North Dakota State Water Commission and the Garrison Diversion Conservancy District

Northwest Area Water Supply Project Final Report – Pre-final Design

Volume 1

June 1995

HOUSTON ENGINEERING
AMERICAN ENGINEERING
MONTGOMERY WATSON

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June 26, 1995

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Subject: Final Report – Pre-Final Design
Northwest Area Water Supply Project

Gentlemen:

Houston Engineering, Inc., along with our associates American Engineering PC, and Montgomery Watson Americas, Inc., are pleased to submit the Final Report for the Pre-Final Design for the Northwest Area Water Supply Project (NAWS).

We have enjoyed working on this project with the State Water Commission staff, in particular David Sprynczynatyk, Dale Frink and Jim Lennington. Their input and cooperation has been invaluable in the development of the Pre-Final design for NAWS.

The scope of services for the Pre-Final Design consisted of thirteen Specific Authorizations which were also referred to as Tasks. The products of six of these Tasks have been assembled and bound in two separate volumes and are presented herewith. The six tasks include:

Task 5: Design Criteria
Task 6: Determine Alternative Water Supply Sources/Systems
Task 7: Pre-Final Design of Selected Plan
Task 9: Cost Estimates
Task 10: Operation Plan
Task 11: Construction Schedule

The contents of each volume are as follows:

Volume 1: Final Report for the Pre-Final Design

Volume 2: Pipeline Plan and Profile for the Pre-Final Design of the Selected Plan

The products of the other seven Tasks, were documented and/or completed as follows:

Task 1: Study Initiation. This was a 23-page document entitled Attachment 1, Specific Authorizations, Northwest Area Water Supply Pre-Final Design Scope of Engineering Services, which was made a part of the Engineering Services Agreement for the NAWS project.

Task 2: Public Involvement. This Task consisted of conducting 10 public information meetings in the NAWS service area.

Task 3: 1993 Community Needs Assessment, July 1993 (Report bound separately).

Task 4: User Commitment. This Task consisted of soliciting Agreements of Intent from potential water users in the NAWS service area.

Task 8: Supporting Data to Environmental Assessment, March 1995 (Report bound separately).

Task 12: Water Service Agreement. This Task consisted of obtaining Water Service Contracts from potential water users in the NAWS service area.

Task 13: Project Management and Final Report. This Task consisted of the preparation of monthly status reports throughout the entire Pre-Final Design period, the completion of the Final Report – Pre-Final Design (Volumes I and II) and the Executive Summary – Pre-Final Design, December 1994 (Report bound separately).

It has been a pleasure working with the State Water Commission and Garrison Diversion Conservancy District on this project and we look forward to our continued involvement as this project moves forward into the final design.

Respectfully, submitted,

HOUSTON ENGINEERING, INC.

Henry A. Trangsrud

Henry A. Trangsrud, Project Manager

Enclosures

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Section 1

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SECTION 1

INTRODUCTION

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Section 1 - Introduction
Section 2 - Design Criteria (Task 5)
Section 3 - Determine Alternative Water Supply Sources/Systems (Task 6)
Section 4 - Development of the Pipeline Delivery System (Task 7)
Section 5 - Lake Audubon Intake (Task 7)
Section 6 - Water Treatment Facilities (Task 7)
Section 7 - Estimates of Construction and O&M Costs (Task 9)
Section 8 - Operation Plan (Task 10)
Section 9 - Construction Schedule (Task 11)

Volume 2 of the Final Report consists of the plan and profile drawings for the East, Parshall and West Systems.

SCOPE OF THE PRE-FINAL DESIGN

The Scope of Work of the individual sections contained in the Final Report of the Pre-Final Design of the NAWS project is described in the following.

Design Criteria

Development of design criteria was prepared to establish a framework for the design of the facilities and components of the NAWS project. The chosen criteria will emphasize cost effectiveness and long-term operation with reduced maintenance costs. The discussions on each criteria parameter presents the reasons behind the selection and the value of each parameter.

This section lists all the necessary design criteria, presents system component selection as needed, and provides enough detail to allow the pre-final design to proceed. The design criteria for the following items are included:

- Usage levels projected to the design year 2020
- Pipelines
- Pump stations
- Water storage facilities
- Water treatment plants
- Raw water intake
- System instrumentation and control

Section 1

HOUSTON ENGINEERING
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- Usage levels projected to the design year 2020
- Pipelines
- Pump stations
- Water storage facilities
- Water treatment plants
- Raw water intake
- System instrumentation and control

Determine Alternative Water Supply Sources/Systems

This section evaluates the utilization of alternate groundwater sources in the Minot area in addition to the Missouri River to serve the NAWS project. The western portion of the NAWS system was not included because the Williston Water Treatment Plant (WTP) relies exclusively on surface water.

The scope of this evaluation has included the following components:

- Review of the 1988 NAWS Study on surface water and groundwater supplies.
- Review of reports and on-going research by the North Dakota State Water Commission (SWC) in the Minot area.
- Identification of the potential service area that could use the Minot/Sundre aquifer water supplies following treatment at the City of Minot's water treatment plant.
- Determination of the most effective combination of groundwater and surface water to provide service to the East System.
- Determination of the quantity of Missouri River water needed to supplement the Minot groundwater supplies for serving the East System.

This evaluation details water quality and quantities available from groundwater and surface water sources in the Minot area. Water quality data are compared against NAWS water quality criteria described in the 1993 Community Needs Assessment Report (July 1993). Water quantities are compared against water demand estimates.

Development of the Pipeline Delivery System

This section describes the pre-final design of the pipeline delivery facilities for the NAWS project. The selected plan includes treatment of water from three locations. These are on the Missouri River, on Lake Sakakawea, and on Lake Audubon, at expanded and upgraded water treatment plants at Williston, Parshall, and Minot, respectively. The pipeline delivery system to service the three separate regional systems include raw water piping, distribution piping, pump stations and reservoirs, and appurtenant facilities.

The objective of this section was to prepare drawings and design reports defining the distribution system which will serve to advance the NAWS project to the phases of final design and construction.

The scope of this section includes detailed development of the following components of the distribution system:

- Pipelines
- Pipeline Hydraulics and Surge Protection
- Corrosion Control
- Pump Stations
- System Communications and Telemetry
- Storage Reservoirs

Lake Audubon Intake

This section describes the intake facilities to supply raw water from the Missouri River to the Minot WTP for the East System. The objective of this evaluation was to investigate alternative sources (Lake Audubon or Lake Sakakawea), alternative intake configurations and the potential for integrated use of the existing Snake Creek Pumping Plant.

The scope of this section includes detailed analysis of the alternatives including siting, costs and facilities layouts. The selected alternative will include the following elements:

- Intake/Pump Station Hydraulics
- Design Criteria
- Facilities Layout and Siting
- Chemical Feed Systems
- Operation and Control
- Cost Estimates

Water Treatment Facilities

This section describes the existing water treatment plants at Minot, Parshall and Williston, and the required modifications, upgrades and expansions to meet the requirements of the Safe Drinking Water Act, capacity demands and the requirements for biota-transfer control for the East System.

The scope of this section includes the detailed development of the necessary modifications of facilities at each of the three WTPs, including:

- Existing Process Descriptions
- Alternative Processes
- Design Criteria
- Preliminary Layouts
- Process Flow Schematics
- Hydraulic Profiles
- Chemical Feed Systems
- Finished Water Pumping
- Intake Modifications at the Parshall and Williston WTPs
- Cost Estimates

Estimates of Construction and O&M Costs

This section summarizes estimates of construction costs for the NAWS systems and provides a detailed segment analysis of the East, Parshall, and West System components and associated construction costs. Operation and maintenance O(&M) costs are also developed for each system. Together with information supplied by the SWC, estimates of user water rates are developed.

The scope of this section includes:

- Capital Cost and Project Cost Summaries
- Segment Analysis and Layout
- Segment Construction Costs
- O&M Cost Criteria
- Operation Costs
- Maintenance Costs
- Financing Costs
- User Costs/Rates

Operation Plan

This section describes the alternatives for management, operation and maintenance of the proposed NAWS facilities. Delineation of the legal authorization and a brief description of the alternatives are presented.

The scope of this section is to present the alternatives that were provided by the SWC.

Construction Schedule

This section develops a preliminary construction schedule based on facility construction priorities developed by the SWC to deliver the largest water users with high quality water at the earliest opportunity.

The scope of this section includes;

- List of Construction Priorities
- Description of the Construction Elements
- Construction Cost Estimates

ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
ADA	American Disabilities Act
ASTM	American Society for Testing and Materials
AWWA	American Water Works Association
cfs	cubic foot (feet)/per second

COE	U.S. Army Corps of Engineers
CRT	Cathode ray tube
D	deflection lag factor
D.I.	ductile iron
D/DBP	Disinfectants/Disinfection By-Products
DBP	disinfection by-product(s)
dBu	decibel measurement for field strength of radio signals
DIP	ductile iron pipe
e	pump efficiency
EBTG	Engineering Biology Task Group
ENR	Engineering News Record
EPA	U.S. Environmental Protection Agency
EPC	equipment protection circuits
ESWTR	Enhanced Surface Water Treatment Rule
FCC	Federal Communications Commission
fps	feet (foot) per second
ft	foot (feet)
ft^3	cubic feet
g	gravitational constant
γ	unit weight of water (62.4 lbs/ft^3)
GDCD	Garrison Diversion Conservancy District
gpd	gallons per day
gpd/ft	gallons per day per foot
gpm	gallon(s) per minute
gpm/ft^2	gallon(s) per minute per square foot
h	total dynamic head (ft)
HAA	haloacetic acids
h_b	minimum expected absolute barometric pressure head (ft)
h_f	friction and minor losses in the suction line (ft)
HP	horsepower
hp	horsepower
HPC	heterotrophic plate count
hr	hour
h_{va}	absolute vapor pressure
h_{va}	the absolute vapor pressure of the liquid at the maximum expected water temperature (ft)
HVAC	heating, ventilating and air conditioning
I.D.	inside diameter
I/O	input/output
K	bedding constant
kw	kilowatt
KWH	kilowatt hours
kwh	kilowatt hours
lbs	pounds
LCR	Lead and Copper Rule
mA	milliamp

MCL	Maximum Contaminant Level
MG	million gallons
mg/l	milligrams per liter
mgd	million gallons per day
MHz	mega hertz
mil	one thousandth of an inch (0.001 inch)
MMI	man machine interface
mo	month
MTU	master telemetry unit
N	pump rotational speed (rpm)
NAWS	Northwest Area Water Supply
NEMA	National Electrical Manufacturers Association
NPDES	National Pollutant Discharge Elimination System
$NPSH_r$	net positive suction head required (ft)
N_s	specific speed
O&M	operation and maintenance
O.D.	outside diameter
OSHA	Occupational Safety and Health Administration
P	internal design pressure (psi)
P_b	average atmospheric pressure (psi)
PLC	programmable logic controller
PRV	pressure reducing valve(s)
psf	pounds per square foot
psi	pound(s) per square inch
psig	pound per square inch gage
PVC	polyvinyl chloride
Q	flow rate (e.g. cfs, gpm)
rpm	revolutions per minute
RTU	remote terminal unit
RWA	rural water association(s)
S	allowable stress
SCADA	Supervisory Control and Data Acquisition
SCCP	pretensioned steel concrete cylinder pipe
sec^{-1}	1/second
SMCL	Secondary Maximum Contaminant Level
sq ft	square feet
SWC	North Dakota State Water Commission
SWTR	Surface Water Treatment Rule
TCR	Total Coliform Rule
TDH	total dynamic head (feet)
TDS	total dissolved solids
THM	trihalomethanes
TM	technical memorandum
TOC	total organic carbon
UPS	uninterruptible power supply
v	velocity

WAPA	Western Area Power Administration
WTP	water treatment plant
z_1	water surface elevation in pump forebay (ft)
z_2	discharge water surface elevation (ft)
z_a	altitude (ft)
µg/l	micrograms per liter

Section 2

HOUSTON ENGINEERING
AMERICAN ENGINEERING
MONTGOMERY WATSON

SECTION 2

DESIGN CRITERIA

INTRODUCTION

Section 2 describes the usage levels and the criteria needed for pre-final design of the pipelines, pump stations, water storage facilities, water treatment plants, raw water intake, and system instrumentation and control systems for the NAWS project. Where necessary, preliminary component selection precedes the design criteria selection and discussion.

USAGE LEVELS

The usage levels used in the NAWS project were divided into four categories: communities with population over 500, communities with population under 500, rural water associations, and rural areas not on rural water systems. The process of determining NAWS water usage levels and the resulting design flow rates for communities with populations over and under 500, and for rural water associations was performed during completion of updated water needs as described in the 1993 Community Needs Assessment Study. All communities and rural water associations within the project area were surveyed to identify those with an interest in obtaining NAWS project water. Project tasks estimated current and projected populations, typical water usage per capita, and peaking factors used for determining the maximum day water demands. Finally, user agreements of intent were obtained from all interested communities and rural water associations.

The results of the Needs Assessment study will be used to determine the design year 2020 water usage levels. These usage levels were calculated using the projected populations of all interested communities and water associations multiplied by the per capita water demands and the appropriate peaking factors. For some communities, water usage levels were calculated based on historical water demands which indicated daily usage and/or peaking factors which were greater than the average. When data were available, the higher than average water usage levels were used for these communities. The study concluded that the following demands and peaking factors were valid for most of the corresponding customers:

Customer Description	Average Daily Use[a]	Average Peaking Factor
Communities with Populations over 500	130[b]	2.5[b]
Communities with Populations under 500	100[b]	2.5[b]
Rural Water Associations	250[c]	2.2

[a] Per capita in gallons per day (gpd).

[b] For communities that demonstrated higher average daily usage and/or higher peaking factors, the higher peaking factors were used.

[c] Per household (gpd).

The rural areas not on rural water systems represent the majority of the area in the NAWS project. The water usage demand for this category includes future rural water systems which have been identified, but not constructed such as the Mountrail Rural Water and Writing Rock Rural Water Users. The usage levels for rural areas not on rural water systems were based on two factors: domestic use and livestock use. The domestic use was based on 75 percent of the farms and rural residences as indicated on the county highway maps for areas not served by existing rural water systems. The water use factors developed for rural water system customers were 250 gallons per day per household and a 2.2 peaking factor. Based on existing rural water systems, it was determined that 25 percent of the farms that will use NAWS water for domestic uses, will also use NAWS water for livestock purposes. The average daily livestock usage for each county was based on generally accepted average daily water consumption for each type of livestock, and information developed by the North Dakota Agricultural Statistical Service which indicated the amount of each type of livestock for each county.

PIPELINES

General

This section presents pipeline and appurtenances design criteria to be used in the pre-final design of the NAWS facilities. The following items are considered in the development of the design criteria:

- Alignment
- Excavation
- Bedding
- Backfill
- Depth of Cover
- Thrust Restraint
- Pipe Materials
- Pipe Pressure Classes
- Fittings and Specials
- Pipe Joints
- Operating Pressure
- Corrosion Protection
- Mainline Valves
- Air Vacuum/Air Release Valves
- Blowoffs
- Pipeline Access Manholes
- Turnout Service Connections
- Vaults
- Roadway Crossings
- Utility Crossings
- Wetlands and Creek Crossings
- Metering Facilities
- Monumentation
- Hydraulic Design

Alignment

Optimum pipeline alignment connects the water users to the water sources with the most direct routes possible at the lowest expense. Route selection will consider the following siting constraints:

- Hydraulic constraints of the system
- Property values
- Paralleling existing roadways or rights-of-way
- Year-round accessibility
- Route geology and topography
- Ease of reclamation

Excavation

Excavation must be in accordance with the requirements established by the Occupational Safety and Health Administration (OSHA). Shoring may be required at certain locations due to space constraints. Shoring design and trench safety will be the responsibility of the contractor.

Construction production is typically optimized where available space allows stringing pipe and pipe zone material, excavation of the trench, and storage of the excavated material all within the available right-of-way. In private easements not adjacent to a roadway, the right-of-way should also accommodate vehicular construction traffic past the construction area. The total right-of-way width needed depends on several factors, including depth of cover, size of the pipe, encountered soils and resultant trench section required under current safety regulations, and size of equipment used by the contractor.

Easements and permits from private parties and public entities will be needed in most locations along the pipeline routes. Section lines in North Dakota are reserved for roads, and use of private land and/or public right-of-way for other purposes will require easements. Easements will be needed from adjacent landowners if the highway right-of-way was not acquired in fee, or by permit from the Department of Transportation (DOT) if it was. Permits will also be required from county commissions when the pipeline is in a section line right-of-way within their jurisdiction even if an easement from the landowner has been obtained.

Where easements are to be acquired, they are typically comprised of some minimum width of permanent easement and the remainder of the required width as temporary easement. The permanent easement needs to be wide enough to accommodate future pipeline maintenance operations. This would allow a segment of the pipe to be excavated for repair and a crane to operate along the pipe length adjacent to the excavation. The anticipated trench width would need to accommodate excavation adjacent to the pipe, to the bottom of the pipe, and trench side slopes per current safety regulations. The permanent easement may also include space for a future parallel pipeline or other

facilities. In general, it is anticipated a 60 foot (ft) permanent easement and a 40 to 60 ft. temporary easement will be required.

The maximum length of open trench will generally be limited to 2,000 feet. This requirement may be more stringent in urbanized areas, or as required by local authorities. Excavation for other below ground structures such as vaults and manholes will generally follow the same requirements for shoring and safety as trench excavation. The bottom of the excavation pit must provide at least one foot of clearance all around the structure.

Bedding

Pipe-bedding and pipe-zone materials consist of all of the materials surrounding the pipe in the trench zone from six inches below the pipe to, depending upon pipe size, six to 12 inches above. This material must be compacted so that pipe walls are supported. The soil structure created in this zone is intended to keep pipe wall flexure within specified limits. Maximum deflection, as a percent of pipe diameter, is specified to be 2.0 for ductile iron pipe and 1.5 for all other pipes. For PVC pressure pipe, the maximum deflection shall not exceed 3 percent of the outside pipe diameter. Unstable trench bottom or side walls must be removed and replaced with stable material to support the pipe zone material.

Pipe zone material is often imported as an approved classified granular material. To protect the exterior pipe coating, imported pipe zone material is typically specified to be sand or small aggregate. Pipe zone backfill for tape-coated steel pipe shall consist of clean sand with a minimum sand equivalent of 30. Pipe zone backfill for mortar-coated steel pipe or pretensioned steel cylinder concrete pipe (SCCP) shall be sand or crushed rock. Select native material may be adequate for tape-coated or mortar coated steel pipe or pretensioned SCCP if it is more or less granular and does not contain large rocks. Pipe zone backfill for ductile iron pipe may be select native material or as required for use with the selected cathodic protection system. If granular bedding material is used, it is important to keep the fines from migrating into the pipe zone material in areas of high groundwater and/or where settlement must be minimized.

Backfill

Trench and structure backfill shall be compacted to a minimum of 90 percent relative compaction. Where the pipeline is constructed under roadways, the backfill shall be compacted to a minimum of 95 percent of ASTM D698 (Standard Proctor). Native excavated materials may be used for backfill above pipe zone provided they are free of organics and other unsuitable material and they are screened to remove particles greater than 6 inches in diameter.

Depth of Cover

The design depth of cover for the NAWS pipeline generally will be a minimum of 7.5 ft to address the factors discussed below:

- external loadings from earth and superimposed loads (i.e., traffic and farm machinery);
- utility conflicts;
- future excavation for utilities;
- pipeline grading to avoid the need for extra air-vacuum/air-release valves; and
- minimum depths of cover as required by the Department of Health.

External Loadings. The pipe structure must be designed to handle the imposed earth loads and superimposed loads above-ground without damage to the pipe, its lining, or its protective coatings. Except in the case of deep or relatively wide trenches, the internal pressures have always controlled pipe design in the past. The current OSHA standards will typically result in wider trenches and increased earth loads on the pipe.

Utility Conflicts. When other utilities are encountered on the pipeline grade, they must either be avoided, generally by installing the proposed pipeline deeper, or the other utility must be relocated. Utilities, such as sewers, which must be maintained on grade are difficult to relocate. In some instances, the pipeline may have to be installed deeper to avoid conflicts with known future utilities.

Future Excavations. Future utility and other excavations in the vicinity of the buried pipeline have the potential of damaging the pipe. The proposed 7.5 foot cover would place the pipe below most such excavations.

Pipeline Grading. Pipeline grading to avoid installation of air-vacuum and air-release valves when small hummocks must be traversed may result in shallower or deeper cover in a few instances. Where necessary, special measures to accommodate shallow cover and provide adequate protection will be considered.

Frost Depths. A minimum depth of cover of 7.5 feet above the pipe will be used to prevent freezing conditions in the pipelines as specified by the North Dakota Department of Health.

Thrust Restraint

Changes in pipe direction (normally ≥15 degrees), cross sectional area, or isolation points develop additional forces which act on the pipe and cause the need for thrust restraints. Thrust restraint of the NAWS transmission pipelines would typically be by use of welded or mechanically restrained joints or thrust blocks depending upon pipe type. Thrust restraints developed by friction between the pipe and surrounding soil depends upon the pipe coating and the pipe zone backfill material used. Thrust restraint should be designed

to accommodate the combination of static and transient pressures, plus a safety factor. Restraint also would be used on steep slopes or where future excavation may allow the pipeline to separate.

At valve vaults, future removal of the valve would require a sleeve coupling or other grooved joint on one side of the valve. Flexibility also is needed where the pipeline enters and exits a vault to accommodate differential settlement. This will be provided by flexible restrained joints or couplings immediately adjacent to the exterior wall. Suitable couplings will be evaluated on a site specific basis.

Pipe Materials

Pipe materials that are appropriate alternatives for the various NAWS transmission pipeline size and pressure requirements include: steel pipe, pretensioned steel cylinder concrete pipe (SCCP), ductile iron pipe (DIP), and polyvinyl chloride (PVC) pipe.

Steel pipe (AWWA C200) is manufactured to a wide variety of sizes and wall thicknesses. A pipe interior cement mortar lining (AWWA C205) dependent on pipe I.D. is recommended. Several exterior coating alternatives will be used depending on field soil corrosivity. Alternative exterior coatings are an 80 mil cold-applied plastic tape (AWWA C214) or a 1-inch thick reinforced cement mortar coating (AWWA C205). Joints may be rubber-gasketed or welded.

Pretensioned steel cylinder concrete pipe (SCCP) (AWWA C303) is standard in sizes to 54-inch diameter and 400 psi pressure class. SCCP is typically constructed with steel end rings designed to contain rubber gaskets or welds.

Ductile iron pipe (DIP) (AWWA C151) is standard in sizes to 54-inch and up to a 350 psi working pressure. A 1/4-inch interior cement mortar lining (AWWA C104) and a polyethylene exterior encasement (AWWA C105) is recommended with dielectric bonding if cathodic protection is required. Push-on rubber-gaskets (AWWA C111) and proprietary restrained joints are available.

Polyvinyl chloride (PVC) pipe (AWWA C900) is standard in sizes to 12-inch and up to a 200 psi pressure class. PVC pipe (AWWA C905) is available also in sizes from 14 to 36 inches and up to a 235 psi pressure class. Push-on rubber-gasketed joints (AWWA C907) are available for nominal sizes of 4 to 8 inches. Fittings for PVC pipe shall be ductile iron and shall conform to the requirements of AWWA C110. PVC pipe fittings shall be mechanical joint.

In the past, prestressed concrete cylinder pipe would have been considered for the higher pressure pipe. Prestressed pipe (AWWA C301) was used for many years, but is now experiencing many failures. Failures are occurring in relatively new as well as older installations. Until the phenomenon causing the failures is better understood and can be accommodated in design, the use of prestressed pipe is not recommended.

In general, pipelines with 12-inch diameter and smaller meeting AWWA Standard C900 will be used. Pipelines with diameters greater than 12-inches will be interior mortar-lined, and either exterior mortar-coated or tape-coated steel; interior mortar-lined and plain or polyethylene encased ductile iron; or SCCP. Both types of ferrous pipes were assumed to have bonded joints and other cathodic protection measures.

Pipe Pressure Classes

The pressure class or thickness of pipe will be determined based on the following:

- handling;
- internal pressure;
- external loads;
- deflection; and
- buckling.

The minimum thickness for handling of welded steep pipe and SCCP shall be the nominal diameter divided by 240. In no case shall the steel cylinder thickness be less than No. 12 gage. For SCCP, the area of the rod reinforcement shall not exceed 40 percent of the total steel area.

The minimum thickness of ferrous pipe and SCCP for internal pressure shall be determined by the following formula:

$$t = \frac{P\,(O.D.)}{2S}$$

where P = internal design pressure (psi)
O.D. = outside diameter (inch)
S = allowable stress (psi)

At the pipe internal working pressure, the allowable stress shall not exceed 50 percent of the yield strength. For tape-coated or mortar coated steel pipe and SCCP, the allowable stress shall not exceed 16,500 psi at working pressure and 22,000 psi at transient pressure. For DIP, the design pressure is 2 times the sum of the working pressure plus a minimum of a 100 psi surge allowance; the allowable stress is 42,000 psi.

PVC pressure pipe shall be designed in accordance with the requirements of Appendix A of AWWA Standard C900. The allowable design stress for use of PVC is a function of a factor of safety which is dependent upon two different pressure conditions: 1) sustained working pressure, which is a long-term pressure condition and relates to the long-term hydrostatic strength of the pipe, and 2) total system pressure, which is a short-term, peak-pressure condition and relates to the short-term hydrostatic strength of the pipe. The factor of safety provided for sustained working pressure in the pressure-class ratings (Table 1, AWWA C900-89) is 3.0 or greater, depending on the dimension ratio of the pipe. The factor of safety for working pressure is a function of the surge-pressure

allowance as well as the allowable design stress. A safety factor of 4.0 is provided for total system pressure.

Deflection will be determined using the modified Spangler equation, with Marston embankment loads. The unit weight of backfill and modulus of soil reaction will be as determined and indicated in the project geotechnical report. The bedding constant (K) is 0.1 and the deflection lag factor (D) is 1.25. The allowable deflection for tape coated steel pipe and DIP is 2.25 percent. The allowable deflection for mortar-coated steel pipe is 1.5 percent. For SCCP, the deflection shall not exceed the square of the inside diameter divided by 4,000. In lieu of the Marston earth loads, AWWA C150 design criteria may be used for DIP. The maximum allowable deflection for PVC pipe after installation shall not exceed 0.03 times the outside diameter. If the calculated deflection exceeds 0.03 times the outside diameter, the pipe class shall be increased or the quality of the pipe zone backfill shall be improved to achieve a higher modulus of soil reaction.

The allowable buckling pressure for steel pipe will be determined in accordance with AWWA M-11. Weld strength and axial stresses will also be verified.

Operating Pressure

Generally, operating pressures in the NAWS water transmission pipelines should remain below 150 psi since valves and other appurtenances with this pressure rating are readily available. Lower pressures allow the use of less expensive pipe. A maximum pressure of 150 psi will be the target in pre-final design, but may be increased to possibly eliminate some reservoirs and pumping stations and thus reduce construction costs.

Fittings and Specials

Tape-coated steel pipe, mortar-coated steel pipe, and SCCP will have fabricated steel specials and fittings conforming to AWWA C208. Reinforcement of outlets will be in accordance with AWWA M-11. Fittings for DIP shall conform to AWWA C110. PVC pipe fittings shall be mechanical joint. Fittings shall be ductile iron and shall conform to the requirements of AWWA C110.

Pipe Joints

The types of pipes being considered for the NAWS transmission pipelines typically use some type of unrestrained rubber-gasketed, push-on joint and are proposed where not restricted by steel pipe size or pressure class. Joints in steel pipe or SCCP typically are restrained by welding. Ductile iron pipe joints are restrained by use of ductile iron restrained coupling adapters. The restraint of PVC pipe will be evaluated on a site specific basis. Depending on the application, thrust restraint may be accomplished with concrete blocks or by using D.I. pipe in critical locations.

Corrosion Protection

Corrosion evaluation and mitigation measures will be considered once the final pipeline alignment is selected. As described earlier, various corrosion protection methods involve the use of: mortar linings; tape or mortar coatings; epoxy; and fusion bonding. All iron and steel pipe joints will be electrically bonded. The use of various cathodic protection methods will depend on the results and recommendations of the corrosion evaluation. Insulation flange kits will be provided at connections to other pipelines and other pipe materials where applicable.

Mainline Valves

Mainline isolation valves will be provided to limit the length of pipeline that must be drained for access and system shutdown, and also to limit spillage of water in the event of an emergency such as a pipeline rupture. Isolation valves are recommended at the pump station(s) and at a maximum spacing of 5 miles and will be incorporated as possible. Butterfly valves or other slow closing valves are suitable for use as isolation valves. The mainline isolation valves will be designed for placement in a manner which will prevent freezing, designed to minimize hydraulic transients, and will include a bypass valve leg.

Air-Vacuum/Air Release Valves

Combination air-vacuum/air-release valves will be installed at all defined high points along the pipeline profile, on the downhill side of all mainline valves, on the uphill side of mainline valves located at high points, and at 1/2 to one mile intervals on long horizontal runs lacking a clearly defined high point. These valves have a large orifice which will vent air from the pipeline during pipeline filling and admit air during draining operations to avoid negative internal pressures which could damage the pipeline. Air-vacuum/air-release valves will be equipped with isolation valves and will be installed in vaults. The vaults will be insulated and will be equipped with other measures to ensure reliable operation in cold weather. Screened standpipe(s) located above the ground also will be provided. Standpipe material will address site specific conditions. For example, if the standpipe is located in pasture land, it will be fenced or constructed of steel. PVC pipe may be acceptable in other locations.

For vacuum relief during pipe draining, the valve will be sized on the full pipeline design flow and a 5 psi allowable differential pressure. For air venting during pipeline filling, a flow corresponding to a filling rate of 1 foot/second and an allowable differential pressure of 2 psi will be used.

Smaller air release valves will be provided at high points and at sudden changes in slopes to release air accumulated during normal system operation. The sizing of small diameter air release valves will be based on 2.0 percent of the design flow and the system pressure.

Blowoffs

Blowoff piping will be provided to facilitate drainage of pipeline segments for maintenance and repairs. Blowoffs will generally be installed at low points in the pipeline profile and on the uphill sides of mainline valves. Blowoff size will allow a reasonable "time-to-drain" for tributary pipe segments; typical diameters are approximately 20 percent of piping diameter.

Where reasonable, blowoffs will be drained to daylight. An energy dissipater will be provided where pipeline drainage may cause erosion. In most cases, it will be necessary to provide a standpipe so that a submersible pump can be inserted to complete draining the pipeline. Water will be directed from the blowoff to appropriate drainage facilities. For most applications, plug, ball, or butterfly valves are proposed as blowoff valves. Rechloramination facilities (permanent or portable) may be required to address regulatory concerns.

Pipeline Access Manholes

For pipelines greater than or equal to 24 inches in inside diameter, manholes or buried access ports with blind flanges will be provided for pipeline access to accommodate initial construction and interior inspection as well as future inspection and maintenance. Recommended locations are on either side of mainline valves, in oversized air-vacuum/air-release valve vaults (when space is adequate), and in manhole access vaults at a maximum spacing of one mile. On larger pipelines (i.e. 24 inches in diameter and larger), manholes shall be 24 inch in diameter and located on the side of the pipe for ease of access, except where combined with air valves. Generally, access vaults will be provided at the nearest road crossing. Buried access manholes will typically consist of a 20-inch diameter side outlet and blind flange and will be marked with a steel post.

Turnout Service Connections

Turnout structures will consist of: isolation valves, flow meter where appropriate, flow control valve, and a bypass. Flow deliveries will be controlled by an automatic control system or by an operator at the distribution system control center. Venturi, multiple pass sonic, turbine or magnetic meters are suitable for the turnouts. Globe pattern valves or butterfly valves may be used for the control valve and also could be used to isolate the connecting pipeline for maintenance. Other equipment may include: a check valve at any turnout where pumping of the turnout flow may occur and sample taps to facilitate water quality sampling.

Vaults

Vaults will be used to house various project facilities. All meters, flow-control systems, mainline valves, major air-vacuum/air-release valves, turnouts, pressure reducing valves (PRV), and specific access manholes will be installed in vaults. Blowoffs will also be

installed in manhole-type vaults on the upstream side of mainline valves. To the extent practical, the various system components will be installed in common vaults.

Vaults containing Supervisory Control and Data Acquisition (SCADA) system components, which would include all automated mainline valve vaults and all flow control stations, would be equipped with appurtenances including:

- hatch and ladder for access;
- electrical power, lights, ventilation, sump pump, and unit heaters;
- intrusion and entry alarms; and
- a battery-powered emergency power supply for the SCADA and local control system.

Precast and cast-in-place concrete vaults, pipeline encasements, cut off walls and other structural concrete shall be 4,000 psi compressive strength. Buried vaults shall be designed for an AASHTO HS-20 live load where applicable. Lateral pressures and bearing capacity shall be in accordance with the project geotechnical report.

In the unlikely instance where vaults would have to be placed in roadways, the top of vaults shall be flush with the finished surface. Outside roadways, vaults shall extend at least one foot above grade. Covers shall be of cast iron or fabricated aluminum, at least 24 inches in diameter.

Roadway Crossings

The NAWS pipeline will cross several major roadways and railways. Pipeline installation within paved highway right-of-ways and at railroad crossings will require a cased crossing using boring, jacking, or tunneling methods. For other non-paved roads, open cut crossings are expected to be generally allowed. In some cases, installation of a cased pipe in the open cut to avoid future roadway excavations for repair of the pipeline could be considered. As a rule, the minimum burial for either the pipeline or the casing would be 7.5 feet below finished grade. Deeper cover may be desirable to avoid pipeline damage from future utility excavations. Future utility installation plans, where known, will be accommodated.

Utility Crossings

At roadway crossings or at other locations, utilities may be encountered which generally parallel the road or cross the pipeline route. These would typically be above the profile of the pipeline, given the greater depth of the pipeline at roadway crossings. One exception may be sewer lines, which may be deeper.

Clearances between sewer lines and the pipeline would be maintained at not less than ten feet horizontal separation (where the lines are parallel) and 1.5 feet vertical separation at crossings. It is assumed that wherever possible the water line would be above the sewer; where adequate separation is not possible, steel pipes would have welded joints and

no-steel pipes would be encased in concrete to prevent contamination of the finished water. Separation from other utilities should be sufficient to protect the existing utility. For larger utilities, encroachment would be reviewed on a case-by-case basis. In some instances, it may be desirable to encroach and protect the pipeline from excavations for the interfering utility by a concrete cradle between the two utilities.

Wetlands and Creek Crossings

Although the NAWS pipeline alignments will try to avoid major wetlands, several water courses will be encountered. Adequate cover must be maintained under stream beds; the pipe will be installed below scour depth, which will be identified in geotechnical investigations during final design. To meet Department of Health requirements, crossing streams greater than 15 ft in width will require: the use of water-tight joints, isolation valves on either side of the crossing, and permanent taps on either side of the source side valve to monitor for leaks. The taps should be located in a buried manhole or valve vault. In addition, riprap should be placed in the stream channel up and downstream of the pipe crossing. Unless special precautions are needed, a typical stream crossing would consist of diverting the stream to allow pipeline installation to midstream and then diverting the stream back to allow completion of the stream crossing. Various lining and piping materials can be used to divert flow and minimize the erosion that takes place in the diversion channels.

Metering Facilities

Metering facilities proposed for the NAWS pipeline will include a "master" meter at the head end of the pipeline, meters at most pump stations, and meters on the connecting pipelines leading to each participant's delivery point. Master and participant meters will allow comparison of inflow against totalized and metered outflow, although it must be recognized that totalized outflow will never exactly equal metered inflow to the pipeline because of meter errors. Participant meters would typically be installed at appropriate locations upstream of rate-of-flow control valves on connecting pipelines after consultation with the customer involved.

Meters used should have the following characteristics:

- ability to read and report rates of flow and totalize flow with accuracy of actual flow rate over the full range of flows contemplated;
- accurate repeatability;
- field calibration capabilities;
- minimum error from SCADA system components; and
- low life cycle costs.

Several types of meters exist. Venturi meters and/or magnetic meters are suitable for use in the NAWS transmission system.

Monumentation

Major valves, air vacuum/release valves, blowoffs, and buried access manholes will be identified with steel marker posts. In those locations where the pipeline does not parallel an existing roadway, marker posts will be placed at the centerline of the pipe or along a line offset from the pipe. The offset distance should be identified on the marker post. The preferred location for marker posts will be along fence lines at approximately one-half mile intervals and wherever the pipe alignment changes. Plastic tape also will be placed in the pipeline trench in urban areas to prevent inadvertent damage to the pipeline from excavation activities. Consideration may also be given to the placement of a metal wire above the pipeline or to the use of a global positioning system to aid in the location of non-metallic pipe.

Hydraulic Design

The total length of transmission pipeline required for the NAWS project may extend more than 700 miles. Hydraulic design of such a long system must not only account for hydraulic construction, but for the economics of construction and operation as well. The design must optimize the total life-cycle cost for the system which is a function of both the energy and installation costs for the various pipe diameters throughout the system.

In general, initial pipe sizes will be chosen to keep the velocities in the 12-inch and smaller pipes to 3 fps or less, at design flows. Pipe velocities in the large diameter pipelines such as the Lake Audubon to Minot pipeline may be higher, but will be less than 6 fps. The design flow will be the combined flows developed by delivering, over a 24-hour period, each customer's maximum day water demand. The pipe diameter which meets the hydraulic constraints and also yields the lowest life-cycle costs will be chosen for service.

After pipe selection, a surge analysis will be performed to determine the appurtenances required to contain transient pressures within the desired range. The desired range runs from a minimum of atmospheric pressure up to a maximum of 1.3 times the normal operating pressure.

One variable required for both the simulation modeling and the life-cycle cost optimization is pipeline friction losses; this variable will be determined by the Hazen-Williams formula. A Hazen-Williams friction factor of 140 was assumed for use with interior mortar-lined steel and D.I. pipe, and a value of 150 was assumed for use with PVC pipe.

Other required variables for use in the life cycle cost analysis are the: interest rate, Engineering News Record (ENR) index, power inflation rate, power cost, pipeline life span, pumping efficiencies, design flows, initial flows, and years required to achieve design flow. For this project, the assumed interest rate will be 8 percent, the assumed power inflation rate will be 5.9 percent (based on conversations with staff from the Western Area Power Administration), and the chosen pipeline life span equals 40 years.

The initial and design flows and years required to achieve the flows vary with reach. An average power cost of $0.025 per kwh was used in the analysis, which is the cost anticipated to be effective January 1, 1994.

BOOSTER PUMP STATIONS

General

The booster pump stations combine with the pipelines and storage reservoirs to form a complete system for transporting water to NAWS customers. Predesign tasks will establish capacities and operation of the stations, select the pump station sites, determine the numbers and types of pumping units required, and the horsepower and electrical supply requirements. This section establishes the various parameters necessary for pump station design. Pump station design will remain consistent between the various stations where practical.

Siting

System hydraulics, route topography, and the need to optimize life-cycle costs dictate the locations and amount of pumping required to deliver the necessary flows. After determining the number of pumping stations needed and their approximate location, final site selection will also depend on the following:

- geology of proposed sites;
- zoning requirements (environmental acceptability);
- relative property values;
- year-round accessibility; and
- availability and accessibility of adequate power supplies.

Pump Station Structures

General design parameters for the various pump station structures are described below.

Booster Stations Along Major Trunk Lines. Booster pump stations located along major trunk lines (e.g. Lake Audubon to Minot) will be housed in buildings designed to protect all pumps, controls, and necessary appurtenances from adverse weather and potential vandalism. On the basis of economics and aesthetics, a concrete masonry building is preferred. Building design features will include adequate insulation and heaters capable of maintaining a temperature sufficient to prevent freezing, ventilation for limiting summer time temperatures inside to meet manufacturer's specifications for the equipment involved, and dehumidification equipment (e.g. wall mounted air conditioning units as feasible). The security measures will include the installation of steel doors with cylinder locks, steel louvers, perimeter fencing, exterior lighting, and possibly alarms. The buildings will not have windows. A standby power system sufficient to power necessary building systems (e.g. electric heaters, lights, etc.) also will be provided.

The building floor will be a concrete slab designed for the special conditions (e.g. vibration, high humidity, and variable loadings) found in pump stations. The foundation and footings will be designed to address site specific soil and frost penetration conditions. Adequate space will be provided for access and repair as well as for adding future pumps if or when necessary. Depending upon the size of the pump station, skylights or hatches and/or a crane generally will be located above each pump to facilitate future pump removal and repair.

Access to the pump stations shall be constructed of an all-weather surface with suitable drainage. The area shall be sloped in a manner to avoid any retention of surface water; no runoff towards any structure will be acceptable.

Other Booster Stations. Smaller capacity pump stations will be located in suitable buildings or concrete vaults equipped with adequate insulation and heating and ventilation systems to meet manufacturer's specifications for the equipment involved. Security measures will include locked access, maintenance hatches, and yard lighting.

The vaults will be either cast-in-place or monolithic pre-cast concrete designed to meet the special conditions which may be encountered. Other vault design features will be similar to those already described for vaults used to enclose pipeline equipment.

Access to the smaller capacity booster stations will consist of a gravel road with allowance for a maintenance truck to turn around. No security fencing will be used. Positive drainage away from the pump station will be provided.

Pump Selection

Two types of pumping units should be considered for the pumping conditions which will be encountered on this project: vertical turbine pumps, and horizontal centrifugal pumps. In the sizes required, the vertical turbine pump is more efficient than a horizontal centrifugal pump. The system head curves which will be encountered make it advantageous to use pumps with steep pump curves which yield small changes in flow for large changes in head. Vertical turbine pumps generally have steeper pump curves when compared to horizontal centrifugal pump curves. Vertical turbine pumps can also be placed in a smaller area and suction head requirements can be more easily accommodated than horizontal centrifugal pumps. Horizontal centrifugal pumps are generally easier to maintain in place, however. They can be disassembled in place without removing them; while the motor, discharge head, and pump must be removed in order to work on vertical turbine pumps. Both types of pumps will be evaluated for their applicability to the specific conditions at each booster station. Consideration also will be given to a standardized pump station component design to facilitate maintenance and reduce the need for many different types of spare parts.

Hydraulic Institute guidelines for pump intake design, as well as manufacturers' recommendations will be used in final design. Each pump discharge line will be equipped with a valve for pump control, a check valve to prevent backflow in the event

of a power failure or other pump shutdown, a butterfly valve for closing the discharge line when a component upstream of the butterfly valve must be removed for servicing, two air valves (to eliminate air pockets in the discharge line), a pressure sensor (for low and high discharge pressure shutdown), and a pressure gauge. Larger pump stations will also be equipped with surge tanks if they are shown as necessary to reduce hydraulic transients. Connections to intake lines and discharge manifolds will be made using flanged connections meeting AWWA specifications unless specifically excepted. Insulating flanged connections will be provided at the intake and discharge sides of the station to electrically isolate the station from the pipeline. Pump station valves and fittings will be designed to withstand the anticipated maximum pressures caused by fluid transients (waterhammer). A preliminary surge analysis is described in Section 4.

Another consideration for pump stations is the number of pumping units to be installed; the more pumps, the greater the flexibility in delivering various flow rates, but at a higher cost. For ease of operation and maintenance, it is generally desirable to have equal size pumping units. Pumps will be cycled on a rotating basis so the pumps are cycled evenly. In the case of malfunction or failure of a pumping unit, it is recommended that one standby pump equal to the capacity of the single largest unit that might be out of service at any one time be incorporated at each pump station. Constant speed pumps are expected to achieve the design objectives with lower cost, higher reliability, and less equipment than variable speed pumps.

