Time After Pumping Started For Constant Rate Test And After Pump Shut Off For Recovery</th><th colspan="2">Time Intervals To Measure Water Levels And Record Data</th></tr>
<tr><th>Pumping Well</th><th>Observation Well</th></tr>
<tr><td>0 to 10 minutes</td><td>0.5 minute</td><td rowspan="4">2 minutes</td></tr>
<tr><td>10 to 15 minutes</td><td>1 minutes</td></tr>
<tr><td>15 to 30 minutes</td><td>2 minutes</td></tr>
<tr><td>30 to 60 minutes</td><td>5 minutes</td></tr>
<tr><td>60 to 120 minutes</td><td rowspan="3">30 minutes</td><td>5 minutes</td></tr>
<tr><td>120 to 240 minutes</td><td>10 minutes</td></tr>
<tr><td>240 to 360 minutes</td><td>30 minutes</td></tr>
<tr><td>360 minutes to end of test</td><td>60 minutes</td><td>60 minutes</td></tr>
</table>

Pumping Test Drawdown Data Sheet

Pumping test drawdown data sheet for ________________________________(include well name)

☐ Pumping well ☐ Observation well, include well ID Tag No.:______and distance to pumping well _____ ft

Type of pumping test: ☐ Constant Rate ☐ Step Drawdown ☐ Other (specify): ________________________

Date and time at start of pumping (mm/dd/yyyy; hh:mm): ______________________________________

Static water level prior to pumping:______ft Water level at end of pumping:______ft

Weather:______________________________________

Time since pumping started	Measured water level (ft)	Draw down (ft)	Discharge rate (gpm)	Specific Capacity (gpm/ft)	Barometric pressure (in/hg)	Cond. (ohms/ cm)	pH	Temp. (C^0)	Remarks (e.g. sample collected, test interrupted, discharge rate change)

Notes: Drawdown is the difference between the measured water level during pumping and the static water level prior to pumping.

Pumping Test Recovery Data Sheet

Pumping test recovery data sheet for ______________________________(include well name)

☐ Pumping well ☐ Observation well, include well ID Tag No.:______and distance to pumping well _____ ft

Type of pumping test: ☐ Constant Rate ☐ Step Drawdown ☐ Other (specify): ________________________

Date and time at start of pumping (mm/dd/yyyy; hh:mm): ____

Static water level prior to pumping:______ft Water level at end of pumping:______ft

Weather:______________________________________

Time since pumping started	Time since pumping stopped	Measured water level (ft)	Residual drawdown (ft)	Barometric Pressure (in/hg)	Remarks or observations

Notes: Residual drawdown is the difference between the measured water level during recovery and the static water level prior to pumping.

Office of Drinking Water Regional Contacts

Regional engineer assignments are subject to change. Contact the appropriate regional office for the name of the engineer assigned to your county. This information can be found online at https://www.doh.wa.gov/CommunityandEnvironment/DrinkingWater/OfficesandStaff

Appendix F: Submittal Outlines for Select Water Treatment Processes

Appendix F.1 **Hypochlorination Facilities for Small Water Systems Using Groundwater or Seawater**

Appendix F.2 **Fluoride Saturator (Upflow Type)**

Appendix F.3 **Arsenic Removal by Coagulation/Filtration**

Appendix F.4 **Arsenic Removal by Adsorbents**

Appendix F.5 **Use of Ozone in Groundwater Treatment**

Appendix F.6 **Reverse Osmosis for Desalination of Seawater or Brackish/Estuarine Surface Water**

Appendix F.7 **Rainfall Catchment Submittal Requirements**

Appendix F.8 **Rainfall Catchment Reliability Analysis - Example**

Appendix F.9 **Plumbing Sample to a Process Analyzer**

Appendix F.10 **Iron and Manganese Treatment by Oxidation-Filtration**

Appendix F.11 **Iron and Manganese Treatment by Sequestration**

Appendix F.12 **Nitrate Removal by Ion Exchange**

Note: Not all water treatment processes are addressed in Appendix F. For general water treatment guidance see Chapter 10 and Appendix A.3.8.

Appendix F.1 Hypochlorination Facilities for Small Water Systems Using Groundwater or Seawater

Submittal Outline

This outline is intended to guide and summarize your design of hypochlorination treatment facilities for small water systems using groundwater or seawater sources treated by reverse osmosis.

F.1.1 General Water System Information

Provide the following general information:

A. Water system name and public water system identification number

B. Owner's name, address, and telephone number

C. Manager's name, address, and telephone number

D. Operator's name, certification level, and telephone number

E. Completed (preliminary) **Purification Plant Rating Criteria Worksheet** (include all existing and proposed treatment processes)

F. Completed **Project Submittal Application Form**

F.1.2 Description of the Water Quality Problem

Describe the source of supply and the purpose and goals of the proposed treatment. For example, is groundwater treatment required to address criteria because of any of the following?

A. WAC 246-290-453(1) – Required 4-log virus inactivation due to fecal indicator in source water.

B. WAC 246-290-451(4) – Required CT6 due to coliform presence in source water, microbial contaminant sources within the sanitary control area, groundwater in hydraulic connection with surface water, and seawater treated by reverse osmosis.

C. WAC 246-290-451(5) – Required free chlorine residual throughout the distribution system.

D. Other treatment objectives – Fe/Mn oxidation/filtration, arsenic oxidation, hydrogen sulfide oxidation, and so forth.

F.1.3 Raw Water Quality

Groundwater quality tends to vary less than surface water quality, but some groundwater parameters can substantially change seasonally. If the design of the disinfection system is driven by an *E. coli* MCL violation or Ground Water Rule triggered *E. coli* source detection, then the priority is to complete the design and construction as soon as possible. Under these circumstances one set of water quality parameter samples (see following table) should be collected in support of the design.

If sufficient time is available, the design engineer should use water quality information collected from both the dry season and from the onset of the wet season before completing the design. Two separate measurements should be performed for the parameters listed below.

Temperature and pH measurements must be performed at the well site (not in a lab) by a qualified person with properly calibrated instruments. All other water quality parameters must be analyzed by a laboratory certified for drinking water analysis. Submit all lab data sheets.

Seawater RO product water is generally very high quality and consistent with very low chlorine demand. Hypochlorination facilities designed for seawater RO systems do not need to sample for the parameters listed below.

Raw Water Quality Table

Water Quality Parameters	Comment
Total Ammonia-Nitrogen (mg/L)	1 mg/L Ammonia-Nitrogen will react with 8-10 mg/L of chlorine.
Dissolved organic carbon (DOC) (mg/L)	DOC is a precursor to the formation of disinfection byproducts as well as exerting a chlorine demand. We found values less than 1.0 mg/l DOC are unlikely to result in exceedances of the TTHM MCL and values less than 2.0 mg/L DOC are unlikely to result in exceedances of the HAA5 MCL. DOC levels can vary seasonally in some groundwater sources. DOH recommends seasonal testing.
Iron (mg/L)	Iron has a secondary MCL of 0.30 mg/L. In addition, 1 mg/L of reduced iron (Fe^{+2}) exerts 0.6 mg/L of free chlorine demand
Manganese (mg/L)	Manganese has a secondary MCL of 0.050 mg/L. In addition, 1 mg/L of reduced manganese (Mn^{+2}) will react with 1.3 mg/L free chlorine
Coliform bacteria	Note the date of any total coliform or E. Coli detections in the ground water source over the previous 12 months.
Bromide (mg/L)	Only applicable to coastal ground water sources.
Chloride (mg/L)	
pH	Applicable when free chlorine is used to achieve 4-log virus inactivation. Temperature and pH measurements must be performed at the well site (not in a lab) by a qualified person with properly calibrated instruments. Temperature should be measured during winter.
Temperature	

<u>Secondary Water Quality Impacts</u>

The introduction of chlorine into a previously unchlorinated system can cause:

- Release of accumulated metals and sediments in the distribution system
- Formation of disinfection byproducts
- Taste and odor concerns

For these reasons, additional water quality analysis should be conducted. The additional analysis will vary from system to system and could include:

- Oxidation-reduction potential (ORP) of untreated and chlorinated water
- Disinfection by-product formation potential

In some cases, destabilization and dissolution of accumulated iron, manganese, arsenic, and other metals may create unwanted aesthetic and/or unsafe drinking water. Engineers and water system staff can evaluate the potential for potential metal release using a variety of methods ranging from simple desktop studies to pipe loop studies using sections of water main pulled from the distribution system.

F.1.4 Hypochlorination System Details

Provide a written description of the hypochlorination treatment operational requirements, including:

A. Injection point.

B. Max system pressure at chlorine injection point.

C. Peak hour demand (Q_{PHD})

D. Average daily water use, gallons/day (Q_{ADD}).

E. Maximum daily water use, gallons/day (Q_{MDD}).

F. Design flow rate of water to be treated at injection point gpm (Q_s), and whether the flow rate at the injection point is constant (for example, Q_s = installed single speed well pump discharge) or variable (for example, VFD booster pump discharge).

G. Target free chlorine residual, mg/L (C_t). If the treatment objective is to provide 4-log virus inactivation, C_t is directly related to available contact time (T). See *Contact Time* discussion below. If the treatment objective is to maintain a free chlorine residual throughout the distribution system, consider the residence time in the downstream storage and distribution system, to the last customer.

H. Estimated chlorine demand (due to ammonia, organics, iron and manganese, or other inorganics), mg/L (C_d).

I. Describe how chlorine demand was estimated.

J. Describe disinfection byproduct formation potential

K. Calculate required chlorine dose, mg/L ($C_s = C_t + C_d$)

F.1.5 Hypochlorination Feed Pump Requirements

This section is devoted to determining the feed pump requirements. ODW recommends that a purveyor purchase a spare feed pump available for immediate installation and use when the operating pump undergoes routine maintenance or fails.

A. Identify sodium hypochlorite stock chlorine strength, in percent available chlorine (e.g., 8.25%) (C_c). If the source of hypochlorite is calcium hypochlorite, the following equations do not apply. Calcium hypochlorite is typically identified as available chlorine in percentage (e.g., 65% available chlorine). See notes below.

B. Amount of stock chlorine to be added to solution tank, in cups (V_c)

C. Volume of feed solution, in gallons (V_f). V_f is the sum of the stock chlorine volume and the volume of added dilution water.

D. Concentration of feed solution, mg/L (C_f)

$$C_f = \left(\frac{(C_c)(V_c)(10{,}000)}{(V_f)(16)} \right)$$

E. Required feed pump rate, gallons/hour (Q_f)

$$Qf = \frac{(Qs)(Cs)(60)}{Cf}$$

Qf = Capacity of chemical feed system, gph

Qs = Maximum system flow rate, gpm

Cs = Desired free chlorine dose, mg/L

Cf = Concentration of feed solution, mg/L

Note: Calcium hypochlorite [$Ca(OCl)_2$] disinfection systems create a hypochlorite solution by dissolving [$Ca(OCl)_2$] into water. One pound of 65% [$Ca(OCl)_2$] provides 0.65 lbs available chlorine. If dissolved into 25 gallons of water (about 210 lbs of water), 1 lbs of [$Ca(OCl)_2$] will produce a solution strength of 0.003 lbs available Cl_2 per lbs of water; equal to C_f = 3100 mg/L.

F.1.6 Hypochlorination Feed Pump Specifications

Describe the make and model of the hypochlorination feed pump(s), and confirm that the selected pump will perform under the range of operating conditions.

A. Identify pump make and model.

B. Identify the pump's discharge pressure range, and confirm that the maximum pressure capacity of the pump is compatible with the maximum discharge pressure at injection.

C. Identify the pump's volumetric discharge range, and confirm that the maximum and minimum capacities are compatible with the maximum and minimum range of flow to be treated at the point of injection.

D. Confirm wetted parts are compatible with chemical solution being pumped.

E. Identify the need for a hypochlorination pump discharge flow modulation. Discharge flow modulation is needed if the flow at the point of injection is variable.

F.1.7 Solution Tank Sizing

Identify the size of the hypochlorite solution tank.

$$RT = \left(\frac{(V_t)(Q_s)(60)}{(Q_f)(Q_{PROD})} \right)$$

RT = Estimated time between tank refills, days

V_t = Size of solution tank, gallons

Q_{PROD} = Expected daily source production, gallons per day

F.1.8 Achieving 4-log Virus Inactivation (CT6) Treatment

Unanticipated environmental conditions, or other factors beyond the control of the utility, may adversely affect source water quality at any time. DOH strongly recommends that all water systems planning disinfection include dedicated contact time facilities capable of achieving 4-log virus inactivation/CT6 disinfection treatment as part of the design - even if the source is currently free of contamination. Please note:

A. Dedicated contact time in this context means contact piping (most preferred) and/or contact storage (less preferred) that is solely dedicated to providing disinfection contact time for the disinfected source, and is not nested in a larger storage tank that is providing operational, standby, or equalizing storage.

B. 4-log virus inactivation treatment triggers are summarized in WAC 246-290-453.

C. CT6 treatment triggers are summarized in WAC 246-290-451.

A purveyor with a groundwater or seawater RO source requiring 4-log virus inactivation or CT6 disinfection treatment before the first customer may achieve this level of inactivation using free chlorine by providing a minimum CT value of 6 provided pH is in the range of 6-9 and water temperature is greater than or equal to 10ºC. Groundwater supplies with measured temperatures below 10ºC or above pH 9 require a CT value greater than 6. See Sections 10.2.1 for further guidance.

Provide the following information:

A. Available minimum contact volume (excluding operational, standby, and equalizing storage), gallons (V)

B. Baffling factor (BF). Use 0.1 for an un-baffled chlorine contact tank with separate inlet and outlet, where tank volume is equal to the lowest daily value. For chlorine contact piping, a length-to-diameter ratio of at least 160 is needed to achieve a baffling efficiency of 1.0 provided that the flow through the pipe is turbulent and not laminar. A length-to-diameter ratio of at least 40 is needed to achieve a baffling factor of 0.7. For shorter pipe segments, the baffling factor can be estimated using the length-to-width ratio and information in *Improving Clearwell Design for CT Compliance* (Crozes et al. 1999).

C. Credited contact time, minutes ($T_{credited}$)

D. Maximum anticipated flow out of the chlorine contact tank and/or through the pipeline to the first customer, gpm (Q)

$$T_{credited} = BF\left(\frac{V}{Q}\right)$$

The free chlorine concentration at the end of "T" must provide for the minimum required CT value.

F.1.9 Checklist of Additional Items

A. Plans showing size and location of:

i. Sampling taps for both raw and treated water.

ii. Sampling tap following contact time piping or storage.

iii. Entry to distribution system sampling location.

iv. Flow meter on the outlet of the tank used for contact time to measure Q when satisfying CT6 or 4-log virus inactivation requirements.

v. Flow meter on the outlet of the contact piping used for contact time to measure Q unless the source meter will measure the same total and instantaneous rate of flow.

vi. All existing and proposed raw and treated water piping, existing treatment (if any), valves, appurtenances, equipment controls and monitoring devices, supports, and cross connection control devices/assemblies.

B. Manufacturer's specifications for:

i. Chemical feed pump

ii. Variable feed pump output control system (if applicable)

C. Solution tank with calibrated volume

D. Flow meter(s)

E. DPD chlorine test kit (see WAC 246-290-451 and -453 for allowable test methods). ODW recommends using only a digital colorimetric testing device employing an EPA-approved analytical method.

F. A continuous chlorine residual analyzer is required at the location for measuring "C" for systems serving more than 3,300 people with a 4-log virus inactivation requirement triggered by WAC 246-290-453.

G. Source of hypochlorite listed under NSF 60 (drinking water additives), except as allowed under WAC 246-290-220.

H. Identify the appropriate ODW monthly treatment plant report form. See **ODW groundwater disinfection report forms** on the ODW web site.

I. Operations and maintenance plan. Confirm the certified operator has reviewed and given the opportunity for input on the design and O&M plan.

J. Disinfection byproducts monitoring plan (not applicable to transient non-community systems).

K. Updated coliform monitoring plan.

Appendix F.2 Fluoride Saturator, Upflow Type

Submittal Outline

This outline is intended to guide and summarize your design of fluoride treatment using an upflow sodium fluoride (NaF) saturator.

F.2.1 General Water System Information

Provide the following general information:

C. Water system name and public water system identification number

D. Owner's name, address, and telephone number

E. Manager's name, address, and telephone number

F. Operator's name, certification level, and telephone number

G. Completed (preliminary) Purification Plant Rating Criteria Worksheet (include all existing and proposed treatment processes)

H. Completed Project Submittal Application Form

F.2.2 Description of the Treatment Objective

Describe the source of supply and the purpose and goals of the proposed treatment. The treatment objective should be consistency of fluoride concentration at all times throughout all portions of the distribution system. The optimal treated water fluoride concentration is 0.7 mg/L. Water systems that fluoridate **must** maintain a fluoride concentration between 0.5 mg/L and 0.9 mg/L in the distribution system (WAC 246-290-460).

F.2.3 Fluoridation System Details

A. Provide a written description of the fluoridation treatment operational requirements, including:

   i. Source type (surface, groundwater, purchased). Note: naturally occurring fluoride concentration in surface water may be more variable than groundwater, demanding more design data and the capacity to recognize changes in natural fluoride concentration in source water.

   ii. Natural fluoride concentration in source water, mg/L (C_n). Provide natural fluoride source water quality data from the last ten years if available.

   iii. Injection point.

   iv. Maximum and minimum system pressure at fluoride injection point, to establish chemical feed pump specification and needed backflow/overfeed protection.

   v. Population served (direct customers and customers served by consecutive systems, if any).

   vi. Average, maximum, and minimum daily source production, gallons/day.

vii. Design flow rate of water to be treated at injection point gpm (Q_s), and whether the flow rate at the injection point is constant or variable.

viii. Target fluoride residual, mg/L (C_t).

ix. Required fluoride dose, mg/L ($C_s = C_t - C_n$)

x. Calculated fluoride feed rate at average, maximum, and minimum source production rate. Note: The maximum pump capacity of the chemical feed pump should be no more than twice the chemical feed pumping rate during average source production.

B. Water softening:

i. ODW recommends water used in saturator first undergo softening if total hardness is greater than 75 mg/L as $CaCO_3$.

ii. Water softener regeneration frequency and salt requirement per regeneration. Note: Operations should strive to maintain a minimum 30-day supply of salt.

F.2.4 Fluoridation Feed Pump Requirements

This section is devoted to determining the NaF feed pump requirements. ODW recommends that a purveyor purchase a spare feed pump available for immediate installation and use when the operating pump undergoes routine maintenance or fails.

$$Qf = \frac{(Qs)(Cs)(60)}{Cf}$$

Qf = Capacity of chemical feed system, gph

Qs = Maximum system flow rate, gpm

Cs = Desired fluoride dose, mg/L

Cf = Fluoride concentration in a saturated solution, mg/L (based on NaF supplier)

F.2.5 Fluoridation Feed Pump Specifications

Describe the make and model of the fluoridation feed pump(s), and confirm that the selected pump will perform under the range of operating conditions.