Electrical Service

Each pumping station's electrical supporting system typically will include: any additional transformers; motor control centers; all pumping motors; corresponding wiring and conduits; lighting, heating, and ventilation components; lighting protection; and other items associated with the instrumentation and control system. Line voltages of 480 volts will supply power to pumps of 800 hp and below, while 2,400 or 4,160 volts will supply power to all bigger pumps (if necessary). The motor control centers will include all the motor starters and disconnect switches and will be designed with safety interlocks for the circuit breaker handles. Reduced voltage starters will be used with the larger units. The electrical cables and conduits will be designed in accordance with the appropriate National Electrical Manufacturers Association (NEMA) area classifications and all conduits will be sized for ultimate service.

Instrumentation and Controls

Instrumentation and controls will be designed to operate the pumps to match the downstream flow characteristics. The pumps or pump stages will automatically cycle on and off in response to signals sent by downstream pressure switches from the SCADA system. The control will result from downstream demands causing pressures to rise or fall to predetermined switching levels. These levels will be set to maintain adequate pressures in the system over the entire range of expected flows.

The control system may be based on either (a) Programmable Logic Controllers (PLCs) which are solid state devices configured to monitor, control, and support display function using flat screen display devices or (b) hand-wired relay logic configured to perform required functions and enclosed in a Local Control Panel (LCP) complete with analog indicators and manual hand stations.

Depending upon location and function in the system, monitoring devices may include:

- water level sensing equipment;
- water pressure sensors;
- flow meters;
- position indicators (valves and gates);
- switches; and
- alarms for intrusion, smoke, and flooding.

Monitoring devices may be local (adjacent to the controller) or remote.

Generally, lead/lag pump operation will be designed for the pump stations. A goal of limiting the maximum number of pump starts to four or five per hour will be the target; pumps rated at 100 hp and larger will be further limited to a maximum of 2 starts per hour. In the event of a low-low level being reached in a pump's respective upstream reservoir, the pump will cycle off to prevent the reservoir from being "pumped dry". Finally, local manual operation and/or remote operation from the system control center will be designed for each pump.

Supervisory Control and Data Acquisition (SCADA) Systems

Where the pumping plant operation is required to be monitored or controlled remotely by a central station, SCADA system equipment will be provided. The system will consist of the Remote Terminal Unit (RTU) located in the pumping plant and a computer at a designated central station. The signal to the central station will be transmitted using a radio based system.

The pump operation is normally initiated by the motor starter mounted in the Motor Control Center (MCC) Room. The starter is controlled by a local switch or push buttons or by the local control automation such as the PLC, RTU, and/or the Process Control Module (PCM). The PCM is also an "intelligent" RTU.

The local control panel normally would house the Programmable Logic Controller (PLC) which is designed to perform the pump station control commands and receive and send commands from the RTU to the central station. Other signals, such as wetwell levels and flow and pressure may send signals directly to the RTU.

The Central Computer System function may consist of display information such as graphics and tables. It can also gather historical data such as: trends of pumping cycles,

measurement of flows and pressures, equipment running time, number of pump starts per hour, and many more features if required.

Emergency Power

Several factors enter into an analysis of the risk of interruption (vulnerability) of design pumping capability. Unfortunately, it is seldom possible to compute an accurate or comparable risk factor. Historical data are generally not representative of future interruptions because the causes of past interruptions may have been partially corrected and future conditions may not have been experienced in the past. In the case of water systems, most short-duration power interruptions are not a serious concern because of the provision of the system storage. Water pumping stations, therefore, do not include standby power provisions.

All pump stations will have on-site, back-up power generators to power key building systems and may be equipped with circuit connections for portable generators.

Design Parameters

Design parameters for the various pump stations including: design discharge, total dynamic head, horsepower requirements, specific speed, net positive suction head, and number of pumping units will be developed using information from previous and future work tasks. Some of the methodology involved is described below. These parameters are important in the selection of the proper pumps and motors, as well as the design of the suction and discharge lines, and wetwells.

Flow Rate. Design flow rates for the East, West, and Parshall systems were developed from information contained in the 1993 Community Needs Assessment. According to data from that report, the maximum daily flow for the East system (i.e. Lake Audubon to Minot and beyond) for the design period is about 29 million gallons per day (mgd); the maximum 24 hour flow for the West system (i.e. Williston and beyond) is about 13 mgd; and the maximum daily flow for the Parshall system is about 1.5 mgd.

Total Dynamic Head. Total dynamic head is the change in energy grade line at the pump. The parameters that affect the change in energy grade line at the pump are related according to the following equation.

$$TDH = z_2 - z_1 + v^2/2g + \text{losses, where}$$

TDH = total dynamic head (feet)
z_2 = discharge water surface elevation (feet)
z_1 = water surface elevation in pump forebay (feet)
$v^2/2g$ = velocity head in the discharge line (feet)
losses = friction and minor losses for manifolding, suction and discharge lines (feet)

The elevation difference between the intake water surface and the highest point in a reservoir is termed the static head. Velocity head is usually a very small amount of the total pumping head. Other head losses occur due to the friction of the water flowing through the pipeline, entering and exiting the pipeline, and flowing through valves and bends. As mentioned before, friction losses for flow through the pipeline will be determined using the Hazen-Williams formula.

Another significant head loss is due to flow through the suction line and discharge manifolding. This value is typically difficult to determine due to the complexity of pipe connections and changes in flow direction. The remaining losses are due to flow through bends and valves (exclusive of those included in the manifold loss figure), and entrance and exit losses. These losses are usually very small when compared to static head and friction losses and will be neglected for preliminary design purposes.

Number of Pumping Units. As described before, multiple pumping units provide reliability and flexibility for pumping flow rates below the design flow rate. A small amount of horsepower in excess of the theoretical horsepower is normally provided to protect against deteriorating pump efficiency and increasing pipe friction, both of which can occur with age. Too few pumping units do not provide sufficient flexibility for pumping lower flows, as a large pump may be required to pump at a rate far from its most efficient operating point. On the other hand, unnecessary complexity and costs can be added to a pump station by providing too many units.

Horsepower. Input horsepower to the pumping units is determined by the following equation. Pump efficiency will be assumed to be 80 percent.

$hp = Q\gamma h/550e$, where

hp = horsepower (foot-pounds/sec)
Q = flow rate (ft^3/sec)
γ = unit weight of water (62.4 lbs/ft^3)
h = total dynamic head (ft)
e = pump efficiency (dimensionless)

WATER STORAGE

General

Storage in a transmission system is generally composed of two components, operational storage and emergency storage. Operational storage is that amount of storage which will allow the system to operate efficiently. In the case of pumping stations, the operational storage requirements are those which allow the pumping station to operate without frequent on-off cycles, while maintaining adequate suction head on the pumps. This is particularly true when pumping stations are in series. Storage should be provided in this

situation to prevent unstable oscillation of flow rates as the pumps try to match each other.

Emergency storage is that storage which is provided to meet demands in the case of the loss of production or transmission capabilities. The most obvious example is the loss of power to either the treatment plant or to any of the pumping systems. Other examples of shutdown include maintenance and repair operations and for pipeline rupture. Because the water from the NAWS system will be pumped to the various users (as opposed to gravity flow), the most reliable alternative would be to provide emergency storage at selected locations along the transmission lines. The amount of emergency storage required is a function of: alternative sources of supply, alternative production facilities, and standby facilities.

The NAWS water delivery system will be designed to deliver, over a 24 -hour period, the combined flow resulting from each customer's maximum day water demand. Actual water demands are subject to both hourly and seasonal fluctuations and will therefore vary from this design flow. Winter months would generally see low demands, while summer peak hourly rates could easily exceed the design flow. The system pumping capacity will be designed to deliver water in fixed increments up to the maximum day water demand. Since the customers to be served by the NAWS system will depend on a constant, uninterrupted water supply, the reservoirs must be able to provide the entire downstream water demand during any emergencies. The amount of emergency storage beyond the operational storage volume of the reservoirs is recommended to be of sufficient capacity to supply the maximum day water demand for 24 hours. This equals the design flow for a 24-hour period. The following paragraphs discuss the reservoir siting and operations, construction, and appurtenances.

Siting

Reservoir siting is a function of the operational parameters of the water delivery system and the topography of the proposed pipeline routes. Reservoirs may be located: on the highest elevation land available along the pipeline route between successive pump stations; at pump stations; or at locations where the largest need exists. Final site selection will be evaluated and determined during design by: the site geology, accessibility, availability of existing electrical services, the cost of land, and applicability for reservoir overflows and other discharges.

Operations

Reservoir operations will account for both normal and emergency conditions. During normal operations, each reservoir's level will rise if upstream pumping exceeds demand. Conversely, the reservoir level will fall if downstream demand exceeds the upstream pumping. In general, the system's pressure switches will be set to allow pumping to approximate closely demand thereby keeping reservoir levels within a preselected range. In the event that a reservoir reaches the maximum level allowable, an altitude valve will close and isolate the reservoir from the upstream supply to prevent overflow. When the

reservoir levels falls to within the normal operating range, the valve will re-open and again allow inflow from the upstream pipe.

The overall tank volumes will equal the operational and emergency storage volumes plus allowances for freeboard and some overflow capacity.

Construction

The reservoirs will be fully enclosed, watertight structures for preventing any possible contamination from outside sources. Several types of reservoirs are suitable for use as storage facilities for the NAWS Project. The more commonly used reservoirs are: above ground steel, reinforced concrete, or post tensioned concrete; partially buried concrete, and buried concrete reservoirs.

Several factors must be considered when selecting a reservoir type, namely: elevation available at the site, construction costs, operation and maintenance, ability to use the site for other purposes, and aesthetics. Above ground reservoirs are able to accommodate different site elevations in order to maintain pressure zone elevations, while buried reservoirs are restricted to those sites with suitable elevations and topography. The construction cost is generally more important so it is given a higher consideration in comparison to the other factors. The maintenance consideration reflects additional expenditures such as routine painting of steel reservoirs which are necessary to maintain the reservoir in operation. The operational consideration reflects the ease of operation or problems which might be encountered during operation. For example, above ground reservoirs will have winter ice concerns which will be reduced for buried or partially buried reservoirs. Dual cells and/or a bypass will facilitate maintenance. The site utilization consideration is a function of the ability to utilize the site for other purposes; thus, a buried concrete reservoir will be superior to an above ground reservoir. Aesthetics is strictly subjective, however, buried and partially buried concrete reservoirs will have a much lower profile than above ground reservoirs, making them more acceptable from an aesthetic viewpoint.

From past experience, above ground circular steel structures are generally the most economical choice for storage volumes below three million gallons but are subject to problems with cold weather operation. Thus, concrete storage reservoirs for large and small storage volumes are the preferred option to address climatic concerns with regards to ice formation inside reservoirs during cold weather. Site topography may dictate whether the reservoir will be above ground or buried. The configuration of each tank will be chosen so as to minimize site earthwork, but will generally have a width or diameter which is two to four times the height. Design and construction of the reservoirs will conform to applicable AWWA Standards.

Appurtenances

Reservoirs will be equipped with the following devices: access manholes for inspection, maintenance, and cleaning purposes; air vents designed to exclude surface drainage,

rainwater, birds, and animals; sampling taps; exterior ladders with appropriate safety devices; at least one and, in some cases, more drains for cleaning and maintenance purposes; devices to indicate water levels; an overflow; and appropriate security devices (e.g. locked manholes and perhaps fencing in certain areas). The reservoirs will use separate inlet and outlets.

Access to the reservoirs will be by gravel or paved roads depending on the location and function of the reservoir in the transmission system. Power and communications and/or remote monitoring equipment also will be provided. Where commercial power service is not readily available, solar powered systems will be considered.

WATER TREATMENT PLANTS

General

The objective of the NAWS Project is to provide safe and aesthetically pleasing water to consumers without interruption and at a reasonable cost. The required quality is measured against the standards set by state and federal governments which regulate drinking water quality. Where applicable, the regulations put forth by the United States Environmental Protection Agency (EPA) and the North Dakota Department of Health will be adhered to.

A variety of water treatment technologies exist to achieve the objectives stated above. Selection of the appropriate technology depends upon: the quality of the raw water source, finished water quality goals, and provisions of the applicable regulations. Other considerations include: costs, reliability, and ease of operation and maintenance. The following paragraphs briefly discuss the project's raw water quality and finished water goals, and present and discuss treatment process selection.

The SWC selected three regional systems as preferred alternatives. Pre-final design tasks will involve upgrades and modifications to the existing treatment plants at Minot, Williston, and Parshall.

Raw Water Quality and Treated Water Requirements

This section presents and reviews available water quality data to be used during pre-final design, and also summarizes the water treatment plant operational constraints associated with water quality.

Raw Water Quality. The Minot Water Treatment Plant (WTP) will obtain raw water from Lake Audubon which in turn receives water pumped from Lake Sakakawea. For purposes of this project, Lake Sakakawea water quality data will supplement the limited data available for Lake Audubon as they are of a similar quality. Sources for these data include the Northwest Area Water Supply Study (1988), the Bureau of Reclamation's quarterly sampling reports, and sampling efforts by the U.S. Army Corps of Engineers.

The data will be further supplemented by ongoing water quality testing performed by Montgomery Watson.

Generally, the waters of the Missouri River in the area of Lake Sakakawea have been characterized as good quality; hard; with a high, but acceptable dissolved mineral concentration; and high in sodium and sulfates. Lake Sakakawea has also been classified as oligotrophic, while Lake Audubon has been classified as eutrophic. As a result of eutrophication, Lake Audubon can and does develop substantial algae blooms throughout the warmer months. Total dissolved solids (TDS) levels in Lake Sakakawea water typically range from 330 to 540 mg/l. In comparison, Lake Audubon water TDS levels range higher than these values, but still at levels below concern. Sodium levels for Lake Audubon range from 97 to 107 mg/l; the relatively high level of hardness has reached 321 mg/l. Hardness levels near the discharge from Lake Sakakawea in Lake Audubon (i.e. the likely site of the raw water intake station) should be lower. Iron and manganese levels for both Lake Audubon and Lake Sakakawea have been very low in the past. A recent total organic carbon (TOC) test taken on Lake Audubon water showed a value of 5.3 mg/l. TOC samples taken at the Garrison Dam sampling site, however, have reached much higher concentrations (up to 9 mg/l), and this is likely the case in Lake Audubon as well. Finally, seven day trihalomethane (THM) and haloacetic acids (HAA) formation tests conducted recently on chlorinated Lake Audubon water recorded levels of 225 and 101 μg/l respectively. Table 2-1 summarizes some general water quality parameters from both Lake Sakakawea and Lake Audubon and contrasts this data with the EPA Safe Drinking Water Act (SDWA) standards. As can be seen from Table 2-1, water quality data addressing all of the standards of concern are not currently available.

Treated Water Requirements. A detailed summary of all existing and proposed drinking water standards was included as an appendix in the 1993 Community Needs Assessment report. To avoid redundancy, the following paragraphs provide only an overview and update of this summary, emphasizing the most significant standards which will affect the pre-final design of the NAWS WTP

The regulations that will impact the treatment requirements for Lake Audubon water are the: Surface Water Treatment Rule (SWTR), Total Coliform Rule (TCR), the Lead and Copper Rule (LCR), the draft Disinfectant/Disinfection By-product (D/DBP) Rule, and the Enhanced Surface Water Treatment Rule (ESWTR). Another draft rule, the Information Collection Rule, also has been proposed. It will increase monitoring requirements, but not directly affect treatment requirements. These regulations include both enforceable primary standards as well as secondary standards which are not enforceable but are still important goals. The primary standards define maximum contaminant levels (MCLs) for various constituents of concern, while the secondary standards propose nonenforceable guidelines for the delivery of aesthetically acceptable water to consumers.

TABLE 2-1

GENERAL WATER QUALITY PARAMETERS FOR LAKE AUDUBON AND LAKE SAKAKAWEA[a]

Water Quality Parameter	Units	Lake Audubon	Lake Sakakawea @ Garrison Dam	Federal MCL[b]
TDS	mg/l	535 - 873	330 - 540	500[c]
Sodium	mg/l	97 - 107	48 - 82	ne[d]
Calcium	mg/l	42 - 47	47 - 65	ne
Magnesium	mg/l	24 - 36	12 - 28	ne
Chloride	mg/l	16 - 17	3.8 - 14	250
Sulfate	mg/l	200 - 374	120 - 250	250[c]
Nitrate (as N)	mg/l	.025 - .04		10
TOC	mg/l	5.3	9	ne[e]
pH	---	8.1 - 8.8	6.5 - 8.5	6.5 - 8.5[c]
Alkalinity, as $CaCO_3$	mg/l	184 - 212	140 - 276	ne
Hardness, as $CaCO_3$	mg/l	257 - 321	170 - 260	ne
Turbidity	NTU	2 - 7	.2 - 10	0.5[f]
Temperature	°C	1.6 - 25.3		ne
Field Conductivity	mhos/cm	869 - 1001	500 - 810	ne

[a]Water quality data collected by U.S. Bureau of Reclamation, U.S. Army Corp of Engineers, and Montgomery Watson.

[b]Maximum Contaminant Level.

[c]Non-enforceable, secondary standard.

[d]Non-existent.

[e]Conventional treatment plants will be required to achieve certain percent reductions under the proposed D/DBP Rule.

[f]Treatment technique based on a filtered water turbidity in 95 percent of the measurements taken each month.

Surface Water Treatment Rule (SWTR)

The SWTR establishes disinfection and filtration requirements for treating surface waters. It's purpose is to protect the public from waterborne diseases which may be caused by microbial pathogens. Compliance with the SWTR requires treatment capable of continuously ensuring 99.9 percent (3-log) *Giardia lamblia* inactivation and 99.99 percent (4-log) virus inactivation. Filtration satisfies a portion of the pathogen inactivation requirement with disinfection satisfying the remainder. The amount of inactivation credit given for filtration is dependent on the treatment technology used. Filtration in a well operated conventional treatment plant (coagulation, flocculation, sedimentation, filtration, and disinfection) will receive credit for 99.7 percent (2.5-log) removal of *Giardia* and 99 percent (2-log) removal of viruses. However, filtration in a direct filtration treatment plant (no sedimentation) receives less inactivation credit: 99 percent removal of *Giardia* and 90 percent (1-log) removal of viruses. Disinfection must meet the remainder of the inactivation requirements with more credit needed in the direct filtration treatment plants.

The SWTR credits disinfection with certain inactivation levels based on the concept of contact time (CT). CT is the product of the disinfection residual concentration in mg/l multiplied by the residual contact time in minutes. The EPA has established tables of CT levels for various disinfectants (e.g. chlorine, chloramines, and ozone) to achieve different levels of inactivation of *Giardia* and viruses under different pH and temperature conditions. A disinfectant concentration must be maintained for a sufficient contact time to achieve the CT level as defined in the tables for the plant operating conditions. The CT available prior to the first service connection is used to calculate the total CT achieved. The most critical conditions expected (i.e. peak flow rate, highest pH values (for free chlorine), and lowest temperature) set the constraining CT value needed for compliance with the regulations.

The SWTR has additional disinfection requirements besides achieving CT compliance. Water delivered to the water distribution system must contain a disinfectant residual of at least 0.2 mg/l. Also, a residual disinfectant concentration must be maintained in water distribution systems so that 95 percent of the monthly samples meet one of the following conditions: 1) a detectable disinfectant concentration is observed; or 2) the heterotrophic plate count (HPC) is less than or equal to 500/ml.

The SWTR also sets requirements for turbidity removal. A filtered water turbidity of 0.5 NTU or less must be achieved in 95 percent of the measurements taken each month and must not exceed 5.0 NTU at any time.

Total Coliform Rule (TCR)

Beyond the SWTR, water treatment plants must meet the TCR provisions. The TCR sets a stringent requirement for the control of coliform bacteria in distribution systems. Under the TCR, one of the key provisions is that coliforms must be absent in at least 95 percent of the distribution water samples.

Lead and Copper Rule (LCR)

The LCR established a treatment technique that includes requirements for corrosion control treatment, source water treatment, lead service line replacement, and public education. The requirements are triggered, in some cases, by lead and copper action levels measured in samples collected at consumers' taps. The lead action level is exceeded if the level of lead in more than 10 percent of the targeted tap samples is greater than 0.015 mg/l (90th percentile). The copper action level is exceeded if the level of copper in more than 10 percent of the targeted tap samples is greater than 1.3 mg/l (90th percentile). Corrosion control methods at the treatment plant may include: pH and alkalinity adjustment, calcium hardness adjustment, or the use of corrosion inhibitors.

Disinfectant/Disinfection By-Product Rule (D/DBP)

A revised draft version of the D/DBP Rule was released in early September 1993. The chief aim of the D/DBP Rule is to set monitoring requirements and MCLs to limit human exposure to disinfectants and disinfection by-products (DBPs). In developing this rule, the EPA used an approach which attempts to balance the chronic health risks from D/DBPs with the acute health risks from microbial pathogens. Under current regulations, total trihalomethanes (THMs) are the only regulated disinfection by-products. Under the proposed D/DBP Rule, MCLs are to be set for THMs, HAAs, bromate, and chlorite. Limits will also be placed on the allowable dosage of chlorine, chloramines and chlorine dioxide. Additional compounds will likely be regulated by future amendments to the D/DBP Rule as health risk data becomes available on other compounds.

The D/DBP Rule also will require conventional treatment plants to install and operate enhanced coagulation for removal of TOC. Enhanced coagulation is defined as " . . . the addition of excess coagulant for improved removal of disinfection by-product precursors by conventional filtration treatment." Certain removal percentages of TOC will be required to be removed and will be based on the raw water alkalinity and TOC levels. Systems will be required to achieve the specified reduction of TOC between the raw water and the treated water prior to continuous disinfection. Systems using ozone will have to achieve the percent reduction before the addition of a residual disinfectant. Based on the limited TOC data available, it appears the Williston and Minot WTPs will be required to practice enhanced softening techniques. It appears the Parshall system will be

exempt from the enhanced softening techniques because it services less than 10,000 people.

The anticipated schedule for the D/DBP Rule is as follows: published for public comment in March 1994; final rule published in December 1996; and rule effective for surface water systems greater than 10,000 people in July 1998.

Enhanced Surface Water Treatment Rule (ESWTR)

As part of the development of the D/DBP Rule process, EPA developed a disinfection by-product regulatory analysis model (DBP-RAM). Using raw water quality data, risk assessments for disinfectants; disinfection by-products, and microbiological indicators; and predictive equations for trihalomethanes and two haloacetic acids, the DBP-RAM tried to predict the health and economic implications of various regulatory scenarios for disinfection by-products. One of the outcomes of the DBP-RAM simulation showed that the existing requirements of the SWTR may not be providing adequate protection against *Giardia*. As a result, EPA staff has developed an "enhanced" SWTR which may increase necessary log removals of *Giardia* and *Cryptosporidium* based on results of more frequent source water monitoring. At present, no treatment requirements exist for *Cryptosporidium*.

The anticipated schedule for the ESWTR is as follows: publish options for public comment in March 1994; finalize ESWTR in December 1996; and make the Rule effective in July 1998.

Treatment Technology Selection

The treatment objective of the three plants to be upgraded and modified during the project will be to satisfy the existing and future drinking water regulations given the existing water quality of Lake Audubon, Lake Sakakawea, and the Missouri River. As previously discussed, the various regulations will require 3-log *Giardia* inactivation, 4-log virus inactivation, coliform bacteria control, and D/DBP control. More stringent removals of *Giardia* and *Cryptosporidium*, and mandated TOC reduction also are possible. Additionally, maximum turbidity levels are set and standards for color, tastes and odors, and other parameters are recommended. The existing water quality as described indicates Lake Audubon water to be of a reasonable quality, but capable of producing tastes and odors and, more importantly, DBP formation problems.

While several basic treatment processes are applicable for treatment of the various raw water sources, continuing to use the existing lime softening processes at all three existing WTPs was selected by the SWC. Based on past experience, this type of plant will provide for reliable and continuous achievement of turbidity standards, and, when treatment is optimized, will meet enhanced softening requirements.

Disinfection of the raw water sources requires special consideration. The SWTR, TCR, and ESWTR emphasize thorough disinfection to eliminate microbial pathogens, while the D/DBP Rule will place limits on the use of disinfectants and the formation of DBPs. The disinfection system proposed at all three WTPs must account for and balance the conflicting goals of these rules. The primary disinfectants include: chlorine, chloramines, chlorine dioxide, and ozone. DBP formation concerns restrict the use of chlorine. High operation costs and a 1.0 mg/l MCL for chlorite make chlorine dioxide an unattractive alternative. While chloramines do not form THMs (if properly mixed) and tend to form a more stable residual, they are weaker disinfectants and require longer contact times and higher concentrations in order to meet the CT requirements of the SWTR. Ozone has proven to be an effective disinfectant with a long history of use. Other advantages include taste and odor control, algae control, and the need for only short contact times. Ozone also is the only primary disinfectant which can inactivate *Cryptosporidium* at reasonable detention times and doses. While the use of ozone may involve the formation of DBPs, at this point it is the recommended disinfectant. Since ozone decomposes rapidly, a secondary disinfectant (chloramines) must be used following ozone disinfection to maintain a disinfectant residual in the transmission system.

Future water quality tests including bench scale and/or pilot plant studies should be performed. This testing will assist in finalizing water treatment requirements. It will also help optimize the chosen processes, determine the necessary additions of chemicals and their dosages, and more accurately predict the level of treatment performance which can be achieved.

Detailed site specific design criteria for all three WTPs are presented in Section 6.

RAW WATER INTAKE

General

Raw water intakes generally consist of three basic components: the intake structure, screening facilities, and pumping facilities. The design and arrangement of these components depend on site specific soil, topographical, climatic, limnological, and water quality conditions.

Lake Audubon Intake

Intake Structure. The intake must be located in an easily accessible location and designed and built to supply a specified quantity of the best available quality of water. The essential factors for the intake system are: reliability, safety, and minimal operation and maintenance costs.

Site selection for the reservoir intake system was be based on the following considerations:

- the stratification characteristics of the water due to seasonal changes in water temperature;
- the water quality of minor streams, algal growth characteristics, and the growth cycle of other microorganisms;
- shore conditions, wind direction, and velocity;
- the conditions of the catchment area, including potential sources of pollution;
- conditions conducive to siltation in the reservoir;
- the ultimate purpose of the reservoir and the impact of joint users on the inlet location;
- the level of sanctioned recreational activities; and
- historical and expected future water level fluctuations.

After evaluation of the items described before, a shore intake on the eastern embankment of U.S. Highway 83 on Lake Audubon was selected to supply raw water to the Minot WTP. Additional information is supplied below.

Shore Intake. Shore intakes should be located so that a minimum water depth of 6 feet is available. The intake velocity limitation is similar to that of the intake tower to reduce problems with debris, ice, and aquatic creatures. While bar screens are normally provided at the intake bay, the low debris level at Lake Audubon will not necessitate their use. Fine screens will be installed just downstream of the bar screen. The clear openings in the screens must be 3/16 - 3/8 inch and, under normal conditions, the maximum flow velocity through the net opening should occur at the lowest submergence level. The fine screens would be protected from icing hazards with the use of bubbler and/or backflushing systems.

Pumping Facilities. The pumping facilities design will generally follow the guidelines described earlier in this technical memorandum. The pumping units will be selected to account for the variations in suction head corresponding to the change in reservoir levels and will be positioned to satisfy the minimum submergence requirements. They will be chosen to operate at or near their best efficiency point with the reservoir at its normal water level. Pre-final design will also investigate the need to provide for injection of a disinfectant (chloramines) for limiting slime growths in the intake structure, the discharge piping, and at other locations to prevent infestation by fresh-water mussels. Splitter walls will be designed for duty between pump suction bells. The facilities will be designed to

be as unobtrusive as possible. Access will be controlled with locked doors and/or fencing and security systems.

The Lake Audubon Raw Water Intake is described in detail in Section 5.

Williston Intake and Parshall Intakes

The intake capacity to the existing low service pumping station is considered adequate to move water into the wetwell of the Williston raw water pump station. An additional pump would be needed as a standby to ensure peak demands (i.e. 13 mgd) could be met.

Modifications to the Parshall Intake are described in Section 6.

SYSTEM INSTRUMENTATION AND CONTROL

General

The NAWS water treatment and delivery system will be large and complex, comprised of numerous pump stations, water storage reservoirs, service connections, many miles of pipe and appurtenances, several water treatment plants (two existing), and at least two intake structures (one existing). An instrumentation and control strategy is needed to effectively integrate these various components together in a system which is simple, reliable, and cost-effective.

The general objectives for installing a sophisticated instrumentation and control system for a treatment plant and water supply system are:

- the continuous production and supply of safe drinking water;
- the automatic execution of corrective measures and automatic response to potentially disastrous situations;
- minimizing the potential for human error;
- the capability to quickly solve analytical problems; and
- the ability to diagnose problems in remotely located equipment before a malfunction occurs.

The following paragraphs describe in conceptual terms what systems are available for the treatment plants, and pipeline pumping and transmission systems, and how the systems will be coordinated.

Water Treatment Plant System. The primary objective of a treatment plant monitoring and control system is to produce the best quality water at a minimum cost. Major benefits derived from the system are:

- data acquisition for process analysis, control, trending, maintenance functions and report generation;
- instant observation of the status of any key process equipment or water quality parameter;
- control of complex, multi-variable processes;
- control of tedious or routine procedures; and
- alarm functions for process equipment malfunctions, including serious conditions such as chlorine gas leaks, overflowing and drain conditions, or exceedance of process and equipment set points or limits.

Control options for treatment plants include manual, semi-automatic, automatic and supervisory. Manual control requires the manual initiation of a function by the operator. Semi-automatic control requires manual initiation of automatic function such as manual initiation of filter backwash, for example. Automatic controls involve the use of sensors, limit switches, timers, analytical instruments, controllers and control logic devices (relays or programmable controllers) to automatically control the process or equipment. A fully automated filter backwash operation is a good example. Supervisory control basically refers to remote controls, either in-plant or remote from the plant. Such control is usually in the form of continuous, automatic monitoring with remote manual setting of control functions. Controlled functions monitored on the computer video display terminal, printer and keyboard, or on the main control board fall into this category.

The pre-final design tasks will evaluate all four levels of the system based on the local conditions, size and complexity of the treatment and water supply systems, the management philosophy of the owner, and the anticipated financial constraints. The final recommendation will be based on owner preference and on sound engineering judgment.

For the purpose of this technical memorandum, it is envisioned that some level of a computer-based monitoring and control system will be used at the various plants to be upgraded. Computer based monitoring and control equipment use digital signals (i.e. on/off electrical pulses) sent along a coaxial cable for transmission of measurement and control signals. A large number of signals can be sent along a single coaxial cable through the use of a remote terminal unit (RTU) located at each major unit process. The RTU receives analog and/or digital signals from nearby sensors and then sequences the transmission of these signals so that a single coaxial cable can be used to send and receive a large number of measurement and control signals. These signals are sent to a central processing unit (CPU) in the control building where they are displayed on a video terminal and possibly logged on a printer or stored on a disk. Control is accomplished by coding commands into the video terminal keyboard. The CPU is a small, real time computer programmed for dedicated use by the plant operating staff with peripheral equipment such as a video screen, keyboard, printer, disk drive, etc.

Another important aspect of the computer based control approach is the use of programmable logic controllers (PLC). The PLC is a small computer dedicated to the control of a particular process or piece of equipment. For example, the filter backwash cycle can be controlled by a PLC. The PLC is programmed to backwash a filter based on a pre-determined time schedule, high filter headloss, or high filter effluent turbidity. Hence, the operator would not normally control the backwash cycle; he would only receive status signals for the PLC to indicate the progress of status of the backwash cycle.

The size of computer system required by a plant is a function of the number of analog and digital signals to be wired to the computer, not the size of the plant. The number of units grows in a stepwise increment as the number of tasks increase. Computer-related tasks are described by the following terms: data logging, report generation, alarm indication, plant graphic display, analog variable displays, manual plant control and automatic plant control. Six computer systems that are combinations of these tasks are generally available and are outlined below:

- report generation only;
- data acquisition and logging, report generation, and alarm indication;
- data acquisition and logging, report generation, alarm indication, and plant graphic displays;
- advanced Display and Data Handling: data acquisition and logging, report generation, alarm indication, plant graphic displays, and analog variable displays;
- manual Plant Control and Advanced Data Handling: data acquisition and logging, report generation, alarm indication, plant graphic displays, analog variable displays, and manual plant control; and
- automatic Plant Control: data acquisition and logging, report generation, alarm indication, plant graphic displays, analog variable displays, and automatic plant control.

The first system is a special case in that it can be performed on a personal computer and requires no hardwired inputs from the plant. The remaining five systems are built around a mini- or micro-computer and can be expanded to handle many additional tasks. The personal computer is not as expandable in the sense that it cannot handle many additional tasks.

The hardware cost of the last five configurations is somewhat dependent on plant size and layout. The memory size is dependent upon the number of analog and digital signals which need to be processed and stored by the computer system.

The mode of analog and digital control and transmission also influences system size and cost. The use of PLCs as opposed to standard relay logic control, increases the computer system cost (since PLCs are normally considered part of the computer system), but reduces the electrical system cost (less relays, circuits and wiring). If the plant size is large and spread out, RTUs are cost-effective since single coaxial cables can be used to transmit the analog and digital signals which would normally require separate wires for each signal.

The needs of a system may be classified into three categories: essential for plant operation, useful for plant operations, and luxury item. The essential items are: plant flow rate control, chemical feed rate control, pump rate control, ozonation system control, filter control, analysis and recording of water quality, control and recording of residual chlorine, and detection of chlorine gas leakage (with an alarm). The useful items are those that reduce the mental and physical activity of the operators or those that can save operational costs. They include: computerized data logging, programmable control of the filter backwashing process, closed-loop automatic control of residual chlorine and the pH of the water being processed, and devices that check the coagulant dosages such as streaming current detectors and automatic plant controls with advanced data handling capability (the latter require extensive custom designed software). All these items are not vital to normal plant operation. Lastly, luxury items are defined as items that make plant operation easier without providing compensatory savings in cost. Prior to final design, the needs and tradeoffs of the various systems will be discussed with State Water Commission staff.

As a final note, modification of an existing, outdated system is usually very difficult, primarily due to outdated sensors, transmitters, and control signalers. In fact, the complete replacement of an old system is often more cost-effective than updating certain parts. However, the owner may wish to keep the existing instrumentation system, including the 7-day recording circular chart and the relay logic control system, as a back-up to the new computerized system and/or a training tool for plant operators.

Pump Station/Transmission Line System. General information about the instrumentation and control systems for the water supply system already has been described in earlier sections of this technical memorandum.

Data Exchange

At this point in the process, it is envisioned the following operational data would be transmitted to the water treatment plant by water supply data sensors along the water supply system it serves. Selected data related to the operation of the pumping station and transmission pipeline system would be made available by a SCADA system. These data would be transmitted to the requesting operator workstation or primary CPU(s) at the water treatment plant at the head of the system (or at an administrative office or other selected location) either automatically, based on predetermined time intervals, or on-demand by the operator. Examples of operational data that might be requested by the operators would include: reservoir water levels, upstream/downstream hourly, daily and

monthly minimum, maximum, and average flow values; pressures along the pipeline; and operational parameters such as set point values for the turnouts.

Computer Control Center

A typical computer control center for a system of this size could include the following facilities:

- A 600 square foot area to house the central computer, system operating area, and two office spaces. It would be manned 8 hours per day, 5 days a week.

- The SCADA central control center would be the primary monitoring and control location for the pumping stations and transmission pipelines.

- The central computer system features at this location could include:

  - Dual redundant main computers -- one functioning as the "primary CPU," and the other as a hot "standby CPU". The standby CPU will automatically assume the primary CPU's functions should the primary CPU fail.

  - Two identical operator workstations to provide a "window" into the operation of the pipeline.

  - A report and an alarm printer to provide a hard copy of the scheduled and on-demand reports, as well as any operational alarm conditions.

  - During off hours, an auto-dialer/voice synthesizer to contact the operator-on-duty in case of the alarm of emergency conditions in the system.

  - One (or more) laptop computer(s) to enable the operator-on-duty to dial and gain access into the system during off-hours.

  - A diagnostics port to allow remote access by authorized personnel for troubleshooting purposes.

System Communications

System communications between the water treatment plant(s) and pumping and transmission pipeline equipment will be by radio telemetry, depending upon location of the various facilities and available utilities. The communication system configuration is discussed in Section 4.

Section 3

HOUSTON ENGINEERING
AMERICAN ENGINEERING
MONTGOMERY WATSON

SECTION 3

DETERMINE ALTERNATIVE WATER SOURCES/SYSTEMS

INTRODUCTION

This section of the memorandum describes water sources which are potentially available to supply the East System portion of the NAWS project.

EXISTING WATER SUPPLY AT MINOT

Minot currently derives it water supply from groundwater aquifers and, depending upon availability and water quality, the Souris River. Records reported to the North Dakota State Water Commission (SWC) for the period from 1985 through 1987 show that groundwater sources supplied an average of 63 percent of the City's water; the remainder was supplied by the Souris River (35 percent), and other internal plant drain, sump pump, and sensor flows (2 percent). Since the onset of the drought in 1988, the City has relied to a much greater extent on its groundwater supplies. For the period from 1988 through 1992, groundwater sources accounted for 92 percent of the City water use. During 1991 through 1992, no surface water was reportedly used. For example, data provided by City staff show that water production in 1992 totaled 2,269 million gallons of which approximately 99.2 percent was from groundwater aquifers; 0.8 percent was from various floor and tank drains, sump pumps and sensor flows which were recycled to the head of the plant; and none from the Souris River. Monthly and total 1992 water production by Minot are summarized in Table 3-1 and shown graphically on Figure 3-1.

Characterization of Groundwater Supply at Minot

Two aquifers, the Minot Aquifer and the Sundre Aquifer, are utilized for water supply by the City of Minot. Production from the aquifers in 1992 totaled 6,900 acre-feet (about 6.2 million gallons per day (mgd) annual average), with maximum summer month (August) groundwater demand in 1992 averaging 8.7 mgd. The historic maximum day demand has been about 15 mgd (10,500 gallons per minute (gpm)). Characteristics of the two aquifers are described below.

Minot Aquifer. The Minot aquifer is that portion of the Souris Valley Aquifer in the vicinity of Minot. The aquifer generally consists of 30 to 50 feet of coarse sandy gravel, overlain by as much as 80 feet of sand. The aquifer is at the surface in the western portion of Minot; in the eastern part of Minot, it is overlain by as much as 80 feet of clay. Transmissivity is high, often in the 250,000 gallons per day per foot (gpd/ft) range, resulting in relatively high capacity wells.

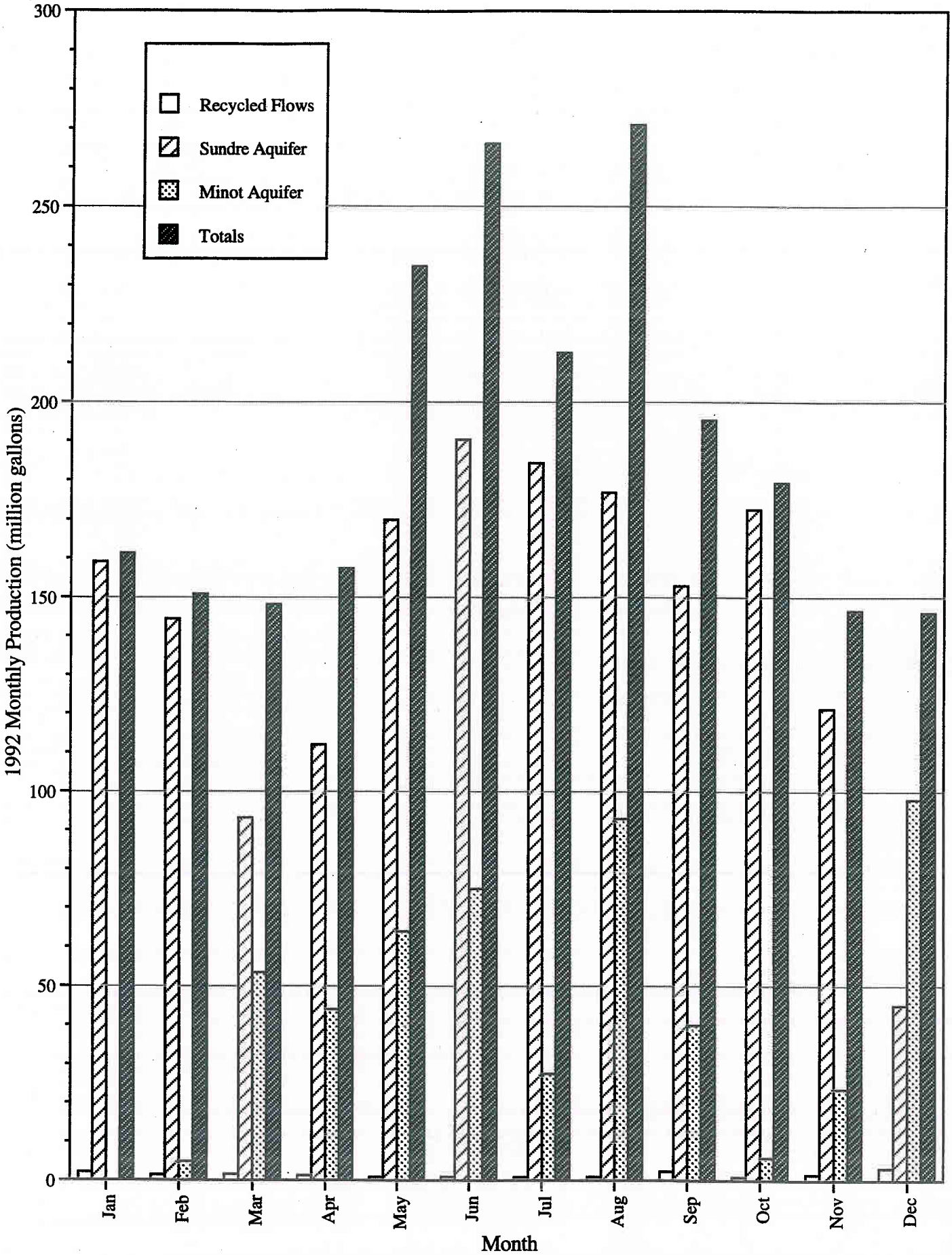
Studies conducted in the 1960s suggested that natural recharge to the Minot Aquifer averages 3 mgd, and is derived primarily from adjacent Souris Valley aquifers (2 mgd) and the Souris River (0.5 mgd), with minor recharge (0.5 mgd) from a combination of

TABLE 3-1

MINOT 1992 MONTHLY WATER PRODUCTION
(gallons)

Month	Souris River	Recycled Flows	Sundre Aquifer	Minot Aquifer	Total
January	0	2,172,000	159,090,000	0	161,262,000
February	0	1,554,000	144,368,000	4,882,000	150,804,000
March	0	1,587,000	93,136,000	53,403,000	148,126,000
April	0	1,277,000	112,038,000	43,978,000	157,293,000
May	0	966,000	169,866,000	63,860,000	234,692,000
June	0	927,000	190,241,000	74,824,000	265,992,000
July	0	932,000	184,321,000	27,392,000	212,645,000
August	0	1,068,000	176,866,000	92,980,000	270,914,000
September	0	2,449,000	152,946,000	39,900,000	195,295,000
October	0	953,000	172,488,000	5,900,000	179,341,000
November	0	1,647,000	121,312,000	23,520,000	146,479,000
December	0	3,173,000	44,944,000	97,857,000	145,974,000
Total	0	18,705,000	1,721,616,000	528,496,000	2,268,817,000

Source: 1992 Annual Water Use Report to the SWC.



1992 MONTHLY WATER PRODUCTION BY CITY OF MINOT
FIGURE 3-1

precipitation, and glacial till and bedrock sources. The "safe yield" from the Minot Aquifer at that time was estimated at about 3 mgd (3,300 acre-feet annually). However, recent studies by the SWC indicate that the sustainable yield has been reduced to about 2 mgd (Steve Pusc, letter of October 19, 1993). It should be noted that the sustainable yield could be increased if artificial recharge is practiced. The potential for artificial recharge of the aquifer using Souris River water is good (provided that recharge water is available), as a previous recharge project (destroyed by river flooding in the 1970s) was able to recharge up to 4 mgd.