A. Identify pump type (diaphragm, peristaltic), make and model.

B. Identify the pump's discharge pressure range, and confirm that the maximum pressure capacity of the pump is compatible with the maximum discharge pressure at the point of injection.

C. Identify the pump's volumetric discharge range, and confirm that the maximum and minimum capacities are compatible with the maximum and minimum range of flow to be treated at the point of injection.

D. Confirm wetted parts are compatible with chemical solution being pumped.

E. Identify the need for a fluoridation pump discharge flow modulation. Discharge flow modulation is needed if the flow at the point of injection is variable.

F. Identify need for an in-line mixer. If the distance from the point of injection to the first service tap is less than 100 feet, an in-line mixer should be included in the design.

G. Feed pump power cord should be specified with a non-standard outlet plug and outlet receptor, or hard-wired to the source pump interlock.

F.2.6 Solution Tank Sizing

ODW recommends the design include a clear solution tank filled manually each day from the fluoride saturator. ODW recommends a maximum 1.25 days (30 hours) of capacity in the fluoride solution tank. The transfer rate from saturator to clear solution tank should be limited to 2 gallons per minute.

Identify the size of the fluoride clear solution tank.

$$RT = \left(\frac{(V_t)(Q_s)(60)}{(Q_f)(Q_{PROD})} \right)$$

RT = Estimated time between tank refills, days
V_t = Size of solution tank, gallons
Q_{PROD} = Expected daily source production, gallons per day

F.2.7 Make-up Water Supply and Cross Connection Control

Describe the make-up water supply and cross connection control measures:

A. Water supply

i. Pipe size, inches

ii. Pipe material

iii. Static pressure, psi

iv. Flow restrictor capacity (2 gpm)

B. Backflow protection. An RPBA is required on the saturator make-up water supply line. See WAC 246-290-490 for requirements concerning cross connection control.

F.2.8 Overfeed Protection

Improper controls on fluoride treatment may lead to an increased risk of fluoride overfeed. The following measures are intended to reduce the risk of overfeed, and should be addressed in the design of any fluoride treatment system (WAC 246-290-110):

A. Fluoride feed pump electrically interconnected with source pump

B. Flow sensing switch in water main interconnected with fluoride feed pump

C. Anti-siphon valve at pump head (not needed if fluoride feed pump is peristaltic type)

D. Anti-siphon valve at injection quill

F.2.9 Checklist of Additional Items

A. Sample tap for raw and treated water (following mixing)

B. Source meter to record total volume pumped

C. Make-up meter to record total solution volume fed

D. Flow proportioned feed pump (if applicable)

E. Feed pump non-standard plug and outlet, or hard-wired to the source pump interlock.

F. ANSI/NSF 60 certification for NaF chemical

G. Dry storage for 30-day supply of chemical

H. Materials Data Safety Sheet (MSDS) information posted where the chemical is stored and used, and/or available where all other MDSD information is kept in the treatment location.

I. Respirator, gloves, apron, goggles provided for handling NaF

J. Fluoride test kit (SPADNS or ISE with ionic strength adjustment) in specifications

K. ODW monthly report forms provided to utility

L. Sample bottles for split sampling provided

M. Plans and specifications

N. Operations and maintenance plan, including

  i. Cleaning the fluoride saturator

  ii. Cleaning fluoride solution inject lines

  iii. Maintaining a complete spare chemical feed pump

  iv. Testing backflow assemblies

  v. Periodic feed pump output calibration (using a calibration column)

  vi. Not adding NaF while the system is operating

  vii. Preparing fluoride overfeed prevention and response plan

  viii. Identify the appropriate ODW monthly treatment plant report form. See ODW fluoridation report forms on the ODW web site.

  ix. Training specific for fluoride treatment for every operator with operational or maintenance responsibility

  x. Confirmation the certified operator has reviewed and given the opportunity for input on the design and O&M plan.

Appendix F.3 Arsenic Removal by Coagulation/Filtration

Submittal Outline

This outline is intended to guide and summarize your design of arsenic (As) treatment facilities for small water systems using groundwater sources. The following factors are known to be important in the successful, reliable operation of an arsenic removal treatment facility using coagulation/filtration.

- Correct coagulant dose applied.
- Adequate oxidation of As(III) to As(V)
- Properly sized filter media.
- Complete and accurate raw water data at time of design.
- Appropriate filtration rate.
- Adequate backwashing frequency, rate, and control. Proper monitoring and control of the backwash recycle return (if applicable).

Because of the many factors affecting design, and because the raw water quality is so critical to selecting an appropriate treatment technique, all facilities must be pilot plant tested at the site, with certain raw water quality tests also performed on site. Submittals for treatment facilities must be prepared by an engineer licensed in Washington State.

Submit all supporting documentation with the design submittal.

F.3.1 General Water System Information

Provide the following general information:

A. Water system name and public water system identification number

B. Owner's name, address, and telephone number

C. Manager's name, address, and telephone number

D. Operator’s name, certification level, and telephone number

E. Completed (preliminary) Purification Plant Rating Criteria Worksheet (include all existing and proposed treatment processes)

F. Completed Project Submittal Application Form

F.3.2 Description of the Water Quality Problem

Describe the source of supply and the purpose and goals of the treatment proposed. Include source number and design flow rate(s).

F.3.3 Raw Water Quality

The raw water quality of most groundwater supplies can vary seasonally and over time. For this reason, a minimum of two separate measurements should be collected for the parameters listed below. In aquifers where the water quality is known to vary significantly, such as some island aquifers, additional raw water quality sampling is recommended. To get accurate data pH measurements must be performed at the well site (not in a lab) by a qualified person with a properly calibrated instrument. All other water quality parameters must be analyzed by a laboratory certified for drinking water analysis. Submit all lab data sheets.

Raw Water Quality Table

Water Quality Parameters	Comment
Arsenic As (Total) (ug/L)	Speciation between As(III) and As(V) must be done in the field, with samples sent to labs. It is not necessary if adequate pre-oxidation is provided. However, it can be useful in troubleshooting treatment.
Arsenic As (III)	
Arsenic As (V)	
Ammonia (mg/L)	An indicator of strong reducing conditions. Interferes with use of Cl_2 as pre-oxidant.
Calcium (mg/L)	High calcium reduces interference from Si lowering the coagulant dose required.
Iron (mg/L)	Iron binds to arsenic so that it can be removed. An iron to arsenic ratio of at least 20:1 is required for effective arsenic removal and can be 100:1 or greater depending upon pH, and competition from Si, PO_4, TOC, etc.
Manganese (mg/L)	Mn removal is frequently desired if it is greater than the secondary MCL of 0.050 mg/L.
TOC (mg/L)	TOC exerts an iron demand, so at concentrations greater than 2.0 mg/L it can significantly impact iron dose and foul filter media.
pH	The ability of arsenic to bind to iron is strongly affected by pH. Ideal range pH 7.0 or less. At pH >8.0, pH reduction may be beneficial.
Phosphate PO_4 (mg/L)	Phosphate is chemically analogous to arsenate [As(V)]. Significant interference occurs at >0.040 mg/L.
Silica Si (mg/L as SiO_2)	Silica can cause significant interference with arsenic binding to iron at pH>7.5 at 20 mg/L and 50 mg/L regardless of pH.

F.3.4 Pilot Testing

Pilot testing is usually necessary to determine that a treatment process is functional, and economically viable, as well as develop the design parameters for the treatment process including:

A. Pre-oxidant type, dose and contact time.

B. Coagulant type, dose and contact time.

C. Filter media type, depth, and loading rate.

D. Backwash frequency, duration, and hydraulic rate.

E. Filter-to-waste duration and rate.

F. Backwash recycle return (flow rate, volume per filter backwash cycle, quality) if applicable.

G. Other process control parameters (if used) such as pH adjustment.

Inadequate arsenic pilot testing may result in treatment process performance inefficiencies or outright failure, delayed implementation of effective treatment, and costly retrofitting or replacement of treatment facilities. For these reasons, pilot plant testing, including the submittal of a pilot testing plan, will usually be required. At a basic level, the project report that summarizes the pilot testing should describe:

H. Pilot plant setup, duration (see Table 10-2), and results as they relate to full scale treatment design.

I. Pilot plant design parameters:

   i. Treatment rate of the pilot plant (gpm/sq.ft.)

   ii. Oxidant and dosage (mg/L) (if applicable)

   iii. Coagulant and dosage (mg/L)

   iv. Length of oxidation and coagulant contact times. Contact time is detention time from point of oxidation or coagulation addition to filter.

   v. Backwash parameters (filter run length/volume (hours or gals), backwash rate (gpm), duration (min)) and recycle return (if applicable).

   vi. pH adjustment (if necessary)

F.3.5 Summarize Coagulation-Filtration Treatment Components

Include schematic drawing of the treatment system identifying:

A. Major system components

B. Process control stations, such as water quality sampling points (raw, post-oxidant, post-coagulant, after each filter, combined filter effluent) flow meter(s), and pressure gauges, chlorine residual analyzer(s), turbidity meter(s)).

F.3.6 Full-Scale Design

The design engineer should cover the following items in the project report and construction documents:

A. Process Control: Document location of and interaction between these process control components:

   i. Sample locations:

      a. Raw water (before any treatment)

      b. After oxidant addition

      c. After iron addition but before filtration

      d. After each filter

e. Combined filter effluent

f. Backwash recycle return (if applicable)

ii. Physical parameter and water quality analysis:

a. The frequency of monitoring should be identified for the flow, oxidant residual, iron, arsenic, pressure, and filtration volume or run time.

b. On-line/continuous water quality instrumentation can improve process control and aid in trouble shooting, so the use of chlorine residual analyzers, pH analyzers, and turbidimeters should be evaluated.

iii. Process control narrative describing:

a. Process control parameters (oxidant residual, target pH, iron concentration), means of process control, and benchmarks for successful operations.

b. Capacity for remote operations

c. Alarm and shutdown conditions

B. pH Adjustment: Document pH adjustment design basis (if applicable)

i. Chemical used, dosage (mg/L), and target pH range.

I.

C. Oxidation: Type, dose, and target residual. and oxidation process design basis

i. Oxidant dose (mg/L)

ii. Target oxidant residual (mg/L)

If ozone is used as an oxidant, see Appendix F.5 for submittal guidance.

iii. Contact Time (sec) between oxidant and iron addition (if added) or oxidant and filter (if not added). Note: Usually 20-60 seconds is needed to convert As(III) to As(V) depending upon oxidant and other water quality parameters.

D. Natural Iron and Coagulant Addition: Document coagulant selection

i. Raw Water Fe/As Ratio.
Note: A Fe/As mass ratio of at least 20:1 is required for effective arsenic removal, and may need to be greater than 100:1 in some cases.

ii. Coagulant:

a. Type, dose (mg/L), and monitoring approach.

E. Flocculation: Document flocculation design basis

i. Time between coagulant addition and filtration vessel.

F. Filter media

i. Type, depth (usually at least 36 inches), effective size, and loading rate.

Note: Filtration rate usually less than 5 gpm/sf for effective filtration, though may be higher for some solid manganese dioxide media provided raw water quality is suitable and if demonstrated through pilot testing.

ii. Expected replacement frequency and sensitivity to oxidants.

iii. ANSI/NSF Standard 61 certification.

G. Backwash: Document backwash design criteria and process

i. Describe design objective
   a. Optimize finished water quality
   b. Minimize backwash volume per cycle
   c. Maximize finished water volume

ii. Identify backwash initiation
   a. Headloss, psi or feet
   b. Time since last backwash, hours or days
   c. Volume of filtered water, gallons

iii. Identify backwash hydraulics:
   a. Flow rate (gpm/sf). Identify the manufacturer's recommended backwash application rate in gpm/sq.ft.
   b. Identify the backwash pump pressure in psi. Attach pump curve. Verify adequacy of system hydraulics for the proposed backwash.
   c. Backwash duration, min
   d. Volume, gal
   e. Verify that no cross connection exists between the backwash source water and the wastewater.

iv. Backwash disposal
   a. Volume of backwash per cycle, average day and peak day.
   b. Describe disposal of backwash, and include all backwash disposal facilities in construction documents.
   c. Confirm that the proposed method of backwash waste disposal is acceptable to the Department of Ecology and the local health department. See Chapter 10 for guidance on permitting water treatment plant waste disposal.

H. Backwash Recycle Return: Document backwash recycle return design basis (if applicable)

i. Backwash storage and return

a. Backwash holding tank volume, gallons

b. Detention time, hours

c. Supernatant recycle return volume, gallons

d. Supernatant recycle return flow, gpm. The recycle return flow should not exceed 10 percent of the total influent to the filters.

ii. Conditioning supernatant recycle return

a. Bag/cartridge filter

b. Chemical addition

c. Turbidity monitoring of supernatant recycle return stream

iii. Identify backwash recycle return initiation

a. Volume of backwash water, gallons

b. Time since last backwash recycle return, in hours or days. Note: Recycling on a volume basis rather than time is recommended, as production varies throughout the year.

I. System Hydraulics

i. Describe source-pumping mode (pumps directly to storage or to distribution).
ii. Define the current installed source pumping capacity in gpm.
iii. Verify that the installed pumping capacity is adequate to meet current design standards with the proposed treatment on line. Discuss all components of the total pumping head (well pump lift, system elevation difference, treatment plant head loss, system head losses, and residual pressure).

F.3.8 Operations and Maintenance

A. Prepare an O&M manual section, which includes:

i. Identify maintenance personnel and operators.
ii. Outline routine inspection and maintenance - daily, weekly, monthly, annually.
iii. Identify major equipment components and their manufacturers.
iv. Identify a record keeping system to track treatment system performance.
v. Disinfection byproduct monitoring plan if chlorine or ozone is used (not applicable to transient non-community systems).

vi. Identify the appropriate ODW monthly treatment plant report form. Obtain the applicable reporting form from the ODW regional engineer.

vii. Confirmation the certified operator has reviewed and given the opportunity for input on the design and O&M plan.

viii. Arsenic treatment optimization goals. See DOH Arsenic Treatment Optimization Program information.

Appendix F.4 Arsenic Removal by Adsorbents

Submittal Outline

This outline is intended to guide and summarize your design of arsenic (As) treatment facilities for small water systems using adsorbents. The cost of adsorbents varies widely along with their performance depending upon the adsorbent type and raw water quality. Breakthrough can occur in as little as a few weeks, or adsorbents can last many months before needing to be replaced. Since very short run times can make the process economically unsustainable, pilot testing is necessary for adsorbents.

Submittals for treatment facilities must be prepared by an engineer licensed in Washington State.

Provide all supporting documentation with the design submittal.

F.4.1 General Water System Information

Provide the following general information:

A. Water system name and public water system identification number

B. Owner's name, address, and telephone number

C. Manager's name, address, and telephone number

D. Operator's name, certification level, and telephone number

E. Completed (preliminary) Purification Plant Rating Criteria Worksheet (include all existing and proposed treatment processes)

F. Completed Project Submittal Application Form

F.4.2 Description of the Water Quality Problem

Describe the source of supply and the purpose and goals of the treatment proposed. Include source number and design flow rate(s).

F.4.3 Raw Water Quality

The raw water quality of most groundwater supplies can vary seasonally and over time. For this reason, a minimum of two separate measurements should be collected for the parameters listed below. In aquifers where the water quality is known to vary significantly, such as some island aquifers, additional raw water quality sampling is recommended. To get accurate data pH measurements must be performed at the well site (not in a lab) by a qualified person with a properly calibrated instrument. All other water quality parameters must be analyzed by a laboratory certified for drinking water analysis. Submit all lab data sheets.

Raw Water Quality Table

Water Quality Parameters	Comment
Arsenic As (Total) (ug/L)	Speciation between As(III) and As(V) must be done in the field, with samples sent to labs. It is not necessary if adequate pre-oxidation is provided. However, it can be useful in troubleshooting treatment.
Arsenic As (III)	
Arsenic As (V)	
Ammonia (mg/L)	An indicator of strong reducing conditions. Interferes with use of Cl_2 as pre-oxidant.
Calcium (mg/L)	High calcium reduces interference from Si leading to longer adsorbent runs.
Iron (mg/L)	Iron in excess of 0.3 mg/L may cause fouling of the adsorbent and may result in the need for excessive backwashing.
Manganese (mg/L)	Mn removal is frequently desired if it is greater than the secondary MCL of 0.050 mg/L.
TOC (mg/L)	TOC in excess of 2 mg/L may cause fouling of the adsorbent and may result in the need for excessive backwashing.
pH	The ability of arsenic to bind to iron is strongly affected by pH. Ideal range pH 7.0 or less. At pH >8.0, pH reduction may be beneficial.
Phosphate PO_4 (mg/L)	Phosphate is chemically analogous to arsenate. Significant interference occurs at >0.040 mg/L.
Silica Si (mg/L as SiO_2)	Silica can prevent arsenic binding to the adsorbent at pH>7.5 at 20 mg/L and 50 mg/L regardless of pH.

F.4.4 Pilot Testing

Pilot testing is usually necessary to determine that a treatment process is functional, and economically viable, as well as develop the design parameters for the treatment process including:

A. Pre-oxidant type, dose and contact time.
B. Volume of water treated to breakthrough, expressed in term of bed volumes.
C. Backwash frequency, duration, and hydraulic rate.
D. Backwash recycle return (flow rate, volume per filter backwash cycle, quality) if applicable.
E. Other process control parameters (if used) such as pH adjustment.

Inadequate arsenic pilot testing may result in treatment process performance inefficiencies or outright failure, delayed implementation of effective treatment, and costly retrofitting or replacement of treatment facilities. For these reasons, pilot plant testing, including the submittal of a pilot testing plan, will usually be required. At a basic level, the project report that summarizes the pilot testing should describe:

F. Pilot plant setup, duration (see Table 10-2), and results as they relate to full scale treatment design.

G. Pilot plant design parameters:

   i. Treatment rate of the pilot plant (gpm/sq.ft. or gpm).

   ii. Oxidant and dosage (mg/L) (if applicable).

   iii. Length of oxidation contact time. Oxidation contact time is detention time from point of oxidation addition to adsorbent vessel.

   iv. Backwash parameters (filter run length/volume (hours or gals), backwash rate (gpm), duration (min)) and recycle return (if applicable).

   v. pH adjustment (if necessary)

H. Estimate the number of bed volumes that can be treated prior to adsorbent media exhaustion and describe the basis for the estimate.

F.4.5 Summarize Adsorbent Treatment Components

Include schematic drawing of the treatment system identifying:

A. Major system components

B. Process control stations, such as water quality sampling points (raw, post pH adjustment, post-oxidant, after each filter, combined filter effluent) flow meter(s), pressure gauges, chlorine residual analyzer(s), turbidimeter(s).