The City of Minot has nine operating wells pumping from the Minot Aquifer. Annual production from the aquifer averaged between about 1000 and 2000 acre-feet between 1940 and 1980, resulting in relatively stable water levels. In the late 1980s, production increased due to growth and drought. Production peaked in 1987 when the City pumped 4,671 acre-feet from the Minot aquifer. The increases in production during the late 1980s resulted in water level declines of about 10 to 15 feet. Production in 1992 totaled about 1,600 acre-feet (529 million gallons), and is detailed by well in Table 3-2.

Water quality from the Minot Aquifer is highly variable, with total dissolved solids (TDS) in samples collected in July 1993 ranging from about 950 to 1500 milligrams per liter (mg/l) as one example. Water quality by well is summarized in Table 3-3.

Sundre Aquifer System. The Sundre Aquifer System occupies a portion of a buried bedrock valley, and extends from about five miles northwest of Minot to beyond the McHenry County border, southwest of Minot. It underlies the Minot Aquifer beneath the City of Minot.

The Sundre is a sand and gravel aquifer, varying from about 30 to 250 feet in thickness, and averaging about 120 feet in thickness. The aquifer varies in width from one to two miles, and is approximately 18 miles long. It is truncated on the southeast end by a low permeability barrier which separates the Sundre from the New Rockford aquifer.

Production from the Sundre Aquifer by the City of Minot began in 1977 to address increasing water supply needs. The City currently operates five municipal wells completed in the Sundre, with individual well yields as high as 3000 gpm. Production in 1987 from the Sundre was about 2,300 acre-feet, but by 1989 had increased to about 3,400 acre-feet, and totaled about 5,300 acre-feet in 1992. These production increases have resulted in 30 feet or more of water level decline. Water level declines and resultant interference with private wells have limited the ability of the City of Minot to develop the Sundre Aquifer. Additional development may be possible if mitigation is provided to private well owners. The 1987 study of the aquifer by the SWC suggested that the sustainable yield from the Sundre could be as high as 6 mgd (6,500 acre-feet annually). A revised estimate by the SWC is anticipated in the near future.

TABLE 3-2

1992 MINOT WATER USE BY SOURCE

	Capacity (gpm)	1992 Use (gallons)	1992 Use (acre ft)	% of Total Water Used	% of Total Ground-Water Used
Sundre Aquifer					
Well A	500	134,286,000	412	6.0%	6.0%
Well B	1,000	332,271,900	1,020	14.8%	14.8%
Well C	3,000	700,009,100	2,148	31.1%	31.1%
Well D	2,700	488,250,300	1,498	21.7%	21.7%
Well E	800	66,798,700	205	3.0%	3.0%
Totals	8,000	1,721,616,000	5,284	76.5%	76.5%
Minot Aquifer					
Well 5	750	157,606,000	484	7.0%	7.0%
Well 6	750	76,600,000	235	3.4%	3.4%
Well 8	800	0	0	0.0%	0.0%
Well 11	600	77,705,000	238	3.5%	3.5%
Well 12	600	70,731,000	217	3.1%	3.1%
Well 13	600	81,673,000	251	3.6%	3.6%
Well 14	950	0	0	0.0%	0.0%
Well 15	800	35,180,000	108	1.6%	1.6%
Well 16	600	29,001,000	89	1.3%	1.3%
Totals	4,300	528,496,000	1,622	23.5%	23.5%
Recycled Flows(a)	Unknown	18,705,000	57	0.8%	
Souris River	2,200	0	0	0%	
System Total	**14,500**	**2,268,817,000**	**6,906**	**100.0%**	

Source: 1992 Annual Water Use Report to the SWC
(a) Does not include recycled filter waste washwater.

TABLE 3-3

MINOT GROUNDWATER QUALITY

	Sampling Date	Lab Conductivity(a)	Ca	Mg	Na	HCO_3	SO_4	Cl	NO_3	Fe	Mn	TDS	Hardness(b)
Sundre Aquifer(c)													
Well A	9/22/92	1490	77	24	230	607	250	53	7.3	1.6	0.21	976	290
Well B	7/7/93	1310	85	25	180	550	240	31	0	2	0.27	877	320
Well C	7/7/93	1670	140	40	190	595	390	60	0.2	2.3	0.46	1130	510
Well D	7/7/93	2340	220	61	270	678	780	47	4.1	3.7	0.71	1750	800
Well E	7/7/93	2680	240	66	340	731	980	45	0.2	3.1	0.35	2070	870
Sundre Average(d)		1898	152	43	242	632	528	47	2.36	2.54	0.40	1361	558
Minot Aquifer(c)													
Well 5	7/7/93	1830	130	48	220	681	250	150	0	2.9	0.4	1180	520
Well 6	7/7/93	1900	110	40	270	745	180	170	0	2.3	0.32	1200	440
Well 8	7/7/93	2420	120	42	370	842	120	340	0	3.2	0.36	1460	470
Well 11	7/7/93	1920	120	46	260	787	160	190	0.7	2.3	0.54	1200	490
Well 12	7/7/93	1620	90	31	240	701	240	66	1.4	2.4	0.45	1070	350
Well 13	7/7/93	1780	42	21	360	848	180	87	2	1.5	0.23	1170	190
Well 14	7/7/93	1560	40	18	310	801	110	75	0.9	1.9	0.28	995	170
Well 15	7/7/93	1520	110	44	170	718	170	74	0	0.6	0.84	976	460
Well 16	7/7/93	1610	52	22	290	818	100	90	2.1	4	0.42	1010	220
Minot Average(d)		1796	90	35	277	771	168	138	0.79	2.34	0.43	1140	368
Average for 1992 Production(e)		2006	155	46	254	717	463	76	1.73	2.83	0.51	1386	576

(a) µmhos/cm.
(b) mg/l as $CaCO_3$.
(c) All units in mg/l, unless otherwise indicated.
(d) Average is not weighted based on production.
(e) Weighted average based on proportion of 1992 production from each individual well.

Water quality from the Sundre is highly variable, with TDS ranging from 800 to 2,200 mg/l, as an example. In general, the water quality from the Sundre Aquifer is poorer than water quality from the Minot Aquifer. Water quality data from City of Minot wells tapping the Sundre Aquifer also are shown in Table 3-3 .

Ability to Expand Groundwater Production with More Wells. Studies of the aquifers by the SWC (Sundre, 1987, and Minot, in progress) have indicated that the sustainable yield from the Minot and Sundre aquifers totals approximately 8 mgd without artificial recharge; with artificial recharge, the sustainable yield would be higher. The 1992 groundwater demand at Minot was approximately 6.2 mgd. Therefore, there appears to be some limited opportunity to expand groundwater production with new wells. However, increases in production would probably occur primarily from the lower quality Sundre Aquifer, which would suggest that the average quality of water produced from the City wells would decrease due to higher levels of sulfate, hardness, sodium, and other dissolved solids.

Water Quality Versus Production Rate. One method to provide the optimum quality groundwater to the Minot system would be to pump those wells (primarily Minot aquifer wells) with better quality water. However, given the limited sustainable yield from the Minot aquifer, increased production from the Minot wells may not be practical. Also, water quality may deteriorate at the Sundre and Minot wells with increased pumping because inflow of poorer quality groundwaters may be induced from adjacent bedrock or till units. Therefore, opportunities to improve significantly the average quality of groundwater produced by adjusting well operations do not appear promising.

Average Groundwater Quality Produced by City of Minot Wells. The average quality of groundwater produced by the City of Minot was estimated based upon the distribution of 1992 groundwater production (by well) and the water quality analyses listed in Table 3-3. As shown in Table 3-3, the average raw water quality is relatively poor, with estimated concentrations of 1386 mg/l TDS, 463 mg/l sulfate, 254 mg/l sodium, and 576 mg/l hardness (as $CaCO_3$).

Ability to Improve Sustainable Yield and Groundwater Quality through Artificial Recharge. Water quality in the Minot and Sundre aquifers might be improved through the artificial recharge of higher quality surface water from the Souris River. However, the quality of Souris River water (as is discussed later) is variable and often relatively poor. In addition, Souris River water is available only on a seasonal basis, with availability expected to decrease in the future as a result of water development projects in Canada. Another consideration is that regulations concerning artificial recharge are becoming more strict and may restrict the ability to recharge deep aquifers through artificial means. Therefore, while artificial recharge shows some promise for increasing the sustainable yield from Minot aquifers, implementation may be problematic.

Characterization of the Souris River Supply

Availability and Amount. Before the onset of the drought in the late 1980's, the Souris River had provided as much as 30 percent of the City of Minot water supply in the winter months and ten to twenty percent of the City supply during summer months. The City has installed capacity for pumping of up to 6,000 gpm (8.7 mgd) of Souris River water, although this capacity is seldom, if ever, fully utilized. Average production from the Souris River by the City of Minot during the period from 1985 up through 1992 averaged about 1,463 acre-feet annually (1.3 mgd average); however, production from this source in 1991 and 1992 was zero (1993 Community Needs Assessment, Appendix C, July 1993).

Gaged flow in the Souris River above Minot has ranged from 12,000 cubic feet per second (cfs) to 0 cfs. This wide range of flows demonstrates the primary problem with the Souris River; flows are often too large or too small for the river to be effectively utilized. Although the Souris is currently controlled by the Lake Darling Dam, and soon will be controlled by the Rafferty and Alameda Dams in Canada, the flows in the Souris are undependable. In addition, the Rafferty and Alameda Dams will allow Saskatchewan to develop Souris River flow as entitled under international treaty. These flows are estimated at 50 to 60 percent of river flow at the U.S. - Canada border; historically, Saskatchewan has taken only about 15 percent of the flow. Compounding the reduction in flows is the fact that some of Minot's water rights are junior to those of the J. Clark Salyer National Wildlife Refuge, the Upper Souris National Wildlife Refuge, and the Eaton Irrigation District. As a result, Souris River flows will not be available to the City of Minot during low flow years, and a net reduction in Souris River flows is anticipated. Therefore, the Souris River is not considered a reliable source for a regional water supply.

Water quality from the Souris River is highly variable, with TDS ranging from 158 to 1,100 mg/l. Taste and odor problems, although treated with potassium permanganate and powdered activated carbon, are reported to be unmanageable in the spring. Souris River water quality is summarized in Table 3-4 below.

TABLE 3-4

SOURIS RIVER WATER QUALITY

Hardness (mg/l as $CaCO_3$)	99-530
TDS (mg/l)	158-1,100
Sulfate (mg/l)	53-310
Sodium (mg/l)	19-210
Iron (mg/l)	0-0.76

Source: Table 5-1, Final Report NAWS Study, November 30,1988.

Suitability of Souris River Water for Use in the NAWS Project. Due to poor quality and unpredictable availability, the Souris River water does not appear to provide a desirable and reliable water supply for the system. While the best use of Souris water is probably for artificial recharge of the Minot Aquifer, with secondary use as a supplemental supply to the City of Minot system, the availability of Souris River water for such uses is expected to diminish upon completion of the Rafferty and Alameda Dams.

MISSOURI RIVER

Characterization of the Missouri River Supply

Availability and Amount. Plentiful supplies of surface water are available from the Missouri River (including Lake Audubon and Lake Sakakawea). The State of North Dakota has taken the position that no water charge will be levied on water withdrawn from the Missouri River or Lake Sakakawea because a given increment of water flowing into the Missouri River and stored in Lake Sakakawea is runoff from North Dakota streams. Lake Audubon is a man-made storage impoundment with water derived from pumping by the Bureau of Reclamation from Lake Sakakawea.

Water Quality. Missouri River water quality is well documented in Section 5.B.2. of the 1988 NAWS Study. In general, the Missouri River water can be characterized as good quality; it is somewhat hard, with moderately high levels of sulfate and TDS. Sodium levels typically exceed the recommended guidance level of 20 mg/l for high risk populations (i.e., individuals with a genetic predisposition to hypertension, pregnant women, and hypertensive patients). Iron and manganese levels are very low in Lake Sakakawea, but are somewhat higher in Lake Audubon. Typical water quality is summarized below in Table 3-5. Comparison of Table 3-5 with Tables 3-3 and 3-4 demonstrates that Missouri River water is generally superior in quality to either the Souris River water or Minot area groundwaters.

WATER SUPPLY NEEDS

Projected Demands

Water supply needs for the East System of the NAWS project have been estimated as part of the 1993 Community Needs Assessment, July 1993. The projected supply requirement estimated for water treatment plant sizing is 30 mgd. Maximum day requirement for the East System is estimated at 29 mgd. The estimated average day requirement is considerably less, about 11 mgd. Demands for specific areas are detailed below.

City of Minot. Maximum day demand for Minot and nearby communities totals 19.59 mgd, and includes 14.82 mgd for Minot, 2.95 mgd for Minot Air Force Base, 0.63 mgd for North Prairie System I, 0.15 mgd for North Prairie System II, 0.08 for the towns of Douglas, Voltaire, and Des Lacs, and 0.95 mgd for rural customers west of Minot and the towns of Burlington, Berthold, and Carpio.

TABLE 3-5

GENERAL WATER QUALITY PARAMETERS FOR LAKE AUDUBON AND LAKE SAKAKAWEA(a)

Water Quality Parameter	Units	Lake Audubon	Lake Sakakawea @ Garrison Dam	Federal MCL (b)
TDS	mg/l	535 - 873	330 - 540	500(c)
Sodium	mg/l	97 - 107	48 - 82	ne(d)
Calcium	mg/l	42 - 47	47 - 65	ne
Magnesium	mg/l	24 - 36	12 - 28	ne
Chloride	mg/l	16 - 17	3.8 - 14	250
Sulfate	mg/l	200 - 374	120 - 250	250(c)
Nitrate (as N)	mg/l	.025 - .04		10
TOC	mg/l	5.3	9	ne(e)
Iron	mg/l	0.02 - 0.79	0.01 - 0.1	0.3(c)
Manganese	mg/l	0 - 0.16	0 - 0.013	0.05(c)
pH	---	8.1 - 8.8	7.5 - 8.6	6.5 - 8.5(c)
Alkalinity, as $CaCO_3$	mg/l	184 - 212	140 - 276	ne
Hardness, as $CaCO_3$	mg/l	257 - 321	170 - 260	ne
Turbidity	NTU	2 - 7	.2 - 10	0.5(f)
Temperature	°C	1.6 - 25.3		ne
Field Conductivity	µmhos/cm	869 - 1001	500 - 810	ne

(a) Water quality data collected by U.S. Bureau of Reclamation, U.S. Army Corps of Engineers, and Montgomery Watson.
(b) Maximum Contaminant Level.
(c) Non-enforceable, secondary standard.
(d) Non-existent.
(e) Conventional treatment plants will be required to achieve certain percent reductions under the proposed D/DBP Rule.
(f) Treatment technique based on a filtered water turbidity in 95 percent of the measurements taken each month.

Rural Water Systems and Towns North of Minot. The water supply pipeline extending north from Minot and the Minot Air Force Base would provide maximum daily use of 3.64 mgd to operational rural water systems, rural users, and communities ranging from Kenmare (on the west) to Bottineau (on the east).

Rural Water Systems and Towns East and Southeast of Minot. The water supply pipelines extending east and southeast from Minot would provide maximum daily

use of 4.38 mgd to: an operational rural water system, rural users, and communities ranging from Deering to Rugby and from Sawyer to Anamoose.

Rural Water Systems and Towns Southwest of Minot. Depending upon where the Missouri River water would be treated, the water supply pipeline extending west from a treated water line or southwest from Minot could provide maximum daily use of about 1.54 mgd to the communities of: New Town, Parshall, Plaza, and Makoti; and some rural users to be served by the Mountrail Rural Water System.

Potential Water Supply Service Areas

A potential water supply distribution system to meet the demands detailed above is presented in Section 4.

Water Quality Criteria

Water quality criteria for the NAWS pre-final design is detailed Section 2. Water quality data for various water sources are compared in Table 3-6 below.

TABLE 3-6

COMPARISON OF WATER QUALITIES
(values in mg/l unless otherwise indicated)

	Lake Sakakawea Raw(a)	Lake Audubon Raw(a)	Minot Groundwater Raw(b)	Minot Treated Water Finished(c)
TDS	330-540	535-873	1386	731
Sodium	48-82	97-107	254	239
Sulfate	120-250	200-374	463	306
Total Hardness (as $CaCO_3$)	170-260	257-321	576	75
Conductivity (µmhos/cm)	500-810	869-1001	2006	1167
Iron	0.01 - 0.1	0.02 - 0.79	2.83	0.017
Manganese	0 - 0.013	0 - 0.16	0.51	0.000

Sources: (a) Table 3-5.
(b) Average for 1992 from Table 3-3.
(c) Single sample obtained on 6/11/92.

Water Quality Issues at Minot

Improvements in Minot Groundwater Quality Resulting from Softening Treatment. Although the raw water quality of Missouri River water is superior to the quality of groundwater from the Sundre and Minot aquifers, lime softening treatment at the City of Minot Municipal Water Treatment Plant results in significant improvement in the quality of water supplied to consumers. In particular, lime softening reduces the average hardness to between 100 and 150 mg/l as $CaCO_3$. The softening process also removes iron and manganese, and lowers the TDS; it is not effective at reducing the concentration of sulfate or sodium.

Suitability of Unsoftened Minot-Area Groundwater for Use in the NAWS Project. Due to relatively poor quality, unsoftened Minot area groundwater does not appear to provide a desirable alternative water supply for NAWS water users located outside the Minot municipal system. The estimated average groundwater quality produced by the Minot well system (Table 3-3) exceeds suggested drinking water standards for the following parameters:

- Sodium — Minot groundwater average - 254 mg/l
  EPA guidance level - 20 mg/l for high risk populations
  North Dakota Dept. of Health - 200 mg/l for normal population
- Sulfate — Minot groundwater average - 463 mg/l
  EPA Secondary Maximum Contaminant Level (SMCL) - 250 mg/l (may increase to an MCL of 400 or 500 mg/l)
- TDS — Minot raw groundwater average - 1386 mg/l
  EPA SMCL - 500 mg/l
- Iron — Minot raw groundwater average - 2.83 mg/l
  EPA SMCL - 0.3 mg/l
- Manganese — Minot raw groundwater average - 0.51 mg/l
  EPA SMCL - 0.05 mg/l

Suitability of Missouri River Water for Use in the NAWS Project. The inorganic chemistry of Missouri River water is superior to Minot area raw groundwater in terms of drinking water standards. Levels of sodium, iron, and manganese are not anticipated to exceed drinking water standards. Levels of sulfate and TDS may occasionally exceed the secondary standards of 250 and 500 mg/l, respectively, but will generally average less than 50 percent of the concentrations found in Minot-area aquifers.

Disinfection By-Products (DBPs). Besides the inorganic parameters discussed above, treatment choices also are impacted by other raw water quality parameters, especially naturally occurring organic matter which is usually measured in aggregate as total

organic carbon (TOC). Interest in TOC concentrations has grown due to its use as a surrogate parameter for disinfection by-product (DBP) precursors. Fewer DBPs (such as trihalomethanes (THMs)) are formed by chlorine or other disinfectants if a portion of the TOC is removed before disinfection.

The DBP issue is important because more stringent limits will be required when the Disinfectants/Disinfection By-Products (D/DBP) Rule becomes effective in July 1998. DBP levels in Minot's finished water were discussed at some length in a memorandum dated November 12, 1993 to SWC staff. The available data show that Minot relies on ground and surface water sources that have the potential for formation of THMs and DBPs. The present treatment plant facilities and operational practices were sufficient to meet the current THM standard of 100 µg/l; additional data review shows that certain operational and/or facility modifications will be necessary to consistently meet the more stringent THM standards of the D/DBP Rule. These changes could include: testing of wells, prudent use of higher quality wells, pretreatment practices to increase TOC and DBP precursor removal, chlorination at a lower pH, consideration of an alternative primary disinfectant, and the use of a chloramine distribution system residual.

More limited data about raw and finished water TOC and DBP concentrations are available for Missouri River water. The three TOC data points which are known to exist show a relatively high TOC concentration in the raw water sampled at each location. Bench scale chlorination experiments conducted on raw Lake Audubon water in summer 1993 showed a total THM level below most of the Sundre and Minot wells, and below the Souris River source. Thus, while limited data about Missouri River water show it is superior to local Minot water supplies in terms of inorganic parameters (e.g., TDS, sodium, and sulfates), it too must be treated carefully to limit the production of DBPs.

Biota Transfer Issue. Biota transfer between the Mississippi and Hudson Bay watersheds is an issue that will affect the design and locations of raw water treatment and conveyance facilities. At this point in the project, it is not known what measures will be required to transport raw water across the watershed divide for treatment at Minot. This issue is being studied by a joint technical committee appointed by the International Joint Commission. The final decision by the International Joint Commission is expected in July or August 1994.

WATER SUPPLY OPTIONS

Potential Combinations of Missouri River and Groundwater for Minot-Area Water Supply

Based upon conversations with City staff, the City of Minot desires to use water supplied by the NAWS East System pipeline as the primary City water supply and supplement this base supply with their existing groundwater production facilities. The available, sustainable supply from groundwater is estimated to average about 8 mgd from Minot-area aquifers (i.e., 2 mgd from the Minot Aquifer and 6 mgd from the Sundre Aquifer).

The following options are considered for meeting water quantity and quality requirements of the Minot-area water supply as part of the NAWS project.

All Missouri River Water. This option would consist of eliminating use of the Sundre and Minot aquifer well fields and relying completely on the Missouri River for Minot water supply. Although the overall quality of the raw water supply would improve in terms of inorganic components, this option would abandon use of a significant groundwater infrastructure system for municipal water supply.

Mix of Missouri River and Minot Groundwater. This option would consist of mixing Missouri River and Minot groundwater and softening the blend at the Minot WTP. A potential mix of 65 percent Missouri River water with 35 percent Minot groundwater has been suggested by City staff. A 50/50 mix of Missouri River and Sundre groundwater is discussed in detail on pages 6-4 through 6-12 of the 1988 NAWS Report. The analysis conducted at that time found that the use of 50 percent Sundre Aquifer water would result in higher TDS, sulfate and sodium levels than would be experienced with use of Missouri River water only. While resulting TDS, sodium, and sulfate levels would exceed secondary standards, the water would not be unsafe. The high sodium levels in the Sundre Aquifer water may make blending with more than 50 percent Missouri River Water desirable. If Minot aquifer water is used in conjunction with Sundre aquifer water, the overall water quality is anticipated to improve slightly.

All Groundwater from Minot and Sundre Aquifers. Studies by the SWC suggest that the sustainable yield from the Minot and Sundre Aquifers is probably about 8 mgd. As the annual demand for the Minot system is less than 7 mgd, it would be possible for the City of Minot to continue to use groundwater and not connect into the NAWS system. However, the NAWS system provides a significant source of higher quality water for municipal and industrial supply that could be used for economic development purposes and as a safeguard against the effects of future droughts. Therefore, the City has indicated a desire to be included in the NAWS project.

QUANTITY AND TIMING

The Minot plant is theoretically capable of treating about 18 mgd (12,510 gpm), although hydraulic limitations within the raw and treated water distribution systems presently limit capacity to about 17 mgd (12,160 gpm). Maximum day demands on the Minot system are currently about 15 mgd (10,425 gpm) . The current pumping capacity of the operating wells and the Souris River pumps are reported to be approximately 20.9 mgd (14,500 gpm). Assuming that the Souris River supply is not available during peak periods, the current groundwater pumping capacity is about 17.7 mgd (12,300 gpm), although groundwater production capabilities may be limited by hydraulic restrictions within the raw water pipeline system and recent water level declines in the aquifer. Therefore, it appears the distribution system capacity must be increased and one or more new wells must be constructed for Minot to be able to treat and supply water to additional customers.

The timing of connecting additional users into either the Minot or NAWS systems will be dependent on pipeline construction schedules. For instance, if pipelines from Minot to outlying areas are completed prior to the completion of the NAWS pipeline and expansion of the Minot WTP; it may be beneficial to connect these outlying areas (at least initially) into the Minot supply. Therefore, if areas of critical shortage are identified, these areas could be connected to the Minot WTP for interim water supply prior to construction of the pipeline supplying Missouri River water.

CONCLUSIONS

Water Supply Availability

Groundwater. Sustainable yield from the Minot Aquifer was recently estimated by the SWC at 2 mgd; previously, the SWC estimated sustainable yield from Sundre Aquifer at 6 mgd. A revised estimate of sustainable yield from the Sundre Aquifer is anticipated in the near future.

Production from the Minot and Sundre Aquifers by the City of Minot during 1988 through 1992 (a period of severe drought) averaged 5.9 mgd. Production in 1992 at the end of the drought averaged about 6.2 mgd. As the estimated sustainable yield from the two aquifers totals about 8 mgd, there appears to be potential for a limited amount of additional groundwater development at Minot.

Souris River. Flows in the Souris are expected to decrease in the future as a result of water development projects in Canada. As a result, the Souris River is not considered a dependable source for the City of Minot or the NAWS Project.

Missouri River. Plentiful supplies of surface water are available from the Missouri River at Lake Audubon or Lake Sakakawea. No limitations on water availability from the Missouri River are expected in the future.

Water Supply Needs

The maximum day requirement for the East System is estimated at 29 mgd. Estimated average day requirements are considerably less, about 11 mgd. The projected supply requirement estimated for water treatment plant sizing is 30 mgd.

Water Quality Comparison

Missouri River raw water is superior to Minot raw and treated groundwater in terms of inorganic parameters such as TDS, sulfate, and sodium. Treated Minot groundwater has lower hardness than raw Missouri River water. The TOC concentration of raw Missouri River water from either Lake Sakakawea or Lake Audubon is expected to be higher than the TOC of Minot groundwaters.

Potential Service Area

The potential service area for the East System is shown schematically on Figure 3-2. This area can be served using all Missouri River water or a combination of Missouri River water and Minot area groundwater.

Quantity and Timing

The Minot area aquifers do not have sufficient sustainable yield to serve the entire East System of the NAWS project. However, the potential exists for expanding groundwater production on a short-term basis to serve outlying areas during construction of the pipeline to supply Missouri River water to Minot and other locations on the East System. Expansion of groundwater production would likely require modification or expansion of existing raw water pipelines and construction of one or more additional wells.

Section 4

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SECTION 4

DEVELOP PIPELINE DELIVERY SYSTEM

INTRODUCTION

This section describes the recommended key components of the NAWS pipeline delivery systems including the raw and finished water pipelines, finished water pump stations, and storage reservoirs. Pipeline routes and pump station and reservoir sites were determined. Typical configurations for the pump stations and storage reservoirs are also presented.

PIPELINE ROUTES

Three regional systems have been selected by the State Water Commission (SWC) to serve the various portions of the project area including: the West System, the Parshall System, and the East System. The selected pipeline routes for each of the three systems are shown in Figure 4-1.

West System

The service area for the West System includes all of Williams, Divide, and Burke Counties and a portion of Mountrail County. This system would use the City of Williston's intake structure on the Missouri River and its existing water treatment plant. The required intake and treatment capacity for this system would be 13 million gallons per day (mgd) with 9 mgd to supply major users (the City of Williston, Ray & Tioga Water Association, and Williams Rural Water Association) and 4 mgd to supply the other small communities and rural users in the service area including: Grenora, Powers Lake, Crosby, Stanley, Columbus, and Bowbells. Several smaller cities and towns within the area along with the proposed Writing Rock Rural Water Association would also be served. The design population of the West System is approximately 27,000 people of which 21,100 are city residents and 5,900 reside in rural areas. Approximately 220 miles of pipeline ranging in diameter from 4 to 16 inches, 10 finished water pumping stations, and five reservoirs would be required to serve the system. Figure 4-1 shows the approximate locations of the facilities and the proposed pipeline routes.

Parshall System

The required intake and treatment capacity for this system would be 1.5 mgd with 1 mgd allotted to supply Parshall, New Town, Makoti and Plaza; and 0.5 mgd to supply other users including the proposed Mountrail Rural Water System. The design population of the Parshall System is about 4,700 people of which about 2,800 reside in the four towns and 1,900 reside in rural areas. Construction of approximately 38 miles of pipeline ranging in diameter from 4 to 10 inches, one pumping station (at the expanded water treatment plant), and one reservoir would be required to serve the system. The locations of key elements of the Parshall system are shown in Figure 4-1.

East System

The service area of the East System includes: the northern portion of McLean County; all of Pierce, McHenry, Renville, and Bottineau Counties; and the eastern two-thirds of Ward County. The major users would include: Minot (which also serves the Minot Air Force Base and a portion of the North Prairie Rural Water System), Kenmare, Rugby, and Bottineau; the existing rural water systems of the Upper Souris and All Seasons Rural Water Associations; and the three proposed rural water systems, one around Lake Metigoshe, one in Pierce County, and one in the northern extension of the McLean-Sheridan System. The design population for the East System is approximately 80,300 people of which about 60,900 are city residents and 19,400 reside in rural areas. The maximum design capacity of the East System is about 28 mgd.

The East System would include 457 miles of pipeline ranging in diameter from 4 to 44-inches, a new intake at Lake Audubon, a raw water pumping station, a raw water storage reservoir, expansion and upgrade of the Minot water treatment plant, 14 finished water pumping stations, and 13 finished water reservoirs. Figure 4-1 shows the approximate locations of the facilities and proposed pipeline routes.

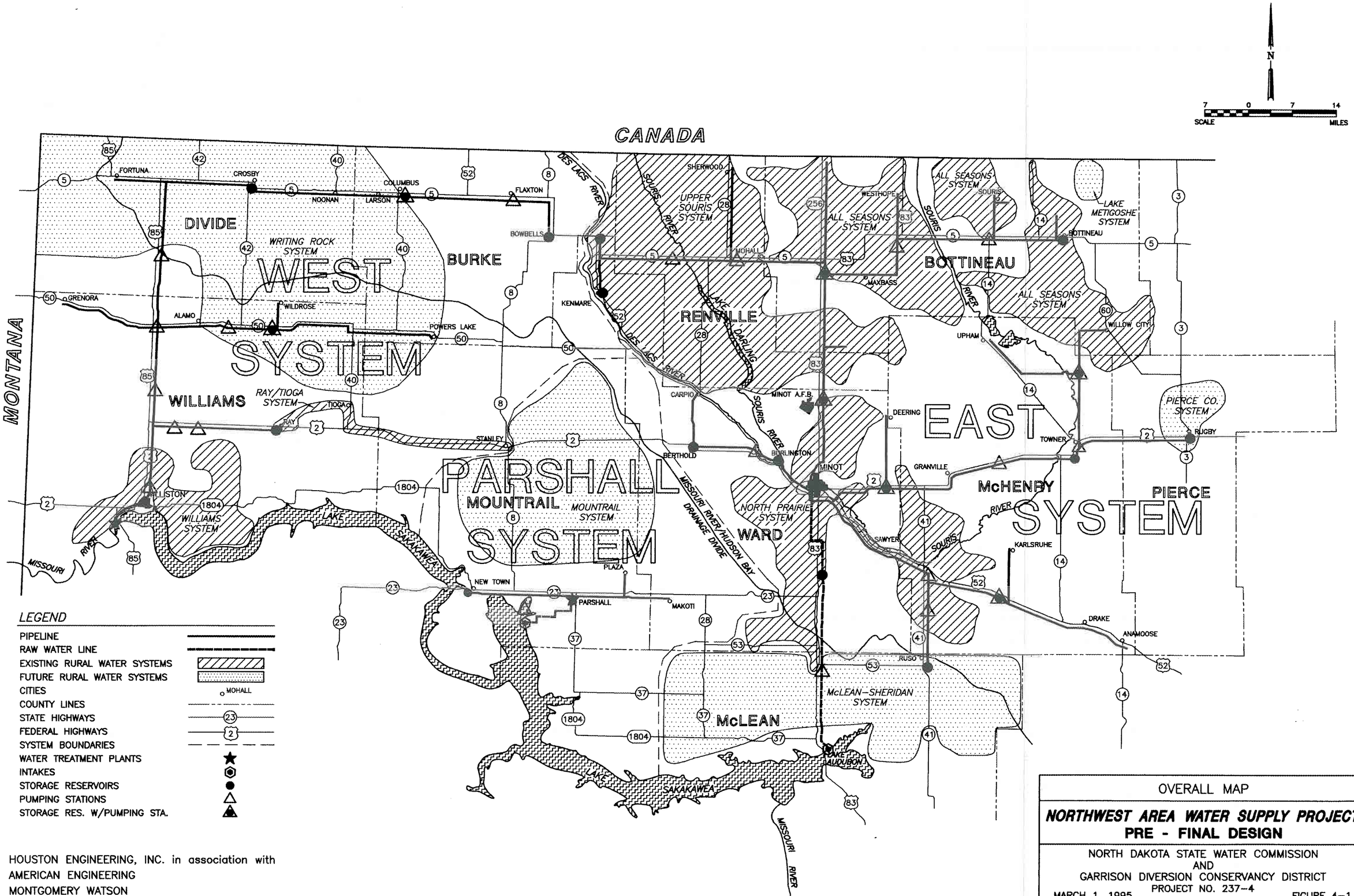
PIPELINE SIZING

Introduction

The pipelines serving the three regional systems were sized to deliver the design population's maximum day water demand. The largest diameter pipeline in the project is the raw water pipeline in the East System which would convey water from Lake Audubon to the upgraded and expanded Minot Water Treatment Plant (WTP) for subsequent distribution to other East System users. With the exception of an additional raw water pipeline to serve the expanded Parshall WTP, all remaining pipelines would convey finished water.

Hydraulics

The pipelines for the three systems were sized for the required maximum day demand flow rates. Pipeline sizing was based on an optimization of pipeline and pumping station capital and operating costs. In general, the pipeline operating pressures in the NAWS water transmission pipelines would not exceed 150 pounds per square inch (psi). In several locations, however, most notably along the raw water line to Minot certain pipe segments would require higher pressure class ratings (200 psi) due to topography. Pressure at turnouts into municipal or rural water systems would have a minimum pressure of 20 psi. The 20 psi minimum pressure (at maximum day flows) will prevent the possibilities of negative pressure in the pipeline. Delivery pressures at the turnouts to system users will vary depending on the delivery point location along the hydraulic gradeline. Pipe sizes were also selected based on friction head loss and flow velocity. In most cases, friction head losses were limited to between 2.5 to 4 feet per 1,000 feet of pipe. A Hazen-Williams friction factor of 140 was assumed for use with steel and ductile



HOUSTON ENGINEERING, INC. in association with
AMERICAN ENGINEERING
MONTGOMERY WATSON

OVERALL MAP

NORTHWEST AREA WATER SUPPLY PROJECT
PRE - FINAL DESIGN

NORTH DAKOTA STATE WATER COMMISSION
AND
GARRISON DIVERSION CONSERVANCY DISTRICT
PROJECT NO. 237-4

MARCH 1, 1995

FIGURE 4-1

iron pipe and a value of 150 was assumed for use with polyvinyl chloride (PVC) pipe. Generally, velocities in finished water transmission lines with diameters greater than 12 inches were kept below 6 fps. Finished water transmission lines with diameters of 12 inches and less were limited to flow velocities of less than 3 fps due to concerns with transient pressure surges and thrust restraint.

Plan and Profile

Plan and profile drawings of the pipeline transmission systems were developed as part of the Pre-final Design Study. The plan and profile drawings are presented in Volume 2 of the NAWS Pre-Final Design.

PIPELINE MATERIALS

Introduction

Four pipe materials are considered as appropriate alternatives for the various NAWS transmission pipelines including: steel pipe (AWWA C200), pretensioned steel cylinder concrete pipe (SCCP) (AWWA C303), ductile iron pipe (D.I.) (AWWA C151), and polyvinyl chloride (PVC) (AWWA C900) pipe. Pipeline materials will be selected based on: type of water being conveyed (i.e. whether raw or finished water), capacity, and economics.

Raw Water Pipelines

Two raw water pipelines would be constructed to supply the existing WTPs at Parshall and Minot. A new 10-inch diameter pipeline would parallel the existing 8-inch diameter PVC raw water pipeline to supply the Parshall WTP. This new raw water pipeline would likely be constructed of PVC with push-on rubber-gasketed joints.

The pipe sizes calculated for the raw water line between the new Lake Audubon intake and the Minot WTP vary between 24 to 42 inches in diameter. The pipeline within the Missouri River basin consists of 38 and 44-inch diameter pipe. This part of the pipeline likely would be constructed of steel or ductile iron pipe with a cement mortar lining and an appropriate exterior coating to reduce corrosion. Recent changes in the AWWA standard for pretensioned steel cylinder concrete pipe have made it less competitive with steel and D.I. Pipe joints would be push-on rubber-gaskets for the ductile iron pipe, and push-on rubber-gaskets or welded joints for the steel pipe. Thrust restraint on the ductile iron pipelines would be accomplished using special coupling adapters. Steel pipe fittings would be restrained by welding. Typically all joints in steel pipelines would be welded for pressure ratings above 225 psi. The use of ductile iron pipe would be limited to Class 250 and below. D.I. pipe manufacturers typically will not guarantee their thrust restraint systems above this pressure rating without special shop and field inspections which are expensive and difficult to implement.

The raw water pipeline to Minot, especially the portion which lies within the Hudson Bay drainage, was the subject of much discussion at the meeting of the Engineering Biology Task Group (EBTG) which met during the winter and spring of 1994 to discuss concerns about the potential for biota transfer between the Missouri River and Hudson Bay drainages. While the EBTG made no recommendations concerning pipeline materials in the portion of the pipeline located in the Missouri River drainage, it did however expect that the pipeline located in the Hudson Bay drainage be constructed of steel with a cement mortar lining and some type of uniform coating on the pipe exterior to reduce corrosion. In early discussions with the EBTG, the sole use of a steel pipeline with all welded joints in the portion of the raw water pipeline located in the Hudson Bay drainage was considered acceptable. Concerns about pipeline leaks from gasketed joints have since been reduced because of the proposed disinfection of the raw water at Lake Audubon and the use of a chloramine residual in the raw water pipeline. Thus, the use of ductile iron pipe as an alternative pipe material may be possible.

Finished Water Pipelines

Finished water transmission pipeline sizes would range from 4 to 24 inches in the NAWS transmission system. Pipeline materials for these pipelines above 14-inches in diameter would be mortar-lined and either mortar or tape coated steel pipe (AWWA C300) or mortar-lined ductile iron pipe (AWWA C151). The standard joint for each pipe type would be the rubber-gasketed push-on joint. The following restraint systems would be utilized: steel pipe would use welded joints and ductile iron would use proprietary restrained joints. Again, above a pressure Class of 225 psi, all steel pipe joints would be welded. D.I. pipe would be limited to pressure Class 250 and below. Pipelines 14-inches and below in diameter would be PVC with mechanical joints (AWWA C110). Concrete thrust blocks would be used for thrust restraint.

SURGE ANALYSIS

Introduction

Hydraulic transients (surges) can cause problems in transmission pipeline systems. Hydraulic transients can occur as both positive (upsurge) and negative (downsurge) pressures that can affect the structural integrity of the pipe and joints from over-pressurization, or cause potential pipe collapse during negative pressure (vacuum) conditions.

Power failure at pumping stations results in the worst case surge conditions, when all pumps immediately stop. Instantaneous power failure is the basis used in the preliminary surge analysis. However, starting or stopping pumps or the rapid opening and closing valves can also create surge conditions. A power failure can result in a rapid downsurge in pressure in the discharge header and piping resulting in a potential in water column separation. Water column separation occurs when the velocity of the water flow is reduced too rapidly and the resulting downsurge in pressure falls below vapor pressure. When this occurs, a vapor pocket is formed. A large vapor pocket is termed column

separation. Column separation can occur at between 25 and 35 ft of negative pressure depending on the elevation, temperature and other characteristics of the water and pipe system. When the vapor pocket collapses and the water columns recombine, a large pressure upsurge is created that can travel through the system at wave speeds of up to 4700 fps.

A preliminary computer analysis was conducted on the pipeline in systems for the East, Parshall, and the West water supply projects. An initial computer evaluation on the pipe systems identified potential problem areas: upsurge pressures and column separation. Subsequent computer evaluations on specific pipe segments, usually between pump stations where surge problems were identified, were used to test conceptual mitigation measures to correct the problem.

Surge Protection Devices

A number of surge protection devices exist to limit or reduce the potential for hydraulic surges. A description of the devices employed in the NAWS project which were shown in the computer simulations to reduce surge problems are described below. In addition, design considerations should also be employed to reduce the formation or consequences of surge formation. Design considerations include avoidance of steep gradients near pumping facilities and valve and equipment selection. Operating procedures to reduce the potential for surges include control of pump start-up and shutdown times, valve closure times, and filling and emptying procedures in the pipelines. Where these measures cannot control surges in the event of a catastrophic system shutdown (power failure), the following devices are used to reduce surges.

Air Release Valves. Air release valves are provided at each pump to permit the removal of air pockets in the pump column and discharge header piping. On larger pumps, a check valve on the air release system is used to prevent air from being sucked back into the valve.

Air and Vacuum Valves. These valves are located on the transmission system at the high points. The combination valves are used to remove air trapped in the line and admit air into the line in the event of negative pressure. Design issues to be addressed concerning surge protection include:

- Sizing of the vacuum valve to admit air quickly to prevent negative pressure or column separation.

- Sizing the air release valve to exhaust air slow enough to prevent over-pressurization should an upsurge be created after column separation. The air in the pipe acts as a compressible cushion in this situation.

Surge Tanks (Surge Arrestor). Surge tanks are provided at the pump stations to reduce surge problems. Surge tanks consist of a steel pressure vessel about half filled with air and water. The tank acts to dampen both positive and negative surges. During pump

shutdown in a power failure, the surge tank, under pressure from the air, discharges water into the pipe. As the resulting upsurge occurs, water is forced back into the tank compressing the air.

Surge tanks have application on the discharge header in pump stations and are also often used on the suction header in booster pump stations. Sizing criteria involves air/water volume relationships, inlet/outlet characteristics, and the size and operating pressure of the discharge header the surge tank is placed on.

Suction Bypass. A suction bypass pipe and a check valve provide a protection device at booster pump stations to reduce the impact of surges. Under normal operating conditions, the check valve prevents backflow from the discharge header to the suction header. Upon power failure however, the resulting down surge in the discharge header results in a lower pressure in this line compared to the pressure in the suction header. The down surge is therefore dampened by flow from the high pressure (suction) side to the pump discharge side.

Computer Model

A computer simulation was used to evaluate hydraulic transients in the transmission systems. The pipeline profiles in the East, Parshall and West Systems were divided into individual pipe systems based on the characteristics of the systems (i.e., pipe size, flow, etc.). These pipes represent actual grades and elevations developed in the Pre-Final Design. Each pipe is then divided into a number of distinct segments. The computer uses flow velocity, acoustical wave speed, and the length of the pipe to determine the number of segments. The calculations are performed under a time marching process and output is printed on time intervals. The model input data required for the pump stations and the pipeline facilities includes:

<u>Pump Station Data</u>

Number of Pumps:	The number of pumps at the pump station, operating in parallel.
Number of stages:	The number of stages in each pump.
Sump elevation:	Water level of constant HGL at the pump suction.
K factor through the pump:	Minor headloss coefficient associated with flow through the pump and adjacent piping.
Steady state speed:	Pump operating rpm.
Moment of inertia:	Rotating inertia associated with each pump/motor combination.

Steady state flow:	Flow rate from each pump in the station.
Discharge pressure:	Gage pressure at the pump station discharge during steady state operation.