F.4.6 Full-Scale Design

A. Process Control: Document location of and interaction between these process control components:

   i. Sampling taps:

      a. Raw water (before any treatment)
      b. After oxidation
      c. Before adsorbent vessel(s)
      d. After adsorbent vessel(s)
      e. Combined adsorbent effluent
      f. Other

   ii. Physical parameter and water quality analysis:

      a. The frequency of monitoring should be identified for the flow, oxidant residual, iron, arsenic, pressure, and filtration volume or run time.

b. On-line/continuous water quality instrumentation can improve process control and aid in trouble shooting, so the use of chlorine residual analyzers, pH analyzers, and turbidimeters should be evaluated.

iii. Process control narrative describing:

a. Process control parameters (oxidant residual, target pH, iron concentration), means of process control, and benchmarks for successful operations.

b. Capacity for remote operations

c. Alarm and shutdown conditions

B. pH Adjustment: Document pH adjustment design basis (if applicable)

i. Chemical used, dosage (mg/L), and target pH range.

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C. Oxidation: Type, dose, and target residual. and oxidation process design basis

i. Oxidant dose (mg/L)

ii. Target oxidant residual (mg/L)

If ozone is used as an oxidant, see Appendix F.5 for submittal guidance.

iii. Contact Time (sec) between oxidant and iron addition (if added) or oxidant and filter (if not added). Note: Usually 20-60 seconds is needed to convert As(III) to As(V) depending upon oxidant and other water quality parameters.

D. Adsorption Process: Document adsorbent and adsorption design basis

i. Prefiltration

a. None

b. Bag/cartridge filter

ii. Adsorbent Media

a. Adsorbent name and manufacturer
b. Configuration (series or parallel)
c. Loading rate (gpm/sf)
d. Depth (inches)
e. Empty bed contact time (EBCT, in minutes). Usually EBCT must be at least 5 minutes. EBCT calculated as [Volume of media (ft^3)*7.48 gal/ft^3/Q (gal/min)].

f. NSF 61 certification

g. Disposal options/requirements upon adsorbent media exhaustion

E. Backwash: Document backwash design criteria and process

ii. Describe design objective

a. Optimize finished water quality

b. Minimize backwash volume per cycle
c. Maximize finished water volume

iii. Identify parameters for backwash initiation

a. Headloss (psi or feet)
b. Run time since last backwash (hours/days)
c. Volume of filtered water (gal)

iv. Identify backwash hydraulics:

a. Flow rate (gpm/sf). Identify the manufacturer's recommended backwash application rate in gpm/sq.ft.
b. Identify the backwash pump pressure in psi. Attach pump curve. Verify adequacy of system hydraulics for the proposed backwash.
c. Backwash duration, min
d. Volume
e. Verify that no cross connection exists between the backwash source water and the wastewater.

v. Backwash disposal

a. Describe constituents, average day, and peak day volume of backwash.
b. Describe disposal of backwash, and include all backwash disposal facilities in construction documents.
c. Confirm that the proposed method of backwash waste disposal is acceptable to the Department of Ecology and the local health department. See Chapter 10 for guidance on permitting water treatment plant waste disposal.

F. Backwash Recycle Return: Document backwash recycle return design basis (if applicable)

i. Backwash storage and return

a. Backwash holding tank volume, gallons
b. Detention time, hours
c. Supernatant recycle return volume, gallons
d. Supernatant recycle return flow, gpm

ii. Conditioning supernatant recycle return

a. Bag/cartridge filter
b. Settling

c. Turbidity monitoring of supernatant recycle return stream

iii. Identify backwash recycle return initiation

a. Volume of backwash water, gallons

b. Time since last backwash recycle return, in hours or days. Note: Recycling on a volume basis rather than time is recommended, as production varies throughout the year.

G. System Hydraulics

i. Describe source-pumping mode (pumps directly to storage or to distribution).
ii. Define the current installed source pumping capacity in gpm.
iii. Verify that the installed pumping capacity is adequate to meet current design standards with the proposed treatment on line. Discuss all components of the total pumping head (well pump lift, system elevation difference, treatment plant head loss, system head losses, and residual pressure).

F.4.7 Operations and Maintenance

A. Prepare an O&M manual section, which includes:

i. Identify maintenance personnel and operators.
ii. Outline routine inspection and maintenance - daily, weekly, monthly, annually.
iii. Identify major equipment components and their manufacturers.
iv. Identify a record keeping system to track treatment system performance.
v. Disinfection byproduct monitoring plan (not applicable to transient non-community systems).

vi. Identify the appropriate ODW monthly treatment plant report form. Obtain the applicable reporting form from the ODW regional engineer.

vii. Confirmation the certified operator has reviewed and given the opportunity for input on the design and O&M plan.

viii. Arsenic treatment optimization goals. See DOH Arsenic Treatment Optimization Program information.

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Appendix F.5 Use of Ozone in Groundwater Treatment

Submittal Outline

This outline supplements the outline provided in Appendix F.3, F.4, and F.10. It is intended to provide general guidance on the use of ozone as a pre-filter oxidant on groundwater sources.

F.5.1 Raw Water Quality

1. Based on actual water quality results, show ozone demand calculations for the water to be treated.
2. Identify the benefits that can be achieved by the proposed ozone treatment facilities.
3. Identify any adverse effects that the proposed ozone treatment system may have on other water quality parameters (disinfection byproduct generation, increased corrosivity, and nutrient source for regrowth bacteria).
4. Discuss the effect of the proposed ozone treatment on the requirements of the federal Safe Drinking Water Act (Groundwater Rule and the Stage 1 Disinfection/ Disinfection Byproducts Rule).

F.5.2 Pilot Plant Testing

Inadequate pilot testing has resulted in treatment process failures, delayed implementation of effective treatment, and costly replacement of inadequate treatment. For these reasons, pilot plant testing, including the submittal of a pilot testing plan, will usually be required. Appendix F.3, F.4, F.10, and Section 10.3 have additional information on pilot testing. At a basic level, the project report that summarizes the pilot testing should describe:

A. Pilot plant setup, duration and results as they relate to full scale treatment design.

B. Pilot plant design parameters:

   i. Treatment rate of the pilot plant (gpm/sq.ft. or gpm).

   ii. Ozone dosage (mg/L).

   iii. Ozone demand and decay coefficients

   iv. Length of oxidation contact time. Oxidation contact time is detention time from point of oxidation addition to coagulant or filter/adsorbent vessel.

C. How long was the pilot plant operated? What seasonal changes in water quality may affect the performance of the proposed treatment plant?

F.5.3 Full-Scale Design

A. Oxidation by Ozone

Provide a written description and specifications of the proposed treatment plant. Include engineering drawings with appropriate labels.

i. Feed Gas Preparation

a. Identify the type of supply gas (air, pure oxygen, other).

b. Describe the method of gas drying. What is the seasonal variation in air moisture and can the gas be dried to a maximum dew point of -60C (-76F)?

c. Describe how feed gas is supplied to the generator (pump, and venturi). Define the operating pressures? (If using a compressor, specify "oil free").

ii. Ozone Generation

a. Identify the type of ozone generator (corona discharge, other). The minimum concentration of ozone in the generator exit gas should not be less than 1 percent (by weight).

b. Define the ozone production rate (g/hr, lbs/day) and ozone concentration (mg/L).

c. Specify a minimum of two generators, each sized to provide 50 percent of peak flow or similar alternative.

d. Verify that the existing power supply can meet the electrical needs of generators. Are the electrical components safety certified?

e. Describe the method of generator cooling.

f. Specify corrosion resistant components in the ozone generator.

g. Has the specified ozone generator been certified by an independent laboratory? If so, list the certifying agent.

iii. Ozone Dissolution / Contact Vessel

a. Describe the method for introducing ozone into the raw water stream (venturi, pump, diffuser, other). Identify operating parameters of method used (such as pressure differential, counter-current flow, mitigation of precipitate formation).

b. Identify the necessary contact vessel required to provide contact time. Include sizing calculations and basis for sizing. Is contact vessel resistant to corrosion?

c. Include a pressure/vacuum relief valve on the contact vessel; show that it is piped to a location for safe discharge.

d. Identify on a drawing controls for cleaning, maintenance and drainage of the contact vessel.

iv. Off-gas Destruction Unit

a. Describe a system which meets safety and air quality standards for treating off-gas from the contact vessel (Washington Industrial Safety and Health Act - WISHA -Chapter 296-841 WAC- the maximum permissible exposure is 0.30 mg/L for 15 minute exposure). If undissolved ozone gas from the contact vessel is recirculated, demonstrate that the ozone concentration (off-gas) in downstream storage vessels is within standards and that the vessel is not subject to excess corrosion.

v. Piping Materials

a. Specify pipe material with demonstrated corrosion resistivity (such as low carbon 304L or 316L stainless for ozone service, non-solvent welded UPVC pipe, Teflon valve seats). Identify a replacement schedule if recommended by the manufacturer.

vi. Ozone Facility Instrumentation

a. Pressure gauges and air flow meters to monitor the ozone generation process (such as at discharge of air compressor, inlet to ozone generators, and inlet to ozone destruct unit).

b. Dew point monitor to measure the moisture of the feed gas.

c. Ozone monitors (or alternate equivalent) to measure ozone concentration in the feed gas, undissolved gas in the contact vessel, ozone residual prior to filtration, ozone residual post filtration and in the off gas from the destruct unit.

d. An ambient ozone monitor (or alternate equivalent) in the vicinity of the contact vessel and generator.

e. An emergency electrical shut-down accessible from outside of the treatment building.

vii. Ozone Facility Alarms

a. Dew point shutdown/alarm.

b. Ozone generator cooling water flow, temperature and power shutdown/alarm.

c. Ambient ozone concentration shutdown/alarm.

F.5.4 Operations and Maintenance

Prepare an O&M manual section, which includes:

A. Identify maintenance personnel and operators.

B. Outline routine inspection and maintenance—daily, weekly, monthly, annually.

C. Identify major equipment components and their manufacturers.

D. Identify a record keeping system to track treatment system performance.

E. Safety reference WISHA, which establishes permissible levels of airborne contamination (chapter 296-841 WAC).

i. Provide the manufacturer's Material Safety Data Sheet for ozone. Post a copy of the data sheet in an obvious place in the treatment house.

ii. Provide a summary of the health effect of exposure to ozone. Post a copy of these health effects in an obvious place in the treatment house.

iii. Identify first aid procedures related to ozone exposure.

iv. If unsafe ozone gas is present, define a procedure for exhausting the building and system shutdown (for example, familiarization with ambient ozone monitor function and procedures, or other). How is building access determined to be safe?

F. Disinfection byproduct monitoring plan (not applicable to transient non-community systems).

G. Miscellaneous

i. Sampling taps for both raw and finished water and after each treatment unit.

ii. Totalizing meter to record total volume treated.

iii. Flow proportioned ozone feed.

H. Confirmation the certified operator has reviewed and given the opportunity for input on the design and O&M plan.

Appendix F.6 Reverse Osmosis for Desalination of Seawater or Brackish/Estuarine Surface Water

Reverse osmosis (RO) treatment of an open sea - seawater source is currently not subject to the requirements of the surface water treatment rule in Washington State (Part 6 WAC 246-290). Estuary and brackish sources of water may be subject to the surface water treatment rule, depending on the degree of surface runoff supplying the proposed intake and the viability of protozoa. All of Washington State's seawater RO systems at the time of this document revision are small systems located in northern Puget Sound.

Check with the DOH regional engineer for the county that your project will be located to determine applicable treatment requirements and process control/monitoring parameters for your specific project prior to initiating design and /or planning.

F.6.1 General Water System Information

Provide the following general information:

A. Water system name and public water system identification number
B. Owner's name, address, and telephone number
C. Manager's name, address, and telephone number
D. Operator's name, certification level, and telephone number
E. Completed (preliminary) Purification Plant Rating Criteria Worksheet (include all existing and proposed treatment processes)
F. Completed Project Submittal Application Form

F.6.2 Permitting (verify with local, state, tribal and federal agencies)

Generally the county planning department will be the lead agency that will issue final approval for the facility. However, many different governmental agencies must review and approve a desalination facility. The following list includes key permits / reviews that will be required. Check with all levels of government including tribal authorities if your project involves tribal lands.

A. Water Right Permit (contact the Department of Ecology for current requirements)
B. Shorelines Permit (county) (includes SEPA)
C. Building Permits (county)
D. Discharge Permit/NPDES (WA Department of Ecology)
E. 401 Water Quality Certification (WA Department of Ecology)
F. Washington Department of Fish and Wildlife (Hydraulic Project Approval – HPA)
G. Washington Department of Natural Resources (Aquatics Resource Use Authorization and Easement)

H. U.S. Fish and Wildlife and the National Marine Fisheries Service (both review for impact to ESA listed species and comment through the USACE permitting process)

I. U.S. Army Corps of Engineers (Joint Aquatic Resource Permit Application-JARPA – Section 404 or section 10 permit) (Section 404 is for placing fill in marine waters and section 10 is for working in marine waters)

J. Franchise from County Public Works (depends on the location of waterlines, etc.)

K. WA Department of Health (Engineering report, construction documents approval)

F.6.3 Pilot Study

Pilot studies are important in determining RO pretreatment requirements and potential fouling characteristics of the raw and pretreated water. A pilot study may not be necessary if other plants with the same raw water quality are operating near the proposed project. Contact the appropriate ODW regional office to discuss. If a pilot study is needed, refer to Section 10.3 for guidance.

Whether a pilot study is conducted or not, the water system's management and operators should visit similar plants to understand of the complexity of the treatment processes and its operational and maintenance requirements in order to inform decision-making and system design.

F.6.4 Project Report

The engineering or project report should cover design issues as described in Chapter 2 and Chapter 10. Redundancy, accessibility, manned and remote operations, alarm conditions, and treatment monitoring and performance expectations should be considered in treatment design. Special attention should be given to the following:

A. Intake and Brine Discharge Considerations

i. Shallow well/infiltration gallery location and design. The intake should be greater than 200 feet from any source of contamination (septic drain field, fuel storage, chemical storage, waste discharge). These types of intakes are not recommended due to significant problems associated with maintenance and with access.

ii. Direct seawater intake should be located to consider ease of maintenance, protection from damage (for example, by boat anchors), and the potential for contamination by fuel spill and sewage discharge. Environmental concerns will drive both intake and discharge design. Redundant piping should be considered when intakes are located in sensitive areas.

iii. Intake pipe fouling from mollusks can be a significant issue affecting ongoing operations depending upon intake location. Design must address this issue. Successful approaches have included alternating the intake line and the brine discharge lines (brine kills the mussels); oversizing intake lines and physical cleaning (requires access ports).

iv. Intake pump considerations. The intake pump, whether submersible (in seawater) or located on a dock, is subject to severe environmental stress. Careful attention to the quality of stainless steel selected is a must, as is attention to the use of different metals in proximity to each other. If a submersible pump designed for seawater cannot be found, a standard pump may be used with frequent monitoring, and inspection and repair of the pump incorporated into the design.

B. Raw water quality including temperature and salinity by season. Puget Sound waters are generally high quality but are subject to seasonal turbidity events, changes in salinity and very cold temperatures. Local areas may have significant algal blooms and be subject to mollusk fouling. This information may need to be acquired from existing nearby seawater RO plants.

C. Treated water design criteria. Total dissolved solids (TDS) and conductivity are usually used to control permeate acceptance. Conductivity is directly measured and then an assumed salinity conversion factor is used to estimate TDS. The conversion factor used must be identified. The design engineer must affirm that the conversion factor is appropriate for the Washington State seawater intake location. Finished water alkalinity and pH goals must also be identified. Finished water TDS following pH and alkalinity must be less than 500 mg/L. TDS of the RO permeate should be less than 350 mg/L.

D. Membrane design criteria and selected membrane characteristics (e.g., membrane type and configuration). ANSI/NSF 61 certification is required. Expected recovery rate (percent), useful life of membrane, and expected membrane replacement schedule must be determined.

E. Membrane cleaning. If cleaning is to be performed onsite, environmental issues for waste disposal must be addressed.

F. High pressure pump design pump sizing. Pump sizing is frequently performed in conjunction with the seawater RO membrane supplier. Consideration for startup and shutdown must be included along with the ability of all components to handle expected operating pressures.

G. Energy recovery systems. A significant portion of the energy used for the high-pressure RO feed pump can be recovered from the concentrated brine stream, thereby reducing energy costs. Use of energy recovery systems is recommended.

H. Pretreatment of the seawater. Pretreatment is the most critical component for a successful treatment system. Puget Sound waters are generally high quality but are subject to seasonal turbidity events, and very cold temperatures. Local areas may have significant algal blooms and be subject to severe mollusk (primarily mussels) fouling. Successful installations to date have pretreatment operating independently of the seawater RO system. Pretreatment systems deliver water to a filtered / treated seawater tank or tanks. The RO high-pressure pump draws from the seawater tank. Pretreatment considerations may include turbidity reduction, Fe/Mn removal, anti-scale stabilization, microbial control, organics reduction and hardness reduction.

I. Brine as a resource. Brine is a filtered water source, not just a waste stream. Store on site to use to backwash a pressure media filter or for use to inhibit marine growth in intake lines.

J. Equipment operation and maintenance. Describe instrumentation and controls, including alarms, telemetry for remote operation, and provisions for protecting instrumentation and electrical components from corrosion.

K. Product water corrosivity. The corrosive nature of RO permeate requires corrosion control treatment for all seawater RO installations. Blending with other sources as the sole means of corrosion control is not acceptable. Alkalinity and pH treatment ranges must be identified. Specific instruments for measuring alkalinity and pH must be identified. These instruments must be compliant with industry standards such as Standard Methods.

L. Disinfection. Disinfection capable of reliably achieving the CT6 disinfection requirements of WAC 246-290-451 in order to achieve 4-log virus inactivation is required. Water temperatures may be very cold which will affect contact time requirements.

M. Material compatibility. Piping and fittings material compatible with seawater and low ionic strength permeate water must be carefully evaluated.

N. Operation and maintenance. The treatment plant design should give extra thought to equipment accessibility for maintenance. Some examples include:
   i. Setting the high pressure pump at waist height with full access for maintenance and repairs (HP pumps typically require frequent maintenance)
   ii. Providing for open space to allow membrane removal from the pressure vessels.
   iii. Locate sampling taps and water meters for easy access and reading.
   iv. Label all treatment plant piping and equipment.

O. Operator qualifications. A certified operator will be required to operate this treatment facility. Operator certification requirements and the availability of qualified operators is a critical component for determining if seawater RO is a viable and sustainable option.

P. Noise. Sound proofing and noise abatement should be evaluated.

F.6.5 Operations and Maintenance Manual

I. Identify maintenance personnel and operators.

II. Outline routine inspection and maintenance - daily, weekly, monthly, annually.

III. Identify major equipment components and their manufacturers.
   i. List spare parts, chemicals, and supplies to be kept on hand.

IV. Establish written procedures for
   i. Water quality and treatment performance data collection and record-keeping.
   ii. Chemical addition and determining chemical dosages.
   iii. Conductivity-TDS meter calibration and maintenance, testing and reporting processes.
   iv. Membrane cleaning/rejuvenation/replacement, including the care and storage of the RO membranes when not in operation. (We recommend using non-chlorinated permeate water).
   v. Plant start-up and shut-down

V. Prepare disinfection byproduct monitoring plan (not applicable to transient non-community systems).

VI. Identify the appropriate ODW monthly treatment plant report form. Obtain the applicable reporting form from the ODW regional engineer.

VII. Confirm the certified operator has reviewed and given the opportunity for input on the design and O&M plan.

F.6.6 Other Design Considerations

A. All components in substantial contact with the product or raw water, including membranes, must be ANSI/NSF Standard 61 certified.

B. All chemicals used must be within their ANSI/NSF Standard 60 approved doses.

C. Use non-corrosive materials (stainless steel, PVC, fiberglass) throughout the treatment plant.

Appendix F.7 Rainfall Catchment Submittal Requirements

Rooftop catchment involves the collection of rainfall from an elevated roof surface. Rainwater collected from ground catchment areas is considered storm water. Depending upon the specific roof characteristics for collecting rainfall, roof top water can be subject to animal and human pathogens, material degradation from roof components, and wind-blown contaminants.

Additionally, most rooftop catchment systems collect untreated rainwater and store the untreated water in large cisterns. This cistern water may be stored for months before the water is used. Additional water quality changes can occur during this protracted storage. For these reasons, roof top catchment systems are treated as surface water sources for purposes of treatment design and treatment operations.

WAC 245-290-130 requires water systems to obtain drinking water from the highest quality source feasible. WAC 246-290-420 requires water systems to provide an adequate quantity and quality of drinking water in a reliable manner. Use of collected rainwater from a roof top catchment system poses significant challenges to satisfying these basic requirements. We know from experience small water systems struggle with demonstrating compliance in operating a small surface water treatment plant. As presented in Appendix F.8, we believe it is difficult to develop a reliable roof top catchment system in most areas of the state without incurring significant construction and ongoing operation/maintenance costs.

Submittal Outline

The design submittal requirements for a new surface water source and potable water treatment facilities are substantial. Even very small rainfall rooftop catchment designs must meet all of the requirements of Part 6 of WAC 246-290. Refer to additional guidance in Appendix A.3.8 and WSDM Chapter 11.

I. General Water System Information

Provide the following general information:

A. Water system name and public water system identification number

B. Owner's name, address, and telephone number

C. Manager's name, address, and telephone number

D. Operator’s name, certification level, and telephone number

E. Completed (preliminary) Purification Plant Rating Criteria Worksheet (include all existing and proposed treatment processes)

F. Completed Project Submittal Application Form

II. Checklist of Additional Items

A. Rainfall is acidic and very low in dissolved solids and alkalinity. As a result, it can leach contaminants from materials it contacts. For this reason all roofing materials that are in contact with rainfall must be certified under ANSI/NSF 61 or NSF P151. This requirement extends to all equipment and appurtenances located on the roof that are substantially contacted by rainfall, including solar panels, satellite dishes, chimneys and vents.

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B. Because of rainfall's corrosive potential, evaluate the need for corrosion control treatment.

M.

C. Address all applicable new source and treatment considerations (e.g., blending with other sources and impact on distribution system).

D. Water rights administered by the Washington State Department of Ecology may be required depending upon the location and size of the project.

E. Provisions for on-going raw water cistern inspection, water quality sampling, and cistern cleaning and maintenance must be incorporated into the design.

F. Address service capacity (source capacity and storage) requirements if the rooftop catchment source is intended to meet part or all of the water system's source firm yield requirements without depending on trucked water. Refer to Appendix F.8.

N.

G. Review the Department of Ecology's policy on rainwater collection. The policy can be viewed on Ecology's Rainwater Collection web page.

Appendix F.8 Rainfall Catchment Reliability Analysis – Example

Introduction

The main points of this reliability analysis are:

- The importance of assessing rainfall data collected over many years.
- The source of supply/reliability analysis must be based on drought conditions. Designing a roof top catchment system based on average rainfall data will result in an unreliable water supply.
- Reliability analysis should be based on the 50-year drought, using monthly (not annual) rainfall data.

Annual Rainfall

The generally accepted equation for rainfall yield from a roof surface is:

> 0.8 (efficiency of rainfall capture) x 0.62 (gallons per inch of rainfall per square foot of roof) x SF (capture area of roof) x inches of rainfall per time period

For example, a building with a 1,500 SF roof capture area receiving an average annual rainfall of 40 inches per year will yield a water supply volume of 0.8 x 0.62 x 1,500 x 40 = 29,760 gallons, sufficient to provide the equivalent of 81 gallons per day (average daily supply). As described below, calculating an average supply from an average annual rainfall does not result in a reliable potable water supply.

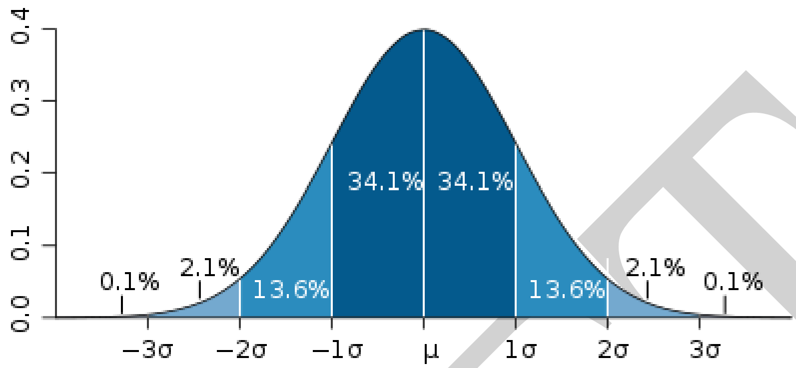
Many locations in Washington have extensive data on monthly and annual rainfall (see Appendix C for weather and rainfall data agency web pages). Rainfall information was reviewed from three locations, each with 62 years of data (1949-2010):

- Olympia
- SeaTac
- Orcas Island

Rainfall information for areas east of the Cascade Range indicate less average rainfall and longer, more severe drought potential compared with the three locations noted above.

We calculated the mean (μ), standard deviation (σ), and -2σ of this data in an effort to assess the feasibility of rainfall rooftop catchment (RRC) for community water systems.

Figure 1
Graphical Depiction of Standard Deviation



Assuming annual rainfall data follows a normal distribution, 2.2 percent of annual values are expected to be below **-2σ**. The probability of annual rainfall occurring below **-2σ** in any given year is approximately 1 in 50.

Below is the annual rainfall distribution curve for SeaTac. Rainfall frequency was measured in 5-inch increments.

Figure 2
Average Annual Rainfall Distribution for SeaTac

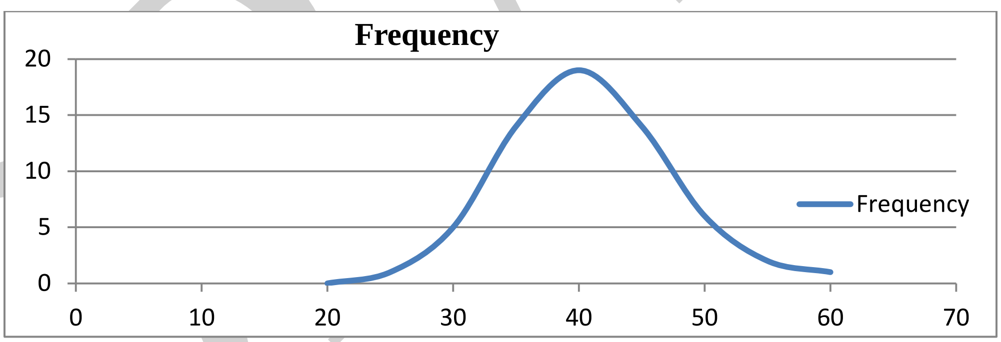


Table 1
Statistical Information on Annual Rainfall in Selected Washington Cities (inches)
1949-2010

Location	μ	Median	σ	Rainfall at -2σ	Range of Measured Rainfall
Olympia	50.63	52.31	8.37	33.89	29.92 to 66.71
SeaTac	38.2	39.73	6.58	25.04	23.78 to 55.14
Orcas Island	28.65	30.42	4.29	20.07	17.07 to 37.21

The annual rainfall for the design year will define the threshold for where we consider RRC to be a viable sole-source of supply to a public water system. Section 5.9.1 and 5.11.1 recommends using a 98 percent level of reliability, equivalent to a 50-year drought, for surface supplies (including RCC).

We can approximate the threshold of low annual rainfall predicted once every 50 years (two percent chance of occurrence in any given year) by applying the **-2σ** value for rainfall at a given location.

Applying the **-2σ** rainfall value assumes a ~ 98 percent reliability standard. This standard implies a 1-in-50 chance that in any given year the rainfall will be less than the **-2σ** rainfall value, and a 1-in-5 chance that the rainfall will be less than the **-2σ** rainfall value during any consecutive 10-year period.

Table 2
Average Daily Water Supply per Dwelling (based on rainfall at -2σ)

Location	Rainfall at -2σ	Ave Daily Supply (1500 SF Roof per Dwelling)	Ave Daily Supply (2000 SF Roof per Dwelling)
Olympia	33.89 in	68 gpd per house	91 gpd per house
SeaTac	25.04	50	67
Orcas Island	20.07	40	53

For reference, Forks, Washington has a **-2σ** rainfall value of 82 inches per year (based on the same 62-year data period: 1949-2010), providing an average daily supply of 165 gpd per dwelling.

Appendix D implies an appropriate design value for indoor use as 200 gallons per dwelling per day in the absence of existing system information.

Monthly Rainfall

Annual rainfall dictates the feasibility of captured rainfall as a public drinking water supply. The distribution of the annual rainfall dictates the reservoir (cistern) size needed to assure a continuous supply of water during periods of little or no rainfall. Data from the three locations were reviewed to determine the number of years in which the aggregate rainfall measured in any consecutive three-month period totaled less than 1.0 inches. The results are tabulated below:

Table 3
Critical 3-Month Dry Periods

Location	Number of Years with 3-Month Total Rainfall Less Than 1.0 Inches	Minimum 3-Month Total Rainfall		
		Rainfall	Year	Equivalent supply[1]
Olympia	4 (1951, 1970, 1998, 2006)	0.45 inches	1970	3.6 gpd
SeaTac	2 (1967, 1987)	0.84 inches	1987	6.9 gpd
Orcas Island	1 (1951)	0.94 inches	1951	7.7 gpd

1. Average daily supply available from rainfall during this 3-month period, based on a 1,500 sf roof and 91 days

There are also longer periods of relatively dry weather. In 1987 there was a five-month period (June – October) with total rainfall of 2.06 to 2.09 inches at these three locations. This level of rainfall equates to an average daily supply of 10 gpd per dwelling during the five-month period.

Based on the history of monthly rainfall, the cistern size needed to overcome the uneven distribution of rainfall during the year at these three locations would have to be on the order of five months' supply, equal to about 30,000 gallons per dwelling unit assuming 200 gallons per day average daily demand. Storing captured rainfall for many months will result in significant degradation of water quality.

Providing for fire flow, disinfection contact time, a factor of safety (e.g., to account for climate change), and accounting for hydraulic inefficiency of the treatment process will only serve to increase the total storage (untreated and treated) necessary.

Conclusion

The annual and monthly variability of rainfall makes harvested rainfall difficult to justify as a sole-source drinking water supply for community water systems in all but the wettest parts of Washington State.

Appendix F.9 Plumbing Sample to a Process Analyzer

The following information was created by and copied with permission from the Hach Company, Terry Engelhardt, Author and Application Development Manager for the Hach Company, 2018.

I. Location, location, location - If the sample is not right the analysis is not right!

1. Is the sample well mixed and representative?
   a. Any chemical addition which can affect the measurement should be added far enough ahead of the sample point that it is well mixed and in solution. For example, if pH is to be measured after lime is added for pH adjustment, the measurement needs to be made far enough downstream that all the lime is in solution.
   b. If a chemical addition will interfere with measurement of the parameter of interest the sample should be drawn before the point of chemical addition. For example, suspended lime particles can cause positive error in particle count measurements following filtration. Measure particle counts in filter effluent prior to lime addition.
2. Where possible avoid turbulent flow conditions – i.e. sample locations near valves, ells, tees, flanges. Turbulent flow conditions may dislodge scale, introduce severe entrained air and cause other sample anomalies which will lead to inaccurate measurement.

II. Keep it short, keep it simple, and don't delay

1. The instrument should be located as close as is practical to the sample location. If the sample to be measured is in the pipe gallery, then the instrument should be there too! Avoid piping sample long distances. Exceptions to the 'keep it short' rule include –
   a. Do not mount instruments in locations where the environment will be hostile to the function and life of the instrument, i.e. areas with corrosive gases or strong electromagnetic fields.
   b. Do not mount instruments in locations that are hazardous or difficult to access for operations and maintenance personnel. All instruments require periodic maintenance. If instruments are difficult or hazardous to access to perform required maintenance, they won't be maintained and they won't work!
2. Use sample lines that are corrosion resistant, small diameter and maintain high sample velocity
   a. Where possible use opaque, black plastic or metallic pipe/tubing compatible with the sample to be measured. The sample line must be opaque to block all light thus discouraging biofilm.
   b. Use the smallest diameter tubing that will deliver adequate volume and maintain high velocity.
      i. Select small diameter sample line to minimize the volume contained in the sample line and to maintain a high velocity.
      ii. Maintaining high velocity in the tubing will discourage accumulation of biofilm and solids which will affect accuracy.

c. Don’t delay - Keep it short, keep it simple, maintain high velocity. See Table 1.

III. Do not mix and match plumbing fittings!

1. Do not come off a steel pipe with a galvanized nipple, connected to a brass valve which will then connect to a copper sample line before getting to the analyzer. This is all too common and creates a galvanic corrosion cell at every point of contact between dissimilar metals.

2. Get to corrosion resistant materials as quickly as possible with minimal changes in the composition of materials of the plumbing fittings. Preferably, install corrosion-resistant metal or plastic materials.

Table 1
Sample Delay in minutes per Foot of Sample Line
(Neglecting friction loss which will further increase the delay)

Sample Flow Rate, ml/min	Sample Flow Rate, gal/min	Copper Tubing (Nominal size, ID)			Schedule 40 PVC (Nominal size, ID)		
		1/4” 0.315	3/8” 0.43	1/2” 0.545	1/4” 0.364	3/8” 0.493	1/2” 0.602
300	0.08	0.051	0.095	0.153	0.068	0.125	0.186
500	0.13	0.031	0.057	0.092	0.041	0.075	0.112
750	0.20	0.020	0.038	0.061	0.027	0.050	0.075
1000	0.26	0.015	0.029	0.046	0.020	0.038	0.056
1500	0.40	0.010	0.019	0.031	0.014	0.025	0.037
2000	0.53	0.008	0.014	0.023	0.010	0.019	0.028
3785	1.00	0.004	0.008	0.012	0.005	0.010	0.015
		Schedule 80 PVC (Nominal size, ID)			Tygon Tubing (Nominal size, ID)		
		1/4”	3/8”	1/2” 0.562	1/4” 0.125	3/8” 0.25	1/2” 0.375
300	0.08	N/A	N/A	0.162	0.008	0.032	0.072
500	0.13	N/A	N/A	0.097	0.005	0.019	0.043
750	0.20	N/A	N/A	0.065	0.003	0.013	0.029
1000	0.26	N/A	N/A	0.049	0.002	0.010	0.022
1500	0.40	N/A	N/A	0.032	0.002	0.006	0.014
2000	0.53	N/A	N/A	0.024	0.001	0.005	0.011
3785	1.00	N/A	N/A	0.013	0.001	0.003	0.006

IV. Go with the flow – Every place for a valve and every valve in its place

1. Controlling flow and pressure is important for all measurements, critical for others. Instruments such as amperometric chlorine analyzers, particle counters and turbidimeters are very sensitive to changes in flow and pressure

2. Valves – so many choices.

   a. Needle valves – avoid. Needle valves typically are incorporated with rotameters. Seems simple, measure flow and control flow all in one. Just say no!

   i. Use a rotameter to measure flow if you wish. They are simple, easy to install and provide sufficient accuracy.

ii. Do not use rotameters with integral needle valves for flow control. Needle valves create air bubbles and easily become fouled with solids carried in the sample stream and corrosion products that may slough off of plumbing fittings.

b. Ball and gate valves are commonly used to throttle (control) flow but they are not designed for flow control. Use ball and gate valves for on/off control. Ball valves are convenient in that they are normally ¼ turn from closed to fully open making operation fast and easy.

c. Globe valves are designed to control flow. Use globe valves to control flow to instruments.

V. Keep it right down the middle

1. Avoid sampling from the top or bottom of a pipe. Sampling from the top will often result in problems with air in the sample. Sampling from the bottom will be non-representative due to solids which will inevitably accumulate at the bottom of the pipe.

2. Typically it is best to sample from the side of a pipe ± 45°.

3. It is desirable to sample from the middle of the pipe rather than from the edge, especially on larger diameter pipe (say, 6” diameter or larger). Samples drawn from the edge of a pipe can be non-representative due to fouling from sediment/biofilm accumulated on the wall of the pipe.

4. Sample probes or sample quills are available from a number of manufacturers. Some of these are fixed in place, others are retractable. The probes/quills are available in a variety of materials. The probe pictured at right is retractable. And, it has multiple ports so the sample is a cross section of the pipe rather than one point – a feature may be desirable in some instances, especially larger diameter pipes.

VI. Pump It Up – or not.

1. Avoid pumping sample where possible. Pumping can change the sample by entraining air, changing sample temperature (thus changing solubility of some substances) contributing corrosion products and/or changing the nature (i.e. size or shape) of suspended material.

2. When pumping is unavoidable:

a. Avoid pumps which cause pulsation (diaphragm, piston, and peristaltic pumps). Pulsations can cause measurement irregularity. If unavoidable, use a pulse dampener after the pump.

b. Centrifugal pumps are typically preferred to positive displacement pumps for providing samples to analyzers due to lower cost and centrifugal pumps are easier to throttle (control flow and pressure).

c. Select a pump with components compatible with the sample. A pump with all composite (non-metallic) wetted parts is desirable. Pumps with metallic wetted parts can create measurement errors by introducing corrosion products.

d. Avoid excessive suction lift. At sea level an ideal pump has a maximum lift of about 34 feet. Considering pump efficiencies, and other factors, even at sea level the practical suction lift is only about 25 feet. At a mile above sea level (e.g., Denver, CO) the practical lift is only about 21 feet. Pump manufacturers will typically specify the maximum lift for their pump. Operating a pump near suction lift design limits will cause cavitation and other problems. The suction side of a centrifugal pump should not be restricted - the valve on the suction side should always be fully open. Where suction lift is too great, one should consider a submersible pump. (Suction head is the measure, in feet, of the pumped liquid level above the centerline of the pump inlet. Suction lift is the measure, in feet, of the pumped liquid level below the centerline of the pump inlet.)