Pipeline Data

Pipe Number:	The individual pipes used to approximate the system profile are numbered consecutively, with data defined below.
D:	Pipe diameter.
L:	Length of the pipe.
F:	Darcy-Weisbach friction factor.
L/A:	The time required for a pressure wave to travel from one end of the pipe to the other.
Junction:	The connection point between pipes is numerically coded, to indicate a regular connection, or such options as vacuum valves, standpipes, surge tanks, booster pumps, etc.
Wave speeds - Initial/ Calculated:	The wave celerity for each pipe is input, based on its diameter and type of material. This celerity is then adjusted mathematically to the "calculated" value. In using the Method of Characteristics as a mathematical procedure, this interpolation of wave speeds is needed to maintain Courant Stability.
% C:	The change of wave speed from the input to the calculated value. In general, wave celerities will vary from theoretical values by as much as 10%, so this wave speed adjustment has little impact on the accuracy of the computations.
Upstream pipe elevation:	Elevation at the beginning of the pipe.
Downstream pipe elevation:	Elevation at the end of the pipe.

Transmission System Evaluation

The East, Parshall and the West System were hydraulically evaluated to determine where surge problems exist within the transmission pipelines (refer to Figure 4-1). The evaluation recommends surge control devices to reduce the surge conditions.

The analysis of surge is presented by pipeline segments, referencing specific pump stations, and pipeline routes along highways, based on the Pre-Final Design plan and profile drawings in Volume 2. The segments for each system are identified in sequential numerical order (i.e., East 1, East 2, etc.). The recommended surge control devices are summarized in the following:

East System

East 1. This initial computer run models a simultaneous power failure shutdown of the raw water Lake Audubon intake pump station and the raw water pump station near Max (RW2). Air valves were included at the intermediate high points in the pipe profile in the modeling, but prove to be ineffective in controlling downsurge. Column separation is predicted in both the 44-inch and 38-inch pipelines, therefore surge arrestors are recommended. Subsegments of the raw water line were modeled separately to determine the effectiveness of the recommended surge control devices.

East 1A

This run models the transients in the 44-inch portion of the system from the Lake Audubon intake to RW2. A 3,000 ft^3 surge arrestor was used on the discharge manifold at the intake pump station (RW1), and an 1,800 ft^3 surge arrestor was used on the suction piping of pump station RW2. Surges are well controlled using these devices, and no negative pressures develop on downsurge. The maximum pressure reaches about 200 psig.

East 1B

This run models the 38-inch pipeline from RW2 to the 3.0 mg reservoir (RWS1). A 2,200 ft^3 surge arrestor was included on the discharge piping of the pump station RW2, and air valves were included at various high points on the pipeline for additional downsurge protection. Minimum pressures fall to -10.4 psig (-24 feet), with no threat of column separation. Maximum pressures reach 156 psig for this portion of the system.

Minot. This computer model is a composite of the four pipelines leaving the Minot Water Treatment Plant, but generally following the profile of the 24-inch line heading north to reservoir FWS1 at Minot AFB. A power failure shutdown of the water treatment plant's finished water pump station (FW1) is assumed, at a flow rate of 8280 gpm. Column separation is predicted, even with air valves located at the high points in the pipeline profile, therefore a surge arrestor is recommended.

Minot A

This computer run duplicates the previous model, but utilizes a 600 ft^3 surge arrestor at the finished water pump station. Minimum pressures fall to -5.2 psig (-12 feet) with no threat of column separation. Maximum pressures on flow reversal reach 165 psig.

East 2. This computer run models transients in the 16-inch and 14-inch pipelines on Highway 83 from the reservoir and pump station at Minot AFB (FW2) north to the 1.0 mg reservoir (FWS2) near the junction of Highway 5. Five air valve locations are included in this model, and their operation is imperative for adequate control of downsurge. Minimum pressures are predicted to fall to -10.6 psig (-24.5 feet) for this run, with no upsurge being generated on flow reversal.

East 3. This computer run models the simultaneous power failure shutdown of pump station FW3 located at storage reservoir FWS2, and pump stations FW4 and FW5 located on Highway 5, feeding the 1.5 mg reservoir (FWS3) near Bottineau. The pump stations FW4 and FW5 are fitted with 8-inch suction bypass pipes, to permit unrestricted flow through the station on power failure. These bypasses prove to be adequate as no negative line pressures develop on downsurge, and maximum line pressures are less than 200 psig.

East 4. This computer run models the simultaneous power failure shutdown of pump station FW3 located at storage reservoir FWS2 and the pipe system the West. This includes pump stations FW6 and FW7 feeding the reservoirs at Kenmare (FWS4) and Upper Souris I (FWS5). For this analysis, 8-inch suction bypass pipes are utilized at pump stations. No negative pressures are predicted on downsurge and maximum pressures are less than 150 psig.

East 5. This computer run models the transients of a power failure shutdown in the 10-inch pipeline from the 0.5 mg reservoir near Burlington (FWS12) to the 0.5 mg reservoir at Berthold. The pump station (FW15) between these two reservoirs is fitted with an 8-inch suction bypass pipe. No negative pressures are predicted on downsurge, and maximum pressures do not exceed 150 psig.

East 6. This computer run models the simultaneous power failure on the 14-inch pipeline from the reservoir pump station (FW8) on Highway 2 at the Deering turnout, east, to the 0.5 mg reservoir at Towner (FWS7), including pump station FW9. Pump station FW9 was fitted with a 10-inch suction bypass pipe. No negative pressures developed on downsurge, and maximum pressures do not rise above 150 psig.

East 7. This computer run models the transients of a power failure shutdown in the 14-inch pipeline from Towner to the 2.0 mg reservoir at Rugby (FWS8). The pump station between these facilities (FW10) was fitted with an 8-inch suction bypass. Air valves were also used at two intermediate high points along the pipeline profile. Minimum downsurge pressures fall to -11.3 psig (-26 feet), and the maximum pressures were less than 150 psig.

East 8. This computer run models the transients of a power failure shutdown in the 6-inch pipeline running north from Towner to pump station FW11, which serves Upham and Willow City. This pump station was fitted with a 4-inch suction bypass. No negative pressures develop and the maximum pressures were less than 150 psig.

East 9. This computer run models the transients of a power failure shutdown in the 10-inch pipeline along Highway 41, running south to the storage reservoir at Ruso (FWS11). The two pump stations in this reach (FW12 and FW13) are assumed to fail simultaneously, and each was fitted with an 8-inch suction bypass. Pressures drop to -10.8 psig (-25 feet) near one of the pump stations, but generally no negative pressures are predicted for the transmission pipeline. Maximum pressures are predicted to rise to 170 psig.

East 10. This computer run models the transients of a power failure shutdown on the 10-inch, 8-inch and 6-inch pipelines running east on Highway 52 from pump station FW12 at the intersection of Highway 41 and 52 to the reservoir FWS10. No negative pressures are predicted for this run and maximum pressures are less than 150 psig.

Parshall System

This computer run models a power failure shutdown of the finished water pump station at the Parshall water treatment plant, and the transient conditions that develop in the 10-inch piping feeding the 1.0 mg reservoir (FWS1) nearly 19 miles away at New Town. Air valves at two intermediate high points appear to control downsurge sufficiently, and no column separation is predicted. Maximum pressures are less than 150 psig.

West System

West 1. This computer run modeled a simultaneous power failure shutdown of the 300 hp pump station (FW2), and pump stations FW10 and FW11 in the branch line along Highway 2 to the town of Ray. Air valves were included at high points in the pipe profile, but these fail to provide adequate protection as column separation is predicted in several areas of the profile. Surge arrestors are recommended at pump station FW2, as well as at pump stations FW10 and FW11. Computer evaluations of the sub-segments of the West 1 system were then analyzed individually with the specific surge control devices.

West 1A

This run models the 16-inch discharge piping from the pump station FW2 to the intersection of Highways 85 and 2, and the 14-inch pipeline along Highway 2 to the suction of pump station FW10. A 400 ft^3 surge arrestor is included on the discharge piping at pump station FW2, and a 200 ft^3 surge arrestor is included on the suction piping at pump station FW10. Surges are well controlled to prevent column separation. Maximum pressures reach about 200 psig.

West 1B

This run models the 14-inch piping from pump station FW10 to pump station FW11. A 120 ft^3 surge arrestor is included on the discharge piping at pump station FW10, and a 200 ft^3 surge arrestor is included on the suction piping at pump station FW11. Column separation is prevented with these proposed surge vessels, and the maximum pressures are under 200 psig.

West 1C

This run models the 14-inch discharge piping from pump station FW11 to the 2.0 mg reservoir near Ray (FWS5). A 250 ft^3 surge arrestor is recommended on the discharge at this pump station. Negative pressures develop near the end of the profile, but column separation is avoided using the surge arrestor and air valves at intermediate high points. Maximum pressures do not exceed 150 psig.

West 2. This computer run models the 14-inch and 10-inch pipelines from the Ray turnout north along Highway 85 and east along Highway 5 to the 0.5 mg reservoir at Crosby (FWS1). A simultaneous power failure shutdown of pump stations FW3, FW4 and FW5 on this line was assumed. An 8-inch suction bypass line (with check valve) was modeled at each pump station to permit unrestricted flow from the suction line to the discharge header of each station. This helps attenuate the downsurge by limiting the drop in discharge pressure at the pump station. These bypasses, in conjunction with air valves at intermediate high points along the pipeline, are shown to prevent column separation in this portion of the system piping. Maximum pressures are predicted to be less than 160 psig.

West 3. This computer run models the 8-inch and 6-inch pipeline from the intersection of Highways 85 and 50 (discharge to pump station FW4), to the 0.5 mg reservoir near Wildrose (FWS4). Suction bypass lines are used at pump stations FW8 and FW9 on this pipeline to limit downsurge and prevent column separation on simultaneous power failure shutdown of both pump stations. An 8-inch bypass was used at pump station FW8 and a 6-inch bypass line is used at pump station FW9. Maximum pressures are less than 150 psig for this portion of the system.

West 4. This computer run models flow in the 8-inch and 6-inch pipeline from the 0.25 mg reservoir (FWS2) on Highway 5 near Columbus to the 0.5 mg reservoir at Bowbells (FWS3). The pump station between these two reservoirs (FW7) is fitted with a 4-inch suction bypass, which controls downsurge on power failure pump shutdown. No negative pressures develop under this situation, and the maximum surge pressures are less than 150 psig.

PRESSURE REDUCING STATIONS AND PRESSURE RELIEF VALVES

Introduction

The need exists to modulate flow and/or pressure at several locations in the NAWS pipeline systems to reduce the pressure differential between upstream and downstream locations. The required facilities are described below.

Raw Water Pipelines

A flow control station would be needed to dissipate pressure in the raw water line from Lake Audubon before the raw water enters the wetwell at the Minot WTP. A sleeve valve would be used to reduce the pressure head. Sleeve valves consist of two fabricated concentric or eccentric cylinders, one of which is provided with a series of holes. The outer cylinder remains stationary, while the inner cylinder is moved by a motor-operated screw to expose as many holes as necessary to reduce the variable upstream pressure to near zero. Sleeve valves may be installed in either the horizontal or vertical position. The most common orientation is vertical, where the valve is located in a stilling well. Unlike other types of energy dissipation valves, the sleeve valve is dynamically balanced for designed flow and pressure conditions and there is essentially no vibration. The raw water pressure reducing station would be located on the 34-inch raw water pipeline at a location to maintain pipe pressure class in the 34-inch pipe at 150 psi.

Finished Water Pipelines

Pressure reducing valves are provided at three locations in the East System and four locations in the West System to reduce system head to allow use of a lower pressure class pipe, and at several turnouts to municipal and rural water systems.

The pressure reducing valves would be installed on the East System at the following locations:

- on the 4-inch diameter line from Berthold to Carpio,
- on the 4-inch diameter line from State Highway 5 to Souris,
- on the 6-inch diameter line from U.S. Highway 2 to Deering.

Pressure reducing valves would be installed at the following locations on the West System:

- on the 14-inch diameter line along Highway 2 to Ray,
- on the 4-inch diameter line, from U.S. Highway 85 along State Highway 50 to Grenora,
- on the 10-inch diameter line along Highway 5 toward Crosby,
- on the 8-inch diameter line along Highway 5 between Crosby and Noonan.

The valves would be installed in turnout vaults to protect against freezing. Although a number of different types of pressure reducing valves could be utilized to accomplish the required pressure reduction, it is again recommended that sleeve valves be used due to their effectiveness and reliability. A smaller pressure regulating valve and other line valves would also be installed with the sleeve valves to supply lower demands and to allow isolation of the larger valves for maintenance purposes.

PRELIMINARY CORROSION EVALUATION

A preliminary corrosion evaluation was prepared for the proposed pipeline routes using soils data from U.S. Department of Agriculture reports and other available information and is presented in Appendix A. In summary, the report concluded that soils in the project area exhibit a wide variation in characteristics (which is typical of glacial till) and are likely to be corrosive to coated steel and D.I. pipe. Corrosion control measures will typically be needed to increase the useful life of the metallic pipeline system. Components of an effective corrosion control plan will include: proper material selection, high performance coatings, and cathodic protection measures specific to the pipeline materials and soils environment. A more detailed corrosivity investigation during design will be necessary to evaluate conditions specific to the pipeline system and to recommend an effective corrosion control plan. For the purposes of this report, it will be assumed that all metallic pipe will require coatings and cathodic protection.

PUMP STATIONS

Introduction

This section presents the design concepts and assumptions used in developing the preliminary design of pumping facilities which will serve the three regional transmission systems. The tasks involved with the predesign of the pumping facilities included:

- Locating the pump stations within the transmission system;
- Determining a pump type and a standard pump station configuration;
- Developing a pump control strategy;
- Evaluating electrical power availability near the proposed pump station locations; and
- Developing a pump station communications and telemetry system.

Pump Station Location

The locations of the various raw and finished water pumping facilities in the NAWS system are shown in Figure 4-1. The East System would have the largest number of finished water pump stations with a total of 14. The raw water pumping system includes the Lake Audubon intake and the Max pump station. The Minot WTP finished water pumping station would be retrofitted to a higher pumping capacity to allow for service to the other consumers in the East System. As shown in Table 4-1, the installed pumping capacity for the East System pump stations would range from 9 to 3800 hp.

TABLE 4-1

PUMP STATIONS

System	Pump Station Location	Total Capacity (gpm)	Pumping Head (ft)	Required Horsepower (hp)	Individual Pump Capacity (hp)
East	RW1 - Lake Audubon	20,850	368	2500	2 @ 350 1 @ 600 2 @ 1250
	RW2 - Near Max	20,850	283	2000	2 @ 250 1 @ 500 2 @ 1000
	FW1 - Minot WTP	19,829	346	2250	3 @ 500[a]
	FW2 - Minot Air Force Base	2,525	331	300	4 @ 100
	FW3 - 3 miles south of the intersection of U.S. Hwy 83 and State Hwy 5	2,359	281	200	4 @ 75
	FW4 - Westhope Turnoff	1,282	224	75	3 @ 40
	FW5 - Souris Turnoff	1,046	406	150	4 @ 50
	FW6 - 1.5 miles east of intersection of Hwys 5 and 28	637	257	60	3 @ 30
	FW7 - 4 miles west of Souris River along Hwy 5	588	317	60	3 @ 30
	FW8 - Turnoff to Deering along U.S. Hwy 2	2,195	236	200	4 @ 75
	FW9 - 14.8 miles west of Towner along U.S. Hwy 2	1,907	315	200	4 @ 75
	FW10 - Intersection of Hwy 14 and U.S. Hwy 2	1,550	223	125	4 @ 40
	FW11 - 11.2 miles west and south of Willow City	164	92	5	3 @ 3
	FW12 - 0.5 miles south of Velva	791	285	75	3 @ 40
	FW13 - 8.6 miles south of Velva on Hwy 41	424	342	50	3 @ 25
	FW14 - 0.7 miles west of the turnoff to Karlsruhe along Hwy 52	302	315	30	3 @ 15
	FW15 - 5.7 miles west of Burlington along U.S. Hwy 2	391	336	50	6 @ 10
Parshall	RW1 - Van Hook Arm Lake Sakakawea	1067	71	25	3 @ 15
	RW2 - Transfer Station	1067	271	100	2 @ 25[b] 2 @ 60
	FW1 - Parshall WTP	1067	346	125	2 @ 25[b] 2 @ 60
West	RW1 - Missouri River at U.S. Hwy 85	9994	63	200	1 @ 50[b] 2 @ 100[b] 1 @ 100

TABLE 4-1 (continued)

PUMP STATIONS

System	Pump Station Location	Total Capacity (gpm)	Pumping Head (ft)	Required Horsepower (hp)	Individual Pump Capacity (hp)
West	FW1 - Williston WTP	8861	165	550	1 @ 100[b] 1 @ 150[b] 1 @ 300[b] 1 @ 300
	FW2 - North of Williston 10 miles south of intersection of U.S. Hwys 85 and 2	2753	316	300	4 @ 100
	FW3 - 5.8 miles north of intersection of U.S. Hwys 85 and 2	1274	300	125	4 @ 40
	FW4 - Intersection of U.S. Hwy 85 and Hwy 50	923	248	75	3 @ 40
	FW5 - 12.8 miles south of intersection of U.S. Hwy 85 and Hwy 5	923	247	75	3 @ 40
	FW6 - Columbus Turnout	214	296	20	3 @ 10
	FW7 - Flaxton Turnout	193	215	15	3 @ 7.5
	FW8 - 4.8 miles east of Alamo turnout along Hwy 50	211	295	20	3 @ 10
	FW9 - 0.8 miles west of turnout to Wildrose	211	112	7.5	3 @ 5
	FW10 - 4.2 miles east of intersection of U.S. Hwys 85 and 2	1480	315	150	4 @ 50
	FW11 - 7.9 miles east of intersection of U.S. Hwys 85 and 2	1480	345	200	4 @ 75

a The existing Minot WTP pump station will be modified to replace three existing pumps with three 6 mgd pumps (500 hp each).

b Existing pumps. Modification to existing facilities will include construction of additional capacity as shown.

The Parshall system would require an expansion of its existing raw water intake and transfer pumps as well as an expansion of the finished water pumping capacity at its expanded water treatment plant. No additional pumping facilities would be required to convey water to other parts of the Parshall system.

Eleven pump stations would be required to deliver water to consumers in the West System including the finished water pumping station at the Williston WTP which would require upgrading. In addition, the raw water pumps at Williston's intake on the Missouri River will have to be retrofitted to a higher capacity.

Pump Selection

Introduction. Pumps will be needed to lift raw water from the water sources into treatment plants and/or transmission lines, and boost line pressure to convey finished water to customers. A number of different types of pumps are available for use in municipal applications. Based on the service required, we would recommend two types of pumps for the NAWS project: vertical turbine pumps and horizontal centrifugal pumps. Both are very efficient, reliable and accepted as standard in the industry. Horizontal centrifugal pumps are cost effective and easy to repair since access is available above ground to the entire unit, however, it is necessary to locate horizontal centrifugal pumps below the hydraulic gradeline to aid in priming. Vertical turbine pumps require much less space to install and provide greater flexibility in many situations.

Based on these considerations, vertical turbine pumps are recommended for use in the new Lake Audubon intake and in locations where water is boosted to design pressure from an in-line storage facility located with or nearby the booster pump station. Booster stations without storage should be evaluated during design with regard to economics and client preference; however, at this time it is envisioned that vertical turbine pumps using cans (vessels) would be used in most booster stations.

Pump Selection. A variety of service conditions will be encountered while addressing the various pumping needs in the three transmission systems. A standard pump station layout is recommended to reduce engineering and construction costs. The complexity and details of individual pump stations will vary, however, depending on their size and service. The following guidelines were used in developing a typical pump station layout. First, except for several large pumps at the new Lake Audubon intake, all other pumps will be 480 or 230 volts which will reduce the pump motor and control system costs. Second, the number of pumping units in any pump station to be constructed along the transmission lines will typically be limited to a minimum of three or to a maximum of five. The number of pumps in each particular station will be a function of: equipment reliability, demand patterns, and available storage. In the smaller pump stations, three similar sized units will be provided to accommodate average and maximum day demands as well as a standby unit which may be substituted for another pump when that pump is unavailable for use.

Where 3-phase power is not available (FW15 in the East System), a number of small pumps are proposed. Typically, motor horsepower sizes are limited to a maximum of 10 hp for single-phase power. The number of pumps in all pump stations will be selected to provide the maximum day demand with the largest pumping unit out of service. Finally, limiting the number of pump sizes within a pump station and also among other pump stations would reduce the necessary spare parts inventory and facilitates maintenance. The number and capacity of pumps to be installed at the various raw and finished water pump stations in the three regional systems are shown in Table 4-1.

Pump Station Layout. Each vertical turbine pump will be mounted in a separate can, and connected to a common suction header. The intake, suction header, and pump cans will be designed to provide uniform flow to the pumps and minimize vortex formation and air entrainment. The Hydraulic Institute guidelines for pump intake design as well as manufacturer's recommendations will be used in final design.

Each pump discharge line will be equipped with a valve for pump control, a check valve to prevent backflow in the event of a power failure or other pump shutdown, a butterfly valve for closing the discharge line when a component upstream of the butterfly valve must be removed for servicing, an air valve (to eliminate air pockets in the discharge line), a pressure sensor (for low and high discharge pressure shutdown), and a pressure gage.

The intake line, suction manifold, and buried portions of the discharge manifold will be lined and coated with cement mortar. The pump cans will also be lined and coated with cement mortar. The above-ground discharge line from each pump will be cement mortar lined and painted on the outside. Connections will be made using flanged connections meeting American Water Works Association (AWWA) specifications unless specifically excepted. Insulating flanged connections will be provided at the intake and discharge sides of the station to electrically isolate the station from the pipeline. Surge control tanks will be required at several pump stations, and the tanks will be located inside the pump building. In other pump stations, a suction bypass with a check valve will be required for surge control. The manifolding, couplings, and valves will be designed to withstand the anticipated maximum pressures caused by fluid transients (water hammer). A typical layout of a canned vertical turbine pump station is shown in Figure 4-2.

Pump Station Automatic Control

The pump stations and reservoirs in the transmission systems have been sized large enough to provide the maximum day demand over a 24-hour period and with flexibility to meet low, average, and high demands. Each pump station in the NAWS system typically will be taking suction either from a reservoir or from a lower pressure zone and will be designed for multiple units to meet the range of flows and pressures. The pumps will be activated or deactivated in response to changes in flow or pressure set points. Each pump will be driven by a constant speed drive. The discharge header will be provided with a pressure relief valve connected to the pump suction header or to the suction reservoir if shut-off pressure of selected pump(s) exceeds the established range of

operating pressure for the higher zone. The pumps also will be controlled by pressure with a low suction pressure or a level cutoff switch, and a high discharge pressure cutoff switch.

Depending on system demands, the lead pump will operate continuously, except in cases where the system can be fed by gravity and it is not necessary for any pumps to be operating. As the demand increases, the flow will increase and the discharge pressure will drop. When the first flow set point is reached, the first lag pump will start. At this point, if the demand stays constant, the discharge pressure may increase causing the bypass relief valve to release flow back to the suction in excess of the demand to maintain the pressure in the system. When the demand increases, the pressure will decrease until the preset low pressure is reached. Decrease in pressure will increase flow until the second flow set point will signal the second lag pump to start. If an additional pump is required, the sequence will follow in the same manner as for the first and second lag pumps. Decrease in demand will signal the last lag pump to stop as soon as the set point is reached. Further decrease in flow will signal the pumps to stop in reverse order. It will be necessary to include adjustable delays in the control program to prevent too frequent cycling.

Equipment Protection Circuits (EPCs) will be provided which shut down equipment upon detection of exceptionally low pressure or low wetwell level conditions, motor winding temperature, or vibration, etc. EPCs shall be hardwired and should function independently of sensors used for steady state operation of equipment.

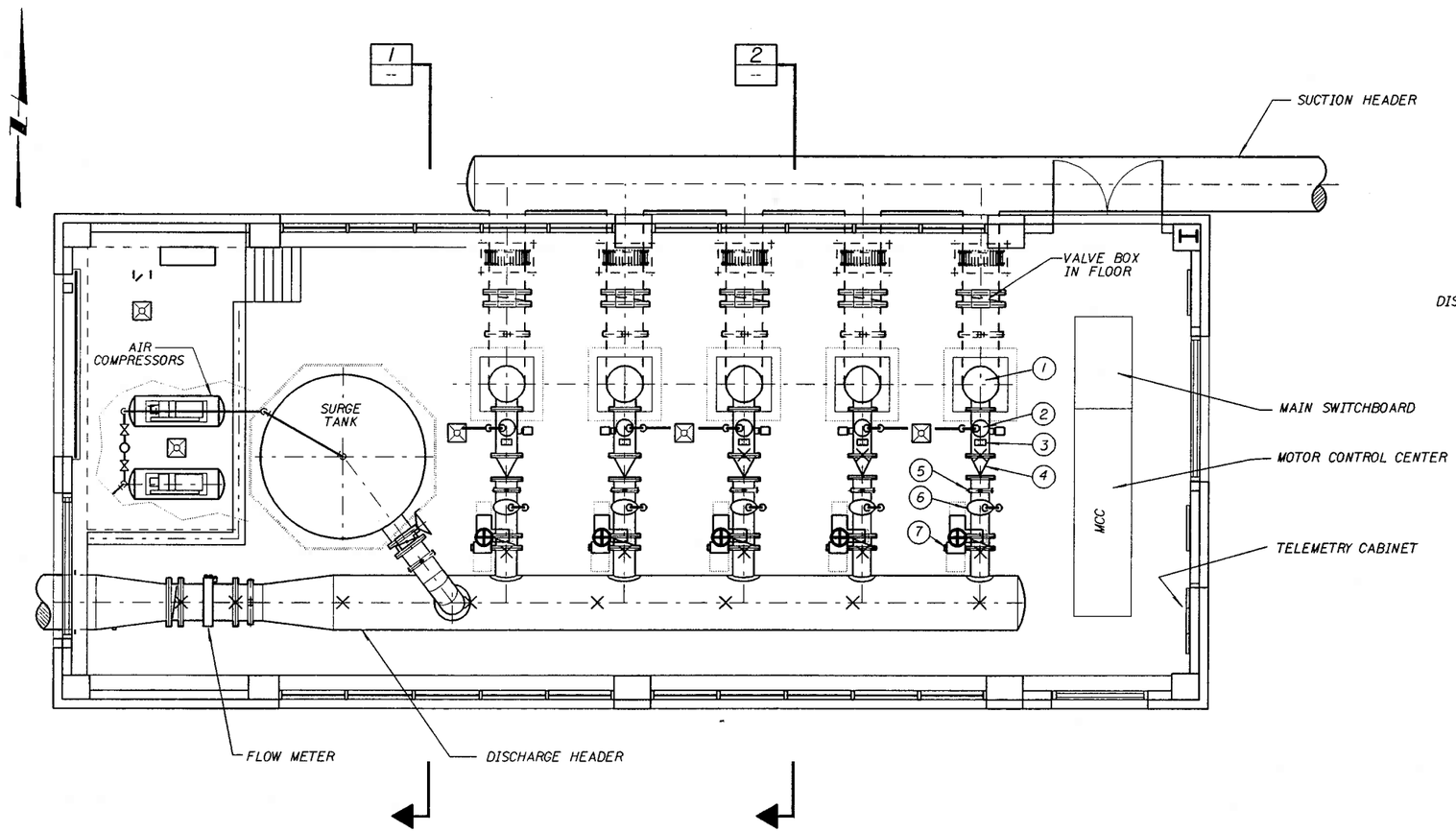
After each pump operation, the pumping sequence would be alternated to provide even pump "run times". Independent high and low level alarm switches would be provided at reservoirs to protect the pumps and alert the operations personnel of a high water level problem.

Electrical Service

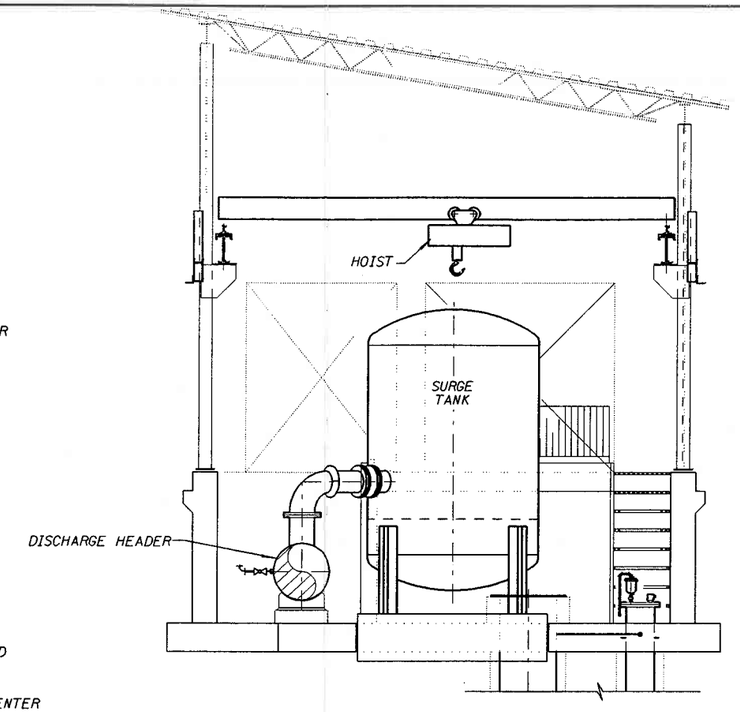
The required horsepower at each pump station along the transmission system is presented in Table 4-1. In general, the most economical electrical service to power the pump stations along the transmission lines will be three phase, 480 volt service. The exception are the individual pumps over 800 hp at the Lake Audubon intake and the booster pump station at Max. These pump stations will require both 480-volt and either 2400 or 4160 volt supplies for the large pumps. During final design of these facilities, replacing the large pumps with multiple pumps smaller than 800 hp will be evaluated. This would allow 480-volt power to be used for the entire pump station.

Power is available to the pump stations from the following Rural Electric Cooperatives (REC):

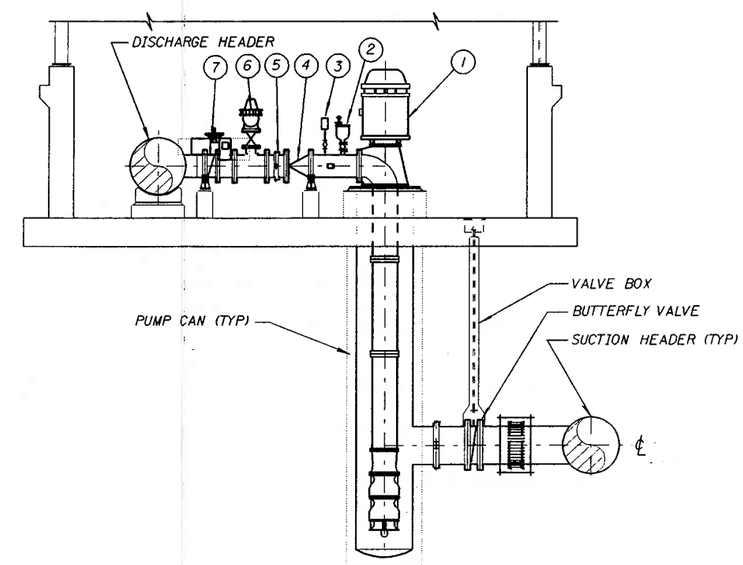
- Burke Divide Electric Cooperative, Columbus, ND
- McLean Electric Cooperative, Inc., Garrison, ND
- Mountrail-Williams Electric Co-op, Williston, ND



PLAN



SECTION 1



SECTION 2

TYPICAL PUMP LAYOUT

1. PUMPING UNIT
2. AIR VALVE
3. PRESSURE GAUGE
4. CHECK VALVE
5. MECHANICAL COUPLING
6. PUMP CONTROL VALVE
7. BUTTERFLY VALVE

TYPICAL PUMP STATION
PLAN AND SECTIONS
FIGURE 4-2

1/4"=1'-0"

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- North Central Electric Cooperative, Bottineau, ND
- Verendrye Electric Cooperative, Velva, ND

The regional RECs were contacted regarding the availability, capacity, and type of power adjacent to the proposed pump station locations. From these conversations, it appears that three pumping stations in the East System and four pump stations in the West System will have only single-phase service. These systems will require that new three phase electrical service lines and/or that the existing single phase lines be converted to provide three phase power to economically service these pumping stations. These pump stations include:

East

- FW5-Souris Turnoff,
- FW9-14.8 miles west of Towner along U.S. Hwy 2,
- FW14-0.7 miles west of the turnoff to Karlsruhe along Hwy 52.

West

- FW3-5.8 miles north of the intersection of U.S. Hwy 85 and 2,
- FW5-12.8 miles south of intersection of U.S. Hwy 85 and Hwy 5,
- FW10-4.2 miles east of intersection of U.S. Hwys 85 and 2,
- FW11-7.9 miles east of intersection of U.S. Hwys 85 and 2.

The RECs contacted indicated the costs of running new three phase power lines varied from about $15,000 to $40,000 per mile. One pumping station in the East System and two pumping stations in the West System will require that single phase power be supplied by new lines. These pump stations include:

East

- FW15-5.7 miles west of Burlington along U.S. Hwy 2.

West

- FW8-4.8 miles east of Alamo turnout along Hwy 50,
- FW9-0.8 miles west of turnout to Wildrose.

Pump station FW15 in the East System will have to rely on six 10 hp pumps to supply system needs, which is at the pump motor hp threshold which would require installation of three phase power. All of the RECs contacted indicated that the cooperatives could supply the line, meter, and transformer to the pumping stations.

Standby Power. Several factors enter into an analysis of the risk of interruption (vulnerability) of design pumping capacity and the need to provide standby power. The risk factors that need to be evaluated include:

- Equipment reliability
- Demand patterns
- Power interruption frequency
- Available storage

It is difficult to compute an accurate or comparable risk factor. Historical data are generally not representative of future interruptions because the causes of past interruptions may have been partially corrected. Also, future conditions may not have been experienced in the past. In the case of water systems most short-duration power interruptions are not a serious concern because of the provision of system storage. As stated in Section 2, no standby power will be provided to the pumping units in the proposed facilities at this time. Auxiliary standby power equipment, sufficient to operate key building systems (i.e. heat and light) will be provided.

SYSTEM COMMUNICATIONS AND TELEMETRY

Introduction

This section describes the methods, equipment, and types of instrumentation and control for the NAWS water supply, treatment, and distribution system facilities. It is proposed to implement a Supervisory Control and Data Acquisition (SCADA) system for each of the three NAWS systems.

A number of alternatives exist to develop a communications telemetry network to link the proposed SCADA system together. These include radio based systems, leased hard wire, buried private wire, etc. Given the size and distances of the proposed control systems, several factors were considered important in the evaluation of the communication systems, including:

- Cost
- Reliability
- Quality of digital/data transmission
- Ownership

Generally, radio based systems are less expensive than the other alternatives, particularly given the distances involved between all of the remote facilities and the central control systems to be located at each of the three regional water treatment plants. Reliability and quality of transmission would need further evaluation for hard wired systems. However, experience has shown that radio based systems have an acceptable degree of reliability and quality in similar installations as that proposed for the NAWS project. Finally, it is recommended the ownership and maintenance responsibility for the entire telemetry system remain with one entity, rather than relying on other entities to maintain part of the communications system.

During final design, detailed analysis may identify certain conditions that dictate the use of other communication telemetry systems where radio based telemetry may not provide

adequate service. However, for the purpose of the Pre-Final Design, the radio based system is recommended as the most cost-effective system.

The key elements of the system pre-final design include a radio survey, selection of radio frequencies, design of the SCADA control concept, delineation of the computer facilities, identification of control room facilities and identification of pumping station and storage reservoir control facilities.

The NAWS system has been divided into three separate systems, each system is associated with a water treatment plant. The systems are defined at the East System, the Parshall System, and the West System. The East System is designated by the prefix "E" on Figure 4-3. This system starts at the water treatment plant, designated "WT" and is SCADA node 1; consequently the designation for this water treatment plant is E-WT-1. Associated with this system are 14 pump stations, 14 reservoirs, and two raw water pump stations. Pump station and storage reservoir names, as used in this section, are used to identify remote telemetry units and SCADA nodes (E-FW14-5, E-FWS4-22, etc.).

The Parshall System consists of one water treatment plant, one raw water pump station and one reservoir. The West System consists of one water treatment plant, 10 pump stations, and 15 reservoirs. The West and Parshall systems are shown on Figure 4-4.

The system has been configured so that the water treatment plant for each system acts as the master telemetry unit (MTU) for the system. Therefore, each water treatment plant will act as SCADA Central for its system. By locating the SCADA Central stations at the water treatment plants, the system demands with respect to pumping and any other necessary changes to the plant flow can be determined and acted upon. Since the water treatment plants are staffed on a more continuous basis and pump stations are responded to on a scheduled or failure basis, the water treatment plant should have the staff available in order to operate the system more efficiently.

Figures 4-3 and 4-4 depict: the configuration of the systems, the pump stations and their associated remote telemetry units (RTUs), and the reservoirs and their RTUs. Proposed radio towers have been included based upon a preliminary radio survey. At pump stations located at reservoirs, only one RTU is provided for both facilities.

In general, the control system will be programmable logic controller (PLC) based. The SCADA system will be used as a window into the PLC control system. The PLC control system will function as both the manual and automatic control system. This is to say that the PLC will be used as an interposing device even in the manual control mode. There will be direct control of motors and valves outside of the PLC control loop; however, there will be no logic associated with any devices in this control mode. PLC redundancy will be addressed by having sufficient spare components on hand to effect a rapid replacement of defective parts. The PLC has been chosen as the primary control device, because it is industrially hardened, and more suitable for industrial control environments than the SCADA system computers. Industrially hardened computers can be purchased;

however, the premium paid for such devices is quite high, and does not appear to be justified in these cases.

SCADA System

Radio Survey. The means for communication between the pump stations, reservoirs, and the water treatment plant has been selected to be primarily radio based. A preliminary computer simulation radio survey was conducted as a component of this project. The purpose of this study was four fold:

(1) Determine if radio telemetry is a feasible alternative to process control for the three defined systems;

(2) Determine the approximate equipment requirements if a radio telemetry system proves to be a feasible alternative;

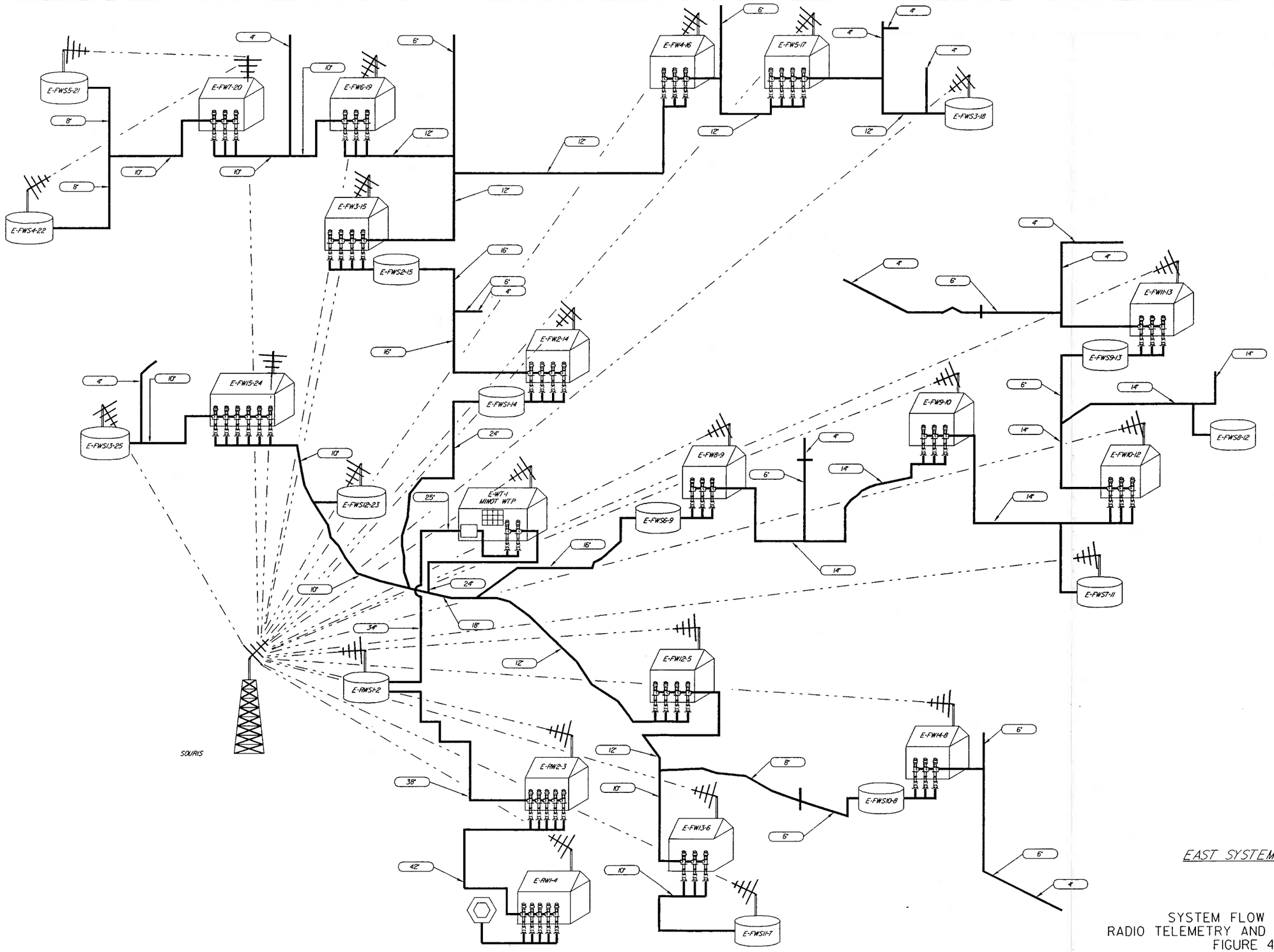
(3) Determine alternatives for frequencies for use in the system; and

(4) Determine what further studies will be necessary for fully designing the system if the radio telemetry system is selected for implementation.

The computer simulation radio survey used a digital elevation modeling database. The United States Geological Survey has developed a digital elevation database for the entire United States. The database uses 3 arc sec elevation data, in which data is taken every 3 sec or approximately 300 feet to develop a topography of the area. Given this database, there are several methods used to interpolate the points in the digital elevation model. The four point method, which is an FCC recommended method, is the least accurate and simply averages the closest four points. This method tends to lose valleys, gullies, hills, and mountain tops. The 12 point averaging method takes slopes from all four directions toward the point. This method is more accurate and picks up valleys and hills better. The 12 point method was used for the study.

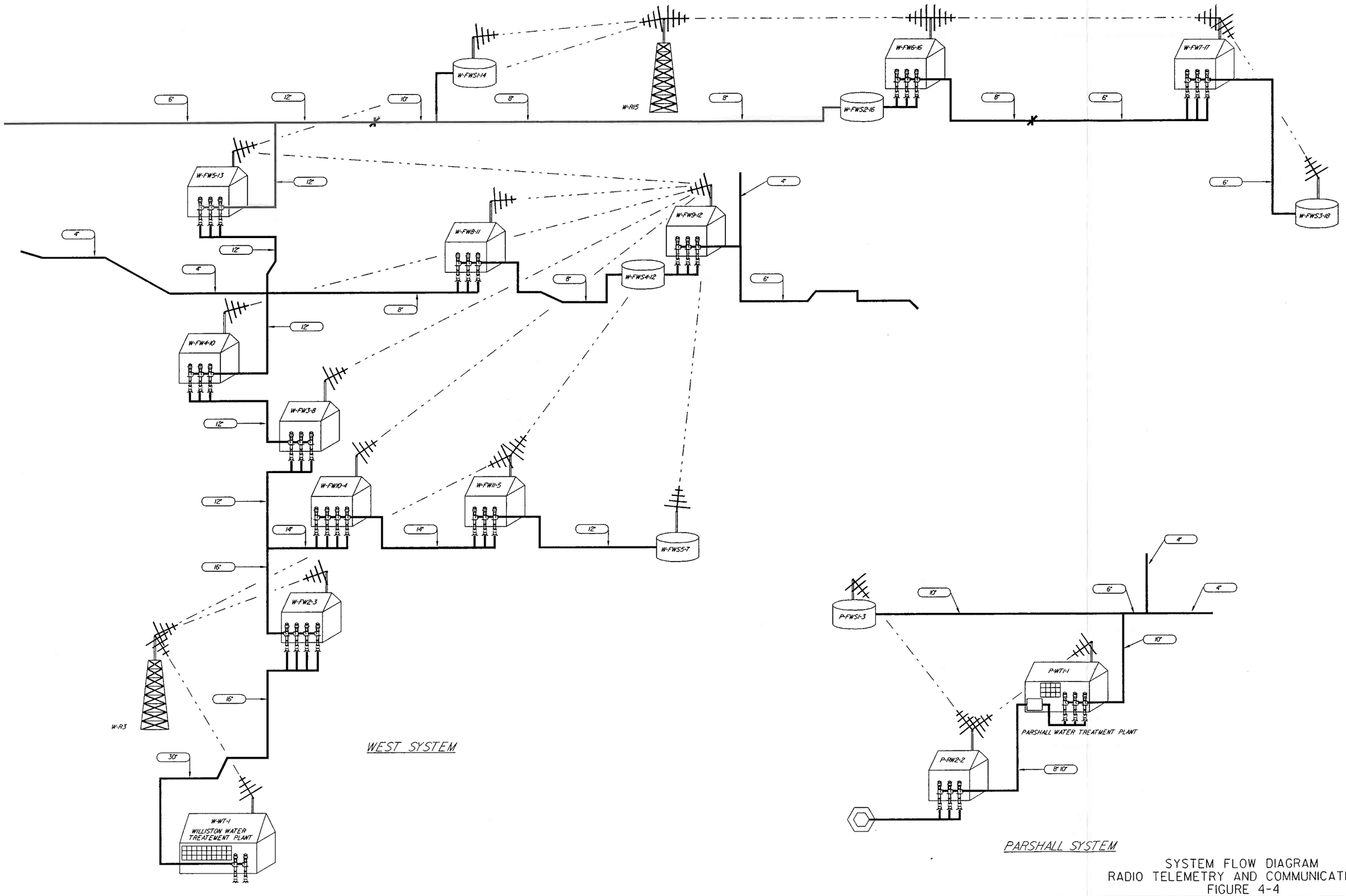
Field strength calculations were made to determine the field strength at the radio receiver, which is critical with regard to how the radio operates, and whether the system will operate without degradation. Field strength calculations take into consideration the gains and the losses in the system as well as any losses caused by intrusion to Fresnel zones. The Fresnel zone is an oblong shaped zone along the line of site of a radio transmission signal that if intrusions into the zone occur, such as land forms, it will affect the signal strength. In general, a 40 dBu field strength would be a good strong signal. Systems can operate as low as 20 dBu; however, any field strength at that level or below should be field tested. The Bullington method, which has long been the standard method of choice and is based upon basic studies applicable to different areas and climates, was used for this study.

Radio Frequencies. For the purposes of this study, the 900 MHz frequency bands were used because they represent the highest degree of attenuation, are the most susceptible to



SYSTEM FLOW DIAGRAM
RADIO TELEMETRY AND COMMUNICATIONS
FIGURE 4-3

/usr2/proj/naws/fig43.dgn 23-JUN-1995 09:00



SYSTEM FLOW DIAGRAM
RADIO TELEMETRY AND COMMUNICATIONS
FIGURE 4-4

/usr2/proj/news/fig44.dgn 23-JUN-1995 09:01

Fresnel zone infringement, and give the most conservative results, which was felt to be a design criteria at this point in the project. The 900 MHz frequencies also offer more protection by the FCC from co-channel users and interference, than do the lower frequencies. Research has indicated that there are still hundreds of channels available in the 450 MHz bands, and in reality as a result of the survey and final detailed design the 450 MHz bands may be selected. At this point in time, and with the information available, it is safer to use the 900 MHz bands, because if the system can function at 900 MHz then it should function even better at 450 MHz.

In general, the assumption has been made for this study that if the path profile shows a line-of-site, then the radio is probably workable. The Bullington method was run assuming a worst case, 900 MHz frequency band, and resulted in strong field paths at 50 miles. Attenuation due to Fresnel zone intrusions can possibly be solved by adjusting tower heights and increasing gains in the antenna system.

SCADA Design. The SCADA system would function utilizing a polling algorithm. The MTU at SCADA Central would sequentially poll, by address, each RTU. While being polled the RTU would then download to SCADA Central the current values of all analog signals, the status of all discrete signals, and update the Input/Output (I/O) table in order to transmit commands from SCADA Central to the RTU. After the polling period was concluded for one RTU, the next RTU in the polling sequence would be polled. An RTU may be polled more than once per complete cycle, by loading the polling table with a particular RTU address more often. In this way, more critical systems are polled more often than less critical systems. For example, a polling sequence for four RTUs may be organized as follows for one polling cycle:

1st	RTU-1
2nd	RTU-2
3rd	RTU-1
4th	RTU-3
5th	RTU-1
6th	RTU-2
7th	RTU-1
8th	RTU-4

In this scheme, RTU-1 is polled four times per cycle of eight or 50 percent of the time; RTU-2 is polled two times per cycle of eight or 25 percent of the time; and RTU-3 and RTU-4 are each polled once per cycle of eight or each 12.5 percent of the time. This polling table will be available to the operator at each MTU so that priorities can be changed as system conditions dictate.

Between each individual poll the SCADA software at the MTU will go into a quiescent mode in which the MTU will look into the system to see if any RTU is reporting a fault. The RTU to MTU communication in this scenario is a report by exception. If any RTU has a fail or alarm condition, it will be allowed to transmit its data between individual polling events, so that an alarm is noted immediately at SCADA Central and the system

does not have to wait until the RTU which is alarmed to be polled in order to log the alarm.

Each pumping station will transmit and/or receive the following conditions during each polling period:

1. Analog values.

    a. Pump station discharge flow rate.
    b. Pump station discharge pressure.
    c. Pump station suction pressure.

2. Discrete values.

    a. Individual pump status running.
    b. Individual pump status stopped.
    c. Individual pump status fail.
    d. Pump station building high temperature.
    e. Pump station building low temperature.
    f. Pump station building intrusion alarm.
    g. Power failure.

3. Commands from SCADA Central to RTU.

    a. Station discharge pressure setpoint.
    b. Local Control System Override.
    c. Command each individual pump on.
    d. Command each individual pump off.

Each reservoir will transmit and/or receive the following information:

1. Analog values.

    a. Reservoir level.

2. Discrete values.

    a. Reservoir high level.
    b. Reservoir low level.
    c. Power failure.
    d. Intrusion alarm.

The SCADA Central system computer will function in the following modes:

1. Change system parameters and setpoints at the RTUs based upon operator entered setpoints that will be based upon system demands and production requirements.

2. Assume manual control of a pump station and command individual pumps to turn on and off.

3. Receive and re-transmit process variable values between RTUs, for instance, reservoir level will be transmitted from the reservoir RTU to SCADA Central's MTU which will store this information and also re-transmit this information to the pump station downstream which is trying to maintain the water level in the reservoir at a setpoint.

4. Log all analog values, trend values, store and archive values.

5. Log alarms, log identification of operator who acknowledges alarms along with time and date.

6. Print alarms on the alarm printer along with operator ID, time, and date.

7. Corrective actions, transfers from manual to automatic and automatic to manual shall be logged on the alarm printer along with the operator ID, time, and date.

8. Log and accumulate run times, failures, number of pump starts per day, etc. This information would be used to generate a report concerning elapsed run time, number of starts, etc. that can be used for maintenance and troubleshooting purposes. Thresholds may be entered into the software to trigger a preventative maintenance work order based upon accumulated run time and/or number of starts, etc.

SCADA Computer Systems. The SCADA Central computer disk memory shall hold all accumulated data for one month as a 30-day continuous data base. After 30 days resident time on the disk, the aged files shall be purged. In this way, the last 30 days of operation are always readily available for reviewing historical trends with search capabilities. Each day the SCADA Central computer will down load that day's collected database to a magneto optical disk data base. The magneto optical disks have large storage capacity and a data shelf life of 30 years. The magneto optical disks will act as the system archiving mechanism.

The 21-inch color graphic CRT for the distribution system will display a system map that locates all RTU sites. The system map shall give a quick overall view of the distribution system. Any pump station or reservoir showing any alarm condition shall flash red. If the alarm is acknowledged and the condition exists,

the pump station or reservoir icon shall be steady red; if conditions are normal, the pump station icon shall show green. The pipeline connectors between pump stations and reservoirs shall show yellow if there is no water flow and shall show blue if there is water flow. All icons shall be outlined in black. Using a mouse, the operator will be able to select any RTU site from the system map screen. The selected RTU site will then graphically display the following information:

1. The status of each pump in the pump station, running, fail, stopped.

2. Pump station operating mode, local, hand, or automatic.

3. The operator shall be able to select between hand or automatic operation by use of the mouse.

4. In hand operation the operator shall be able to manually start and/or stop any pump in the station.

5. Discharge pressure, discharge flow rate, totalized flow, and suction pressure shall be displayed for the pump station.

6. Elapsed run time for each pump shall be displayed.

The color graphic CRT screen shall be able to be divided into quadrants so that up to four different screens can be displayed at the same time, by operator selection using a mouse. Additionally, the operator can select any one screen to occupy the entire screen.

The color graphic CRT screen will also display on-line real time as well as historical trends for all analog values. Up to six variables shall be able to be trended on any one trend screen. The real time value of the variable shall be displayed on the trend screen.

A color ink jet type printer shall allow any screen and/or trend to be printed upon operator demand. Printed colors shall match those displayed on the color graphic CRT screen.

East SCADA Telemetry System

Research into available towers in the area for use as a repeater for the East SCADA System resulted in locating the Souris River Telephone Tower. The tower is 480 feet tall and can be used as a repeater site for the East system. There is space on the tower for an antenna for the SCADA system at an elevation of approximately 400 above grade line. The cost of rental for the tower is $0.25 per foot, or $100.00 per month. This repeater site would be preferable, in that from this one site essentially all of the proposed SCADA nodes can be covered. The following summarizes the line of site path study for the East System SCADA sites shown in Figure 4-3.

RTU NO.	TO	TOWER HEIGHT IN FEET AT RTU	PATH RATING *
E-WT-1	SOURIS	20	A
E-RWS1-2	SOURIS	20	A
E-RW4-3	SOURIS	40	A
E-RW1-4	SOURIS	40	A
E-FW14-5	SOURIS	60	A
E-FW13-6	SOURIS	60	B
E-FWS11-7	SOURIS	20	A
E-FW14-8	SOURIS	20	A
E-FW8-9	SOURIS	20	A
E-FW9-10	SOURIS	60	B
E-FWS7-11	SOURIS	60	B
E-FW10-12	SOURIS	200	B
E-FW11-13	SOURIS	120	B
E-FW4-14	SOURIS	40	B
E-FW3-15	SOURIS	120	B
E-FW4-16	SOURIS	100	B
E-FW5-17	SOURIS	100	B
E-FWS3-18	SOURIS	60	B
E-FW6-19	SOURIS	60	B
E-FW7-20	SOURIS	80	A
E-FWS5-21	E-FW7-20	20	B
E-FWS4-22	E-FW7-20	20	B
E-FWS14-23	SOURIS	80	B
E-FW15-24	SOURIS	40	A
E-FWS13-25	SOURIS	40	A

* Path rating code: "A"- Excellent path, "B"-Good path needs to be tested, "C"- Marginal path needs to be tested; "D"- No path.

The digital elevation model study predicts that there is no reliable communications path between the Souris River Tower and RTU sites E-FWS5-21 and E-FWS4-22; therefore, E-FWS5-21 and E-FWS4-22 have been configured as sites that will communicate directly to E-FW7-20 which will act as a store and forward site. Store and forward communications use the RTUs as mini-repeaters to link remote sites to the central computer system. E-FW7-20 does have direct communications with SCADA Central, located at E-WT-1. Also, since the levels in E-FWS5-21 and E-FWS4-22 are controlled by E-FW7-20, using E-FW7-20 as a store and forward site will make the level data at the reservoirs available to E-FW7-20's PLC, for control on a closer to real time basis which should enhance this system's response time.

The study resulted in 7-20 foot poles, 5-40 foot poles, 6-60 foot poles, 4-80 foot poles, 4-100 foot towers, 4-120 foot towers, and 1-200 foot tower.

Parshall SCADA Telemetry System

The following summarizes the line of site path study for the Parshall system SCADA sites also shown in Figure 4-4.

RTU NO.	TO	TOWER HEIGHT IN FEET AT RTU	PATH RATING *
P-WT-1	P-RW4-2	80	A
P-FWS1-3	P-RW4-2	20	B
P-RW4-2	---	100	

* Path rating code: "A"- Excellent path, "B"-Good path needs to be tested, "C"-Marginal path needs to be tested; "D"- No path.

Based upon the results of the digital elevation model, P-RW4-2 would be used as a store and forward site for P-FWS1-3. Due to the limited communications needed to and from P-FWS1-3, the store and forward communications scenario should not prove to be too cumbersome.

The study resulted in 1-20 foot pole, 1-80 foot pole, and 1-100 foot tower.

West SCADA Telemetry System

Research into available towers in the area resulted in the following findings:

The KOTANA Communications Tower is available for the West System; however, this tower did not appear to have the coverage desired.

There is a company which is in the process of placing a tower near Crosby, that appears to have sufficient coverage to make the West telemetry system much more feasible in terms of cost, equipment, coverage, propagation time, etc. Further information concerning this tower is not currently available. Because no additional documented information was available, the West telemetry system has been configured based upon current data.

There is a distinct possibility that the best solution for the West telemetry system radio communications would be to erect a new tower at an optimum site and height to cover the entire system.

While the digital elevation model study resulted in a feasible system, it does have some undesirable propagation delays due to the store and forward communications paths. This configuration requires more in the way of repeaters and towers; however, for the purposes of this pre-final design report, the system appears to be feasible. The following summarizes the line of site path study for the identified West telemetry system SCADA sites shown in Figure 4-4.

RTU NO.	TO	TOWER HEIGHT IN FEET AT RTU	PATH RATING *
W-WT-1	WR3	60	A
W-***-2	DELETED	***	**
W-FW4-3	WR3	200	B
WR3	W-FW11-5	80	A
W-FW10-4	W-FW9-12	200	A
W-FW11-5	W-FW9-12	80	B
W-***-6	DELETED	***	**
W-FWS5-7	W-FW9-12	60	B
W-FW3-8	W-FW9-12	130	B
W-***-9	DELETED	***	**
W-FW4-10	W-FW9-12	200	B
W-FW8-11	W-FW9-12	80	B
W-FW9-12	W-FW11-5	200	B
W-FW5-13	WR15	80	B
W-FW5-13	W-FW9-12	80	A
W-FWS1-14	WR15	60	A
WR15	---	130	B
W-FW6-16	WR15	120	B
W-FW7-17	W-FW6-16	120	B
W-FWS3-18	W-FW7-17	20	A

* Path rating code: "A"- Excellent path, "B"-Good path needs to be tested, "C"- Marginal path needs to be tested; "D"- No path.

Based upon the results of the digital elevation model study, and as is shown on Figure 4-4, two repeater stations appear to be needed. These repeater stations are designated as WR3 and WR15. To improve system response time, the following RTU nodes may be best if configured as repeaters:

W-FW6-16
W-FW5-13
W-FW9-12
W-FW11-5

The store and forward transmission time can average up to 2 minutes per RTU, which if a signal were to be transmitted from W-FWS3-18 to W-WT-1, the store and forward path would involve the following RTUs:

W-FWS3-18 <--> W-FW7-17 <--> W-FW6-16 <--> WR15 <--> W-FW5-13 <--> W-FW9-12 <--> W-FW11-5 <--> WR3 <--> W-WT-1.

An estimated propagation delay for this communication path could be 16 minutes, which would definitely slow down the SCADA system. Incorporating repeaters as referred to above will definitely improve the system communication time by up to 8 minutes.

Communication between W-FWS3-18 and W-FW7-17 would be configured as a store and forward system. Also, since the level in reservoir FWS3-18 is controlled by W-FW7-17, using W-FW7-17 as a store and forward site will make the level data at the reservoir available to W-FW7-17's PLC for control on a closer to real time basis, which should enhance the system response time.

The study resulted in 1-20 foot pole, 3-60 foot poles, 5-80 foot poles, 4-120 foot towers, 4-130 foot towers, and 4-200 foot towers. Up to 60 feet in height, a wooden pole may typically be used to support an antenna; at 80 feet, a wooden pole may be used depending upon conditions, above 80 feet, a steel tower is typically used.

Control Room

The control room at each water treatment plant shall have dual purposes. The control room shall act as SCADA Central for the water distribution system as well as a SCADA system for monitoring the water treatment plant operation. The MTU for the distribution system will be located in this room. The room will also house a graphic panel on the wall that will represent the distribution system. The control console will be configured as a wrap around desk, with a graphic panel incorporated into the slanted surface in front of the operator. This console would be located in front of the graphic board representing the distribution system, so that while sitting at the console the operator will be able to have a simultaneous quick overview of the plant operation and status via the console, and that of the distribution system on the wall in front of him. At each end of the console would be located a 21-inch color CRT monitor, one which would act as the window into the distribution SCADA system and the other which would act as the window into the plant SCADA system. Either monitor could be assigned to either system through the plant computer. This type of configuration allows redundancy in that one monitor could perform two purposes in the event of a monitor failure.

The basic equipment that would be located in the control room is listed below:

1. Distribution system MTU.
2. Distribution system mosaic graphic panel located on the wall.
3. Distribution system color graphic monitor.
4. Distribution system color ink jet printer.
5. Distribution system dot matrix alarm printer.
6. Distribution system computer with archiving capabilities.
7. Plant operations programmable logic controller (PLC) or remote PLC rack.
8. Plant operations mosaic graphic panel incorporated into console.
9. Plant operations color graphic monitor.
10. Plant operations color ink jet printer.

11. Plant operations dot matrix alarm printer.
12. Plant operations computer with archiving capabilities.
13. Control room console.
14. Control room Engineer's work station.
15. Control room laser printer for report generation.

The mosaic graphic panels, which consists of approximately 1" square plastic tiles that snap into a metal grid, would be selected because they allow for modifications as the system changes and develops. The use of the laminated graphic boards essentially prohibits any modifications after the board has once been fabricated; however, the mosaic tile approach permits tiles to be removed and replaced with relative ease. The distribution system graphic panel, that would be located on the wall in front of the operations console, would display a system map of the distribution system. Each pump station would have pilot light indicators, which are located in the cell behind the tile so that the mosaic face remains clean and smooth. The pump station graphic would represent the number of pumps at the pump station and each pump would have a red and green light associated with it. The green light to indicate a running status and the red light to represent a fault condition. Adjacent to each pump station graphic would be located digital displays that would echo the discharge pressure and flow rate from the pump station. Reservoirs would also be depicted on this graphic panel. The reservoir graphics would have high level and low level alarm pilot lights. Adjacent to the reservoir graphic would be a numeric display indicating reservoir level. There would be no chart recorders or other types of recording devices on this graphic panel, because all data logging, trending, and archiving would be resident in the distribution system computer.

The mosaic graphic panel that is incorporated into the console would display the status of various in-plant functions. This panel would also have some selector switches and pushbuttons that would allow the operators to access and control devices within the plant. A plant programmable logic controller (PLC) would reside in this console, along with man machine interfaces (MMIs), that are directly connected into the PLC for control purposes. The intent of the operator pilot devices and MMIs is to place the plant control system functions directly into the PLC, and the plant operations computer would act as a SCADA window into the PLC. In this way, the plant SCADA system can be collecting data, trending information, and archiving process variables while the plant PLC is controlling the system.

Dot matrix printers would be used for alarm logging, because the alarm log would use continuous form computer paper, so that the alarm conditions and logs are automatically and sequentially logged on a continuous strip of paper.

The Engineer's work station allows a Engineer to make on line modifications to the SCADA systems and plant PLC controls without interfering with operations. Additionally, the terminal serves another purpose, that being a personal computer that can be used for data analysis, word processing, and reporting. Given the special functions of this computer, a laser printer would be attached to it for report generation, etc.

The overall system configuration is essentially redundant, in that there are two SCADA terminals. This configuration allows for continuous operation, in a down graded mode, if one of the computers were to go down. In this event, the plant computer could be used to monitor the distribution system while the distribution computer was being repaired.

The entire control room would be supplied conditioned power through an uninterruptible power supply (UPS). This UPS would be sized to accommodate the entire control room power load, so that in the event of a power failure at the plant, and before the in-plant generation were to come on line, the UPS would supply continuous power, so that the control and SCADA system were always on line.

The above described control rooms would be typical of the West and East Systems. The Parshall System would incorporate the distribution system into the plant SCADA system because there are only two remote sites, and the cost of a separate distribution system SCADA system could not be justified.

Pumping Station SCADA Equipment

Each pumping station will have an RTU which will communicate with SCADA Central via an antenna and radio link. Some pump stations, by virtue of their geographical location, would also act as a repeater and/or store and forward site for other pump stations. Due to geography all SCADA sites, most likely, will not be able to communicate directly with SCADA Central; this is especially true of the Western System. Communications for this system would relay information from one pump station to the next, which would then re-transmit this information to SCADA Central. This store and forward scenario adds delay times into the SCADA system and consequently could affect system response. The detailed design phase of the project will have to address the final communications configuration.

Each pump station will contain a PLC that will be used for local control. The SCADA System will send information from SCADA Central to the PLC and information from the pump station will be routed through the PLC to the RTU and uploaded to SCADA Central. By using the PLC to control all functions within the pump station, all logic is resident in the PLC. If there were a communications problem or SCADA system problem, the pump station could revert to a default mode of operation, or simply maintain status and continue to operate without the SCADA system input. With this configuration, a SCADA system failure will not disable the distribution system.

Each pump station typically will be equipped with the following devices:

1. Pump Station Discharge Flow Meter, with both a local readout and 4-20 mA output used for control and data acquisition.

2. Pump Station Discharge Pressure transmitter with a 4-20 mA output used for control and data acquisition.

3. Pump Station Suction Pressure transmitter with a 4-20 mA output used for control and data acquisition.

4. Pump Station High Discharge Pressure switch, to be used as a safety control element, SCADA reporting, and backup to the discharge pressure transmitter.

5. Pump Station Low Suction Pressure switch, to be used as a safety control element, SCADA reporting, and backup to the suction pressure transmitter.

6. Each pump will have a pump control valve with fully opened and closed limit switches, to indicate valve position.

7. Pump station MMI to be used for local control setpoints, local run time indications, local fault conditions, pump sequencing, local flow rate and totalization information. This device will act as the operator's main interface into the pump station controls for local control as well a parameter setpoints.

The PLC and SCADA systems will have their power conditioned using a small on-site UPS. This device will also be used to power the equipment during a power outage, as well as filter noise and transients out of the power supply.

The pump station will have a manual ability that will allow the operators to start and stop pumps, open and close valves in a manual manner. This mode of operation is totally manual and will require that the operator start and stop pumps and open and close valves in the proper sequence. The operators will also have a manual control mode available through the PLC which will automatically control the valves as the motors are started and stopped.

An intrusion and a fire detection system will be included in each pump station that will report status of these conditions to SCADA Central.

Water Storage Reservoir SCADA Equipment

Each reservoir will have a level transmitter that will produce a 4-20mA signal that is proportional to reservoir level. This level information will be sent to the local RTU at the reservoir, which in turn will transmit this value to SCADA Central, which will display, trend and archive the data. SCADA Central will also retransmit this information to the appropriate pump station, etc. for control purposes.

As a safety back up to the level transmitter, separate discrete high and low level signals will be transmitted to SCADA Central via the local reservoir RTU. These signals will be used as alarms at SCADA Central as well as be retransmitted to the pump stations for control purposes.

In areas which contain both a reservoir and pump station, all signals would be transmitted through a single RTU.

Where utility power supply may be unavailable or too costly to install at the reservoir, the power supply for the reservoir may be generated using a photo-voltaic system. The photo-voltaic solar cells will be used to generate power and charge a battery. The battery will be used for evening and low light ride though. On such systems, the RTU and associated equipment will be supplied with direct current.

RESERVOIRS

Introduction

Storage reservoirs will be provided in the water supply and transmission systems sized to provide a storage volume equal to the maximum day demand for each community. Generally, storage reservoirs will be located at or near each distribution supply point. However, in certain areas of the distribution system serving several small communities, storage will be provided at a common reservoir. The location of the storage reservoirs was coordinated with the hydraulic design of the transmission system piping and pumping stations. Where practical, reservoirs were located in common with booster pump stations.

Types of Storage Reservoirs

Two basic storage configurations were used in the transmission system:

1. In-line storage facilities where the entire pipeline flow would be pumped into a storage reservoir and the hydraulic grade line would be allowed to go atmospheric. The water could then flow by gravity or be pumped or boosted up to design pressure in the continuing transmission system.

2. Off-line storage facilities where the reservoirs would be fed a portion of the flow from the main distribution system. These facilities would be terminal distribution points at communities or consolidated storage serving several communities.

At terminal storage reservoirs, no additional water supply facilities would be provided beyond the storage reservoir to hook up to local distribution systems. At consolidated storage sites, the off-line storage reservoir would be operated to maintain the maximum day demand of the communities being served. Should the main supply be interrupted, the stored volume would be used to feed the distribution system downstream of the storage facility.

Storage Reservoir Construction

Two types of storage reservoir construction are proposed to be used in the NAWS transmission system including:

- Concrete storage reservoirs
- Steel tanks

It is recommended that all storage reservoirs 1.0 MG and larger be of concrete construction. The reservoirs in the system from 0.25 to 0.5 MG were assumed to be insulated steel tanks, however, concrete tanks can be used if required.

Concrete Storage Reservoirs. Concrete storage reservoirs are the preferred option to address climatic concerns with regards to ice formation inside reservoirs during cold weather. Concrete reservoirs are proposed for use with all in-line storage applications where pumping facilities (vertical turbines) are located in common with the reservoir. Concrete storage reservoirs may be buried, partially buried, or ground level structures. They may be constructed either with conventional techniques or may be post tensioned. In general, post tensioned construction is more cost effective than conventional construction techniques at reservoir volumes greater than 2 million gallons (MG). A typical plan and sections of a conventional reinforced concrete storage reservoir is shown in Figure 4-5.

Concrete construction techniques will follow specific design requirements for hydraulic structures including water stop/construction joint spacing, and alternate, non-adjoining pour requirements.

Steel Tanks. Steel storage reservoirs on concrete foundations are usually ground level tanks or can be partially buried using concrete stub walls. Steel tank configurations are usually cost effective for storage volumes of 1.0 MG or less, or where other requirements do not dictate that concrete tanks are needed. Steel tanks are typically not the preferred option because of higher maintenance costs which may result from interior and exterior corrosion and from abrasion of ice on interior coatings.

If steel tanks are selected for use in certain locations, concrete foundations shall be constructed as per concrete requirements set forth for concrete reservoirs. The steel tank, usually fabricated as a circular tank, shall be designed for hydraulic service with interior and exterior coatings, an exterior overflow and insulation. A typical plan and section of a steel storage reservoir on a concrete foundation is shown in Figure 4-6.

Elevated steel storage tanks may be applicable for small storage requirements in off-line applications such as a terminal facility, but it is recommended these applications be limited in the NAWS system. Disadvantages of this type of tank are related to cold weather considerations, and the requirements for insulation to prevent freezing. Tank elevation will need to be coordinated with the distribution head required and the delivery

pressure of the NAWS distribution system feeding the tank; the use of pressure reducing valves (PRV) may be required on the feed line to the tank.

General Design Criteria

General design criteria applicable to all types of storage systems include:

- Emergency Overflows. All storage reservoirs shall be equipped with emergency overflows to prevent pressurization of the structure should the maximum water surface level be exceeded. Discharge of overflows will need to be addressed on a case by case basis during design, and may require a National Pollutant Discharge Elimination System (NPDES) permit application if the discharge is to a receiving water.

- Cold Weather Application. Storage reservoir design will take into account cold weather application for all above ground systems. This is especially critical for off-line storage facilities where water does not pass through the facility, but rather, is held in reserve. Reservoir insulation and heat tracing on exposed piping will be required.

- Valving. All reservoirs shall include isolation valves on the inlet and outlet piping. Exterior valves shall be located in vaults for ease of access outside the reservoir.

- Water Level Monitoring. Storage reservoirs shall be equipped with appropriate instrumentation to monitor water surface operation levels in the reservoir.

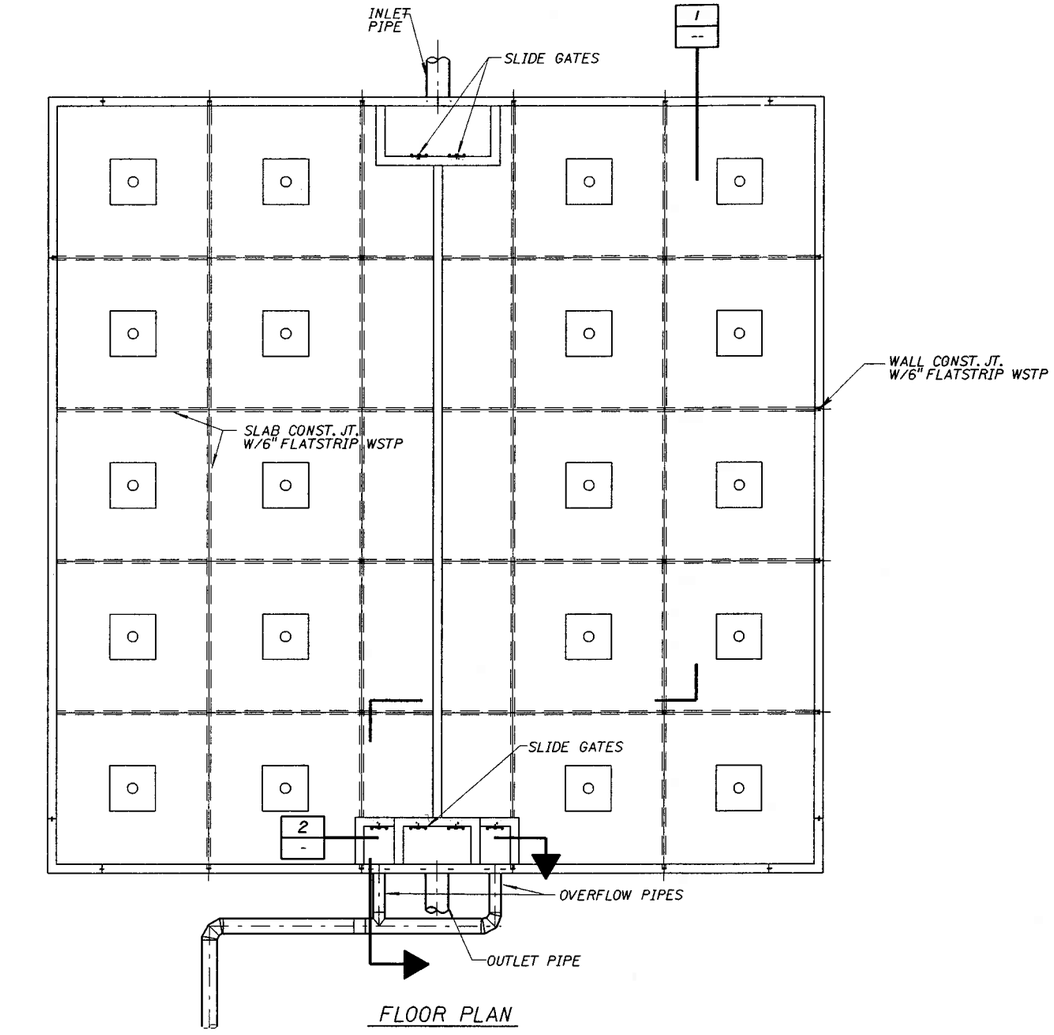
- Hydraulic Testing. All storage reservoirs shall be hydraulically tested prior to being placed in service.

- Ten States Standards. Requirements of the Ten States Standards shall be incorporated into the design of storage reservoirs, including, but not limited to groundwater elevation, dewatering requirements, utility separation distances, etc.

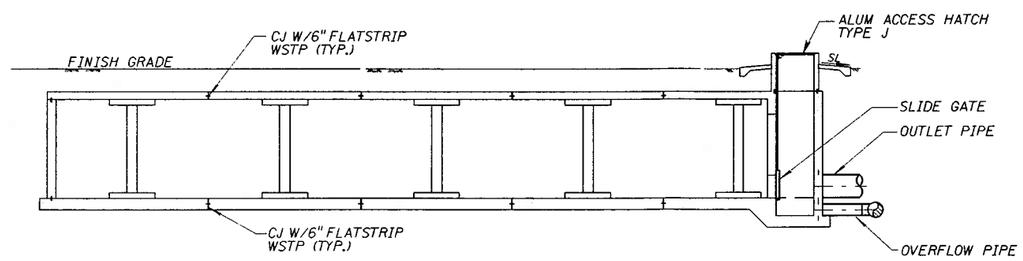
- Dewatering.. All storage reservoirs shall be designed to provide the means to achieve the complete dewatering of the storage volume.

NAWS STORAGE REQUIREMENTS

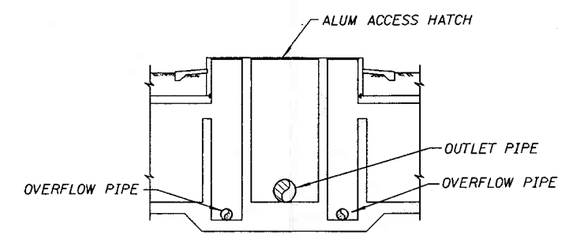
Storage is required on all three of the regional water transmission systems: the East System, the West System, and the Parshall System. Finished water storage volume requirements will be equal to the maximum day demand for all users on each system. On the East System, a 3 MG reservoir will be needed on the raw water supply line from Lake Audubon to Minot to control static head requirements in the downstream piping system



FLOOR PLAN



TYPICAL SECTION 1



OVERFLOW SECTION 2

TYPICAL CONCRETE RESERVOIR
CONVENTIONAL CONSTRUCTION
PLAN AND SECTIONS
FIGURE 4-5

1" = 10'-0"

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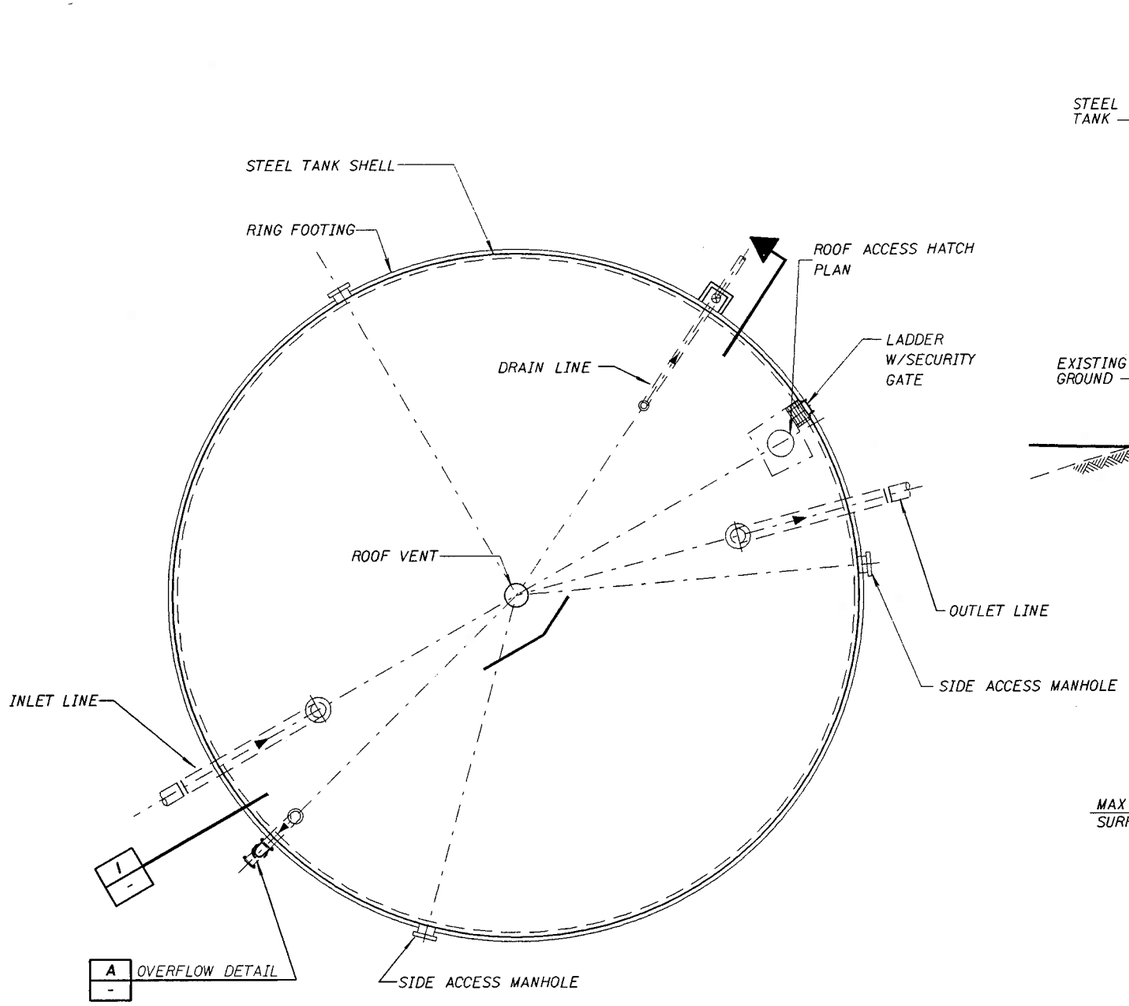
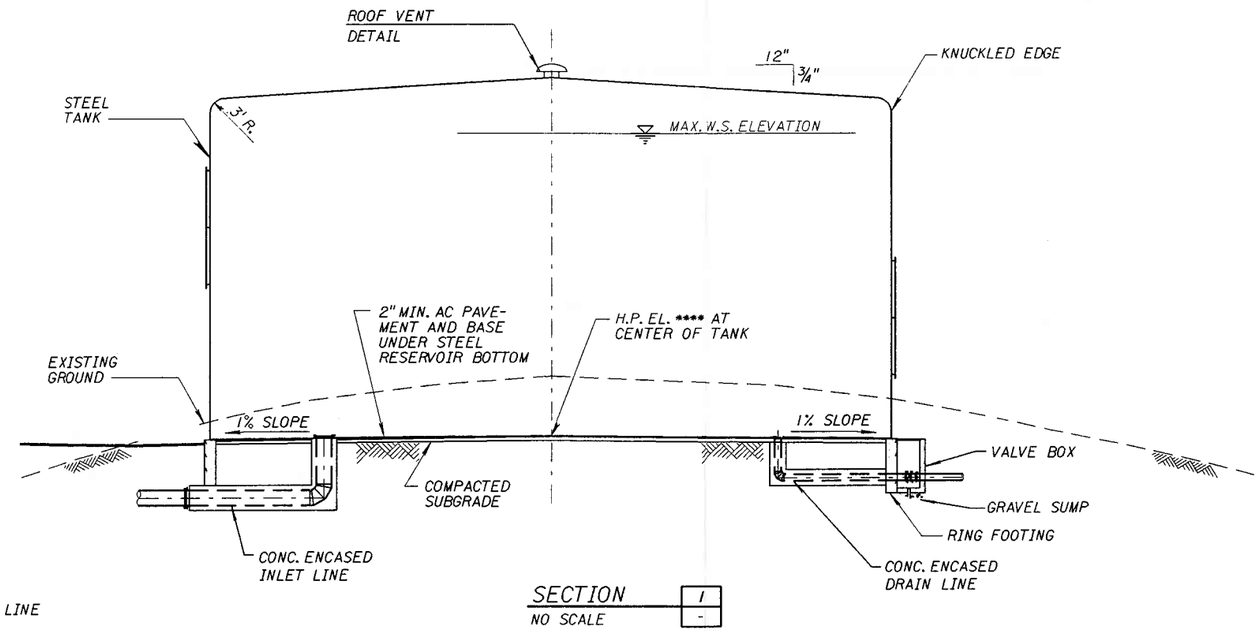


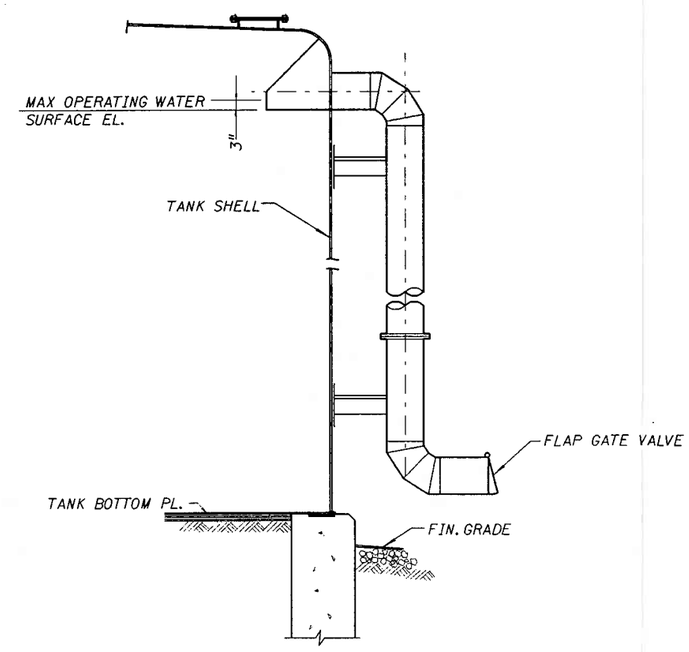
STEEL RESERVOIR PLAN

NO SCALE



SECTION 1

NO SCALE



OVERFLOW DETAIL A

NO SCALE

TYPICAL STEEL TANK
PLAN, SECTION AND DETAILS
FIGURE 4-6

1" = 1'-0"

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from the divide to Minot, and as operational storage to assist in balancing the delivery of water to the Minot WTP.

A summary of the locations and required storage volumes for the various reservoirs in the three transmission systems is shown in Table 4-2.

TABLE 4-2

STORAGE RESERVOIRS

System	Reservoir Location	Storage Volume (MG)	Configuration	Reservoir Construction/ Type
East	RWS1-U.S. Hwy 83 south of Minot	3.0 (raw water)	In-line	Concrete
	FWS1-Minot AFB	6.5	In-line	Concrete w/P.S.
	FWS4-3 miles south of the intersection of U.S. Hwy 83 and State Hwy 5	1.0	In-line	Concrete w/P.S.
	FWS3-Bottineau	1.5	Off-line	Concrete
	FWS4-Kenmare	0.5	In-line	Steel-terminal
	FWS5-Upper Souris/ Renville Co.	0.5	In-line	Steel-terminal
	FWS6-Deering turnoff	0.4	In-line	Concrete w/P.S.
	FWS7-Towner	0.5	Off-line	Steel
	FWS8-Rugby	2.0	In-line	Concrete-terminal
	FWS9-Willow City/ Upham	0.25	In-line	Steel w/P.S.
	FWS10-Karlsruhe turnoff	0.5	In-line	Concrete w/P.S.
	FWS11-Ruso	0.5	In-line	Steel-terminal
	FWS14-Burlington	0.5	Off-line	Steel
	FWS13-Berthold	0.5	Off-line	Steel
Parshall	FWS1-New Town	1.0	Off-line	Concrete-terminal
West	FWS1-Crosby	0.5	Off-line	Steel
	FWS4-Columbus	0.5	In-line	Concrete w/P.S.
	FWS3-Bowbells	0.25	In-line	Steel-terminal
	FWS4-Wildrose	0.5	In-line	Concrete w/P.S.
	FWS5-Ray	2.0	In-line	Concrete-terminal

COST ESTIMATES

Cost estimates for the pipelines, hydraulic controls, pump stations, SCADA/telemetry systems and reservoirs were prepared using information from similar facility designs, equipment and material suppliers, and recent bids from projects of similar complexity. Construction cost estimates for these facilities are broken down for the East, Parshall, and West systems. Pipeline construction costs are broken down to represent large diameter pipelines (14 to 44-inch diameter), and small diameter pipelines (4 to 14-inch diameter). Pipeline construction costs estimates are presented in Tables 4-3 through 4-5.