Appendix F.10 Iron and Manganese Treatment by Oxidation - Filtration

Submittal Outline

This outline is intended to guide and summarize your design of iron (Fe) and manganese (Mn) treatment facilities for small water systems using groundwater sources. Refer to Appendix F.5 if your Fe/Mn treatment process will use ozone as a pre-filter oxidant.

Various surveys of iron and manganese treatment facilities in the USA have shown that only 50 to 60 percent of the facilities produced water that met drinking water standards for iron (Fe) and manganese (Mn). The following factors are known to be important in the successful, reliable operation of a Fe/Mn removal treatment facility using oxidation/filtration.

- Correct oxidant dosage is applied.
- The oxidation pH is sufficiently high and oxidation time is sufficiently long to ensure conversion of soluble Fe/Mn to the oxidized state.
- Properly sized filter media.
- Complete and accurate raw water data at time of design, including an understanding of iron complexation with humic substances or with silica.
- Appropriate filtration rate.
- Adequate backwashing frequency, rate, and control. Proper monitoring and control of the backwash recycle return (if applicable).

Submittals for treatment facilities must be prepared by an engineer licensed in Washington State. All supporting documentation **must** be included with the design submittal (WAC 246-290-110).

I. General Water System Information

Provide the following general information:

A. Water system name and public water system identification number

B. Owner's name, address, and telephone number

C. Manager's name, address, and telephone number

D. Operator’s name, certification level, and telephone number

E. Completed (preliminary) Purification Plant Rating Criteria Worksheet (include all existing and proposed treatment processes)

F. Completed Project Submittal Application Form

II. Description of the Water Quality Problem

Describe the source of supply and the purpose and goals of the treatment proposed. Include source number and design flow rate(s).

III. Raw Water Quality

The raw water quality of most groundwater supplies can vary seasonally and over time. For this reason, a minimum of two separate measurements should be collected for the parameters listed below, ideally capturing seasonal changes in raw water quality. In aquifers where the water quality is known to vary significantly, such as some island aquifers, additional raw water quality sampling is recommended. To get accurate data, temperature, ferrous iron and pH measurements must be performed at the well site (not in a lab) by a qualified person with a properly calibrated instrument. All other water quality parameters must be analyzed by a laboratory certified for drinking water analysis. Submit all lab data sheets.

Raw Water Quality Table

Water Quality Parameters	Comment
Total Iron (mg/L)	The concentration of total iron can affect the filter run time, treated water quality, and economics of the treatment process.
Ferrous (Fe^{+2}) Iron (mg/L)	Ferrous iron exerts an oxidant demand requiring higher concentrations of oxidant addition.
Manganese (mg/L)	The concentration of total manganese can affect the filter run time, treated water quality, and economics of the treatment process.
Hardness (mg/L as $CaCO_3$)	Calcium and magnesium can compete with biding sites on ion exchange resins.
Alkalinity (mg/L as $CaCO_3$)	Higher alkalinity waters require more chemical addition to adjust the pH if needed.
Ammonia (mg/L)	Ammonia can impact the treatment process if chlorine is used as an oxidant.
TOC (mg/L)	TOC can cause fouling of filter media, exert an oxidant demand, and lead to the formation of disinfection byproducts. TOC can impact treatment performance at 1.0 mg/L and be especially problematic at concentrations greater than 2.0 mg/L.
Temperature (ºC)	Temperature can impact the kinetics of the treatment process. Usually not a significant factor.
pH	If air or chlorine are used as an oxidant, the oxidation of manganese is very slow at pH less than 8.0.

IV. Pilot Testing

Pilot testing is usually necessary to determine that a treatment process is functional and economically viable, as well as helpful in developing the appropriate design parameters for the treatment process including:

A. Oxidant type, dose and contact time.
B. Optimal pH and adjustment (if necessary).
C. Filter media type, depth, and loading rate.
D. Backwash frequency, duration, and hydraulic rate.
E. Filter-to-waste duration and rate.

F. Backwash recycle return (flow rate, volume per filter backwash cycle, quality) if applicable.
G. Other process control parameters (if used) for parameters such as pH adjustment.

We recognize many small systems installing Fe/Mn treatment may choose to skip the pilot test step and go straight to a full-scale design (see Section 10.3.5). Design engineers and water system owners who choose to skip the pilot study step do so at their own risk. Design engineers should discuss with the regional engineer their justification for skipping the pilot test step before proceeding to full scale design.

Adequate piloting provides greater assurance of treatment performance. At a basic level, the project report that summarizes the pilot testing should describe:

H. Pilot plant setup, duration (see Table 10-2) and results as they relate to full scale treatment design.
I. Pilot plant design parameters:
    i. Treatment rate of the pilot plant (gpm/sq.ft.)
    ii. Oxidant and dosage (mg/L) (if applicable)
    iii. Length of oxidation and coagulant contact times. Contact time is detention time from point of oxidation or coagulation addition to filter.
    iv. Backwash parameters (filter run length/volume (hours or gals), backwash rate (gpm), duration (min)), and recycle return (if applicable)
    v. pH adjustment (if necessary)

V. Summarize Oxidation-Filtration Treatment Components

Include schematic drawing of the treatment system identifying:

A. Major system components
B. Process control stations, such as water quality sampling points (raw, post-oxidant, after each filter, combined filter effluent) flow meter(s), and pressure gauges, chlorine residual analyzer(s), turbidity meter(s)).

VI. Full-Scale Design

The design engineer should cover the following items in the project report and construction documents:

A. Process Control: Document location of and interaction between these process control components:
  i. Sample locations:
    a. Raw water (before any treatment)
    b. After oxidant addition

c. After each filter
d. Combined filter effluent
e. Backwash recycle return (if applicable)

ii. Physical parameter and water quality analysis:
a. The frequency of monitoring should be identified for the flow, oxidant residual, pressure, and filtration volume or run time.
b. On-line/continuous water quality instrumentation can improve process control and aid in trouble shooting, so the use of chlorine residual analyzers, pH analyzers, and turbidimeters should be evaluated.

iii. Process control narrative describing:
a. Process control parameters (oxidant residual, target pH), means of process control, and benchmarks for successful operations.
b. Capacity for remote operations
c. Alarm and shutdown conditions

B. pH Adjustment: Document pH adjustment design basis (if applicable)
i. Chemical used, dosage (mg/L), and target pH range.

C. Oxidation: Document oxidation process design basis
i. Oxidant type
ii. Oxidant dose (mg/L)
iii. Target oxidant residual (mg/L)

If ozone is used as an oxidant, see Appendix F.5 for submittal guidance.

iv. Contact Time (sec) between oxidant and filter.

D. Filter media
i. Type, depth (usually at least 36 inches), effective size, and loading rate.
Note: Filtration rate usually less than 5 gpm/sf for effective filtration, though may be higher provided raw water quality is suitable and if demonstrated through pilot testing.
ii. Expected replacement frequency and sensitivity to oxidants.
iii. ANSI/NSF Standard 61 certification.

E. Backwash: Document backwash design criteria and process
vi. Describe design objective
a. Optimize finished water quality
b. Minimize backwash volume per cycle

c. Maximize finished water volume

vii. Identify backwash initiation

a. Headloss, psi or feet
b. Time since last backwash, hours or days
c. Volume of filtered water, gallons

viii. Identify backwash hydraulics:

a. Flow rate (gpm/sf). Identify the manufacturer's recommended backwash application rate in gpm/sq.ft.
b. Identify the backwash pump pressure in psi. Attach pump curve. Verify adequacy of system hydraulics for the proposed backwash.
c. Backwash duration, in minutes. We recommend a visual means of confirming adequacy of BW duration, such as a segment of clear pipe.
d. Volume, gal
e. Verify that no cross connection exists between the backwash source water and the wastewater.

ix. Backwash disposal

a. Volume of backwash per cycle, average day, and peak day volume of backwash.
b. Describe disposal of backwash, and include all backwash disposal facilities in construction documents.
c. Confirm that the proposed method of backwash waste disposal is acceptable to the Department of Ecology and the local health department. See Chapter 10 for guidance on permitting water treatment plant waste disposal.

F. Backwash Recycle Return: Document backwash recycle return design basis (if applicable)

i. Backwash storage and return

a. Backwash holding tank volume, gallons
b. Detention time, hours
c. Supernatant recycle return volume, gallons
d. Supernatant recycle return flow, gpm. The recycle return flow should not exceed 10 percent of the total influent to the filters.

ii. Conditioning supernatant recycle return

a. Bag/cartridge filter
b. Chemical addition
c. Turbidity monitoring of supernatant recycle return stream

iii. Identify backwash recycle return initiation

a. Volume of backwash water, gallons

b. Time since last backwash recycle return, in hours or days. Note: Recycling on a volume basis rather than time is recommended, as production varies throughout the year.

G. System Hydraulics

i. Describe source-pumping mode (pumps directly to storage or to distribution).

ii. Define the current installed source pumping capacity in gpm.

iii. Verify that the installed pumping capacity is adequate to meet current design standards with the proposed treatment on line. Discuss all components of the total pumping head (well pump lift, system elevation difference, treatment plant head loss, system head losses, and residual pressure).

VII. Operations and Maintenance

A. Prepare an O&M manual section, which includes:

i. Identify maintenance personnel and operators.

ii. Outline routine inspection and maintenance - daily, weekly, monthly, annually.

iii. Identify major equipment components and their manufacturers.

iv. Identify a record keeping system to track treatment system performance.

v. Disinfection byproduct monitoring plan if chlorine or ozone is used (not applicable to transient non-community systems).

vi. Identify the appropriate ODW monthly treatment plant report form. Obtain the applicable reporting form from the ODW regional engineer.

vii. Confirmation the certified operator has reviewed and given the opportunity for input on the design and O&M plan.

viii. Fe and Mn test kits in specifications.

Appendix F.11 Iron and Manganese Treatment by Sequestration

Sequestration (also called stabilization, chelation, or dispersion) by application of a polyphosphate enables soluble iron and manganese to remain in solution, even in the presence of an oxidant like chlorine. It is intended to preserve the water's physical aesthetic by preventing the formation of particulate iron and manganese and its attendant turbidity. Sequestration does not remove iron or manganese, and will not re-suspend particulate iron or manganese back into their soluble form.

Design Limitations

There are several limitations to sequestration design engineers should recognize, including:

- Sequestering is not recommended when combined iron/manganese level is above of 0.5 milligram per liter (mg/L), and will not be approved if the combined iron/manganese level is in excess of 1.0 mg/L, with the manganese at no more than 0.1 mg/l as Mn.
- Addition of sequestering agents such as the polyphosphates (hexametaphosphate, trisodium phosphate) must be done prior to any oxidation influence.
- Concentrations of polyphosphate cannot exceed 10 mg/L as PO_4 in the distribution system.
- The polyphosphate must be listed under ANSI/NSF Standard 60 and the dose must fall below the NSF-approved dose.
- Because polyphosphate is a bacterial nutrient and can lead to bacterial growth in distribution lines, industry standards dictate post-sequestration chlorine disinfection shall be provided and a detectable chlorine residual shall be maintained throughout the distribution system.
- Polyphosphate addition to the drinking water supply will increase the phosphate concentration in wastewater effluent. Design engineers should consult with the local wastewater treatment authority on the advisability of adding polyphosphate to the drinking water supply.
- Polyphosphates can increase lead solubility (Holm and Schock, 1991). For this reason, if polyphosphates are added to a water supply, additional tap sample monitoring under the Lead and Copper Rule will likely be required.
- To prevent oxidation of the iron or manganese before they are stabilized, the polyphosphate should be added into the well near the suction side of the pump to minimize oxidation by aeration.
- The application point for the disinfectant should be more than 10 feet downstream of the pump discharge. A greater distance may be required by manufacturer's recommendations.
- Sequestering agents are effective in cold water, but lose their capability in heated or boiled water. It should be recognized that this form of treatment may not resolve customer concerns for hot water portions of domestic service.

- ODW will require installation of Fe/Mn removal treatment if we determine that sequestration is ineffective at mitigating aesthetic water quality issues.

Pilot Testing for Sequestering – Laboratory Bench Scale Tests

When sequestering is considered for iron/manganese control, we recommend the following steps to determine the dosage of sequestering agent needed for proper iron/manganese control:

1. Treat a series of 1-liter samples with a standard chlorine solution to determine the chlorine dose required to produce the desired free chlorine residual. The minimum target free chlorine value should be 0.2 mg/L.
2. Prepare a standard sequestering agent solution by dissolving 1.0 gram of agent in a liter of distilled water in a volumetric flask.
3. Treat a separate series of five 1-liter samples with varying amounts of the sequestering agent. One milliliter (ml) of the standard sequestering agent solution (prepared as per step 2 above) is equivalent to a 0.1 percent solution. One ml of this stock solution in one liter of sample is equivalent to 8.34 pounds of sequestering agent per million gallons, equal to one part per million (ppm). Add 1, 2, 3, 4, and 5 ml dosages to the 1-liter samples and stir until the sequestering agent dose is well-mixed. Continue to stir while adding the previously determined chlorine dosage to avoid localized high chlorine concentrations.
4. Observe the series of treated samples against a white background to note the degree of discoloration. The proper dose of sequestering agent is the lowest dose that delays noticeable discoloration for a 4-day period. This dose cannot exceed 10 mg/L as PO_4 or the NSF-approved maximum dose for the polyphosphate sequestering agent.

Initial water quality testing should include:

Raw Water Quality Table

Water Quality Parameters	Comments
Ferrous Iron (Fe^{+2}) (mg/L)	Sequestration is not recommended when the concentration is greater than 0.5 mg/L.
Manganese (Mn^{+2}) (mg/L)	Sequestration is not recommended when the concentration is greater than 0.1 mg/L.
Hardness (mg/L as $CaCO_3$)	Hardness ions (calcium and magnesium) can bind with the sequestering agent.
Alkalinity (mg/L as $CaCO_3$)	Higher alkalinity waters require more chemical addition to adjust the pH if needed.
Temperature (ºC)	Sequestering is not effective for hot water and the sequestering agent breaks down more rapidly in warm water, though temperature is usually not an issue for most groundwater supplies.
pH	Sequestering is more effective at a lower pH (less than 7.5) since iron and manganese are more readily oxidized at a higher pH.

Note: *Samples for the above bench test should be collected freshly, kept away from direct sunlight to avoid heating, and maintained at room temperature for the duration of the test.*

Public Notification

The sequestration design submittal should include a public notification for distribution to consumers. The notification should inform consumers that they may still experience discoloration and particulate problems with the hot water portion of their home plumbing. In addition, customers located in more remote portions of the water distribution system should be informed that discoloration and particulate matter may still pose aesthetic problems if water in their portion of the distribution system is not routinely flushed.

Distribution System Related Problems

Occasionally, complaints about aesthetic concerns are not directly attributable to source water levels of iron or manganese. Existing water systems should examine the nature of any consumer complaints to determine if the problem is water source or distribution system related.

Corrosion within the distribution system may contribute to aesthetic problems at consumers' taps, including red, orange, or brown-colored water, odor, and/or particulate matter. Before initiating a sequestration design, the design engineer should determine the observed aesthetic problems are not the result of distribution system corrosion. If distribution system corrosion is determined to be the problem, treatment options should be targeted at ways to mitigate problems associated with water corrosivity (e.g., orthophosphate addition, pH or alkalinity adjustment).

Recommended References

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Appendix F.12 Nitrate Removal by Ion Exchange

Submittal Outline

This outline is intended to guide and summarize the design of nitrate (NO_3) treatment facilities for small water systems using groundwater sources. Refer to Section 10.2.5 and DOH 331-309 for additional guidance.

Ion exchange is the most common treatment process employed in Washington State for the removal of nitrate in drinking water. The following factors are known to be important in the successful, reliable operation of a nitrate removal treatment facility using ion exchange.

- Complete and accurate raw water data at time of design, including an understanding of seasonal changes in raw water quality.
- Clearly defined O&M practices and clearly defined role of the operator to fulfill O&M responsibilities.

Submittals for treatment facilities must be prepared by an engineer licensed in Washington State. All supporting documentation **must** be included with the design submittal (WAC 246-290-110).

I. General Water System Information

Provide the following general information:

A. Water system name and public water system identification number

B. Owner's name, address, and telephone number

C. Manager's name, address, and telephone number

D. Operator’s name, certification level, and telephone number

E. Completed (preliminary) Purification Plant Rating Criteria Worksheet (include all existing and proposed treatment processes)

F. Completed Project Submittal Application Form

II. Description of the Water Quality Problem

Describe the source of supply and the purpose and goals of the treatment proposed. Include source number and design flow rate(s).

III. Raw Water Quality

The raw water quality is critical to determining whether ion exchange is an appropriate treatment process, sizing equipment, resin selection, brine regeneration frequency, waste volume generated, and the need for post-ion exchange pH adjustment/corrosion control. Nitrate ion exchange treatment facilities should be pilot tested at the site, capturing raw water seasonal variability. Certain raw water quality tests should also performed on site.

The raw water quality of most groundwater supplies can vary seasonally and over time. For this reason, a minimum of two separate measurements should be collected for the parameters listed below, capturing seasonal changes in water quality. In aquifers where the water quality is known to vary significantly, such as some island aquifers, additional raw water quality sampling is recommended. To get accurate data, temperature, ferrous iron and pH measurements must be performed at the well site (not in a lab) by a qualified person. All other water quality parameters must be analyzed by a laboratory certified for drinking water analysis. Submit all lab data sheets.

Raw Water Quality Table

Water Quality Parameters	Comment
Nitrate (mg/L)	The concentration of nitrate and how much it varies seasonally affects the design of the treatment process
Total Organic Carbon (mg/L)	TOC can foul ion exchange resins and compete for adsorptive sites on the resin. Pretreatment for TOC removal may be necessary, especially at concentration of TOC greater than 2.0 mg/L.
pH	Anion exchange can cause a significant drop in pH, making the water more corrosive.
Sulfate (mg/L)	Sulfate competes with nitrate for binding with the ion exchange resin, creating the need for more frequent regeneration.
Iron (mg/L)	Iron can foul ion exchange resins, so pretreatment may be required if the combined concentration of iron and manganese exceeds 0.1 mg/L.
Manganese (mg/L)	Manganese can foul ion exchange resins, so pretreatment may be required if the combined concentration of iron and manganese exceeds 0.1 mg/L.
Alkalinity (mg/L as $CaCO_3$)	Anion exchange removes carbonate from the water which decreases the pH and dissolved inorganic carbon usually making the water more corrosive.
Hardness (mg/L as $CaCO_3$)	High hardness can lead to mineral precipitation, thereby fouling the ion exchange resin.
TDS (mg/L)	A useful general parameter for assessing the potential for competition and fouling of the resin.
Turbidity (NTU)	High turbidity can foul the resin leading to increased headloss and the need for more frequent regeneration.