The estimated construction cost for hydraulic controls for the East and West systems are presented in Table 4-6. The hydraulic controls include surge arrestors and suction bypass facilities to control hydraulic transients, and pressure reducing valve stations to control the hydraulic grade line.

Pumping station construction cost estimates were developed for the booster pump stations on the transmission systems. These stations included the pump stations associated with storage reservoirs, in-line boosters, and the raw water booster station near Max. The pump station associated with the Lake Audubon raw water intake is discussed in Section 3. The modifications to the Parshall raw water intake are discussed in Section 4. The pump station construction costs are presented in Table 4-7. The SCADA telemetry system construction costs are presented in Table 4-8. Reservoir construction costs are presented in Table 4-9. Reservoir costs were developed for buried concrete reservoirs and insulated steel tanks.

TABLE 4-3

EAST SYSTEM PIPELINE
COST ESTIMATE

Description	Length, ft	Construction Cost
Large Diameter (14 to 42 inches)		
42 inch	64,000	$ 7,420,000
38 inch	83,000	8,400,000
34 inch	79,000	8,120,000
24 inch	16,700	1,050,000
22 inch	84,600	4,650,000
18 inch	23,630	1,205,000
16 inch	114,180	4,681,000
14 inch	322,890	11,465,000
		$46,991,000
Small Diameter (4 to 12 inches)		
12 inch	393,730	$7,087,000
10 inch	342,820	4,629,000
8 inch	95,460	955,000
6 inch	401,320	2,408,000
4 inch	384,800	1,539,000
		$16,618,000
TOTAL CONSTRUCTION COST		**$63,609,000**

TABLE 4-4

PARSHALL SYSTEM PIPELINE COST ESTIMATE

Description	Length, ft	Construction Cost
Small Diameter (4 to 12 inches)		
10 inch	155,910	$2,105,000
8 inch	41,750	418,000
6 inch	42,240	253,000
4 inch	16,500	66,000
TOTAL CONSTRUCTION COST		**$2,842,000**

TABLE 4-5

WEST SYSTEM PIPELINE COST ESTIMATE

Description	Length, ft	Construction Cost
Large Diameter (14 to 16 inches)		
16 inch	72,540	$ 2,974,000
14 inch	41,500	1,473,000
		$ 4,447,000
Small Diameter (4 to 12 inches)		
12 inch	293,460	$5,282,000
10 inch	78,380	1,058,000
8 inch	171,600	1,716,000
6 inch	403,020	2,418,000
4 inch	99,600	398,000
		$10,872,000
TOTAL CONSTRUCTION COST		**$15,319,000**

TABLE 4-6

HYDRAULIC CONTROL CONSTRUCTION COST

Description	Quantity	Construction Cost ($)
East System		
Surge Arrestor at RW1	1	$70,000
Surge Arrestors at RW2	2	125,000
Surge Arrestor at FW1	1	40,000
10-inch Suction Bypass	1	5,000
8-inch Suction Bypass	9	36,000
4-inch Suction Bypass	1	1,000
PRV Station on 34-inch Pipe	1	200,000
PRV station on 6-inch Pipe	1	19,000
PRV Stations on 4-inch Pipe	2	26,000
Subtotal		$522,000
West System		
Surge Arrestor FW2	1	$35,000
Surge Arrestors FW10	2	45,000
Surge Arrestors FW11	2	55,000
8-inch Suction Bypass	4	16,000
6-inch Suction Bypass	1	2,000
4-inch Suction Bypass	2	2,000
PRV Station on 14-inch Pipe	1	40,000
PRV Station on 10-inch Pipe	1	32,000
PRV Station on 8-inch Pipe	1	25,000
PRV Station on 4-inch Pipe	1	13,000
Subtotal		$265,000
TOTAL CONSTRUCTION COST		**$787,.000**

TABLE 4-7

BOOSTER PUMP STATION CONSTRUCTION COST

Description	Horsepower	Construction Cost ($)
East System		
RW2	2000	$1,000,000
FW2	300	400,000
FW3	200	325,000
FW4	75	170,000
FW5	150	250,000
FW6	60	150,000
FW7	60	150,000
FW8	200	325,000
FW9	200	325,000
FW10	125	225,000
FW11	5	50,000
FW12	75	170,000
FW13	50	130,000
FW14	30	100,000
FW15	50	130,000
Subtotal		$3,900,000
West System		
FW2	300	$400,000
FW3	125	225,000
FW4	75	170,000
FW5	75	170,000
FW6	20	70,000
FW7	15	70,000
FW8	20	70,000
FW9	7.5	50,000
FW10	150	250,000
FW11	200	325,000
Subtotal		$1,800,000
TOTAL CONSTRUCTION COSTS		**$5,700,000**

TABLE 4-8

SCADA TELEMETRY SYSTEM CONSTRUCTION COST

Description	Quantity	Construction Cost ($)
East System		
RTUs	25	$875,000
MTU/Central Computer	1	100,000
Antenna Poles (<80 ft high)	20	40,000
Antenna Towers (>80 ft high)	5	20,000
Subtotal		$1,035,000
Parshall System		
RTUs	3	$105,000
MTU/Central Computer	1	50,000
Antenna Poles (<80 ft high)	2	4,000
Antenna Towers (>80 ft high)	1	4,000
Subtotal		$163,000
West System		
RTUs	17	$595,000
MTU/Central Computer	1	100,000
Antenna Poles (<80 ft high)	9	18,000
Antenna Towers (>80 ft high)	8	32,000
Repeater Tower	1	100,000
Subtotal		$845,000
TOTAL CONSTRUCTION COSTS		**$2,043,000**

TABLE 4-9

STORAGE RESERVOIR CONSTRUCTION COST

Description	Storage Volume (MG)	Construction Cost ($)
East System		
RWS1	3.0	$ 900,000
FWS1	6.5	1,825,000
FWS2	1.0	400,000
FWS3	1.5	550,000
FWS4	0.5	250,000
FWS5	0.5	250,000
FWS6	0.4	225,000
FWS7	0.5	250,000
FWS8	2.0	700,000
FWS9	0.25	150,000
FWS10	0.5	250,000
FWS11	0.5	250,000
FWS12	0.5	250,000
FWS13	0.5	250,000
Subtotal		$6,500,000
Parshall		
FWS1	1.0	$400,000
West System		
FWS1	0.5	$250,000
FWS2	0.5	250,000
FWS3	0.25	150,000
FWS4	0.5	250,000
FWS5	2.0	700,000
Subtotal		$1,600,000
TOTAL CONSTRUCTION COST		**$8,500,000**

Section 5

HOUSTON ENGINEERING
AMERICAN ENGINEERING
MONTGOMERY WATSON

SECTION 5

LAKE AUDUBON INTAKE

INTRODUCTION

This section describes the development of the Lake Audubon Intake. The raw water supply for the East System of the NAWS project will be drawn from Lake Audubon through an intake/pumping facility. The facility will include: an intake channel, intake structure, fine screens, pumps, and chloramination equipment. The raw water supply will be delivered to the Minot WTP through a raw water pipeline.

The selection of the Lake Audubon source involved an evaluation of a number of alternatives, including an intake in Lake Sakakawea, an integrated facility using components of the U.S. Bureau of Reclamation's Snake Creek Pumping Plant, and a Lake Audubon facility. A number of different subalternatives were developed and evaluated to determine: the feasibility, relative cost, and advantages and disadvantages of the various options. A detailed description of this analysis was prepared in a December 27, 1993 letter to American Engineering, P.C. Only a brief summary description of the alternatives is presented in the following.

SUPPLY ALTERNATIVES

The purpose of evaluating alternative supply sources was to determine if Lake Sakakawea water, which is generally perceived as having somewhat better raw water quality, could be used to supply the project. For comparison purposes, construction cost estimates were developed for each alternative. These costs included intake structures, fine screening facilities, a pump station, and appurtenant equipment.

Lake Sakakawea Intake and Pump Station

Two alternative locations were evaluated for a Lake Sakakawea intake which would be able to draw water from elevation 1790 or lower. Alternative 1 was located on the west embankment of U.S. Highway 83 directly south of the Snake Creek Pumping Plant. Alternative 2 was located on a peninsula approximately two miles west of the Snake Creek Pumping Plant.

Lake Sakakawea intake options would have to accommodate a wide range of water surface fluctuations. The average fluctuation would be about 25 feet ranging from elevation 1850 to 1825. However, during drought conditions, the lake elevation could drop substantially below this average operating pool range. The minimum pool elevation expected is 1775.

Alternative 1. The Lake Sakakawea west embankment location is about 800 feet south of the southern embankment of the Snake Creek Pumping Plant. Access to the site would

be through the existing parking lot of the pumping plant. Substantial embankment work would be required to construct the access road to the facilities site. The depth of the structure to reach elevation 1790 would be approximately 65 feet. It was not feasible to consider drawing from a lower elevation because a natural topography feature in the lake bottom about 2 miles away restricts flow to the embankment below elevation 1790. Therefore, a 1,000 foot intake line was assumed to reach the elevation 1790 pool. The construction cost of a facility in this location was estimated at $8.5 million.

Alternative 2. The intake, located on the peninsula approximately two miles west of the Snake Creek Pumping Plant, would require an intake line of over 4,000 feet to reach an elevation of 1765. This elevation would provide water at the minimum drought pool level in Lake Sakakawea. With the additional length of force main necessary to connect this facility to the pipeline route to Minot, Alternative 2 was projected to cost over $19 million.

While slightly better raw water quality could be obtained from Lake Sakakawea, the costs of providing water from this source would be significantly higher than the Lake Audubon alternatives. Increased pumping costs would also be experienced, especially during drought periods when the lake elevation is low. Because the potential costs of these alternatives far exceed those of the Lake Audubon options, they were eliminated from further consideration.

Integrated Facilities

A number of conceptual options to integrate the NAWS intake into the existing Snake Creek Pumping Plant facilities were developed for discussion with the Bureau of Reclamation. The benefits to the NAWS project would be the use of an existing structure to serve as the intake for the NAWS pump station, and the ability, with structural modifications to part of the existing Snake Creek discharge structure, to draw water from either Lake Audubon or Lake Sakakawea. The Lake Sakakawea water supply availability would, however, be dependent on lake elevation and operation of the Snake Creek Pumping Plant. While construction costs of these alternatives were projected to be more in line with the NAWS Lake Audubon intake alternatives, several issues that could impact the feasibility, cost, or construction schedule of the NAWS project were identified as listed below:

- Agreement by the Bureau of Reclamation to allow use and modifications to the existing Snake Creek Pumping Plant for the NAWS project would be required.

- Site access and available area adjacent to the Snake Creek Pumping Plant facilities may be limited and may not accommodate the NAWS facility.

- The depth of construction to access the existing Snake Creek Pumping Plant facilities could impact the existing site, and increase construction

costs more than assumed in the alternative analysis depending on where the NAWS pumping facility was located.

- Modification of the existing discharge gate structure to isolate the north discharge tube would require a modification in the operation of the Snake Creek facilities.

- Relocation of the existing sewer from the Snake Creek discharge structure would be required.

- Drawing water from Lake Sakakawea at lake elevations above approximately 1835 would result in passing flow through existing pump equipment.

- Operation of the existing Snake Creek facilities could have impacts on the NAWS facilities which would result in additional construction costs or special facilities to handle pressurized flow in the existing 11 foot diameter discharge pipe.

Four conceptual alternatives were developed for integrating the NAWS and the Snake Creek Pumping Plant facilities. A detailed description and conceptual layouts of the alternatives was prepared in a December 27, 1993 letter to American Engineering, P.C. Table 5-1 presents the estimated construction costs for each of the integrated alternatives. These costs reflect canned pump station construction. Construction of wetwells and the addition of fine screening would add approximately $650,000 to those alternatives capable of accepting these facilities. A summary description of the alternatives is presented below.

Alternative 3. This alternative would directly tap the existing north, 11 foot diameter discharge pipe. Vertical turbine pumps would be placed into the pipe. While this would be the simplest integration for the NAWS facilities, it has several disadvantages. First, pumps placed inside a pipe in this configuration would not result in the best hydraulic design and would probably preclude operation of the Snake Creek pump which discharges to this line. Second, minimum water surface elevations in both lakes of 1838 would be required to use either lake source. Finally, fine screening could not be added to this alternative, and the facilities would use a portion of the existing Snake Creek Pumping Plant parking lot.

Alternative 4. Alternative 4 would involve a horizontal tap into the 11 foot diameter discharge line below the spring line of the existing pipe. This alternative could accommodate either a canned pump station construction or wetwell construction with fine screening. This alternative would be located the same as Alternative 3; therefore, the site impacts would be similar. This type of construction would permit use of either lake source at slightly lower lake elevations (approximately 1835).

TABLE 5-1

INTEGRATED PUMPING FACILITIES[1]
COST COMPARISONS

Facilities Component	ALTERNATIVE 3 West Embankment Canned Pump Station, Vertical Pipe Tap	ALTERNATIVE 4 West Embankment Canned Pump Station, Horizontal Pipe Tap	ALTERNATIVE 5 East Embankment Canned Pump Station, Outlet Structure Tap	ALTERNATIVE 6 Snake Creek Pumping Plant Facilities Modification
PUMP STATION				
Earthwork/Demolition	$13,000	$29,000	$77,000	$100,000[a]
Concrete	included	included	included	40,000
Steel Work	---	---	---	15,000
Pump Cans	55,000	100,000	55,000	---
Building	226,000	226,000	226,000	---
Pumps/Electrical	570,000	570,000	570,000	570,000
Pipe, Fittings, Mechanical	124,000	124,000	124,000	175,000
Force Main/Pipe Allocation	18,000	18,000	38,000	126,000
Site Work	included	included	49,000	---
Riprap	---	---	100,000	---
SUBTOTAL	$1,006,000	$1,067,000	$1,239,000	$1,026,000
OTHER WORK				
Flap Gate Modification	$94,000	$94,000	$94,000	$94,000
Sewer Relocation	15,000	15,000	15,000	15,000
SUBTOTAL	$109,000	$109,000	$109,000	$109,000
CONSTRUCTION COST	**$1,115,000**	**$1,176,000**	**$1,348,000**	**$1,135,000**

[a] Demolition inside Snake Creek Pumping Plant.

NOTES:
1. Cost estimates were developed at a master planning level of detail of -15% to +30%.
2. Wetwell construction with mechanical fine screening adds $650,000 to Alternatives 4 and 5, and deletes the pump can costs.
3. Wetwell construction not applicable to Alternative 3.
4. Fine screening cannot be added to the Lake Sakakawea source and would require a major modification to the existing Lake Audubon discharge structure to accommodate screening to this source.

Alternative 5. Alternative 5 would tap the concrete discharge structure on the east side of the U.S. Highway 83 embankment at elevation 1830. Either lake source could be used at water surface elevations above 1830. Either a canned pump station or a wetwell/fine screening configuration could be used. Issues associated with this option include: the necessity to construct a site to build the NAWS facilities, access, and similar modifications to operations of the Snake Creek Pumping Plant as identified in the previous alternatives.

Alternative 6. Alternative 6 represents a major modification to the Snake Creek Pumping Plant, wherein the NAWS pumping facilities would replace one of the three, large existing pumps inside the structure. This alternative would provide supply of Lake Sakakawea water at much lower water surface elevations, but would disable completely one of the existing pumps. This option would also limit hydraulic pumping flexibility because the space available for installing pumps is limited. Because of the range of flow to be provided, additional costs for variable speed drives would be needed. Issues concerning joint operation, coordination, and other administrative costs also detract from the feasibility of this alternative.

Integrated Alternative Summary. Based on the alternative analysis, Alternative 5 appears to be the most feasible option for integrating the Snake Creek and NAWS facilities. This option would allow addition of fine screening equipment and would be the least disruptive to the existing Snake Creek Pumping Plant facilities. The estimated construction costs for the various alternatives do not provide a definitive difference to distinguish one alternative from another. Previous work on the conceptual layout of a stand alone intake/pump station on Lake Audubon indicates that Alternative 5 would be similar to the stand alone facilities except for the integrated use of the existing discharge structure for the NAWS intake structure.

NAWS Intake Structure on Lake Audubon

A separate intake on Lake Audubon would not impact the Bureau's existing facilities, however the sole source of water for the NAWS project would be limited to Lake Audubon. A benefit of this source would be a much lower range of water surface fluctuations, normally about 5 feet, ranging from elevation 1843 to 1848. The minimum water surface elevation expected is 1835.

Alternative 7. This alternative would involve construction of a stand alone NAWS intake/pump station approximately 120 feet north of the existing Snake Creek discharge structure on the east embankment of U.S. Highway 83. The facilities would include: a concrete intake structure situated on the bank, a canned or wetwell pump station, modifications to the access road, and enlargement of the embankment to site the structure. A channel also would need to be constructed at invert elevation 1830 to connect to the existing discharge channel of the Snake Creek facilities. The estimated construction cost of this facility using canned pump station construction is presented in Table 5-2. The estimate shown below was prepared for comparison to the other six

alternatives and does not include various components which will be described later in this section.

TABLE 5-2

LAKE AUDUBON INTAKE PUMP STATION CONSTRUCTION COST

Earthwork/Channel Excavation	$227,000
Concrete: Pump Station/Intake	181,000
Pump Cans	55,000[a]
Building	226,000
Pumps/Electrical	570,000
Pipe, Fittings, Mechanical	124,000
Site Work	49,000
Embankment/Rip Rap	100,000
CONSTRUCTION COST	$1,532,000

[a]Wetwell construction with mechanical fine screening adds $650,000 and deletes pump can costs.

Recommended Alternative

The evaluation of alternatives indicates no distinct advantage of drawing the NAWS water supply from Lake Sakakawea. Alternative 5 would permit the ability to draw water from both Lake Sakakawea and Lake Audubon, but only when lake elevations are above 1830. Costs for integrated facilities would likely be less than for an intake on Lake Audubon, however, the cost differences are not significant. Integrating the NAWS facilities into the Snake Creek Pumping Plant would require major modifications in how the Snake Creek facilities are operated. This would likely involve issues and costs that extend beyond the relative comparison of construction costs. The feasibility of the integrated facilities and the Lake Audubon intake was reviewed with the North Dakota State Water Commission. The review resulted in a directive from the Commission to proceed with the Lake Audubon Intake/Pump Station. While the integration of the facilities may be technically feasible, the other issues regarding joint operation, costs, and potential schedule delays provides no definitive reason to pursue an integrated intake/pump station for the NAWS project.

NAWS EAST SYSTEM INTAKE/PUMP STATION

The NAWS intake/pump station must supply the maximum daily demand of 28 mgd for the East System to the Minot WTP for treatment and distribution. The prefinal design of the NAWS Lake Audubon intake/pump station includes a siting analysis, evaluation of Lake Audubon operating criteria and characteristics, hydraulics, design criteria, and development of facilities layouts.

NAWS Intake Siting Analysis

Siting of the NAWS intake on Lake Audubon involved an analysis of a number of issues including:

- Relative location of the intake to deep water in Lake Audubon.
- Topography of the bank and reservoir bottom.
- Access.
- Location relative to power supply.
- Wind, wave action, and potential for sediment plume interference.
- Raw water pipeline route and constructability.

A siting evaluation was conducted using topographic maps of the Lake Audubon shoreline in the proximity of the Snake Creek Pumping Plant and U.S. Highway 83. Although access on the north shore area of Lake Audubon exists to serve recreational facilities, the only close access near the water is on the embankment separating Lake Audubon from Lake Sakakawea. Another key factor is the distance from the intake/pump station to deep water (approximately elevation 1830).

Two potential sites were selected that met the criteria identified above. One site was on the east embankment approximately 120 feet north of the existing Snake Creek Pumping Plant discharge structure on Lake Audubon; the other site was located on the north shore about 170 feet from and adjacent to the embankment. Both locations are near the proposed pipeline route and offer reasonable access to water; however the north shore facility would require construction of an access road to service the facility. The existing embankment is rip-rapped and would not be as subject to erosion or sediment plumes as would a structure located on the north shore line. The embankment structure also is positioned at the 1835 lake bottom contour and it may be possible to access the 1830 depth just offshore from the intake or by constructing an underwater channel to connect into the existing Snake Creek discharge channel. The existing road along the embankment provides a direct route to the north roughly paralleling U.S. Highway 83 for a distance of approximately 2,000 feet. This location also has a power supply directly available. Based on the factors evaluated above, the site on the east embankment appears to be the most feasible location and is recommended. The pre-final design layout and criteria development are presented for siting of the NAWS intake and pump station on the east embankment.

LAKE AUDUBON OPERATIONAL CRITERIA

The physical conditions that will affect the operation of the intake/pump station include: reservoir operating levels, climatological conditions, ice formation, and sediment potential.

Lake Audubon Lake Levels

Since 1975 when Lake Audubon's water surface was raised from an average elevation of 1834 to elevation 1847, the lake stage levels have fluctuated seasonally over a small range, with a slight decreasing trend starting in 1987 through the present. Recent annual water surface fluctuations are about four feet. The following seasonable water surface ranges were reported:

Season	Water Surface Elevation Ranges
Summer	1846 to 1847
Winter	1843 to 1844

The maximum pool level for Lake Audubon is 1850. The absolute minimum expected pool level is 1835.

Based on the operating ranges and the minimum pool level, the design criteria for the intake structure invert was set at elevation 1830. This would provide about five feet of operating water stage during absolute minimum pool conditions.

Lake Audubon pool elevations are controlled by the U.S. Bureau of Reclamation using the Snake Creek Pumping Plant. During the spring, Lake Audubon is filled to spring target water surface elevations. It is assumed that Lake Audubon's water surface elevations and lake level fluctuations will be maintained at or near current levels for the foreseeable future. If lake level operation plans change, the intake on Lake Audubon should still be able to draw water down to elevation 1835 or below, however, water quality likely would be impacted at this shallow condition.

Prevailing Wind Direction

The prevailing wind direction on Lake Audubon is from the northwest. Since the intake location is at the northwest end of the lake, the structure will be protected from the prevailing wind. The secondary wind direction is from the southeast. This will require that issues concerning: wave generation, ice flow during spring breakup, sediment plume circulation, and surface debris be addressed during design.

Placement of the intake on the rip-rapped embankment will provide protection from wave erosion of the bank and direct sediment influence adjacent to the structure. Sediment and turbidity may still be an issue from erosion of the unprotected north shore of the lake and lake circulation patterns.

Preliminary wind speed information was obtained from the Bureau of Reclamation. Higher wind speeds are experienced in the winter (January through March) when the lake is frozen. Return period frequency analysis for one hour duration winds are as follows:

Return Period	Wind Speed (mph)
2 year	30
5 year	36
10 year	40
25 year	45

Southeast winds will have an effect on the site in terms of wave generation. Given the shallow conditions of the lake and a fetch of about 2 miles, wave heights in excess of 2.5 to 3.0 feet are expected.

Sediment/Turbidity Plumes

The location of the intake structure on the rip-rapped Snake Creek embankment should mitigate direct sediment plumes caused by bank erosion and waves. However, since the structure is located only 250 feet from the unprotected north shore line of Lake Audubon, lake circulation patterns may carry turbidity plumes near the intake.

Debris

Large debris and trash are not expected at the intake site. The primary source of water for Lake Audubon is Lake Sakakawea pumped through the Snake Creek Pumping Plant. Lake Audubon is a shallow, manmade impoundment that has a very limited drainage area. Large debris such as tree limbs are not expected to be a problem. However, vegetation, weeds and aquatic plants will be present. For this reason, and the sheet and frazil ice conditions described in the next section, trash racks will not be provided and only fine screening is recommended to remove this material prior to pumping. A screen mesh of 3/8-inch openings would remove material of the size and nature expected in Lake Audubon. This screen will also be effective in removing smaller size debris such as twigs and trash that may be introduced in the reservoir.

Ice Conditions

The winter time climatological conditions in North Dakota will result in the formation of both sheet ice and frazil ice. These conditions will influence the design and operation of the intake and pump station. Winter temperatures range well below freezing, with temperatures of -30°F or below commonly experienced. Ice formation, which will influence the structural design and the operational requirements of the intake, is described below.

Sheet Ice. Sheet ice forms in Lake Audubon several feet thick. Sheet ice formation occurs during the lower winter lake levels (1843 to 1844). Filling of the reservoir to spring target levels occurs after the melt.

The structural design of the intake facilities exposed to the sheet ice at low lake levels will be taken into account. Sheet ice loads will be an issue during spring break-up when wind driven ice flows will bear upon the structure. For this reason, a bank situated facility is recommended instead of a free standing facility which would be sited in the water and connected to the shore by a bridge or earthen causeway.

The intake structure minimum draw-off elevation of 1830 provides a total minimum water depth of about 12 feet under normal operating conditions. This should be adequate to accommodate winter time operation of the facility. A compressed air bubbler system will be incorporated at the entrance of the intake forebay to retard the formation of sheet ice at the structure.

Frazil Ice. The potential exists for the formation of frazil ice at the intake structure. The approach velocities of water through the structure should be limited to about 0.5 feet per second or less in the intake forebay, channels, equipment (fine screens), and wetwell at winter lake levels. Velocities will be limited to one foot per second or less through control gates or isolation gates. The fine screening chamber will be isolated from the exterior by a concrete wall and the isolation gate. Compressed air will be supplied to the screening chambers to reduce frazil ice formation and sheet ice formation. The compressed air will transmit some warmth to the chamber. Under extreme cold temperature conditions, the water may tend to form slush. If this becomes an operational issue, steam injection in the screening area could be added.

Frazil ice is a particular problem at trash racks exposed to cold temperatures. Special design considerations normally include material and coating alternatives, addition of air or steam, and approach velocity limitations. Because of the frazil ice potential and the extreme damage potential from sheet ice movement, the trash racks were eliminated from the intake structure. Although large debris is not expected, the intake forebay is designed to accept a trash rack should it prove to be needed. Sheet ice and frazil ice formation would control design criteria for these devices.

Site Constraints

The east side of the Snake Creek embankment is a protected rip-rapped site with steep sideslopes to about elevation 1835. The site is relatively close to the Snake Creek Pumping Plant discharge channel, which is at elevation 1830. While the site has direct access, the building pad area is extremely limited. The NAWS intake and pump station facility will require a portion of the embankment slope and construction of a level building area for the pumping and chemical feed facilities. This can be accomplished by constructing additional embankment fill and/or relocating the existing access road.

Construction of the intake channel in Lake Audubon and the portion of the intake structure located below the normal high water level will require a 404 permit. Construction of the intake/pump station and the access road modifications may require either permits and/or review by State and Federal agencies including the Corps of

Engineers, the Bureau of Reclamation, Federal Department of Transportation, and the State Transportation Department.

Figure 5-1 shows the general site layout for the NAWS raw water intake pumping station facilities on Lake Audubon. Special design considerations are described below:

- Embankment rip-rap size: 2 ft diameter above low water, 1 ft diameter below the low water elevation (1835).

- Embankment construction: engineering fill compacted to soils consultant recommendations.

- Construction dewatering: interlocking sheet piles with dewatering pumps and plastic membrane barriers or equivalent, or earthen embankments if permitted by the State, Environmental Protection Agency (EPA), and the Corps of Engineers.

- Intake channel construction: barge mounted dredge or track-hoe depending on water surface elevations. Construction methods will likely have to comply with EPA and State water quality measures and requirements.

- Raw water pipeline: the pipeline route for the 42-inch raw water pipe leaving the pumping facilities will be located within the limits of the existing access roadway at a minimum depth of 7.5 ft.

INTAKE/PUMP STATION HYDRAULICS AND CRITERIA

Hydraulics

The intake/pump station hydraulics were developed using the minimum pool elevation of 1835 and a maximum day demand of 28 mgd. All pump station hydraulics were calculated on the discharge centerline pressure not exceeding 150 psi (346 ft of head). The discharge pressure relates to the optimization analysis conducted to determine the cost trade-off for pumping and the pressure class and size of the raw water pipeline. Intake hydraulics and design criteria for the facility are shown below:

Intake Facility Component	Design Criteria	Extreme Low Pool El = 1835	Winter Normal Pool El = 1843
Entrance channel, velocity	–	0.33 fps	–
Entrance structure, velocity	<0.5 fps	0.62 fps	0.33 fps
Fine screen, velocity	<2.0 fps	1.42 fps	0.54 fps
Wetwell approach velocity	<1.0 fps	0.30 fps	0.11 fps

At the extreme, low pool elevation of 1835, the entrance structure velocity exceeds the design criteria: however, this is a winter time criteria related to frazil ice formation. The maximum winter day demands are expected to be significantly less than 28 mgd, and the normal winter low pool elevations are expected to be greater than elevation 1835.

The layout of the pump station wetwell hydraulics are designed around the Hydraulic Institute Standards. The total maximum headloss through the intake structure is 1.03 ft, including a maximum water surface differential of 1.0 ft across the mechanical screen.

The pumps are required to lift raw water from Lake Audubon to a booster station located near the town of Max. The maximum friction loss through the 42-inch diameter force main at 28 mgd is 88 ft. The estimated headloss through the pump station discharge manifold is 5.8 ft. Thus, the total friction loss at the pump discharge is about 94 ft. The maximum total dynamic head at the pump discharge is 346 ft (150 psi). Therefore, the static head is 252 ft.

The maximum hydraulic grade line elevation from Lake Audubon to the Max Booster Station is presented below:

Location	Hydraulic Grade Line Elevations
Lake Audubon (minimum pool)	1835
Pump Wetwell (dirty screen)	1834
Pump Discharge Head	2203
Entrance to 42-inch Force Main	2197
Max Booster Station	2109

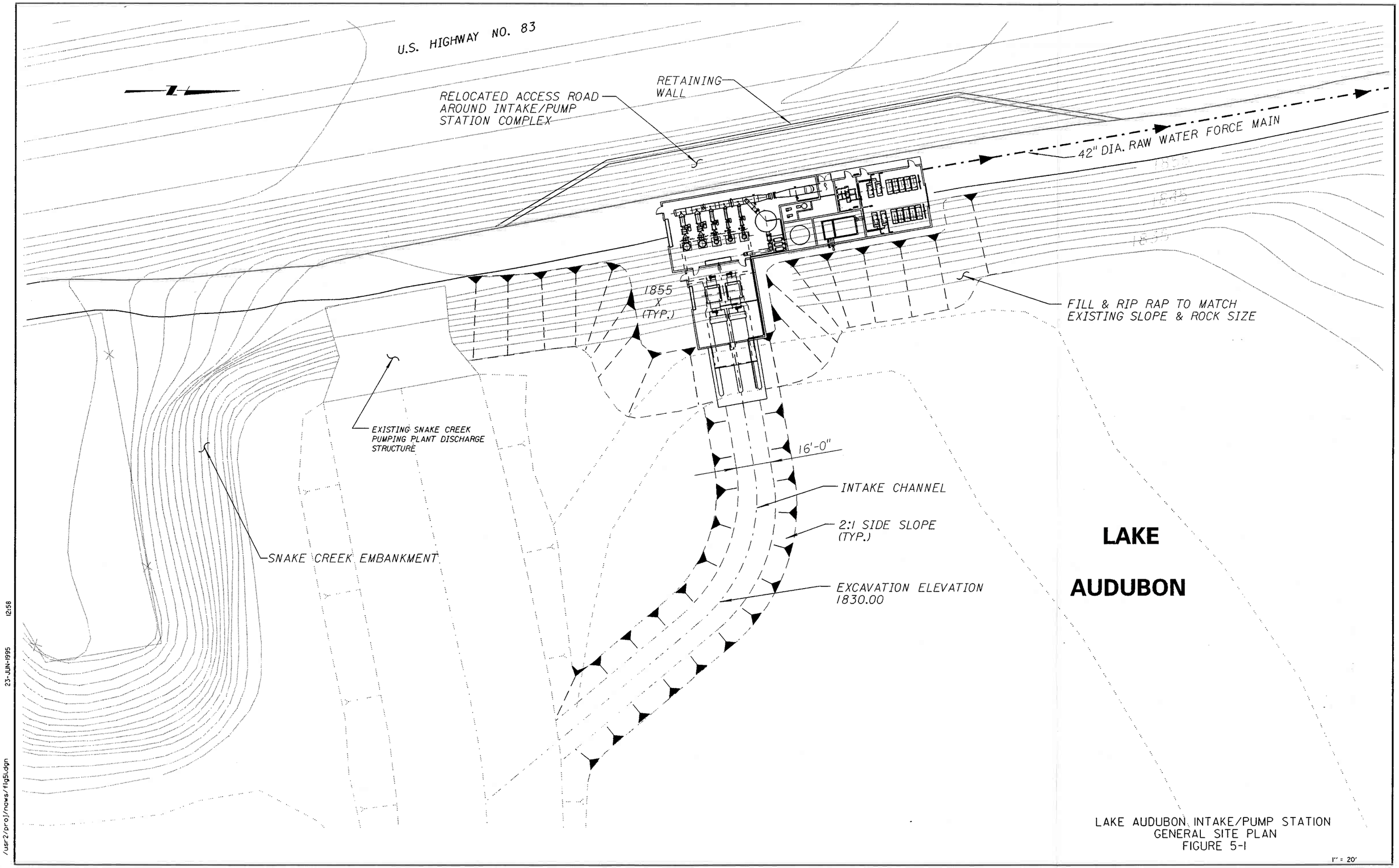
The total dynamic head for each pump (including the static lift from the minimum wetwell level to pump discharge centerline) is 368 ft which was used to design the horsepower requirements for the individual pumps.

Pumping Facilities Design Criteria

Table 5-3 presents the design criteria for the intake pump station facilities.

Chemical Feed Facility Criteria

Discussions concerning biota transfer issues resulted in a requirement that the raw water supply achieve a designated contact time with chloramines before reaching the divide between the Missouri River and Hudson Bay drainage basins. Therefore, a chemical addition facility is included as part of the Intake/Pump Station structure.



LAKE AUDUBON INTAKE/PUMP STATION
GENERAL SITE PLAN
FIGURE 5-1

1" = 20'

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TABLE 5-3

LAKE AUDUBON INTAKE/PUMP STATION DESIGN CRITERIA

Description	Units	Criteria
Capacity		
Maximum day	mgd	28
Average	mgd	11
Minimum	mgd	4
Raw Water Intake Structure		
Intake Channel (dredged or excavated)		
Width	ft	16
Average depth	ft	4
Side slope	ft:ft	2:1
Length	ft	280
Intake Structure		
Type: concrete bank structure without trash rack		
Intake width/per bay (two bays)	ft	7
Intake velocity		
minimum pool	fps	0.66
winter pool	fps	0.35
Invert elevation	ft	1830
Raw Water Pumping Station		
Type: Concrete wetwell with traveling mechanical screens and vertical turbine constant speed pumps		
Number of screens	no.	2
Screen panel width	ft	4.5
Screen mesh	in	3/8 x 3/8
Maximum velocity through screen	ft/sec	1.42
Screen depth (invert to floor)	ft	25
Number of pumps	no.	5
Capacity (mgd)	mgd	4, 4, 7, 15, 15
Horsepower (brake)		
4 mgd pumps (2)	hp	350
7 mgd pump	hp	600
15 mgd pumps (2)	hp	1250

The chemical addition facilities will first add chlorine then aqueous ammonia to form chloramines in the raw water supply. The facilities include equipment for the storage and feeding of chlorine and ammonia, and a chlorine scrubber. The maximum chloramine dose will be 4 mg/l.

The chlorine room and the chlorine feeders will be located in isolated rooms with outside access. The chlorine scrubber and the aqueous ammonia tank will be located within concrete containment walls. The on-site chemical storage volumes will equal the maximum day demand for approximately 30 days. The design criteria for the chemical feed system is presented in Table 5-4.

TABLE 5-4

LAKE AUDUBON INTAKE/PUMP STATION CHEMICAL FEED DESIGN CRITERIA

Description	Units	Criteria
Ammonia (Aqua Ammonia, Liquid, 29% soln. 2.2 lbs/gal)		
Dosage	mg/l	0.6 to 1.2
Feed rate	gal/day	9 to 127
Metering pumps	no.	2
Storage tank	no.	1
Capacity	gal	4,000
Days Storage	days	30
Chlorine (Gaseous, ton cylinders)		
Dosage	mg/l	2.0 to 4.0
Gas feed rate	lbs/day	65 to 1,000
Chlorinators	no.	3
Capacity, each	lbs/day	300, 700, 1,000
Ton cylinders	no.	14
Days storage	days	28

Disinfectant contact time will be provided in the 42-inch and 38-inch diameter raw water pipeline to the divide. At 28 mgd, this equates to a contact time of 390 minutes. The final design criteria and disinfectant application points will be developed during final design and will be based on chlorine demand data, raw water disinfection efficiency, and the results of the Chloramine Challenge Study.

INTAKE OPERATION AND CONTROL

The raw water intake/pump station will supply flow to the Minot WTP through a transmission system which includes the force main, a booster pump station, and a 3.0 million gallon storage reservoir. Other than the reservoir, which will provide some dampening effect, the system will be required to pump raw water very near the treatment plant flow rate. This will require that control of the intake and the booster station near Max be established with the plant.

The intake/pump station will draw water at elevation 1830 to 1835 (gate opening elevations). Since Lake Audubon is relatively shallow (14 to 17 feet), multiple draw-off elevations would likely not be beneficial, and are not provided.

The chemical feed system will be paced to flow rate. A flow meter is provided on the discharge force main.

INTAKE PUMP STATION LAYOUT

Figures 5-2 and 5-3 present the pre-final design of the facilities, including an overall plan and section through the intake/pump station.

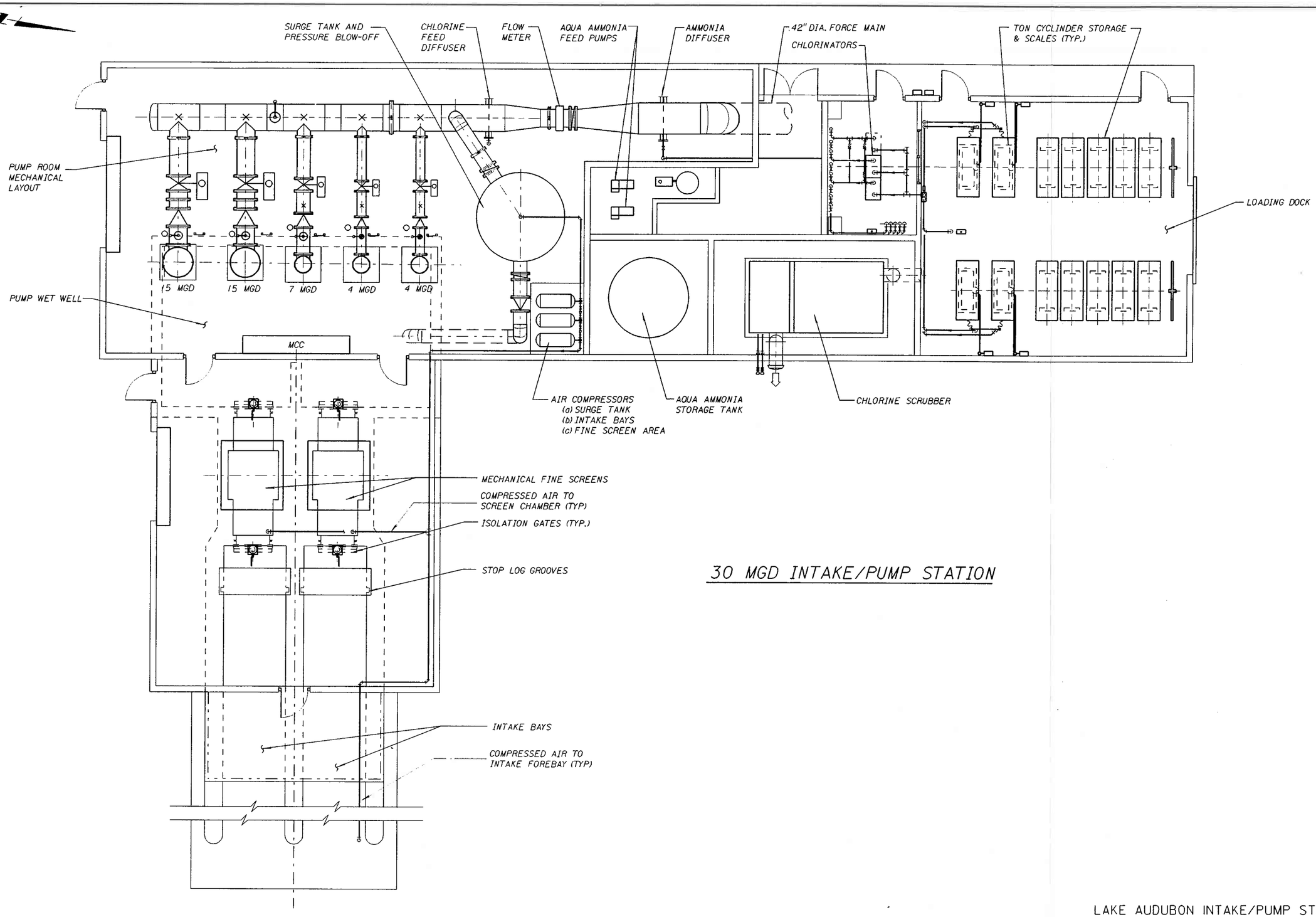
COST ESTIMATES

A cost estimate for the Lake Audubon Intake/Pump Station was prepared using recent construction bids from projects of similar complexity, and from information supplied by equipment and material suppliers. The construction cost estimate also includes provisions of the chemical feed facilities needed to provide disinfection of the raw water. The costs for the intake/pump station are presented in Table 5-5.

TABLE 5-5

LAKE AUDUBON INTAKE/PUMP STATION CONSTRUCTION COST

Description	Construction Cost
Earthwork/Channel Excavation/Embankment	$330,000
Concrete	400,000
Building	360,000
Screening Facilities	480,000
Pumps/Electrical	580,000
Mechanical/Pipes/Fittings	170,000
Instrumentation	40,000
Chlorine System/Scrubber	210,000
Ammonia Feed System	80,000
Site Work	50,000
CONSTRUCTION COST	**$2,700,000**



30 MGD INTAKE/PUMP STATION

LAKE AUDUBON INTAKE/PUMP STATION
FLOOR PLAN
FIGURE 5-2

3/16" = 1'-0"

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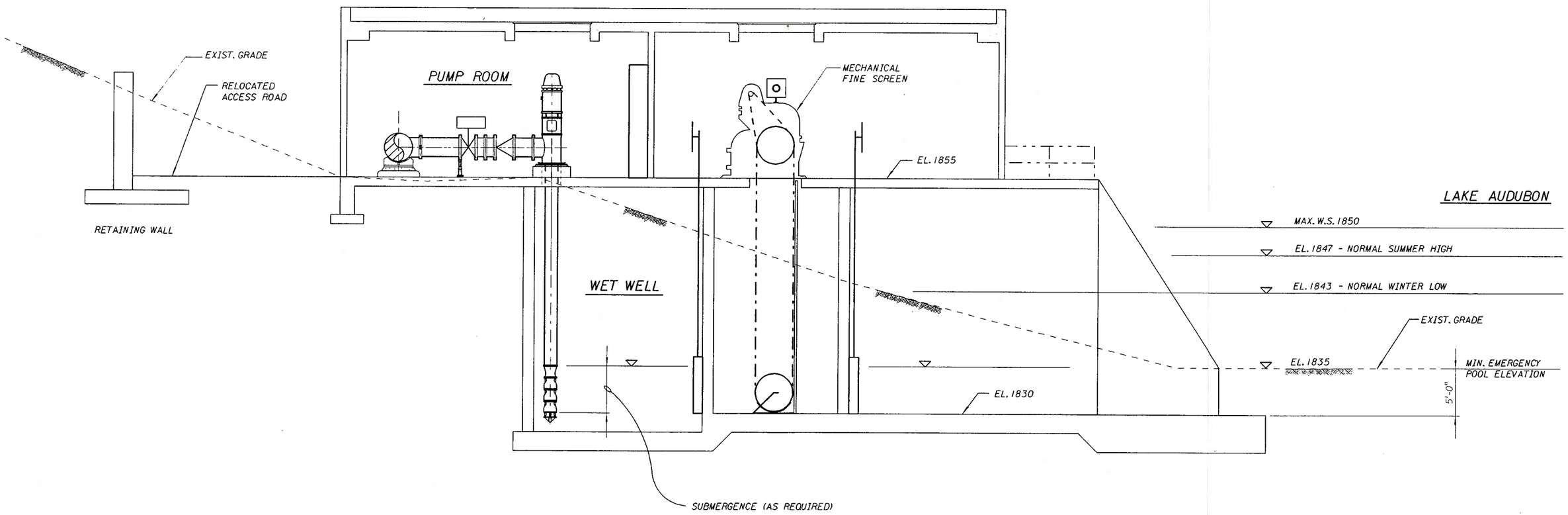

LAKE AUDUBON INTAKE/PUMP STATION
SECTION
FIGURE 5-3

3/16" = 1'-0"

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Section 6

HOUSTON ENGINEERING
AMERICAN ENGINEERING
MONTGOMERY WATSON

SECTION 6

WATER TREATMENT FACILITIES

INTRODUCTION

The purpose of this section is to identify water treatment facilities necessary for the NAWS project. Extensive discussion and analysis have resulted in the selection of the following locations for NAWS water treatment plant (WTP) facilities:

- Existing Minot WTP
- Existing Williston WTP
- Existing Parshall WTP

The existing WTPs will require a significant capital investment in order to be incorporated into the NAWS system. Capital improvements to these facilities can be placed in the following categories:

- Expansion of facility capacity to meet the increased requirements of the NAWS demand projections.

- Rehabilitation to update and modernize existing facilities that are in some cases 40 years old. These include improvements needed to bring facilities into compliance with current building and fire codes.

- Process modifications necessary to meet the new State and Federal water quality regulations currently in place or projected in the near future.

In addition, for the Minot WTP, capital improvements have been included which are necessary to meet biota transfer concerns as developed by the Engineering Biology Task Group. The following pre-final design information is presented herein for each of the three water treatment facilities:

- Process flow schematics
- Hydraulic profiles
- Preliminary layouts
- Design criteria

A brief description of each process element for the three WTPs is also provided to explain the need for modifications to existing facilities or construction of new facilities.

MINOT WTP

The City of Minot WTP was originally constructed in 1952 with a capacity of 6 mgd. In 1962 an additional 12 mgd of capacity was added, bringing the current process capacity

to 18 mgd. While the process capacity is 18 mgd, the plant is limited hydraulically by the finished water piping and has a practical maximum capacity of 14 mgd. The Minot WTP must be expanded to a reliable process and hydraulic capacity of 28 mgd in order to satisfy the demands of the NAWS East System.

Water Supply

The Minot WTP currently treats surface water from the Souris and Des Lacs Rivers and groundwater from the Souris and Sundre Aquifers. The existing river intake has a capacity of about 8.7 mgd while the combined well field capacity is approximately 17 mgd. Typically, in the winter the use of river water constitutes approximately one-third of the raw water supply and 10 to 20 percent during the summer. The NAWS East System will provide up to 28 mgd of Missouri River water diverted from Lake Audubon to be treated at the Minot WTP.

Process Description

Both the surface and groundwater sources of supply used by Minot are characterized by hardness levels ranging from 200 to over 700 mg/l (as $CaCO_3$). The existing main treatment process at the Minot WTP includes softening in solids contact units, recarbonation, filtration, disinfection with chlorine, and distribution through the high service pumping station. Potassium permanganate and powdered activated carbon (PAC) added at the river intake have been used in an attempt to control the severe tastes and odors which can occur in the surface water sources.

Historically, the Minot WTP has utilized the lime-soda ash softening process to reduce total hardness to a level between 80 and 90 mg/l. In the NAWS project, the basic elements of softening will be kept in place at the Minot WTP; however, the addition of new solids contact units capable of treating surface water through a conventional process (coagulation, flocculation, sedimentation) will provide added flexibility. The process train at the Minot WTP will be arranged to allow the following alternatives:

- Separate treatment of groundwater (up to 18 mgd capacity) and surface water (up to 10 mgd capacity) to provide optimized pretreatment for each source, followed by blending prior to filtration.

- Treatment of blended groundwater and surface water at capacities of up to 28 mgd.

- Split stream treatment with clarification of surface water and softening of groundwater, followed by blending prior to filtration.

- Sequential treatment of blended or separate groundwater and surface water process streams with clarification followed by softening at capacities of up to 12 mgd.

- Two stage softening of blended or separate groundwater and surface water process streams with intermediate recarbonation at capacities up to 12 mgd.

At the present time, it is proposed that ozone will be used for primary disinfection followed by the addition of chloramines (combined chlorine and ammonia) for a distribution system disinfectant residual. The use of ozone is proposed for the following reasons:

- To provide required disinfection (CT compliance) without the expense of additional yard piping, clearwells, and baffling which would be required to allow CT compliance at a 28 mgd capacity with chlorine.

- To minimize the formation of disinfection by-products (DBPs) and eliminate the risk of noncompliance with more stringent water quality standards while treating Missouri River water.

- To provide a positive and reliable means of removing taste and odors in the surface water supplies.

It should be noted that bench scale testing will be necessary during design to determine the need for ozone and to target required dosages. Pilot scale testing may be necessary to demonstrate the ozone process to State regulatory agencies.

During discussions conducted as a part of the Engineering Biology Task Group meetings it was reported by members from the City of Minot that bromate formation using ozone on the City's groundwater supplies may be a problem. This issue needs further evaluation during bench/pilot scale studies. Flexibility has been provided in the pre-final design facilities to allow for separate disinfection of groundwater with an alternate disinfectant should it be necessary.

The proposed Minot WTP process flow diagram is shown in Figure 6-1. The corresponding hydraulic profile is shown in Figure 6-2. A general site plan and new treatment facility building plan are shown in Figures 6-3 and 6-4, respectively.

Process Elements

The following is a brief description of the process elements and modifications proposed for the Minot WTP. Complete design criteria are shown in Table 6-1.

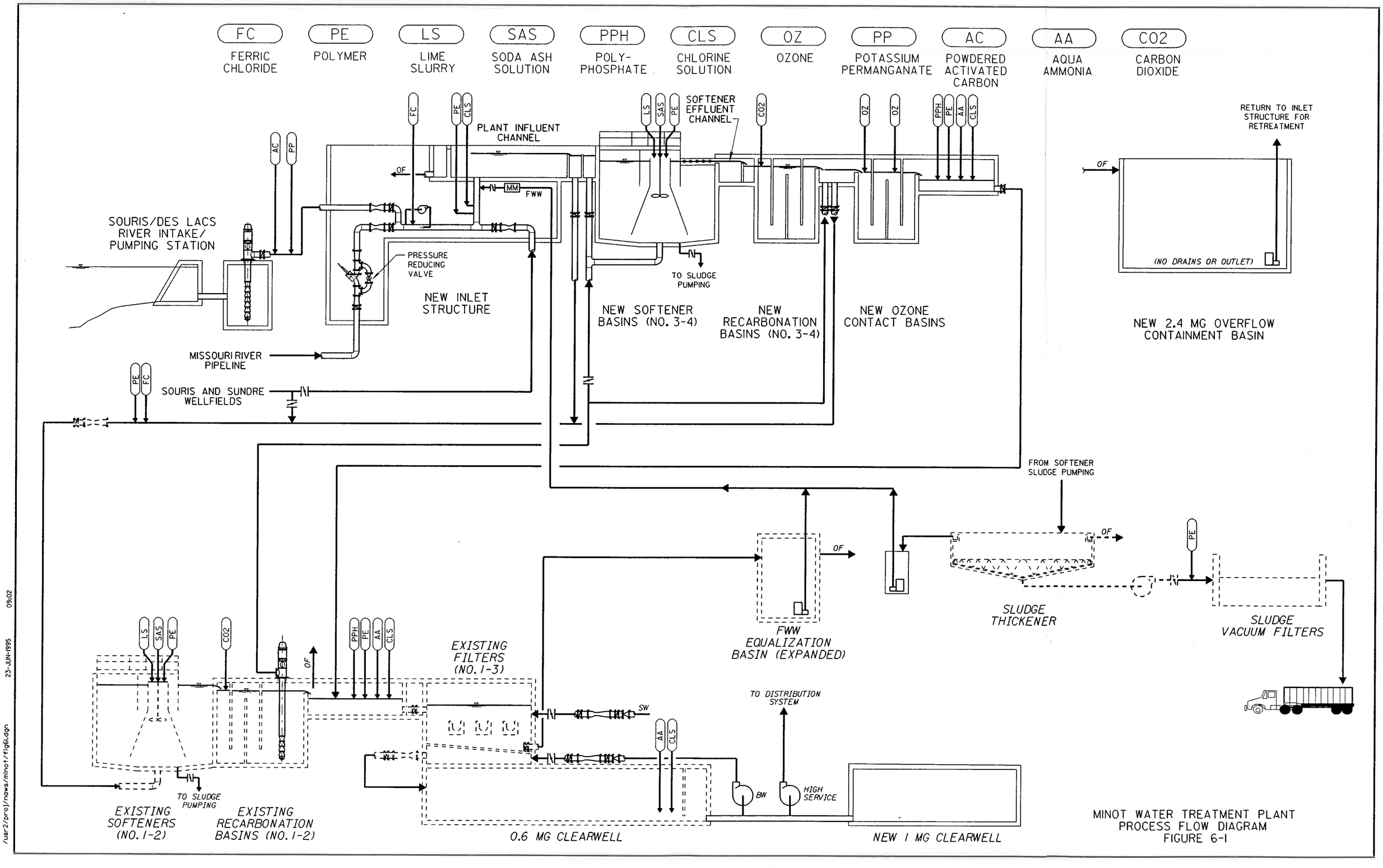
Inlet Structure. A new inlet structure would be constructed which would incorporate the following functions:

- Flow control for the Missouri River pipeline.

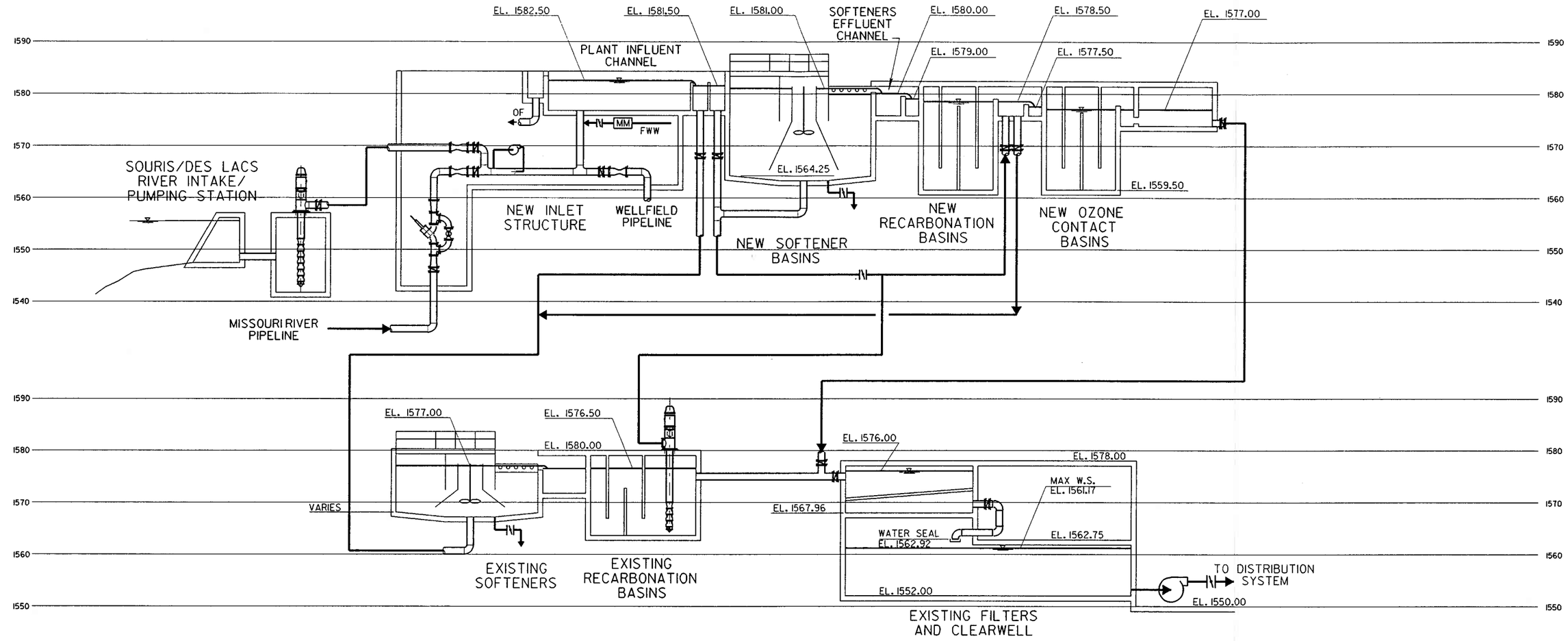
TABLE 6-1

MINOT WTP
DESIGN CRITERIA

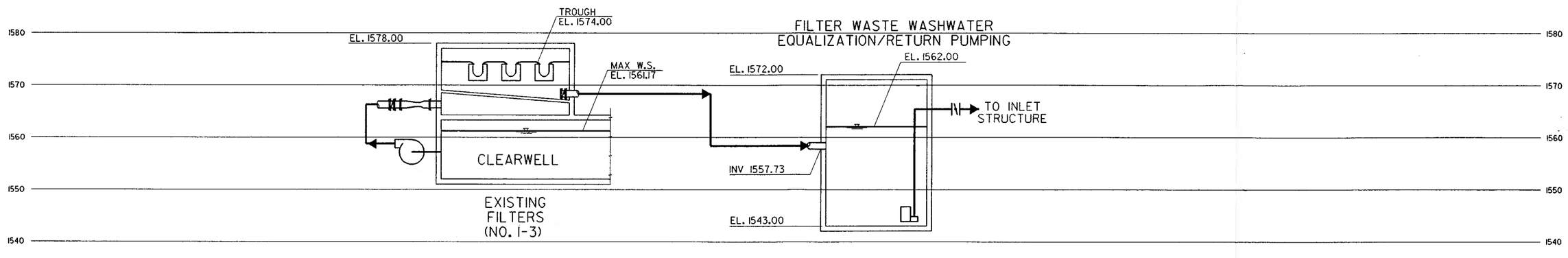
Description	Units	Current	Proposed
Plant Capacity			
Design Capacity	mgd	18	28
Raw Water Supply			
Souris River Intake	mgd	8.6	8.6
Missouri River Pipeline	mgd	-	28
Groundwater	mgd	17	17
Flash Mix (Coagulation)			
Existing Flash Mix (Abandon)			
Flash Mix (New)			
Type: Pumped Diffusion			
Location: Inlet Structure (Pre-softening)			
Number	No.	-	1
Mixing Energy	sec^{-1}	-	750
Pump Power	hp	-	10
Softening Basins			
Type:			
Basins 2, 3, 4: Solids Contact Units (IDI Accelators, IS Type)			
Basin 2: Solids Contact Unit (Walker Process)			
Number	No.	2	4
Dimensions			
Basin 1 (Existing)	ft x ft	60 x 60	60 x 60
Basin 2 (Existing)	ft x ft	84 x 84	84 x 84
Basin 3 & 4 (New)	ft x ft	-	60 x 60
Water Depth (Sidewall)			
Basins 1, 3 & 4	ft	16.75	16.75
Basin 2	ft	19	19
Available Area			
Basins 1, 3 & 4 (Each)	sq ft	3,400	3,400
Basin 2	sq ft	5,800	5,800
Total All Basins	sq ft	9,200	16,000
Volume			
Basins 1, 3 & 4 (Each)	gal	270,000	270,000
Basin 2	gal	1,000,000	1,000,000
Total All Basins	gal	1,270,000	1,810,000
Design Flow			
Basins 1, 3 & 4 (Each)	mgd	6	6
Basin 2	mgd	12	12
Total All Basins	mgd	18	30
Surface Loading Rate (at Design Flow)			
Basins 1, 3 & 4	gpm/sq ft	1.2	1.2
Basin 2	gpm/sq ft	1.4	1.4



MINOT WATER TREATMENT PLANT
PROCESS FLOW DIAGRAM
FIGURE 6-1



MAIN PROCESS FLOW HYDRAULIC PROFILE (28 MGD)



FILTER BACKWASH HYDRAULIC PROFILE (10.4 MGD)

SCALE IN FEET
0 10' 20'
SCALE VERTICAL
SCALE HORIZONTAL : NONE

MINOT WATER TREATMENT PLANT
HYDRAULIC PROFILE
FIGURE 6-2

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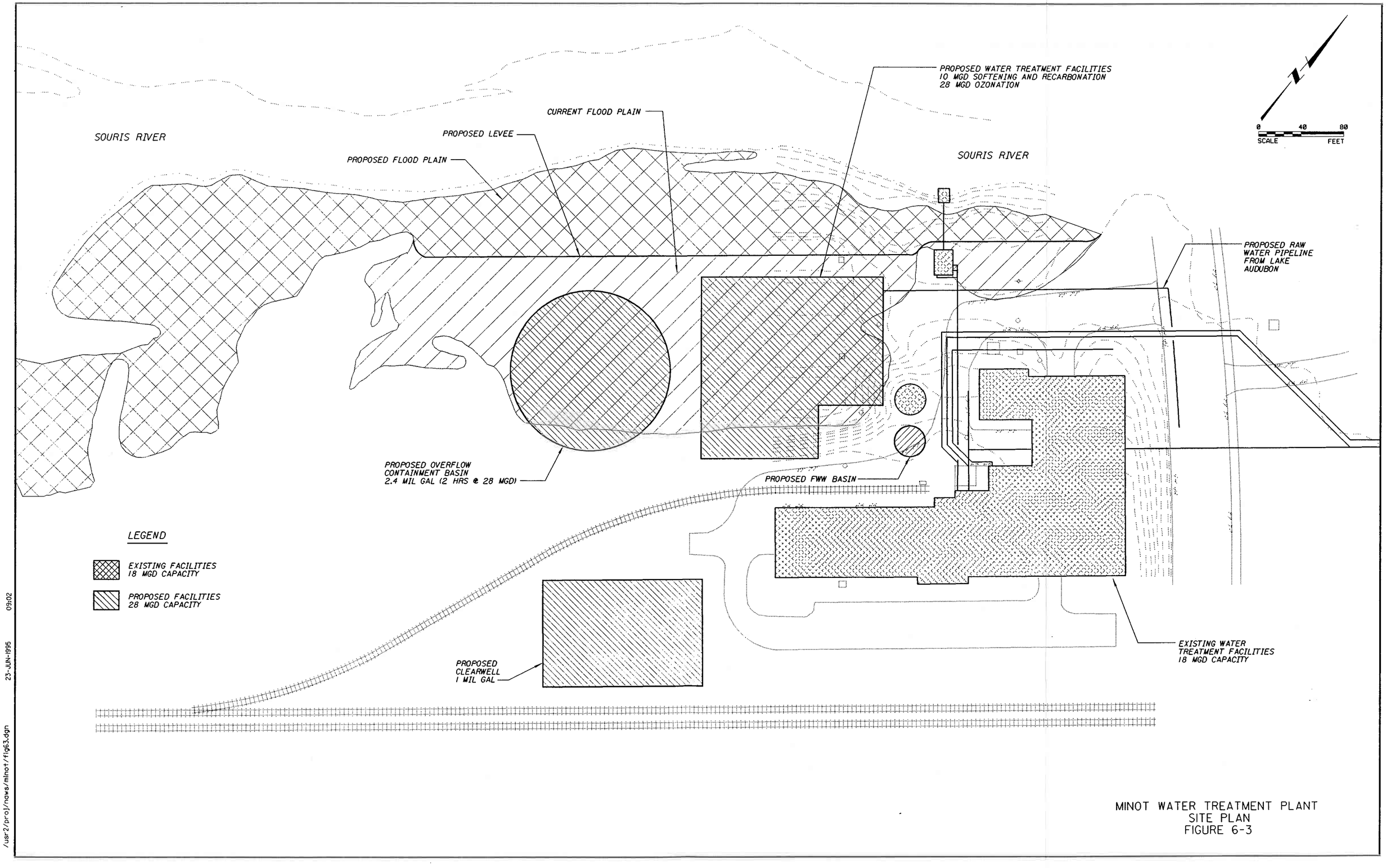

MINOT WATER TREATMENT PLANT
SITE PLAN
FIGURE 6-3

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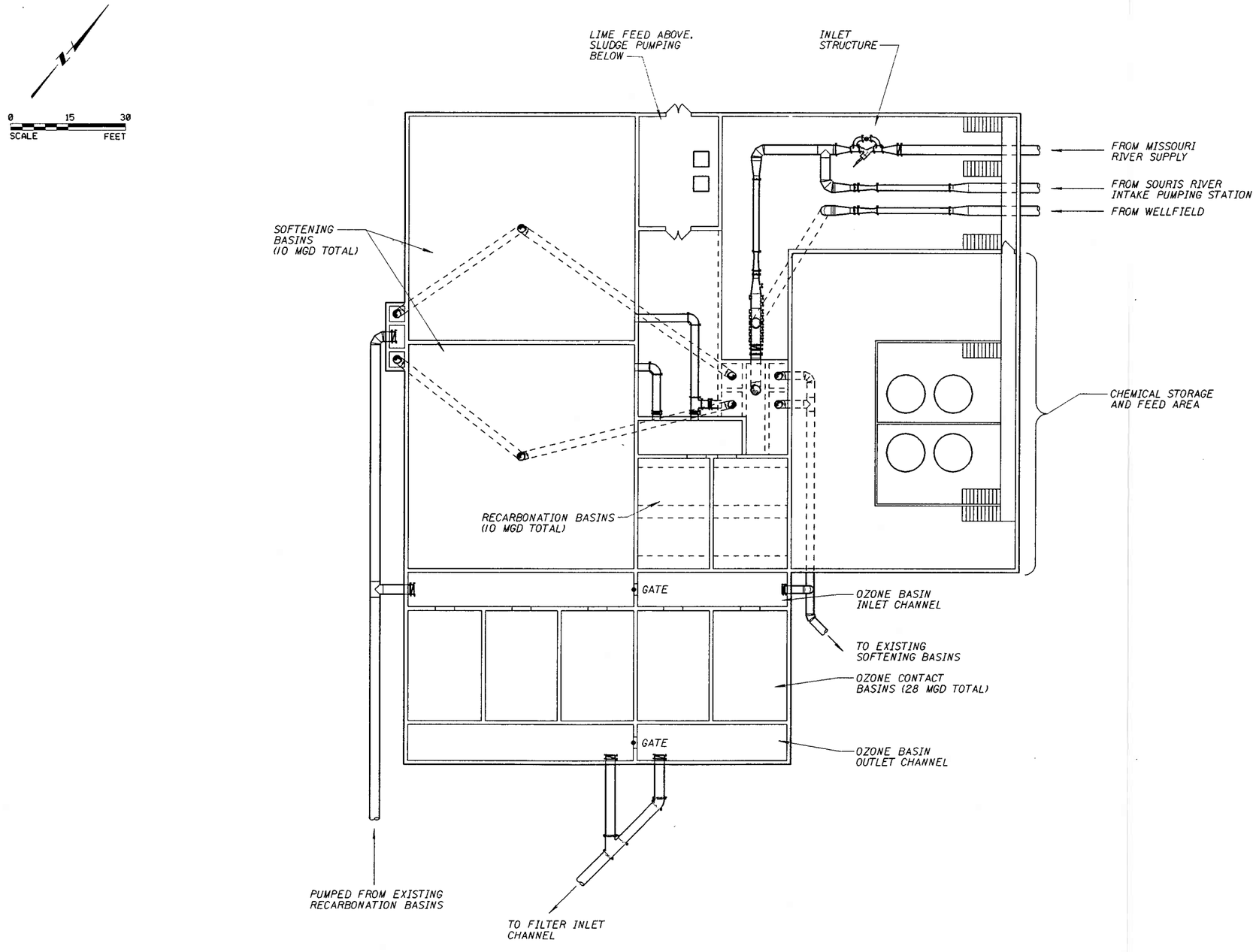


MINOT WATER TREATMENT PLANT
NEW FACILITY BUILDING
FIGURE 6-4

/usr2/proj/nows/minot/fig64.dgn 23-JUN-1995 09:02

TABLE 6-1
(Continued

Description	Units	Current	Proposed
Detention Time			
Basins 1, 3 & 4 (Each)	hrs	1.0	1.0
Basin 2	hrs	2.0	2.0
Recarbonation Basins			
Type: Rectangular, baffled concrete basins			
Number	No.	2	4
Basin Dimensions (w x l)			
Basin 1 (Existing)	ft x ft	20 x 58	20 x 58
Basin 2 (Existing)	ft x ft	14 x 84	14 x 84
Basins 3 & 4 (New)	ft x ft	-	20 x 30
Water Depth			
Basin 1	ft	14	14
Basin 2	ft	14	14
Basin 3 & 4	ft	-	18.5
Volume			
Basin 1	gal	120,000	120,000
Basin 2	gal	120,000	120,000
Basin 3 & 4 (Each)	gal	-	83,000
Design Flow			
Basin 1	mgd	6	6
Basin 2	mgd	12	12
Basins 3 & 4 (Each)	mgd	-	5
Detention Time (at Design Flow)			
Basin 1	min	28	28
Basin 2	min	14	14
Basins 3 & 4 (Each)	min	-	23
Ozone Contactor Basins			
Type: Two-stage, rectangular, baffled concrete basins			
Number of Basins	No.	-	5
Basin Dimensions (w x l)	ft x ft	-	20 x 25
Water Depth	ft	-	18
Volume (Each)	gal	-	70,000
Detention Time	min	-	18
Filters			
Type: Gravity, granular media, rate-of-flow control			
Number	No.	12	12
Filter Dimensions (w x l)	ft x ft	20 x 18	20 x 18
Filter Area (Each)	sq ft	360	360
Total Filter Area	sq ft	4,320	4,320
Filtration Rate			
All Filters In Service	gpm/sq ft	2.9	4.5
One Filter In Backwash	gpm/sq ft	3.2	4.9

TABLE 6-1
(Continued)

Description	Units	Current	Proposed
Clearwell			
Type: Concrete basin, existing clearwell is below filters			
Number	No.	1	2
Dimensions			
Existing	ft x ft	64 x 136	64 x 136
New	ft x ft	-	100 x 150
Water Depth	ft	9.2	9.2
Volume			
Existing	mil gal	0.6	0.6
New	mil gal	-	1.0
Total	mil gal	0.6	1.6
Detention Time	hrs	0.8	1.4
Percent of Max. Daily Production	%	3	6
Finished Water Pumps			
Type: Split-case horizontal with flooded suction			
Number	No.	8	8
Capacity (Total Rated)	mgd	17.3	28

TABLE 6-1
(Continued

Description	Units	Current	Proposed
Detention Time			
Basins 1, 3 & 4 (Each)	hrs	1.0	1.0
Basin 2	hrs	2.0	2.0
Recarbonation Basins			
Type: Rectangular, baffled concrete basins			
Number	No.	2	4
Basin Dimensions (w x l)			
Basin 1 (Existing)	ft x ft	20 x 58	20 x 58
Basin 2 (Existing)	ft x ft	14 x 84	14 x 84
Basins 3 & 4 (New)	ft x ft	-	20 x 30
Water Depth			
Basin 1	ft	14	14
Basin 2	ft	14	14
Basin 3 & 4	ft	-	18.5
Volume			
Basin 1	gal	120,000	120,000
Basin 2	gal	120,000	120,000
Basin 3 & 4 (Each)	gal	-	83,000
Design Flow			
Basin 1	mgd	6	6
Basin 2	mgd	12	12
Basins 3 & 4 (Each)	mgd	-	5
Detention Time (at Design Flow)			
Basin 1	min	28	28
Basin 2	min	14	14
Basins 3 & 4 (Each)	min	-	23
Ozone Contactor Basins			
Type: Two-stage, rectangular, baffled concrete basins			
Number of Basins	No.	-	5
Basin Dimensions (w x l)	ft x ft	-	20 x 25
Water Depth	ft	-	18
Volume (Each)	gal	-	70,000
Detention Time	min	-	18
Filters			
Type: Gravity, granular media, rate-of-flow control			
Number	No.	12	12
Filter Dimensions (w x l)	ft x ft	20 x 18	20 x 18
Filter Area (Each)	sq ft	360	360
Total Filter Area	sq ft	4,320	4,320
Filtration Rate			
All Filters In Service	gpm/sq ft	2.9	4.5
One Filter In Backwash	gpm/sq ft	3.2	4.9

TABLE 6-1
(Continued)

Description	Units	Current	Proposed
Filter Media			
Type: Dual Media			
Anthracite Coal			
Depth	in	24	20
Effective Size	mm	-	1.0
L/d		-	508
Sand			
Depth	in	6	10
Effective Size	mm	-	0.5
L/d	L/d	-	508
Total			
Depth	in	30	30
L/d		-	1016
Filter Backwash			
Type: Water Wash			
Backwash Pumps	No.	1	2
Pump Capacity (Each)	gpm	3,000	7,200
Backwash Rate (Maximum)	gpm/sq ft	8	20
Duration	min	10	8
Type: Surface Wash, Rotating Arm			
Surface Wash Supply	gpm		360
Surface Wash Rate	gpm/sq ft		1.0
Duration	min		5
Backwash Volume			
Filter Drawdown	gal	-	10,000
Surface Wash (5 min.)	gal	-	2,000
Backwash (8 min)	gal	-	58,000
Filter-to-Waste (30 min at 1 mgd)	gal	-	20,000
Total	gal	-	90,000
Filter Waste Washwater (FWW) Handling			
Equalization Basins			
Type: Circular concrete basin with submersible pumps			
Number	No.	1	2
Diameter (Each)	ft	32	32
Water Depth	ft	18	18
Volume (Each)	gal	108,000	108,000
Backwashes Held (Total)	No.	1	2
FWW Reclamation Pumps (To Head of Plant)			
Type: Submersible			
Number	No.	3	3
Capacity (Each)	gpm	-	600
Time To Empty FWW Basin	min	-	50
Percent of Plant Flow	%	-	9

TABLE 6-1
(Continued)

Description	Units	Current	Proposed
Clearwell			
Type: Concrete basin, existing clearwell is below filters			
Number	No.	1	2
Dimensions			
Existing	ft x ft	64 x 136	64 x 136
New	ft x ft	-	100 x 150
Water Depth	ft	9.2	9.2
Volume			
Existing	mil gal	0.6	0.6
New	mil gal	-	1.0
Total	mil gal	0.6	1.6
Detention Time	hrs	0.8	1.4
Percent of Max. Daily Production	%	3	6
Finished Water Pumps			
Type: Split-case horizontal with flooded suction			
Number	No.	8	8
Capacity (Total Rated)	mgd	17.3	28

- Raw water metering for the three separate water supplies (Souris/Des Lacs River, Missouri River, and groundwater).

- Raw water chemical addition points, including flash mix for coagulation.

- A plant influent channel with flow splitting weirs to provide equal distribution of flow between softening basins.

Flash Mix. A pump diffusion flash mix system would be provided to allow for efficient coagulation and formation of microfloc prior to the softening process. A pumped diffusion flash mix system is recommended because it has the advantages of a lower energy requirement, higher efficiency, non-proprietary parts, and lower maintenance costs than alternative mechanical systems. A mixing energy value (G) of 750 sec^{-1} would be provided.

Chemical Feed. The following chemical feed systems would be provided at the expanded Minot WTP. Chemicals which are currently used at the plant are shown underlined:

- Ferric chloride - used as a primary coagulant.

- Polymer (liquid and dry) - used as a coagulant aid, flocculant aid, filter aid, and sludge conditioner.

- Lime (quick lime) - used in softening for removal of carbonate hardness.

- Soda Ash (sodium carbonate) - used in softening for removal for noncarbonate hardness.

- Polyphosphate - used to prevent precipitation of carbonate hardness.

- Chlorine - used as a disinfectant .

- Aqua Ammonia - used with chlorine to form a chloramine distribution system disinfectant.

- Ozone - used as a primary disinfectant, for control of disinfection by-products, and for control of tastes and odors.

- Potassium Permanganate - currently used for taste and odor control.

- Powdered Activated Carbon (PAC) - currently used for taste and odor control.

- Carbon Dioxide - used for recarbonation (pH adjustment and stabilization of precipitation potential).

All chemical feed systems will need to be upgraded to handle the delivery requirements of the increased plant capacity. Where possible, existing chemical feed locations will be used for upgraded systems; however, additional chemical storage and feed areas will be needed in the new water treatment facilities building for new chemical feed systems (e.g., ferric chloride, aqua ammonia, and ozone). In addition, all chemical feed systems will be upgraded to comply with current code requirements (e.g., Uniform Fire Code, Article 80 - Hazardous Materials). As an example, a chlorine scrubber will be required to prevent release of gas from the chlorine storage and feed facilities.

Softening. The existing softening basins are solids contact units of the following capacities:

- Basin No. 1 has a 6 mgd capacity (IDI Accelator, IS type).
- Basin No. 2 has a 12 mgd capacity (Walker Process with sludge scrapers).

Two additional solids contact units identical to the 6 mgd capacity Basin No. 1 are proposed as Basins 3 and 4. At 6 mgd, Basins 1, 3, and 4 would have a surface loading rate of 1.2 gpm/sq ft, while Basin No. 2 would have a surface loading rate of 1.4 gpm/sq ft. The new basins (Nos. 3 and 4) would be configured to allow operation either in series or parallel. Similarly, Basin Nos. 1 and 2 can now be operated either in series or in parallel.

Recarbonation Basins. Currently, the Minot WTP has two recarbonation basins, located in the process between the softening basins and the filters. Recarbonation Basin No. 1 was constructed at the same time as the 6 mgd softening basin; at this flow rate it has a detention time of 28 minutes. Basin No. 2 was constructed at the same time as the 12 mgd softening basin; at this flow rate it has a detention time of 14 minutes. Two new basins (Nos. 3 and 4) are proposed to match the additional capacity of the new softeners (5 mgd each) with a detention time of 23 minutes.

Ozone Contactor. The ozone contactors are proposed at a location downstream of the recarbonation process and upstream of the filters. This location allows prior reduction in ozone demand through the clarification process with subsequent removal of some ozonation by-products (e.g. aldehydes) across the filter prior to distribution. However, because insufficient head is available (<0.5 ft) between the existing recarbonation basins (Nos. 1 and 2) and the existing filters, it will be necessary to install low lift pumps in the existing recarbonation basins to pump water to the new ozone contact basins.

The proposed ozone contact basins will be two stage, rectangular, baffled concrete basins. Five basins will be provided, each with a hydraulic capacity of approximately 6 mgd. A detention time of 18 minutes will be provided for both for disinfection and taste and odor control. The ozone generation equipment will be located directly over the contact basins. Subject to additional analysis during design, it is proposed that the most cost-effective system will be medium frequency generation of ozone from gaseous oxygen (GOX) produced on-site by the pressure swing adsorption (PSA) method with

supplemental capacity and redundancy provided by on-site storage of liquid oxygen (LOX).

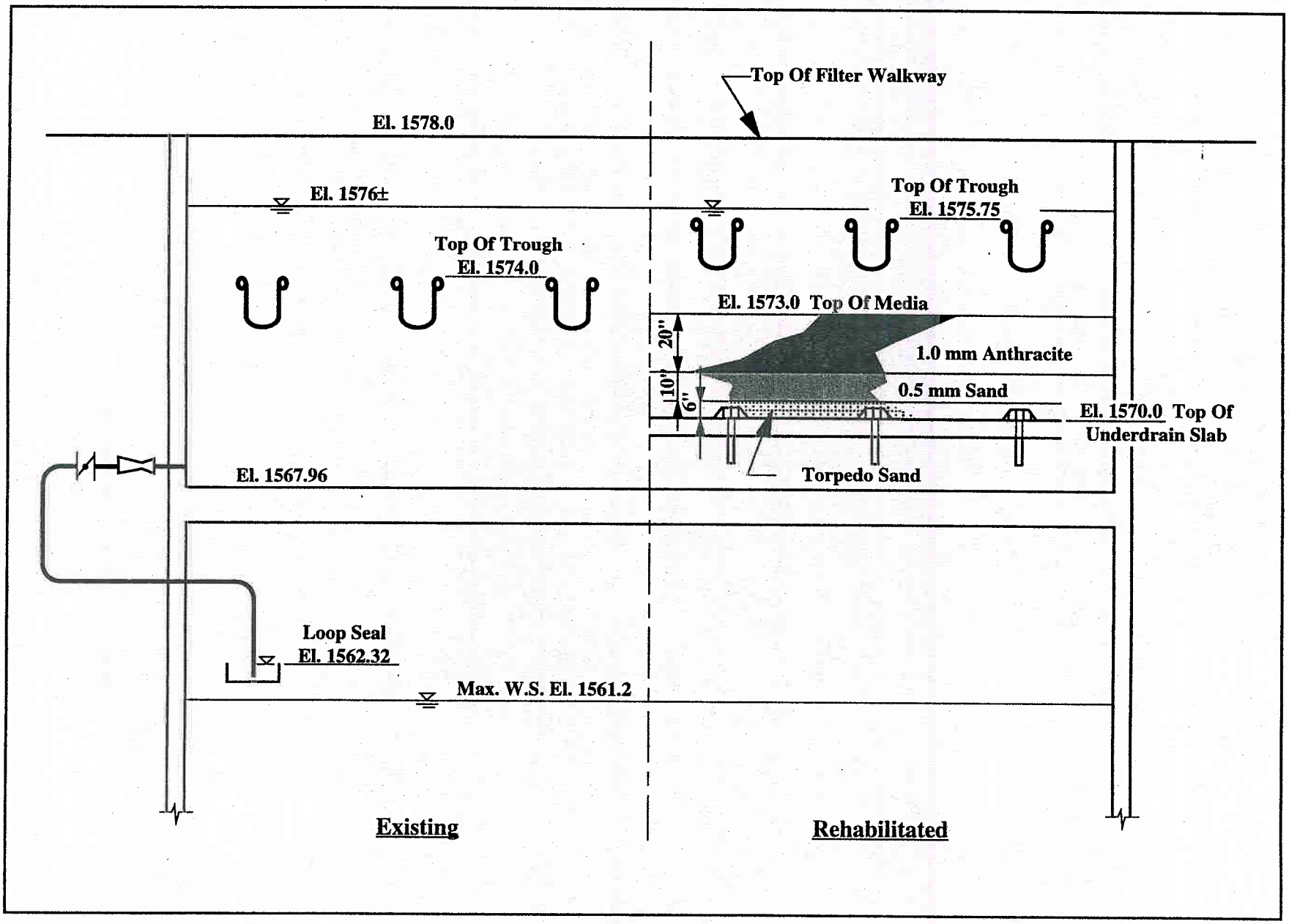
Filters. It is proposed to use the existing 12 filters at the Minot WTP without the addition of any new filters. At a capacity of 28 mgd, this will require a filtration rate of 4.5 gpm/sq ft with all filters in service or 4.9 gpm/sq ft with one filter out of service for backwash. These filtration rates would normally be considered low for a dual media filter, especially in service at a softening plant; however, they should be considered maximum rates at the Minot WTP because of air binding problems. Air-binding is a phenomenon in which large amounts of air bubbles accumulate in the filter bed during the later stages of filtration. The pressure in the upper portion of the filter bed may be reduced to below atmospheric pressure due to the creation of negative pressure. When this occurs, the air in the water no longer remains dissolved, but accumulates within the filter bed, rapidly increasing the headloss throughout the bed. If a high water level (i.e., ≥6 ft) is maintained in the filter above the filter bed, air-binding will be delayed.

One major difficulty with the existing filters is the shallow depth of the filter boxes; only 10 ft is provided from the bottom slab to the top deck. A section showing the elevations of critical elements in the existing filters and their proposed elevations is shown in the attached Figure 6-5. Although a decision as to whether or not the existing underdrains can be left intact will be made during final design, it is apparent at this time that the filter washwater troughs will need to be raised to allow an adequate backwash rate. In addition, the surface wash system should be evaluated for replacement.

The relative water surface elevations in the filter (el. 1576) and in the clearwell (el. 1563) with a 3 ft allowance for effluent pipe and underdrain/media losses (cleanbed headloss) provide approximately 10 ft of operating headloss for solids accumulation. Unfortunately, the 3 ft depth of water above the media with an allowance of 1 ft of negative pressure into the media leaves only 4 ft of operating headloss before air binding becomes a problem. In other words, the design of the existing filter boxes means that filter runs will always be limited by air binding.

Filter Waste Washwater Handling and Disposal. Currently, filter waste washwater (FWW) is stored in an equalization basin prior to being pumped to the head of the plant for retreatment. The FWW volume generated by a single backwash (including filter drawdown, surface wash, backwash, and filter-to-waste) is approximately 90,000 gallons. The single existing FWW equalization basin has a capacity of 108,000 gallons. In order to provide storage capacity for two backwashes it will be necessary to construct a new FWW equalization basin of equal volume to the existing basin. Otherwise, the inability to backwash filters would be a limiting factor in plant capacity.

Clearwell. The existing 600,000 gallon reinforced concrete clearwell is located directly beneath the filters. The existing clearwell provides a operating storage volume of only 3 percent of maximum day flow at the current plant capacity of 18 mgd. In order to provide a reasonable amount of operating storage it will be necessary to construct a new clearwell. Construction of a 1,000,000 gallon clearwell (1.6 million gallons total) will



MINOT WTP FILTERS
FIGURE 6-5

provide approximately 6 percent of maximum day capacity in operating storage at a flow rate of 28 mgd.

Finished Water Pumping Station. The existing finished water pumps are split case horizontal centrifugal pumps with flooded suction piped directly from the plant clearwell. Eight pumps are presently installed with a combined capacity of approximately 17.3 mgd. In order to reach the proposed plant capacity of 28 mgd it will be necessary to replace three of the pumps with larger units; 6 mgd pumps are proposed.

Sludge Handling and Disposal. Currently, sludge from the softening basins is routed to a sludge thickener where it is stored prior to being dewatered using vacuum filters. The dewatered sludge is trucked off site for landfill disposal. Because the vacuum filters are currently operated in a batch mode it will not be necessary to provide additional sludge dewatering capacity for an increased plant flow rate. The existing facilities can be operated for a longer period of time to handle the additional solids to be dewatered.

Currently, the sludge is disposed of at the municipal landfill. Measures to eliminate the possibility of biota transfer from the dewatered sludge to the Hudson Bay drainage are currently in place. The municipal landfill is constructed as a Class A landfill and includes containment berms, lined landfill cells, and leachate collection system with leachate disposal evaporation ponds. Some additional measures to the sludge handling procedures may also be required to address biota transfer concerns. This could include washing the under carriage and tires of trucks after they are loaded and covering the trucks.