IV. Pilot Testing

Pilot testing is usually necessary to determine that a treatment process will be functional and economically viable, as well as helpful in developing the appropriate full-scale plant design parameters including:

A. Necessity for pre-treatment such as softening or pre-filtration
B. Necessity for post-treatment pH adjustment
C. Resin type

D. Resin bed volume

E. Flow rate

F. Empty bed contact time

G. Process control to start regeneration cycle

If a pilot test is not conducted then the design engineer should submit at least two complete sets of raw water sample results reflecting seasonal variability to the ion exchange equipment manufacturer. Request a written assessment of removal efficiency, treatment capacity (volume) and salt use between regeneration cycles.

H. Describe the pilot plant setup, duration (see Table 10-2), and results as they relate to full scale treatment design.

I. For each resin type tested describe pilot plant test results including:

   i. Bed volume

   ii. Surface loading rate through the reactor

   iii. Raw and treated water nitrate concentration, pH, and sodium concentration

   iv. Bed volumes treated before regeneration

   v. Predicted waste volume (sum of backwash and rinse, including softener if applicable), expressed as a percent of water treated during the previous exchange cycle.

   vi. Predicted salt use per gallon of water treated during exchange cycle (including softener, if applilcable).

J. Describe how long the pilot plant was operated and the seasonal changes seen in water quality that may affect the performance of the proposed treatment plant or require operational adjustments.

V. Summarize Ion Exchange Treatment Components

Include schematic drawing of the treatment system identifying:

A. Major system components.

   i. We recommend redundant treatment facilities for sole-source drinking water supplies.

   ii. Pre-treatment softening and/or filtration (if needed).

   iii. We recommend providing a treated water storage reservoir with a volume equal to the average daily demand located prior to entry to the distribution system. The storage reservoir can serve as a buffer between an undetected treatment failure and consumers, prevent some nitrate MCL violations, and provide a source of water for backwashing and rinsing.

   iv. Post-treatment corrosion control/pH adjustment (if needed).

B. Process control, such as water quality sampling points, flow meter(s), clock, pressure gauges, in-line analyzer(s).

VI. Full-Scale Design

A. Process Control: Document location of and interaction between these process control components:

i. Sampling taps:
   a. Raw and treated water
   b. Blending point (if treated and untreated water are blended prior to entry to distribution)

ii. Water quality analysis:
   a. Nitrate
   b. pH

iii. Other process control parameters:
   a. Flow rate
   b. Volume treated
   c. Pressure
   d. Time

iv. Process control narrative describing:
   a. Process control parameters (time, volume, nitrate concentration, pH), means of process control, and benchmarks for successful operations.
   b. Capacity for remote operations
   c. Alarm and shutdown conditions

B. Treatment Process

i. Prefiltration

ii. Softening prior to nitrate ion exchange

iii. Resin Tanks
   a. Number of tanks
   b. Flow path (series or parallel)
   c. Total resin bed area (sq ft)
   d. Loading rate (gpm per sq ft)
   e. Resin depth (ft)
   f. Empty bed contact time (EBCT, in minutes).

iv. Resins
   a. Resin type, trade name, and manufacturer

b. Manufacturer’s published removal efficacy based on raw water quality parameters
c. Manufacturer’s published limitations of use (e.g., pressure, loading rate, raw water quality, chlorine)
d. NSF 61 certification
e. Life expectancy (e.g., number of regeneration cycles) and assumed decline in resin efficiency during each year of operation.

C. Backwash/Regeneration: Document backwash design criteria and process

i. Identify backwash initiation
   a. Time since last backwash, hours or days
   b. Volume of filtered water, gallons

ii. Identify backwash hydraulics:
   a. Flow rate (gpm/sf). Identify the manufacturer's recommended backwash application rate in gpm/sq.ft.
   b. Identify the backwash pump pressure in psi. Attach pump curve. Verify adequacy of system hydraulics for the proposed backwash.
   c. Backwash duration, min
   d. Volume
   e. Verify that no cross connection exists between the backwash source water and the wastewater.

iii. Backwash/Brine Rinse disposal (include softener if applicable)
   a. Describe constituents, average day, and peak day volume of backwash.
   b. Calculate average daily salt disposal (lbs/day)
   c. Describe disposal of backwash, and include all backwash disposal facilities in construction documents.
   d. Confirm that the proposed method of backwash waste disposal is acceptable to the Department of Ecology and the local health department. See Chapter 10 for guidance on permitting water treatment plant waste disposal.

D. System Hydraulics

i. Describe source-pumping mode (pumps directly to storage or to distribution).
ii. Define the current installed source pumping capacity in gpm.
iii. Verify that the installed pumping capacity is adequate to meet current design standards with the proposed treatment on line. Discuss all components of the total pumping head (well pump lift, system elevation difference, treatment plant head loss, system head losses, and residual pressure).

E. Corrosion Control

  i. Assess the corrosivity of the treated drinking water supply.

VII. Operations and Maintenance

A. Prepare an O&M manual section, which includes:

  i. Identify maintenance personnel and operators.
  ii. Outline routine sampling, inspection and maintenance - daily, weekly, monthly, annually.
  iii. Identify major equipment components and their manufacturers.
  iv. Identify a record keeping system to track treatment system performance.
  v. Confirmation the certified operator has reviewed and given the opportunity for input on the design and O&M plan.

Appendix G: Guidance for Leachable Contaminants Testing

The “leachable contaminants test” (a.k.a. “soak test”) can be used to better ensure that contamination from leachable components is identified and addressed when new facilities are brought on-line. This test is predominately used when there is some question about the “quality of workmanship” associated with installation of facilities, or with the materials that would be in substantial contact with drinking water.

If the initial use of a facility presents, or could present, a water quality concern (for example, storage reservoirs primarily, but could include other facilities as well), ODW can require testing of the water before service is initiated to consumers (see Section 7.6.2).

Authority for such monitoring comes from WAC 246-290-300 (1)(a)(i) if contamination is, or could be, suspected in the water system. For example, suspicion over contractor workmanship, illicit materials used, vandalism, or any number of reasons that would suggest the need to ensure appropriate water quality before using a facility in contact with the water to be provided.

For the various structures that may be of concern (such as usually new or recoated storage facilities, and some treatment facilities), testing of the water quality should be performed under defined protocols before putting those structures into service. The concerns that could be associated with various projects and the recommended procedures for conducting leachable components tests are outlined below.

Concern Associated with Paints and Coatings

There may be concerns regarding organic chemical contamination resulting from improper selection or application of paint coatings used for water storage facilities. Experience has shown that in some instances the level of organic contaminants found in drinking water can be elevated due to leaching of the coating materials. This may lead to taste/odor problems or, possibly, health concerns.

Because it is difficult to correct coating problems after their discovery, considerable care should be exercised in the selection and application of coating materials. If contamination exceeding a maximum contaminant level is found the storage facility **<u>must not</u>** be placed into service until contamination is reduced to levels below MCL (WAC 246-290-320).

Here are some precautions:

1. Only coating products that meet ANSI/NSF Standard 61 (see WAC 246-290-220) are to be used for potable water contact surfaces.

2. Only experienced and competent applicators should apply the coatings. The coating manufacturer's recommendations should be followed closely, particularly for ventilation and curing. For forced-air curing, the air should be drawn from the lowest part of the tank because many volatile organic vapors are heavier than air. If there is any doubt about the adequacy of the curing conditions, additional curing time with continued forced air ventilation is suggested. Experience shows that the curing time suggested by the manufacturer is adequate only under the optimal conditions specified by the manufacturer. Longer curing periods are needed if temperature and humidity parameters are not optimal. After the curing period, the tank must be washed and properly disinfected before filling.

Concern Associated with Concrete Construction

Some petroleum-based form-release agents used in the construction of concrete water storage facilities can be a source of organic contamination. Concerns about contamination may result from the improper selection and use of fuel oil or lubricating oil as form-release agents. Special precautions are needed to minimize the hazards associated with use of these materials.

If these products have been used, or if contamination is suspected or found, the storage facility must not be placed into service until the contamination is reduced to acceptable levels (see WAC 246-290-300).

Because it is difficult to remove all traces of petroleum contaminant from concrete, considerable care should be exercised in the selection, thinning, and use of form-release materials. Some of the important precautions are indicated below:

1. Forms should be cleaned prior to use.
2. Only form-release agents that meet ANSI/NSF Standard 60, **OR**, in some instances, food-grade vegetable oils, must be used for potable water contact surfaces (see WAC 246-290-220).
3. Thinning of form-release agents should only be done with ANSI/NSF Standard 60 approved materials, or food-grade vegetable oils.

Following the curing period, the tank **must** be washed and disinfected before filling (WAC 246-290-451).

Concern Associated with Treatment Unit Media or Membranes (Alternate Technologies)

Natural filter sand, gravel, anthracite, ilmenite and garnet may be approved prior to being placed in service, provided they have been tested for leachable contaminants.

This applies particularly to native mineral products, which are subjected only to mechanical processing, such as crushing, screening, and washing.

Vehicles used for transportation of filter media could be a significant source of contamination. Because it is difficult to remove all traces of contamination from media after it is discovered, considerable care should be exercised during the processing and transportation of filter media. Some of the important precautions are indicated below:

1. Vehicles for transporting media should be cleaned of potential contaminating substances prior to use.
2. Following media placement, treatment units should be washed, disinfected (slow sand filters excepted), rinsed and tested for coliform bacteria density before being placed in service.

Leachable Contaminant Testing Procedure

Whenever the water contact surfaces of a storage facility have been coated, or whenever "Leachable Contaminant Testing" is considered appropriate for any type of project, purveyors will be directed to take the following steps prior to putting the facility into service:

Following a period of immersion/contact time, water in the tank, vessel, basin, or treatment unit (hereafter termed, “contact facility”) must be sampled to determine the level of any leached chemicals. Although negotiable, the period for storage tank surface contact with the water is recommended at seven days, and for filter media at 24 hours. The minimum immersion contact period should at least equal, but preferably exceed the maximum anticipated operating detention times under normal operations. The operator should try to maximize the ratio of the wetted surface contact area to the volume of water in the tank. This would suggest that the tank does not need to be filled completely to the overflow level because the area-to-volume ratio continually diminishes as a circular, rectangular, or square tank is filled. It is recommended, however, that at least 10 to 20 percent of the tank volume be used for testing. There should be enough water in the contact facility to account for all contact surfaces where “quality of installation or materials handling” is a concern. Samples of the water collected after the appropriate contact period must be analyzed by a laboratory certified by the Department of Ecology. The analyses should include contaminants that reasonably may be assumed to contribute to potable water contamination by the material being evaluated. Test methods may include, but are not limited to:

- Complete inorganic chemical (IOC) analysis.
- Volatile organic chemical (VOC) analysis.
- General synthetic organic chemical (SOC) analysis, including phthalates and polycyclic aromatic hydrocarbons.
- Phthalates.

The minimum analytes required for some particular situations are:

a. For storage facilities having organic coatings, such as paints and sealants, the sample should be analyzed for VOCs. If the product contains phthalates, that parameter should also be analyzed.

b. For concrete construction, the sample should be analyzed for VOCs and polynuclear aromatic hydrocarbons.
c. For polyethylene, PVC, hypalon, or other flexible-material type storage facilities where fabrication involves solvents and glues, the sample should be analyzed for VOCs and phthalates.
d. For filter media, including occlusion-type alternate technology filters, the sample should be analyzed for VOCs and regulated IOCs.

Note: *The SOC test is primarily for pesticides. It only needs to be considered if concern exists over possible contamination by these types of organic compounds. This may be a concern if the material were transported, or stored in a way that would expose it to these SOCs.*

Note: *When reporting the test results to ODW, identify the sample purpose on the Water Sampling Information sheet as "Investigative." This will ensure the test results—particularly those with detections—are not treated as compliance samples under the source monitoring requirements. A copy of the testing results should be submitted directly to the ODW regional engineer reviewing the project.*

Before delivering water from a storage facility to consumers, the purveyor must evaluate test results for compliance with the current maximum contaminant levels (MCLs). ODW will not allow any storage facility or treatment component to be placed into, or remain in, service if its use would result in, or could be expected to result in, delivery of public drinking water that contained any contaminant(s) exceeding any current MCL.

If contaminants are found in the sample at levels above ODW's monitoring trigger levels, but below MCLs, additional samples should be collected at least quarterly. Such monitoring will remain in place until the contaminant concentrations fall below detection or are determined to be reliably and consistently below the MCL.

Note: *The source susceptibility rating will not change as long as the source of contamination is determined to be independent of the source water quality.*

*ODW **may** advise retesting, testing for additional analytes, or both when contamination exceeding the laboratory detection level is found.*

Appendix H: Slow Sand Filtration

Submittal Outline

This outline is intended to supplement design guidance for slow sand filtration provided in Chapter 10 and 11, and in Appendix A.3.8.

O. General Water System Information

Provide the following general information:

G. Water system name and public water system identification number
H. Owner's name, address, and telephone number
I. Manager's name, address, and telephone number
J. Operator's name, certification level, and telephone number
K. Completed (preliminary) Purification Plant Rating Criteria Worksheet (include all existing and proposed treatment processes)
L. Completed Project Submittal Application Form

I. Pilot Plant Testing

Inadequate pilot testing has resulted in treatment process failures, delayed implementation of effective treatment, and costly replacement of inadequate treatment. For these reasons, pilot plant testing, including the submittal of a pilot testing plan, is required. At a basic level, the project report that summarizes the pilot testing should describe:

A. Pilot plant setup, duration and results as they relate to full scale treatment design.

B. Pilot plant design parameters:

   v. Pre-treatment (if any).
   vi. Impacts of seasonal water quality changes.
   vii. Treatment rate of the pilot plant (gpm/sq.ft.).
   viii. Sand specification and depth.
   ix. Support gravel specification and depth.
   x. Terminal headloss.

II. Design Guidance

Based on pilot testing results, the design must consider the following (WAC 246-290-110):

A. Suitability of slow sand filtration for the available raw water supply.
B. Potential for, and evaluation of, pre-treatment needs.
C. Impact of seasonal water quality changes.
D. Suitability of specific filter sand and support gravels intended to be used.
E. Performance demonstrated at proposed hydraulic loading rate.
F. Length of time needed to commission (ripen) a new filter.

G. Rate of headloss development and length of time between filter scrapings.
H. Ripening time and proposed ripening indicators (e.g. turbidity, coliform counts, time, etc.).

III. Other Design Guidance

Address the following in the project report and construction documents:

A. Filter effluent rates (Hydraulic Loading Rates, or HLR) must not exceed 0.10 gpm/ft^2 (0.24 m/hr) (WAC 246-290-654).
    a. For cold temperatures (water temperatures less than 5°C) - a flow rate not exceeding 0.05 gpm/ft^2 is recommended.
    b. For warmer temperatures higher filtration rates (up to 0.10 gpm/ft^2 maximum) may be needed to maintain adequate dissolved oxygen (D.O. >6 mg/l) within the schmutzdecke. Under low D.O. conditions, metals can be mobilized.
    c. Effect of cold water temperatures on performance and need for covering or other mitigation for freezing weather.

B. Filtration area and number of filters needed to meet system demand, including during cleaning/ripening.
    a. Provide a minimum of two filter beds, each filter capable of meeting peak day demands. If more than two filters are used, maximum day demands should be able to be met with the largest filter out of service.

C. Optimal flow control strategy.
    a. Ensure design provides for continuous operation without significant filter effluent flow rate variation. Any filter effluent flow variations assumed in the design should be gradual to limit detachment of particles from the sand, with less than 50 percent flow change in any 24-hour period.

D. Filter-to-waste and filter backfilling capabilities.
    a. Provide filter-to-waste, or return the filter effluent to the headwater when system demands are low. Intermittent or ON/OFF operation should not be used as a means of rate control.

E. Cleaning frequency and most appropriate method(s). If scraping is the intended filter cleaning method:
    a. Sand quantity removed annually through scraping/cleanings, and duration between resanding.
    b. Address removal, handling, cleaning and stockpiling arrangements for scraped sand.

c. Consideration should be given to allow scraped sand to be washed and stored on-site so it can be reused. Storage area should be covered, and protected from contamination.

F. Assessment of appropriate automation and supervisory control and data acquisition (SCADA) systems.

G. Operator time needed for normal operations, monitoring, reporting and maintenance tasks

IV. Checklist of Additional Items

A. Influent water should be introduced to the supernatant water with enough clearance above the sand (at least 12 inches) to prevent turbulence scouring. Energy dissipating structures may also be used to prevent sand scour.

B. The filtered water elevation must be maintained at or above the sand bed level in order to prevent air binding within the filter. An effluent weir is a simple and effective way to accomplish this.

C. Consider the following for improved safety and ease of operations:

   a. Avoid confined space entry
   b. Provide adequate head space
   c. Eliminate trip hazards
   d. Prevent situations that might make maintenance activities more difficult.

V. Operations and Maintenance

Operations and maintenance manuals and monitoring data collection, recording and reporting procedures should be drafted as part of the design phase in order to facilitate a clear understanding of how the plant will be operated and filter performance monitored. Operations staff should be consulted early in the design phase and also encouraged to visit other slow sand plants.

Table-top exercises for start-up, normal operations, and cleaning procedures should be conducted with designers and operations staff in order to identify design-related operational problems in time to be corrected prior to construction.

Provide the following information in the O&M manual and submit to ODW:

A. Identify maintenance personnel and operators.

B. Outline routine inspection and maintenance - daily, weekly, monthly, annually. Include specific design criteria and specifications that will trigger certain O&M activities, such as resanding or scraping filters.

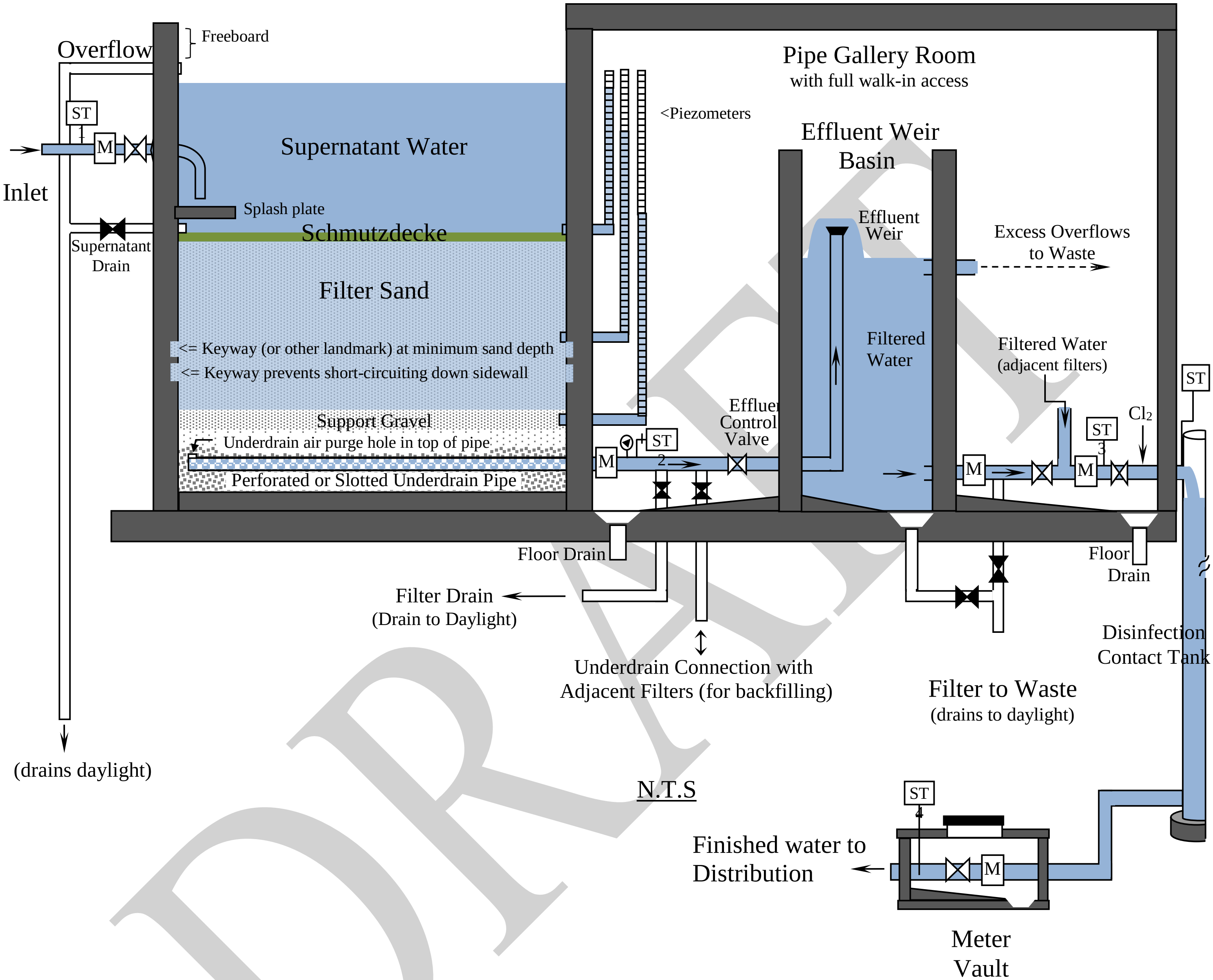
C. Identify major equipment components and their manufacturers.

D. Identify a record keeping system to track treatment system performance.

E. Identify the appropriate ODW monthly treatment plant report form. Obtain the applicable reporting form from the ODW regional engineer.

F. Confirmation the certified operator has reviewed and given the opportunity for input on the design and O&M plan.

Recommended Slow Sand Filter
General Plant Layout



M - Meter
ST - Sample Tap
ST1 - Raw Water monitoring point – turbidity, coliform
ST2 - Individual Filter Effluent (IFE) monitoring – turbidity, coliform
ST3 - Combined Filter Effluent (CFE) turbidity monitoring point
ST4 – Distribution Entry Point monitoring for chlorine, temp, and pH
⧓ - Valve: normally closed (NC)
⋈ - Valve: normally open (NO)

Recommended Raw Water Quality Limits
following roughing filtration pre-treatment, if present

Parameter	Limit	Notes
Turbidity	< 10 NTU (colloidal clays are absent)	Operation is more efficient with lower, consistent turbidity in the 5-10 NTU range. Most slow sand plants successfully treat water (after any pre-treatment such as roughing filters if applicable) with a turbidity of less than 10 NTU (Slezak and Sims, 1984), which is recommended for an upper limit in designing new facilities. Colloidal clays may penetrate deeper into the filter bed causing higher effluent turbidity and may cause long-term filter clogging. Roughing filters can provide up to 50-90% of turbidity removal (Wegelin et al., 1996).
True Color	< 5 platinum color units	The source of color should be determined. Color from iron or manganese may be more effectively removed than color from organics. True color removals of 25% or less were reported by Cleasby et al. (1984). The point of consumer complaints about water aesthetics is variable over a range from 5 to 30 color units, though most people find color objectionable over 15 color units (USEPA, 1999). The Secondary Maximum Contaminant Levels (SMCL) for color is 15 color units, which is also identified as a maximum level for slow sand filtration under the Recommended Standards for Water Works, 2012 Edition. Pre-ozonation or granular activated carbon may be used to reduce color.
Coliform Bacteria	< 800/100 ml (CFU or MPN)	Coliform removals through slow sand filters range from 1 to 3-log (90 - 99.9%) (Collins, M.R. 1998).
Dissolved Oxygen	> 6 mg/l	Dissolved oxygen is critical for maintaining a healthy schmutzdecke for proper filtration. Potential problems resulting from low DO include tastes and odors, dissolution of precipitated metals such as iron and manganese, and increased chlorine demand (Ellis, 1985). Aim for a filtered water DO at or above 3 mg/l.
Total Organic Carbon (TOC)	<3.0 mg/l (low TOC prevents DBP issues)	Recommendations for dissolved organic carbon (DOC) concentrations range from < 2.5 – 3.0 mg/l in order to minimize the formation of disinfection byproducts (DBP) in the finished water. DOC removal in slow sand filters is < 15-25% (Collins, M.R. 1989). About 90% of TOC is DOC (USEPA, 1999). Total Organic Carbon (TOC) removal is variable and may range from 10 – 25% (Collins et. al, 1989; Fox et al, 1984). Determining DBP formation potential may provide additional information by simulating DBP formation in the distribution system due to the addition of disinfectants in the presence of organics.
Iron & Manganese	Each < 1 mg/l	Slow sand filters remove iron and manganese by precipitation at the sand surface. This can enhance organics removal, but too much iron and manganese precipitate can clog the filters. The Secondary Maximum Contaminant Level (SMCL) for iron is 0.3 mg/l and the SMCL for manganese is 0.05 mg/l. Iron and manganese removal in a slow sand filter can be > 67% (Collins, M.R. 1998).
Algae	< 200,000 cells/L (depends upon type)	By providing greater surface area for particle removal, certain types of filamentous algae may enhance biological activity and be beneficial for filtration, but in general, the presence of algae reduces filter run length. Filter clogging species are detrimental to filtration and the presence of floating species may shorten filter run length due to the associated poorer-quality raw water (see the table below for common algal species). Microscopic identification and enumeration is recommended to determine algae species and concentration.

Classification of Common Algal Species[1]

Filter Clogging[2]	Filamentous	Floating
Tabellaria Asterionella Stephanodiscus Synedra	Hydrodictyon Oscillatoria[3] Cladophora Aphanizomenon Melosira	Protococus Scenedesmus Synura Anaaoena[3] Euglena

Footnotes:

[1]Table adapted from Table 10.2, Water Treatment Plant Design, AWWA/ASCE/EWRI, 2012.
[2]Diatoms of all species can generally can cause clogging due to their rigid inorganic shells.
[3]Can also release algal toxins (Microcystin and Anatoxin-a, among others).

Filter Sand Specification
Recommended Range

Parameter	Limit
Effective Diameter (d10)	0.2 – 0.35 mm (No. 70 Sieve = 0.212 mm; No. 45 Sieve = 0.355 mm)
Uniformity Coefficient (UC)	1.5 – 3.0
% Fines passing #200 sieve (75 µm)	< 0.3% by Wt.
Acid Solubility	< 5%
Apparent Specific Gravity	$\geq$ 2.55
Sand bed depth, initial	> 31 inches > 36 inches is recommended to allow for a sufficient number of scrapings before resanding is needed
Minimum operating sand bed depth prior to resanding. This minimum sand bed depth is in addition to the amount of sand anticipated to be removed due to cleanings throughout the life of the filter.	19 inches A horizontal keyway incorporated into the walls along the entire perimeter of the filter with the bottom of the keyway at the 19-inch level serves the dual purpose of indicating the absolute minimum sand bed depth while preventing raw water from seeping down the side walls of the filter to the under drains. A second keyway or scribe mark situated at the 21 to 23-inch level can indicate when the bed is approaching the minimum level.

Delivery/Installation

Sand should be washed thoroughly to remove deleterious materials like clay fines and organics prior to placement. Keep sand clean and only store on a clean, hard, dry, covered surface until placement. Refer to ANSI/AWWA Standard B100-09 or latest revision for additional storage and handling information.

Underdrain Design Parameters

Recommended Specification

Parameter	Limit	Notes
Maximum velocity in laterals	0.75 fps (0.23 m/sec)	According to the 2012 Edition of the Recommended Standards for Water Works (Ten States Standards), each filter should be equipped with a main drain and an adequate number of lateral underdrains to collect the filtered water. The underdrains should be placed as close to the floor as possible and spaced so that the maximum velocity of the water flow in the underdrain will not exceed the value stated.
Maximum velocity in main drain	0.75 fps (0.23 m/sec)	
Spacing of lateral drain pipes	36 inches (91.4 cm)	Although spacing lateral drains up to 79" (2 m) may be satisfactory due to the low hydraulic resistance in the support gravel, a smaller spacing increases the uniformity of flow through the drains (Hendricks, et al., 1991, p. 112). Recommended maximum lateral spacing from Ten States Standards (2012)
Spacing of bottom lateral drain holes	4 – 12 inches (0.1 – 0.3 m) Placed as close to the filter floor as possible and secured in place to prevent movement.	The underdrain system should ensure uniform flow through the overlying sand bed. This is achieved by having a uniform distribution and sufficient number of collection orifices and designing the ratio of the orifice area/conduit (pipe) area such that the headloss within the underdrain pipe is negligible relative to the orifice (Hendricks, et al., 1991, p. 108). This yields a headloss through the drain holes much greater than the headloss in the laterals and main drains to ensure the even flow distribution. The diameter and spacing of the underdrain pipes and the diameter of the orifices should be determined theoretically by hydraulic calculations.
Diameter of drain holes	1/4-inch (6.35 mm)	Include air release holes or slits at the top near the midpoint of the main drain and each lateral. Alternatively, slotted drain pipe may be used where the width of the slots is in the 5/64" – 5/32" (2-4 mm) range, provided the headloss through the slots is determined to be much greater than the laterals and main drains.
Material	PVC or other noncorrosive material meeting ANSI/NSF Standard 61)	

Support Gravel

Support gravel should conform to published design guidelines (see endnote references, below). An example of a 5-layer support gravel system is provided for filter sand with an effective size of 0.2 mm – 0.35 mm using design guidelines from Appendix D of ANSI/AWWA Standard B100-09 and commercially available gravel sizes according to standard sieve sizes under ASTM E11-13. The gravel support using 5 layers as shown below will work if the orifices in the under drain pipe are less than or equal to 1/4" in diameter. 4 layers are adequate with 1/8" (3.175 mm) diameter drain orifices and a bottom gravel layer of 1/2" x 1/4" (12.7 x 6.35 mm).

Support Media[1]	Passing Screen Size (largest particle)[2]	Retaining Screen Size (smallest particle)[2]	Depth of Layer[3]	Criteria (The actual ratio demonstrating how well the criteria has been met is provided in parentheses)
Layer 1 - Top Layer ("very coarse" sand)	No. 10 Sieve (2 mm)	No. 20 Sieve (0.85 mm)	3 inches (76.2 mm)	Within Layer 1: The largest particle size in layer 1 is less than or equal to 2 times the size of the smallest particle size in layer 1. (2.4) Between Layer 1 and the Filter Sand: The smallest particle size in layer 1 is between 4 and 4.5 times the smallest effective size of the filter sand. (4.2)
Layer 2 (No. 6 Sieve x No. 12 Sieve)	No. 6 Sieve (3.35 mm)	No. 12 Sieve (1.7 mm)	3 inches (76.2 mm)	Within Layer 2: The largest particle size in layer 2 is less than or equal to 2 times the size of the smallest particle size in layer 2. (2.0) Between Layers 1 and 2: The largest particle size in layer 2 is less than or equal to 4 times the smallest particle size in layer 1. (3.9)
Layer 3 (1/4" x No. 6 Sieve)	1/4" (6.3 mm)	No. 6 Sieve (3.35 mm)	3 inches (76.2 mm)	Within Layer 3: The largest particle size in layer 3 is less than or equal to 2 times the smallest particle size in layer 3. (1.9) Between Layers 2 and 3: The largest particle size in layer 3 is less than or equal to 4 times the smallest particle size in layer 2. (3.7)
Layer 4 (1/2" x 1/4")	1/2" (12.5 mm)	1/4" (6.3 mm)	3 inches (76.2 mm)	Within Layer 4: The largest particle size in layer 4 is less than or equal to 2 times the smallest particle size in layer 4. (2.0) Between Layers 3 and 4: The largest particle size in layer 4 is less than or equal to 4 times the smallest particle size in layer 3. (3.7)
Layer 5 - Bottom Layer (3/4" x 1/2")	3/4" (19.0 mm)	1/2" (12.5 mm)	3 inches minimum and such that the gravel completely surrounds and provides for at least 1" (25.4 mm) of cover over the laterals and main drain to provide for a level surface for upper gravel layers	Within Layer 5: The largest particle size in layer 5 is less than or equal to 2 times the size of the smallest particle size in layer 5. (1.5) Between Layers 4 and 5: The largest particle size in layer 5 is less than or equal to 4 times the smallest particle size in layer 4. (3.0) Between Layer 5 and the Underdrain: The smallest particle size in layer 5 is at least twice the size of the underdrain orifice size. (2.0)

Delivery/Installation

Support media should be washed thoroughly to remove deleterious materials like clay fines and organics prior to placement. Keep media clean and only store on a clean, hard, dry, covered surface until placement. Refer to ANSI/AWWA Standard B100-09 or latest revision for additional storage and handling information.

Footnotes

[1]Refer to ANSI/AWWA Standard B100-09 or latest revision for more detailed specifications.
[2]No more than 8% by dry weight of particles should be greater than the passing screen size and no more than 8% by dry weight of particles should be smaller than the retaining screen size.
[3]The thickness of each layer of support gravel should be at least 3 times the diameter of the largest particles. For practical reasons, the thickness of each layer should be 2-3 inches for coarse sand and gravel up to 1/2" (12.7 mm) in size. Keep gravel clean and only store on a clean, hard, dry, covered surface until placement. Gravel should be washed thoroughly to remove deleterious materials like clay fines and organics prior to placement. Layers should be placed to a uniform thickness, leveled, and washed in succession according to ANSI/AWWA Standard B100.

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Appendix I: Ultraviolet Disinfection

Executive Summary

This appendix identifies specific technical and regulatory issues associated with the approval and use of ultraviolet (UV) disinfection treatment for drinking water systems. It identifies the following conclusions and requirements:

- ODW will require that each typical reactor design undergo third party dosimetry-based validation testing prior to being approved for use in Washington State.
- 186 mJ/cm^2 is the minimum required applied dose for UV disinfection systems designed to provide 4-log inactivation of viruses.
- 40 mJ/cm^2 is the minimum required reduction equivalent dose where UV disinfection is used for compliance with the Surface Water Treatment Rule.
  Note: *This assumes virus inactivation is accomplished in conjunction with another disinfectant.*
- Where UV is used to comply with the Long Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR), the validated dose will be as specified by the rule and the dosimetry-based validation testing for a proposed UV reactor.
- The quality of the raw water can greatly affect inactivation of target microorganisms with UV, and must meet specific criteria.
- Acceptable operations and maintenance procedures must be established during the design stage of UV disinfection system approval. UV reactors are complex systems and are not suitable for water systems without appropriately certified operators.
- UV reactors are validated using certain component parts. As a result, changes in UV reactor components may decrease the effectiveness of the reactor to inactivate pathogens. As a result, disinfection credit awarded to a UV reactor is limited to the parts used in the original validation. Any modification to the UV reactor, such as replacement of lamps, sleeves, ballasts, sensors, UV transmittance monitors, or controls, with parts other than those originally specified, may result in loss of UV disinfection credit.

Background and Purpose

There has been increasing interest in the potential use of UV light for disinfection purposes. There are several reasons for the interest, including a desire to avoid or eliminate the use of chlorine as a disinfectant, concern over formation of disinfection byproducts associated with chlorine, and a perceived simplicity of operation of UV systems. Of even greater interest has been the relatively recent confirmation that UV light is extremely effective against the protozoan pathogens *Giardia lamblia* and *Cryptosporidium*.

The purposes of this document are to outline the technical issues involved with using UV light and identify those considerations ODW will evaluate when reviewing specific UV disinfection system proposals.

Identifying Technical Issues

This document identifies several technical issues that are important to the use of UV light as an appropriate disinfection technique. In addition, multiple scenarios under which UV may be applied have been identified. These are:

- Groundwater sources that ODW has determined do <u>not</u> require disinfection (WAC 246-290-250(4)).
- Groundwater sources that <u>do</u> require disinfection (WAC 246-290-451).
- Surface water sources that install UV disinfection to comply with the LT2ESWTR.
- Surface water sources that meet the limited alternative to filtration criteria.
- Other surface water sources where UV is used for compliance with the Surface Water Treatment Rule.

Different technical approaches and design criteria are used in addressing these scenarios. The technical issues are presented in five categories: reactor validation, public health criteria, water quality, design, and operations. These categories are discussed below. Much of the information presented in these documents is also provided in the Ultraviolet Disinfection Guidance Manual for the Final Long Term 2 Enhanced Surface Water Treatment Rule (UVDGM) (USEPA 2006).

Reactor Validation

UV disinfection is a technically complex process and there are few water systems in the United States that have installed UV disinfection for pathogen inactivation credit. The two predominant factors that affect the performance of a UV disinfection system are the irradiance distribution in the reactor, and the hydraulic flow characteristics of the reactor. In combination, these provide a fluence, or dose distribution that is unique to the water quality treated, and the specific operating conditions (for example, flow rate, lamp age, cleanliness of quartz sleeves).

In contrast to chlorine disinfection, where the disinfectant concentration is relatively consistent and easily measured, the approximate UV analogue, irradiance, varies significantly throughout the reactor, and can only be measured in those specific locations where a calibrated sensor is fixed. In chlorine disinfection systems, reactor hydraulic residence times are relatively long, and can be characterized through field-testing with physical measurements (for example, tracer studies).

The hydraulic residence time through UV reactors is on the order of a few seconds, and cannot be readily field-determined. These complications render it impractical for a design engineer or utility operator to actually determine the *true* fluence distribution for any given UV reactor under normal operating conditions. At best, some manufacturers may utilize sophisticated computational models to estimate the fluence distribution in their reactors. Design engineers and operating utilities must be sure that UV systems have been validated as effective under known operating and water quality conditions, and actual installations must be operated within specified parameters consistent with the validation. Therefore, ODW will require that each reactor design undergo dosimeter-based validation testing prior to being approved for use in Washington State.

UV reactors **must** be validated to establish the range of operating conditions that qualify for pathogen inactivation credit. Detailed, defined testing protocols have been developed in Germany (DVGW 2006) and Austria (ÖNORM 2001; ÖNORM 2003). Reactors certified at an approved DVGW [*Deutsche Vereinigung des Gas und Wasserfaches*(German Organization of Gas and Waterworks)] testing facility using the DVGW W294 protocol or ÖNORM [*Österreichisches Normungsinstitut*(Austrian Standards Institute)] facility using the ÖNORM M5873-1 or ÖNORM M5873-2 protocol will be considered as providing a reduction equivalent dose of 40 mJ/cm^2 within the validated conditions. For other validation efforts, Chapter 5 of the final UV Disinfection Guidance Manual (UVDGM - USEPA 2006) will be used to review proposed validation protocols. A full-scale reactor must be used for all validation testing.

There are two different UV dose-monitoring strategies commonly used to control UV reactors and confirm that the reactor is providing the required dose within the validated range of operations. The sensor setpoint approach is one UV dose monitoring strategy. The other strategy is called the calculated dose approach. The DVGW W 294 and ÖNORM M5873 protocols use the sensor setpoint approach. Guidelines for the calculated dose approach are outlined in the UVDGM (USEPA 2006). The dose-monitoring strategy affects how a reactor validation is conducted as well as how the reactor is monitored after it is installed.

It will be the responsibility of the UV equipment manufacturer to submit complete validation information to ODW for review and approval. This responsibility includes submitting the details of the testing protocol to ODW prior to the start of the testing. This validation will require verification of the conditions under which at least the minimum validated dose is provided. A third party acceptable to ODW must conduct the validation, not the manufacturer or water system.

Public Health Criteria

Disinfection requirements for ground and surface water systems are addressed in chapter 246-290 WAC. These are discussed in turn below.

Groundwater Sources Requiring Disinfection

WAC 246-290-250 (4) requires groundwater sources to be continuously disinfected unless acceptable source water quality (for example, historical absence of coliform organisms) and an acceptable sanitary control area are provided. This general requirement is consistent with the more specific requirements found in WAC 246-290-451, which lists conditions under which disinfection of a groundwater supply is required; and in WAC 246-290-453, which requires 4-log inactivation of viruses in groundwater supplies under certain specified conditions, including the detection of E. coli in a triggered source sample.

When a groundwater source is contaminated, or threatened with contamination, the source is subject to a multiplicative combination of chlorine residual (C) and contact time (T) that results in a product (termed CT) of at least 6 mg/L-min. This is identified in the *Surface Water Treatment Rule Guidance Manual* (USEPA 1990) with use of free chlorine as the required level to provide 4-log inactivation of viruses in water at 10^{o}C, pH 6-9. If temperature and/or pH are outside the values necessary to provide 4-log virus inactivation at CT6, the Department may require a higher CT value. The values in EPA's guidance manual are based on hepatitis A virus inactivation data and an applied safety factor of 3. The Groundwater Rule source disinfection treatment requirements in WAC 246-290-453 requires 4-log inactivation of viruses in groundwater supplies under certain specified conditions, including the detection of E. coli in a triggered source sample. WAC 246-290-453 also imposes a higher standard for monitoring, reporting, and public notification in the event of a treatment technique violation.

In January 2006, EPA finalized LT2ESWTR. While the focus of the LT2ESWTR is on surface water sources, the rule also established UV dose requirements for viruses. In November 2006, EPA finalized the Ground Water Rule. Treatment equipment installed to comply with this rule **must** provide 4-log inactivation or removal of viruses. For UV disinfection that means the minimum required validated dose is 186 mJ/cm^2. The reduction equivalent dose for a given UV reactor will likely be greater because of the uncertainties inherent in the validation of UV reactors.

Groundwater Sources Not Required to Disinfect

ODW recommends installations provide a UV dose of at least 186 mJ/cm^2 and that water systems meet the same design criteria applied to sources required to disinfect. Water systems that install UV treatment units that have not been validated will be required to install validated equipment (or use another ODW -approved disinfectant) if disinfection is required in the future, and may be required to conduct additional source water sampling for coliform bacteria as required by the Ground Water Rule.

Surface Water

Surface water treatment regulations prescribe treatment techniques that, either in combination with filtration or alone, must achieve identified levels of inactivation and/or removal of pathogens. All surface water systems must provide a minimum of 3-log removal and/or inactivation of *Giardia lamblia* and 4-log inactivation and/or removal of viruses, and control of pathogenic bacteria. These requirements are prescribed in the federal Surface Water Treatment Rule (40 CFR 141.70 through 141.75), and chapter 246-290 WAC, Part 6. The Interim Enhanced Surface Water Treatment Rule (IESWTR) and Long Term 1 Enhanced Surface Water Treatment Rule (LT1ESWTR) impose a 2-log *Cryptosporidium* <u>removal</u> requirement on water systems that are required to filter. With the promulgation of LT2ESWTR, EPA formally recognized the effectiveness of UV for inactivation of *Giardia lamblia* and *Cryptosporidium*. The LT2ESWTR public health protection requirements are **in addition to** those required by the Surface Water Treatment Rule, IESWTR, and the LT1ESWTR. The different UV disinfection standards for disinfection of surface water sources are explained in more detail below.

Limited Alternative to Filtration

To meet the limited alternative to filtration (LAF) standards, the water system **must** provide **<u>greater</u>** removal or inactivation of pathogens for the surface water source than would be provided by the combination of chlorination and filtration. Where UV disinfection is used to meet LAF requirements, ODW has established that a minimum design dose of 40 mJ/cm^2 be used.

Long Term 2 Enhanced Surface Water Treatment Rule

The LT2ESWTR was developed to provide additional public health protection from pathogens present in surface water, especially *Cryptosporidium*. The rule requires **additional** treatment for some sources based on their source water *Cryptosporidium* concentrations and treatment currently provided. UV disinfection is one option that water systems have to comply with the additional treatment requirements.

UV light inactivates *Cryptosporidium* at relatively low doses compared to several other surface water pathogens. Therefore, the doses required are expected to be less than 40 mJ/cm^2 in most cases. The design and operation requirements for UV disinfection installed to comply with the LT2ESWTR are identified in the rule itself and associated UVDGM. The design engineer is expected to consult the LT2ESWTR and UVDGM as part of the design process if UV disinfection is installed for compliance with this rule.

Surface Water Treatment Rule

When the Surface Water Treatment Rule was finalized, it focused on the use of chemical disinfectants to inactivate waterborne pathogens. At the time, the most disinfection resistant pathogen regulated was *Giardia lamblia*. In 2001, ODW recognized research that indicated UV was effective at inactivating *Giardia lamblia* at relatively low doses, and established a minimum UV dose of 40 mJ/cm^2 when disinfection is used to comply with the Surface Water Treatment Rule. This is the same UV dose required by the widely accepted German and Austrian standards, which considered the sensitivity of a several pathogens along with their ability for enzymatic repair of their damaged nucleic acids in establishing a minimum required UV dose. This minimum reduction equivalent dose is more than sufficient to provide 1-log credit for inactivation of *Giardia lamblia*.

Surface water sources **must** also provide 4-log inactivation or removal of viruses. Because viruses are more readily inactivated by chemical disinfectants than *Giardia lamblia*, water systems that provided at least 0.5-log inactivation of *Giardia lamblia* with a chemical disinfectant readily met the 4-log virus inactivation requirement. There are some viruses, especially adenoviruses, which are more UV resistant than *Giardia lamblia*. Adenoviruses were not used to establish the German UV disinfection standard of 40 mJ/cm^2. However, the LT2ESWTR used adenoviruses to establish the UV dose tables for viruses in the rule. Based on this information, reduction equivalent doses greater than 40 mJ/cm^2 will be required for virus inactivation credit.

Disinfection **must** be continuously provided for surface water sources (WAC 246-290-662(1)). In this case, continuous is defined as a period of 15 or more minutes. If a water system fails to provide the required UV dose for 15 or more minutes, they must contact ODW. Failure to provide the required UV dose on a surface water source more than one day per month is considered a treatment technique violation (WAC 246-290-662(4)(b)).

Water Quality

Water quality can significantly influence the effectiveness of UV disinfection. Reductions in effectiveness can result from direct absorbance of UV radiation by the water as well as the various constituents in the water, by the shielding of organisms often associated with higher turbidities, and by the formation of scales (fouling) on lamp sleeves. The following list identifies several water quality parameters to be considered. Specific recommendations or requirements regarding these parameters are shown in italics.

- **Iron and Manganese:** Ferric iron strongly absorbs UV radiation. It can negatively affect a reactor's ability to inactivate microorganisms by "consuming" the UV before it can be absorbed by microorganisms. Oxides of iron and manganese can cause scaling on the quartz sleeve that would reduce the UV irradiance that enters the water column. Iron and manganese concentrations as low as 0.1 mg/L and 0.02 mg/L can cause significant fouling of quartz sleeves (Mackey et al. 2001; Chen 2009; Black and Hill 2009). Iron or manganese exceeding the secondary contaminant levels of 0.3 mg/L, or 0.05 mg/L, respectively, must be removed prior to UV application.
- **Hardness:** Hardness greater than 140 mg/L as $CaCO_3$ can cause scaling on the quartz sleeve that would reduce the UV irradiance that enters the water column (Mackey et al. 2001; Black and Hill 2009). Solubility calculations for carbonates of calcium and magnesium can be conducted to provide a preliminary screening of the likelihood of precipitation. If it appears possible, then the utility should perform pilot testing using the same lamp proposed for the full-scale application.
- **Total Organic Carbon:** Many naturally occurring organic materials in water strongly absorb UV radiation. Like iron, they can negatively affect a reactor's ability to inactivate microorganisms. The nature and amount of the specific organic carbon in the water strongly affects UV disinfection effectiveness. This should become evident through UV transmittance measurements.
- **Turbidity:** Turbidity is a measure of a solution's ability to scatter light, generally as a result of particulate matter. Turbidity alone cannot be directly correlated to a predictable effect on UV system effectiveness, and in some cases, turbidity in excess of 5 NTU has not resulted in diminished UV inactivation performance (Passatino and Malley 2001). However, turbidity control to less than 5 NTU must be provided. In groundwater, turbidity is often a result of iron or manganese precipitation, and removal of these inorganics may eliminate the turbidity problem.
- **UV Transmittance** (of the water): UV transmittance is a measure of water's ability to transmit ultraviolet radiation, and is a function of the factors identified above, as well as some water treatment chemicals (Cushing et al. 2001). The UV transmittance of raw water directly affects the ability of UV light to adequately disinfect raw water. Validation testing must be performed consistent with the UV transmittance of the water for which treatment is proposed.

Most utilities do not have historical records of many of the above parameters. A utility considering UV should begin sampling on at least a monthly basis for each of the above. More frequent testing may be required if significant variation in water quality is expected. It may not be possible to predict the fouling characteristics of any particular water, and pilot testing may be appropriate in some cases. While piloting will likely not be required, opting out of piloting may increase the risk that the facility will not operate as expected.

Design

It is important in the design of a UV disinfection system to consider a range of issues, including:

- Inlet and outlet conditions - correlate to match the validation conditions (or be hydraulically more conservative).
- Reactor isolation – Drip tight reactor isolation valves. If reactors are flooded with chemicals for cleaning, provide additional isolation such as double block-and-bleed valve arrangements.
- UV system operation to be consistent with the identified flow rate (positive flow control may be required).
- Accommodation of “Start/Stop” operation typical of many small and medium size treatment systems to account for warm-up/cool-down requirements of some UV systems (a “flow-to-waste” cycle may be needed).
- The possibility and control of hydraulic shock or potential for significant hydraulic transients.
- The need for reliability and redundancy of a treatment system (may require parallel units).
- Recognition that power fluctuations can shut down some UV systems (while not having the same effect on pumps), and the need for alarm and automatic shut-off features to prevent untreated water from entering the distribution system (an uninterruptible power supply may be applicable).
- Because UV lamps contain mercury, completion of an assessment of a particular treatment system’s vulnerability to lamp breakage and mercury release (included in an emergency response plan).
- The need for a reliable, stable, calibrated UV irradiance sensor system installed and monitored for operational control. The sensor system, which may be one or several individual sensors, is integrally included in the validation process, and must be well described in both the validation test report and the specific design report for the project.

Additional initial design considerations that will be evaluated in the review of any specific proposal can be found in the UVDGM (USEPA 2006).

UV Disinfection Design Checklist

Other recommended guidance for preparing UV disinfection submittals, in addition to what is in this appendix, includes the *Ultraviolet Disinfection Guidance Manual for the Final Long Term 2 Enhanced Surface Water Treatment Rule* (USEPA 2006), and AWWA Standard F110 (AWWA 2016). Consult these resource in addition to the items below when preparing project reports and construction documents for UV disinfection facilities.

(see next page)

Project Reports for UV installation projects should include the following at a minimum:

- **Reactor validation report** submitted and approved by ODW. The reactor validation must identify the operating conditions (flow, UV intensity, and UV lamp status) for which the minimum UV dose is provided.
- Describe CT requirements and the design factor of safety.
- **Water quality data** over a sufficient duration to adequately characterize the source water, usually monthly for a year, unless otherwise agreed to by DOH. Key water quality parameters include:
  - UV transmittance (daily)
  - Turbidity (daily)
  - Iron and manganese (Iron - at least weekly to capture transient events)
  - Hardness
  - Total organic carbon
- **UV reactor dose-response monitoring strategy**. If the calculated dose approach is used, the reactor validation must include measurements of UV transmittance and an empirical dose-monitoring equation developed through the validation testing.
- **A description of the hydraulics** including inlet and outlet conditions to be similar to or more conservative than the ones used in the reactor validation.
- **Provisions for reactor isolation** including adequate valves to prevent short-circuiting and allow for maintenance.
- **Provisions for redundancy** including providing more than one UV reactor to allow for chemical cleaning and equipment maintenance. A redundant reactor may be needed to ensure that design flows can be met.
- **Power quality analysis** including analysis of sub-second power interruptions and voltage sags for the location of a proposed UV facility. Inclusion of an uninterruptible power supply or power conditioning equipment as appropriate.
- **Lamp breakage response plan** that defines emergency response actions that will be taken, including notifying ODW if a lamp breaks. The potential for hydraulic transients should be evaluated because they may cause the quartz sleeves that house UV lamps to fail.
- **Monthly operating and monitoring report form** that is acceptable to ODW. The report form must identify the conditions for which the minimum required UV dose can be provided. Sensor checks and UV transmittance monitor checks, as appropriate, may be included on the form or as part of a separate report.
- **Describe start/stop operational operations** including flow-to-waste, flow recirculation, and other ways to minimize the amount of inadequately disinfected water entering the distribution system during reactor start-up.

- **Describe design assumptions, instrumentation, and data to be used** to continuous monitor and calculate UV efficacy (flow, UV intensity, and UV absorbance) and clearly identify alarms and shutdown conditions. See Policy F.13.

- Operations and maintenance plan, including

  - Testing/calibrating sensors, meters, and alarms
  - Cleaning procedure
  - Lamp replacement procedure
  - Training for every operator with UV reactor operational or maintenance responsibility
  - Confirmation the certified operator has reviewed and given the opportunity for input on the design and O&M plan.

- New UV reactor commissioning process

  - UV manufacturer certifies in writing that UV system is correctly installed prior to starting up the UV system.
  - Verify that upstream piping is free of rocks or debris that could damage sleeves and lamps.
  - Prepare lamp break response procedure. Procedure must include mercury release response and cleanup procedure.
  - Review UV system O&M Manual Standard Operating Protocol prior to startup.
  - Calibrate instruments, sensors, and meters supplied as part of the UV system and that will be used during testing, including UVT analyzers, UV intensity sensors, and power consumption meters.
  - Conduct dry testing first, with a follow-up period of wet testing. UV Supplier shall identify tests that shall be completed with a dry reactor and those that require wet testing. Include ancillary equipment such as flow meters and modulating valves.
  - Test UV system under all design conditions. Verify that the UV reactor is adjusting power to maintain target disinfection levels at varying flows and UVTs. Verify that the correct information is recorded and displayed for continuous monitoring and monthly reporting. Verify all alarm set points. Verify that the values reported on the UV control panel(s) match the values displayed and recorded in the SCADA system.
  - Test the UV system under a power failure scenario to demonstrate proper shutdown or flow diversion response.
  - Verify correct operation of sleeve cleaning system, if included.
  - Run the UV system for several days to verify proper performance under normal operation.

- Issue written acceptance.
- Complete applicable portions of monthly operational reports and submit to DOH.

Credit: Kim Ervin, PE, Jacobs Engineering

Operations

As with all treatment systems, UV disinfection equipment requires regular monitoring and maintenance. ODW has developed report forms for UV disinfection reporting that may be used for operational records. If disinfection credit is granted, a water system **must** submit a report to ODW on a monthly basis (WAC 246-290-480, 666, and 696). The following is a list of the minimum monitoring and reporting requirements. Individual projects may require variations on how these data are presented, however the elements identified below will be basic.

UV Disinfection Monitoring/Reporting Elements

Parameter	Monitoring Frequency	Reporting Requirement
Flow	Continuous	Peak Daily
Irradiance	Continuous	Minimum Daily Value (Amount of time below minimum allowable levels if using sensor setpoint control)
Dose	Continuous (Only applicable for calculated dose control)	Minimum Daily Value if using calculated dose control. (Amount of time below minimum required dose)
UV Transmittance (UVT)	Continuous (Continuous UVT monitoring only required for calculated dose control. A daily grab sample should be taken when sensor set point control is used.)	Minimum Daily Value. UVT during minimum calculated dose. Weekly comparison to bench-top reading. Date of most recent calibration
Power	Continuous	Daily Lamp Status OK
Lamp Operating Time	Continuous	Cumulative operation hours (note when lamp is changed)
Alarms	Continuous	Note high priority alarm conditions occurring during the month
Cumulative Number of Off/On Cycles	Continuous	Monthly total
Sensor Status	N/A	Monthly comparison of working and reference sensors. Note when system sensor calibrated by factory (min. annually)

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