Spill Containment. In order to eliminate the possibility of biota transfer to the Hudson Bay drainage from the Minot WTP it will be necessary to construct an overflow containment basin on-site to contain any overflows or spills from the plant process. A circular pre-stressed concrete basin without a roof slab is proposed as the most cost effective solution. To contain an overflow of two hours duration at the full plant capacity of 28 mgd would require a basin of approximately 2.4 million gallon capacity. A pumping station will be required to recover the contents of the containment basin for retreatment.

It will also be necessary to construct flood protection levees along the south bank of the Souris River both to protect the new plant facilities to be constructed within the current flood plain and also to eliminate the possibility of biota transfer through flooding of the plant site. The design flood elevation at this location is 1560.

Instrumentation and Electrical. In addition to the new electrical service and distribution equipment required to handle the new load at the expanded plant (especially ozone generation facilities), it will be necessary to rehabilitate and replace existing electrical equipment which has passed its useful life (typically considered to be 20 years). An evaluation of existing electrical systems will be conducted during design.

Significant instrumentation will be added with the new process equipment. If possible, the new instrumentation can be interfaced with the existing Healy Ruff SCADA system which controls both distribution (33 remote sites) and existing Minot WTP functions (e.g., chemical system flow pacing). However, a more straight forward approach may be to operate the Minot WTP with its own PLC based distributed control system (DCS) with only critical process parameters reported to the existing SCADA system.

Support Facilities. The increased capacity of the Minot WTP, together with increasing water quality monitoring requirements and handicapped access codes will require some modifications to plant support facilities. The following facilities are proposed:

- Public accessible (handicapped) rest rooms.
- Men's locker/shower room.
- Women's locker/shower room.
- Laboratory expansion.
- Conference room.
- Kitchen/break room.

WILLISTON WTP

The City of Williston WTP was originally constructed in 1961 with a design capacity of 6 mgd. The plant can be operated at rates above the design capacity, although historically 7 mgd has been the process capacity limit of the existing facilities. The last major improvements to the Williston WTP were the construction of a filter waste washwater equalization basin and chlorine storage and feeding facilities in 1980. The Williston WTP will be expanded from its present 7 mgd capacity to 13 mgd in order to satisfy the demands of the NAWS West system.

Water Supply

The Williston WTP currently treats surface water from the Missouri River. The river intake is located in the Missouri River channel and typically takes water directly from the river although as the water level in Lake Sakakawea rises, water is taken from the reservoir itself. Because of it diversion directly from the Missouri River, the Williston WTP treats much higher raw water turbidity than the Minot and Parshall WTPs.

Process Description

The Missouri River water treated at the Williston WTP typically has a turbidity of less 100 NTU although it is expected that turbidity can exceed this value during the spring runoff. The raw water hardness ranges from 200 to 250 mg/l (as $CaCO_3$). The existing main treatment process at the Williston WTP includes mechanical screening, presedimentation, softening in solids contact units, recarbonation, filtration, disinfection with chlorine, and distribution through the high service pumping station. Powdered activated carbon (PAC) is used to attempt control of the severe tastes and odors which can occur in Missouri River water.

Historically, the Williston WTP has utilized a lime-soda ash softening process to reduce total hardness to a level between 110 to 120 mg/l. However, in an effort to reduce operating expenses, the use of soda ash has been discontinued resulting in higher finished water hardness levels. Some consumer complaints have been received as a result of hardness fluctuation. At the present time, the Williston WTP has only a single softening basin. The addition of two new solids contact units capable of treating surface water through a conventional clarification process (coagulation, flocculation, sedimentation) will provide added flexibility. The process train at the Williston WTP will be arranged to allow the following alternatives:

- Softening of the Missouri River water at capacities up to 13 mgd.
- Sequential treatment with conventional clarification followed by softening in separate basins at capacities of up to 6 mgd.
- Two-stage softening with intermediate recarbonation at capacities up to 6 mgd.
- Split stream treatment with clarification and softening process in parallel, followed by blending prior to filtration.

At the present time, it is proposed that ozone will be used for primary disinfection followed by the addition of chloramines (combined chlorine and ammonia) for a distribution system disinfectant residual. The use of ozone is proposed for the following reasons:

- To provide required disinfection (CT compliance) without the expense of additional yard piping, clearwells, and baffling which would be required to allow CT compliance at a 13 mgd capacity with chlorine.
- To minimize the formation of disinfection by-products (DBPs) and eliminate the risk of noncompliance with more stringent water quality standards while treating Missouri River water.
- To provide a positive and reliable means of removing tastes and odors.

The proposed Williston WTP process is shown in Figure 6-6. The hydraulic profile is shown in Figure 6-7. A site plan and new treatment facility building plan are shown in Figures 6-8 and 6-9, respectively.

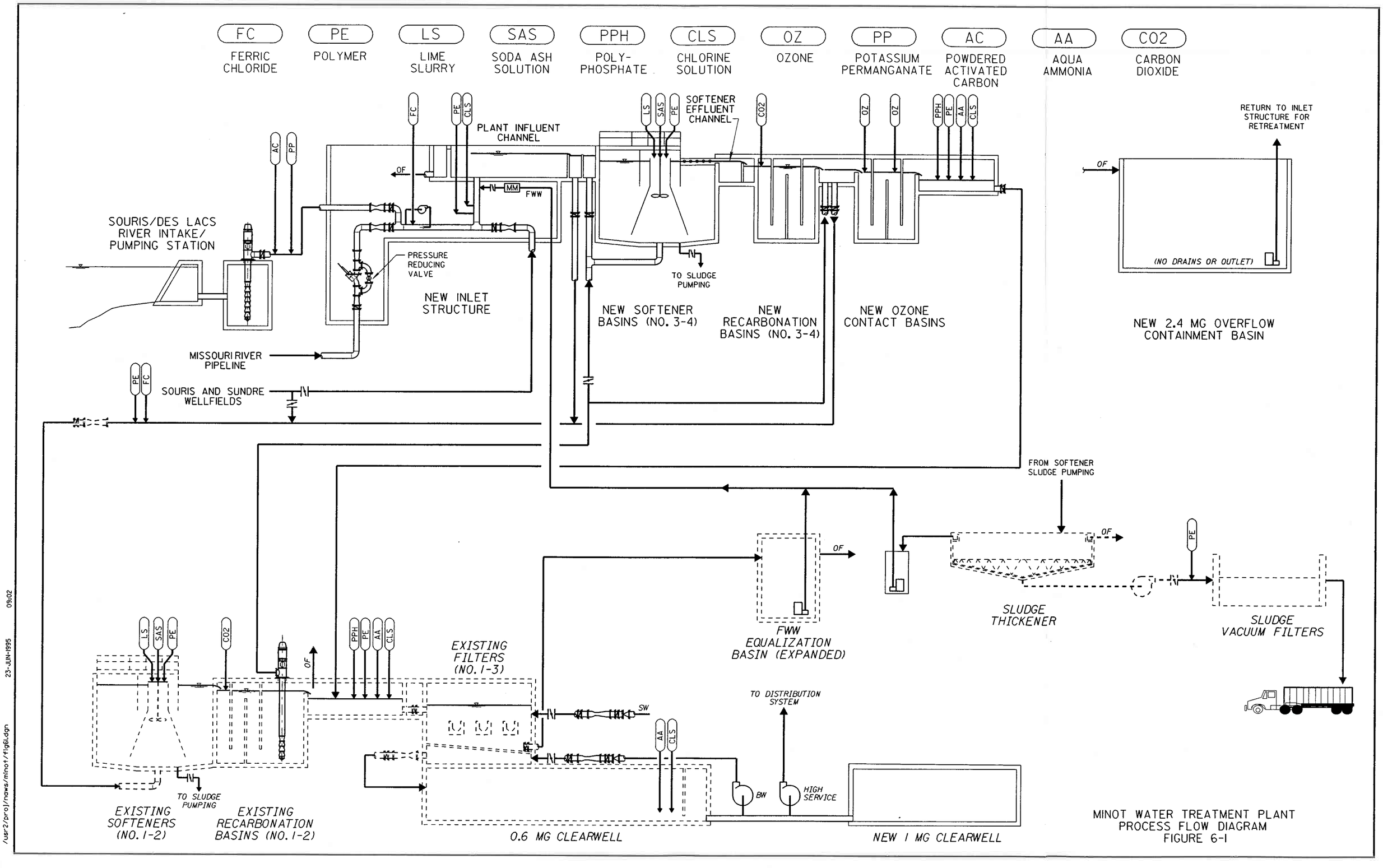
Process Elements

The following is a brief description of the process elements and modifications proposed for the Williston WTP. Complete design criteria are shown in Table 6-2.

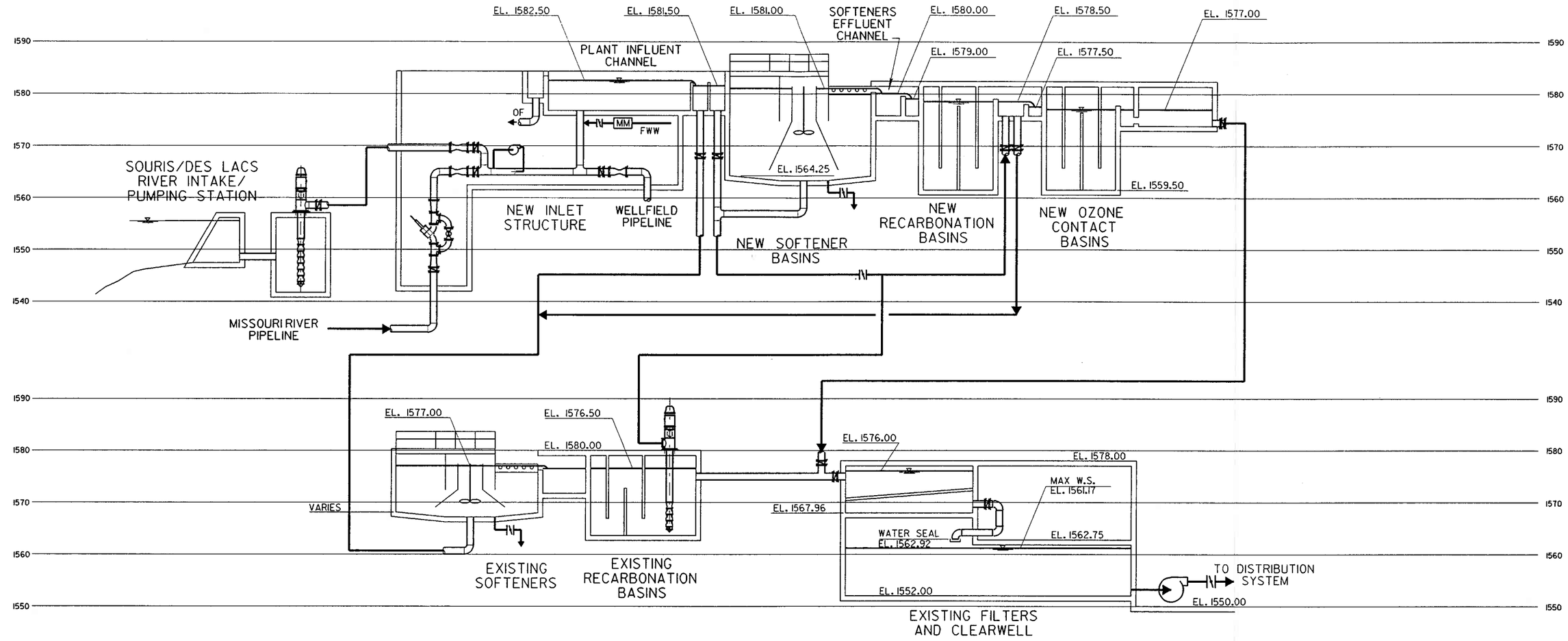
TABLE 6-2

WILLISTON WTP
DESIGN CRITERIA

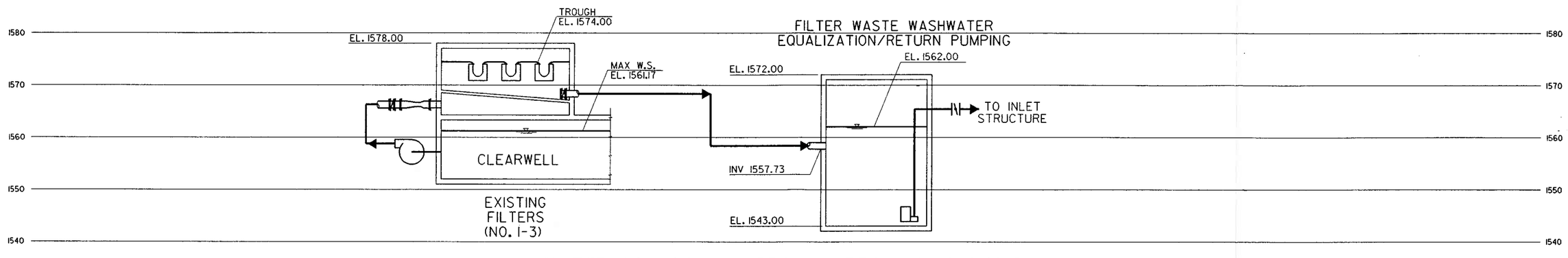
Description	Units	Current	Proposed
Plant Capacity			
Design Capacity	mgd	7	13
Raw Water Intakes			
Type: Free-standing tower with multiple inlet slide gates			
Number of Intake Depths	No.	3	3
Raw Water Pumping Station			
Mechanical Screens			
Type: Traveling screens with spray cleaning			
Number	No.	1	1
Screen Mesh	in x in	-	3/8
Raw Water Pumps			
Type: Submerged suction, horizontal centrifugal located in dry well			
Number	No.	3	4
Fixed Speed Capacity	mgd	3.6, 7.2, 7.2	3.6, 7.2, 7.2
Variable Speed Capacity	mgd	-	7.2
Presedimentation Basin			
Type: Circular, center feed concrete with mechanical scraper			
Number	No.	1	1
Diameter	ft	150	150
Water Depth (sidewall)	ft	12.5	12.5
Volume	mil. gal.	1.9	1.9
Surface Loading Rate	gpm/sq ft	0.27	0.51
Detention Time	hrs	6.5	3.6
Flash Mix (Coagulation)			
Existing Flash Mix (Abandon)			
Type: Flash Mix Box w/Mechanical Turbine Mixer			
Locations: Pre-softening			
Flash Mix (New)			
Type: Pumped Injection			
Number	No.	-	2
Location: Prior to Presedimentation and Prior to Softening			
Mixing Energy	sec^{-1}	-	750
Pump Power (each)	hp	-	5
Softening Basin			
Type: Solids Contact Unit (IDI Accelator, IS Type)			
Number	No.	1	3
Dimensions	ft x ft	53 x 53	53 x 53
Water Depth (sidewall)	ft	16.5	16.5



MINOT WATER TREATMENT PLANT
PROCESS FLOW DIAGRAM
FIGURE 6-1



MAIN PROCESS FLOW HYDRAULIC PROFILE (28 MGD)



FILTER BACKWASH HYDRAULIC PROFILE (10.4 MGD)

SCALE IN FEET
0 10' 20'
SCALE VERTICAL
SCALE HORIZONTAL : NONE

MINOT WATER TREATMENT PLANT
HYDRAULIC PROFILE
FIGURE 6-2

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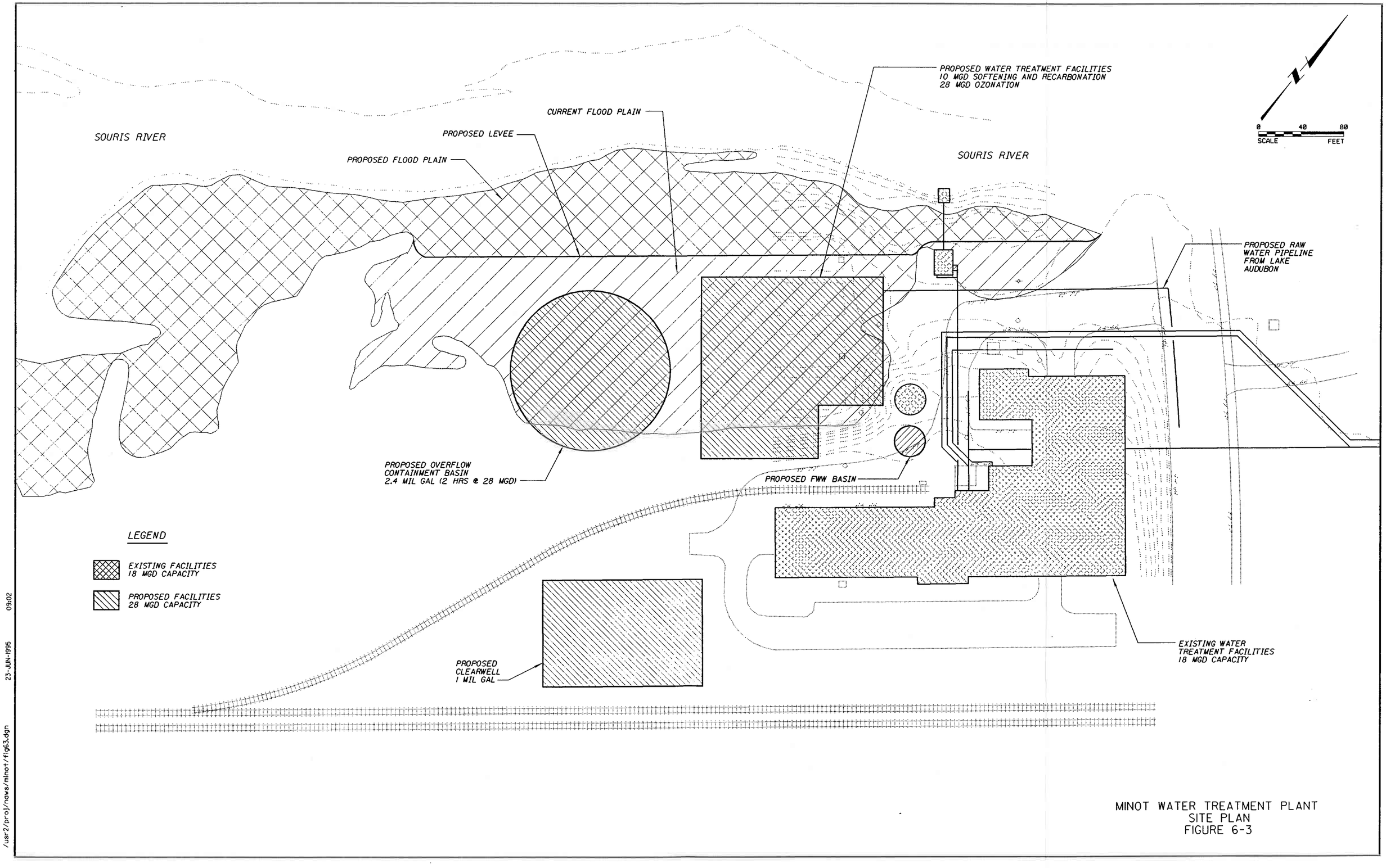

MINOT WATER TREATMENT PLANT
SITE PLAN
FIGURE 6-3

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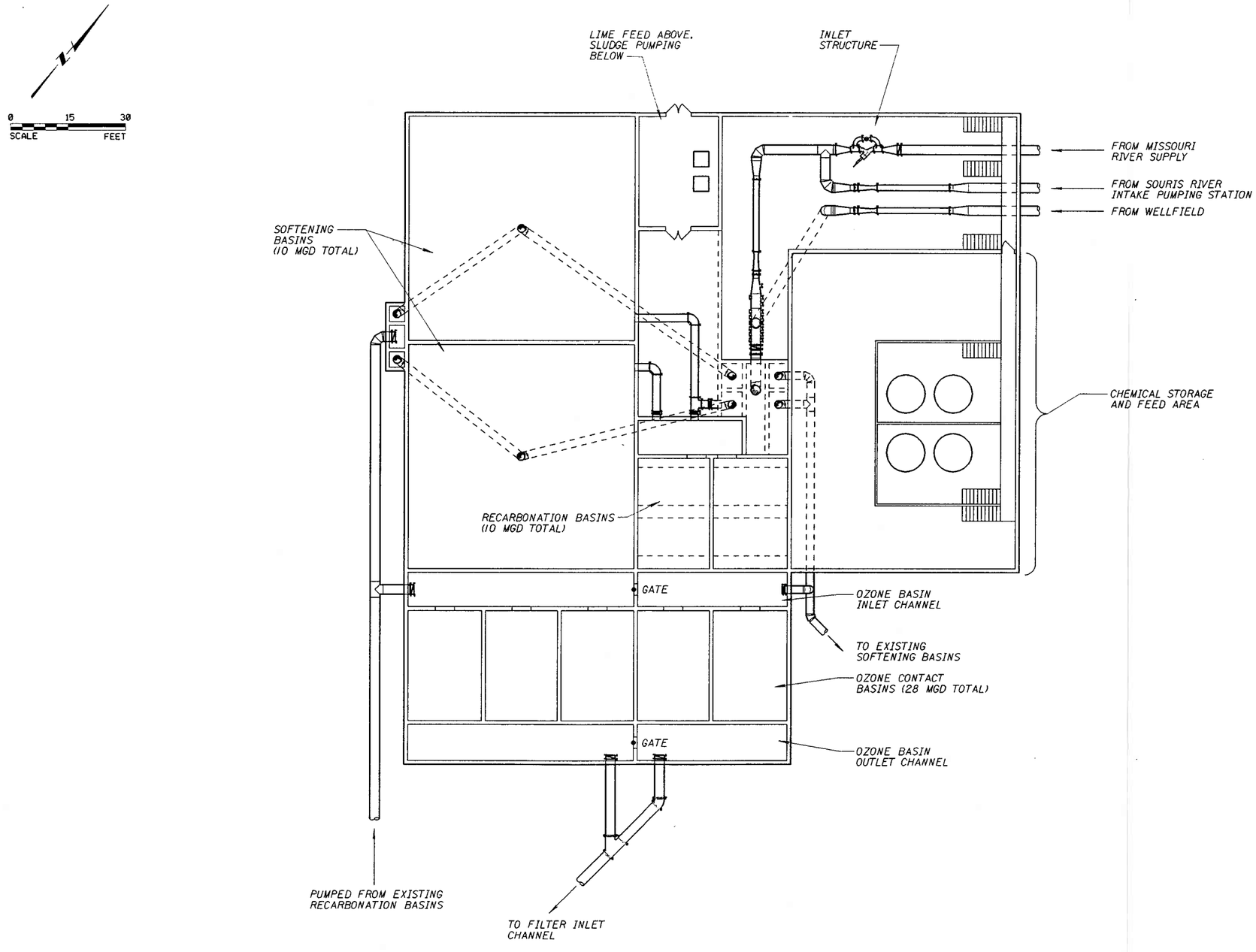

MINOT WATER TREATMENT PLANT
NEW FACILITY BUILDING
FIGURE 6-4

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TABLE 6-2
(Continued)

Description	Units	Current	Proposed
Available Area (each)	sq ft	2,200	2,200
Volume (each)	gal	340,000	340,000
Design Flow			
Per Basin	mgd	6	4.3
Total	mgd	6	13
Surface Loading Rate (@ Design Flow)	gpm/sq ft	1.9	1.4
Detention Time	hrs	1.2	1.9
Recarbonation Basin (Existing)			
Type: Rectangular, concrete basin			
Number	No.	1	2
Basin Dimensions (w x l)	ft x ft	10.5 x 53	20 x 30
Water Depth	ft	16	18
Volume (each)	gal	67,000	82,000
Detention Time	min	16	20
Ozone Contacter			
Type: Two-Stage, rectangular, baffled concrete basins			
Number of Basins	No.	-	2
Basin Dimensions (w x l)	ft x ft	-	20 x 30
Water Depth	ft	-	18
Volume (each)	gal	-	82,000
Detention Time	min	-	20
Filters			
Type: Gravity, granular media, rate-of-flow control			
Filters No. 1-3 (Existing)			
Number of Filters	No.	3	3
Cells Per Filter	No.	2	2
Cell Dimensions (w x l)	ft x ft	13.0 x 28.0	13.0 x 28.0
Cell Area (each)	sq ft	364	364
Filters No. 4-5 (New)			
Number of Filters	No.	-	2
Cells Per Filter	No.	-	1
Filter Dimensions	ft x ft	-	13.0 x 28.0
Filter Area (each)	sq ft	-	364
Total Filter Area	sq ft	2,184	2,912
Filtration Rate			
All Filters In Service	gpm/sq ft	2.2	3.1
One Filter In Backwash	gpm/sq ft	3.3	4.1

TABLE 6-2
(Continued)

Description	Units	Current	Proposed
Filter Media			
Filter No. 1-3 (Existing)			
Anthracite Coal			
Depth	in		20
Effective Size	mm		1.0
L/d			508
Sand			
Depth	in		10
Effective Size	mm		0.5
L/d			508
Total			
Depth	in		30
L/d			1016
Filter No. 4-5 (New)			
Anthracite Coal			
Depth	in		30
Effective Size	mm		1.0
L/d			762
Sand			
Depth	in		15
Effective Size	mm		0.5
L/d			762
Total			
Depth	in		45
L/d			1,524
Filter Backwash			
Type: Water Wash			
Backwash Pumps	No.	1	2
Pump Capacity (each)	gpm	5,500	7,300
Backwash Rate	gpm/sq ft	15	20
Type: Surface Wash, Rotating Arm			
Surface Wash Pumps	No.	1	2
Storage Tank	No.	1	1
Storage Tank Volume	gal	50,000	50,000
Pump Capacity (each)	gpm	350	350
Surface Wash Rate	gpm/sq ft	1.0	1.0
Filter Waste Washwater (FWW) Handling			
Equalization Basin (Existing)			
Type: Rectangular concrete basin with circular mechanical scraper			
Number	No.	1	1
Dimensions	ft x ft	40 x 40	40 x 40
Water Depth (sidewall)	ft	10	10
Volume	gal	120,000	120,000

TABLE 6-2
(Continued)

Description	Units	Current	Proposed
Backwashes Held	No.	1	1
Equalization Basin (New)			
Type: Rectangular concrete basin with sloped floor			
Number	No.	-	1
Dimensions	ft x ft	-	35 x 35
Water Depth (min. sidewall)	ft	-	12
Volume	gal	-	120,000
Backwashes Held	No.	-	1
Equalization Basins (Total)			
Number	No.	1	2
Volume	gal	120,000	240,000
Backwashes Held	No.	1	2
FWW Reclamation Pumps			
Type: End Suction Centrifugal			
Number	No.	2	4
Capacity	mgd	0.3, 0.6	0.3, 0.3, 0.6, 0.6
Time To Empty FWW Basins	hr	3.3	3.3
Percent of Plant Flow	%	15	13
Clearwell			
Type: Concrete basin, partially below filters			
Number	No.	1	1
Water Depth (Varies)	ft	17	17
Area	sq ft	4,725	4,725
Volume (max.)	gal	600,000	600,000
Detention Time (max.)	hrs	2.4	1.0
Percent of Max. Daily Production	%	10	4
Finished Water Pumps			
Type: Vertical Turbine			
Number	No.	3	4
Capacity	mgd	2.2, 4.0, 7.2	2.2, 4.0, 7.2, 7.2

Raw Water Intake. The raw water intake currently in use is a free standing tower with multiple inlet gates. The three inlet gates are positioned at different elevations to allow selection of intake elevation. Normally, the middle gate is left open because the top gate tends to pick up floating debris while the bottom inlet produces silt and sand. No modifications are proposed to the raw water intake structure except to improve the ability to backflush the intake as necessary to control formation of frazil ice.

Raw Water Pumping Station. The raw water pumping station includes a wetwell with a traveling mechanical screen and a pump dry well containing the raw water pumps. The mechanical screen is essential for removal of floating debris, however it is only semi-operable and will be replaced with new equipment. The three existing pumps, located in the dry well, are horizontal centrifugal pumps with flooded suctions. Combined pumping capacity is 18 mgd. A new 7.2 mgd pump with variable speed drive to allow adjustment of flow over a 10 to 1 turndown range will be installed in an empty slot available for its use.

Grit Basin. The original design of the Williston WTP included installation of a grit chamber. Unfortunately, the grit chamber proved to be ineffective at removing grit and its use was abandoned. However, lack of a grit chamber allows the high sediment loads present in the river during the spring runoff to pass to the presedimentation basin where the sand and silt can settle to a depth which will stop the scraper drives. It will be necessary to install a properly designed grit chamber of the rectangular, horizontal flow type with baffled inlet and outlet. Provisions for removing grit and sediment to the sludge lagoons by means of grit pumps or hydraulic eductors should be installed.

Presedimentation. The existing 1.9 million gallon presedimentation basin is a circular unit with a mechanical scraper. Currently, silt and sediment is scraped to the unit center and discharged back to the Missouri River. The existing basin operates at a surface loading rate of 0.27 gpm/sq ft. Instead of constructing an additional presedimentation basin, it is proposed to increase the surface loading rate through the existing unit at a flow rate of 13 mgd to 0.5 gpm/sq ft. To do so will require the consistent application of a coagulant and/or coagulant aid. In order to accomplish this a flash mix and chemical application point for coagulant and polymer will be installed between the grit chamber and the presedimentation basin. The addition of coagulants will provide more efficient removal of solids through the presedimentation basin; however, it will be necessary to discharge the sludge from the presedimentation basin to the lagoons rather than to the river.

Inlet Structure. An inlet structure is proposed to provide the following functions:

- Raw water metering.
- Raw water chemical addition point, including presoftening flash mix for coagulation.

- A plant influent channel with flow splitting weirs to provide accurate distribution of flow between the softening basins.

Flash Mix. The existing flash mix box with mechanical turbine mixer is ineffective and will be abandoned. Two new pump diffusion flash mix systems will be installed, one prior to presedimentation and one (located in the inlet structure) prior to softening. Pumped diffusion flash mix is recommended because it has the advantages of lower energy requirement, higher efficiency, nonproprietary parts, and lower maintenance costs than alternative mechanical systems. A mixing energy value (G) of 750 sec^{-1} is proposed.

Chemical Feed. The following chemical feed systems are proposed for the Williston WTP. Chemicals which are currently used at the plant are shown underlined.

- Ferric chloride - used as a primary coagulant.
- Polymer (liquid and dry) - used as a coagulant aid, flocculant aid, filter aid, and sludge conditioner.
- Lime (quick lime) - used in softening for removal of carbonate hardness.
- Soda ash (sodium carbonate) - used in softening for removal of noncarbonate hardness.
- Polyphosphate - used to prevent precipitation of carbonate hardness.
- Chlorine - used as a disinfectant.
- Aqua ammonia - used with chlorine to form a chloramine distribution system disinfectant.
- Ozone - used as a primary disinfectant, for control of disinfection by-products, and for control of tastes and odors.
- Powdered activated carbon (PAC) - currently used for taste and odor control.
- Carbon dioxide - used for recarbonation (pH adjustment and stabilization of precipitation potential).
- Fluoride - used for prevention of dental caries.

All chemical feed systems will need to be expanded to handle the delivery requirements of the increased plant capacity. In addition, all chemical feed systems will be upgraded to comply with current code requirements (e.g., Uniform Fire Code, Article 80 - Hazardous Materials). As an example, a chlorine scrubber will be required to prevent release of gas from the chlorine storage and feed facilities. Because of the major increase

in capacity relative to existing facilities, a new chemical storage and feed area will be provided in the new treatment facility building for all chemical systems with the exception of the existing lime and soda ash feed system used for the existing softening basin.

Softening Basin. The single, existing softening basin is a solids contact unit with the capacity of 7 mgd (IDI Accelator, IS Type). Two additional solids contact units identical to the existing basin are proposed as Basins No. 2 and 3. At a combined flow of 13 mgd, these basins will have a surface loading rate of 1.4 gpm/sq ft. The new basins (Nos. 2 and 3) will be configured to allow operation either in series or in parallel.

Recarbonation Basins. Currently, the Williston WTP has a single recarbonation basin located in the process between the softening basin and the filters. At a flow rate of 7 mgd, the existing recarbonation basin (No. 1) has a detention time of 16 minutes. Two new basins (Nos. 2 and 3) are proposed. In order to eliminate flow splitting difficulties between the existing softening basin and the two new softening basins prior to recarbonation, it is proposed to size the new recarbonation basins for the total capacity of the plant (13 mgd). At this flow rate (6.5 mgd each), the recarbonation basins will have a detention time of 15 minutes.

Ozone Contactors. The ozone contact basins will be constructed at a location downstream of the recarbonation process and upstream of the filters. This location allows prior reduction in ozone demand through the clarification process with subsequent removal of some ozonation by-products (e.g., aldehydes) across the filter prior to distribution. However, because of the limited head available (1 ft) between the existing recarbonation basins and the existing filters, it will be necessary to install low lift pumps in the existing recarbonation basin to pump water to the new recarbonation and ozone contact basins.

In order to use the existing presedimentation basin to serve the new softening basins at the higher hydraulic gradeline that will be required to allow ozonation between softening and filtration, it will be necessary to raise the water surface in the presedimentation basin by approximately 3.5 feet. It appears that this will be possible by modifying the effluent launders, changing the access to the basin center well, and modifying the sludge scraper drives.

The proposed ozone contact basins will be two-stage, rectangular, baffled concrete basins. Two basins will be provided, each with a capacity of 6.5 mgd. A detention time of 20 minutes will be provided for disinfection and taste and odor control. The ozone generation equipment will be located directly over the contact basins. Subject to additional analysis, it is proposed that the most cost-effective system will be medium frequency generation of ozone from gaseous oxygen (GOX) produced on-site by the pressure swing adsorption (PSA) method with supplemental capacity and redundancy provided by on-site storage of liquid oxygen (LOX).

Filters. The existing Williston WTP has three filters, with two cells per filter to minimize backwash capacity requirements. Two new filters are proposed, each having the same dimensions as one of the existing filter cells. The construction of two filters rather than a single filter with two cells will increase the opportunity to shutdown the existing filters during periods of low flow (e.g., during the winter) so that all plant operations can be conducted at the new facility building.

The existing filters have a filter box depth of only 11 ft which limits the operating performance. The relative water surface elevations in the filter (El 1895) and in the clearwell (El 1880) with a 3 ft allowance for effluent pipe and underdrain/media losses (clean bed headloss) provides approximately 12 ft of operating headloss for solids accumulation. However, the 4.5 ft depth of water above the media with an allowance of 1 ft of negative pressure into the media leaves only 5.5 ft of operating headloss before air binding becomes a problem. In other words, the design of the existing filter boxes means that filter runs will always be limited by air binding.

This difficulty can be eliminated in the design of the new filter boxes by providing additional box depth to allow a greater water depth over the media and take advantage of the large differential between the filter water surface and the clearwell water surface. Filter runs can thus be extended for the new filters by eliminating air binding as the filter performance constraint. These problems cannot be addressed in the existing filters; however, the effects of air binding can be minimized by maintaining filtration rates at or below approximately 4 gpm/sq ft. The current filtration rate with all filters in service is 2.2 gpm/sq ft at 7 mgd. At 13 mgd, with the construction of the two new filters, the total filtration rate will be 3.1 gpm/sq ft with all filters in service or 4.1 gpm/sq ft with one filter out of service for backwash. Section views of the existing, rehabilitated, and new filters are shown in Figure 6-10.

Improvements to the existing filters will include the following:

- Replacement of existing filter media and underdrains
- Raising the backwash troughs to allow higher backwash rates without media loss
- Replacing the surface wash system
- Replacing backwash supply pump and providing redundancy

Filter Waste Washwater Handling and Disposal. Currently, filter waste washwater (FWW) is stored in an equalization basin prior to being pumped to the head of the plant for retreatment. The FWW volume generated by a single backwash is approximately 90,000 gallons. The single existing FWW equalization basin has a capacity of 120,000 gallons. In order to provide storage capacity for two backwashes, it will be necessary to construct a new FWW equalization basin of equal volume to the existing basin. Otherwise, the inability to backwash filters would be a limiting factor in plant capacity.

Clearwell. The existing 600,000 gallon reinforced concrete clearwell is located directly beneath the filters. The existing clearwell provides an operating storage volume of 10

percent of maximum day flow at the current plant capacity of 7 mgd. At the expanded capacity of 13 mgd it will provide four percent of maximum day flow for operational storage. The use of ozone for disinfection will allow the entire volume of the clearwell to be used for operational storage.

Finished Water Pumping Station. The existing finished water pumps are vertical turbine units with the pump bowls located within the plant clearwell. Three pumps are presently installed with a combined capacity of approximately 13.4 mgd. An additional pump will be added to provide redundancy for the capacity of the largest existing pump (7.2 mgd).

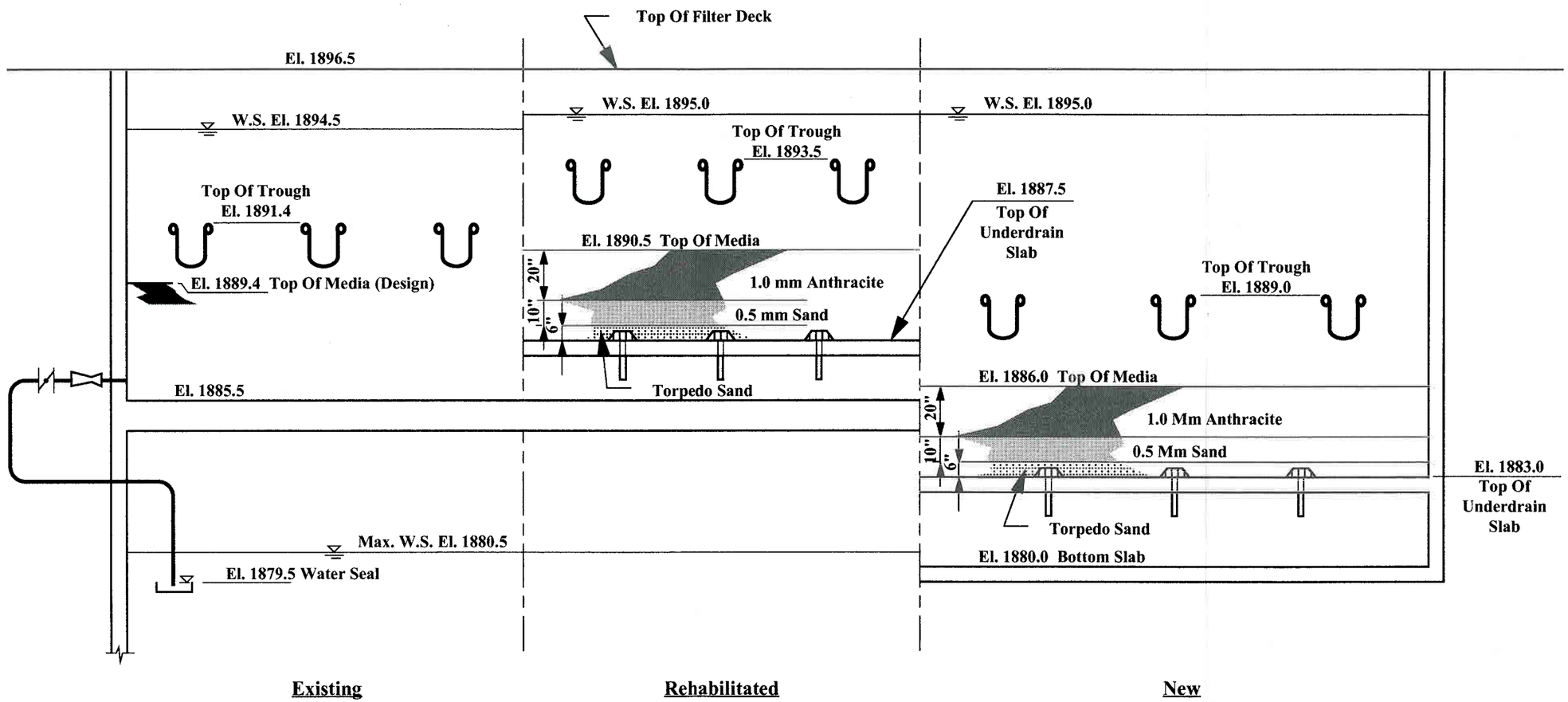
Sludge Handling and Disposal. Currently, sludge from the softening basins is routed to a sludge pumping pit and then pumped through an existing 6-inch force main to the off-site sludge lagoons. If operated to a water depth of six feet (one foot of freeboard) the four lagoons each provide a storage volume of five million gallons, or a total storage capacity of 20 million gallons. The existing lagoons are unlined and percolation and evaporation have minimized the need to return decant to the plant for retreatment.

It is proposed to line the existing sludge lagoons sides and bottoms with fiber reinforced concrete to allow them to operate as a dewatering system with subsequent removal of sludge by front end loaders for ultimate disposal off-site. The existing lagoons have never been cleaned and experience has shown that an unlined lagoon (particularly one with a large depth of sludge) is difficult or impossible to dewater and clean out with ordinary equipment. In addition, decant structures with adjustable weirs will be provided to allow a rapid decanting of standing water after a lagoon is taken off-line for dewatering of the sludge. Ultimate sludge disposal off-site can be as an agricultural land application or to a municipal landfill or dedicated monofill.

Instrumentation and Electrical. In addition to the new electrical service and distribution system to handle the new load at the expanded plant (especially ozone generation facilities), it will be necessary to rehabilitate and replace existing electrical equipment which has passed its useful life (typically considered to be 20 years). An evaluation of existing electrical systems will be conducted during design. Significant instrumentation will be added with the new process equipment. All new facilities and key existing facilities will be controlled through new PLC based distributed control systems (DCS). The plant DCS system can interface with the distribution SCADA system which will be provided in the Williston WTP control center; however, it is not proposed that the SCADA system will control the WTP.

Support Facilities. The increased capacity of the Williston WTP, together with increasing water quality monitoring requirements and handicap access codes will require some modifications to the plant support facilities. The following facilities are proposed to be included in the new building:

- Public accessible (handicapped) rest rooms
- Men's locker/shower room



WILLISTON FILTERS
FIGURE 6-10

- Women's locker/shower room
- Laboratory
- Conference room
- Control room
- Kitchen/break room
- Administrative offices

PARSHALL WTP

The City of Parshall WTP was constructed in the mid-1980's with a capacity of 400 gal/min (approximately 0.5 mgd) to treat both surface water diverted from the Van Hook Arm of Lake Sakakawea as well as local groundwater. The local groundwater supply has been abandoned and the sole source of water to the WTP is surface water. The Parshall WTP will be expanded to a capacity of 1.5 mgd in order to satisfy the demands of the Parshall portion of the NAWS system.

Process Description

Typically, the raw water turbidity entering the Parshall WTP is less than 2 NTU. The existing main treatment process includes softening in a solids contact unit, stabilization with sulfuric acid, filtration, disinfection with chlorine, and distribution through the high service pumping station. Potassium permanganate has been added at the river intake in an attempt to control taste and odors. Normally, softening objectives can be met with the use of lime only and soda ash is utilized only in the summer.

At the present time, it is proposed that ozone will be used for primary disinfection followed by the addition of chloramines (combined chlorine and ammonia) for a distribution system disinfectant residual. The use of ozone is proposed for the following reasons:

- To provide required disinfection (CT compliance) without the expense of additional yard piping, clearwells, and baffling which would be required to allow CT compliance at 1.5 mgd capacity with chlorine.

- To minimize the formation of disinfection by-products (DBPs) and eliminate the risk of noncompliance with more stringent water quality standards while treating Missouri River water.
- To provide a positive and reliable means of removing taste and odors in the surface water supply.

The proposed Parshall WTP process is shown in Figure 6-11. The hydraulic profile is shown in Figure 6-12. The proposed facilities will operate in parallel to the existing solids contact unit and filters which are not shown in Figures 6-11 and 6-12. A preliminary site plan is shown in Figure 6-13 for the proposed parallel facilities at the Parshall WTP site.

Process Elements

The following is a brief description of the process elements and modifications proposed for the Parshall WTP. Complete design criteria are shown in Table 6-3.

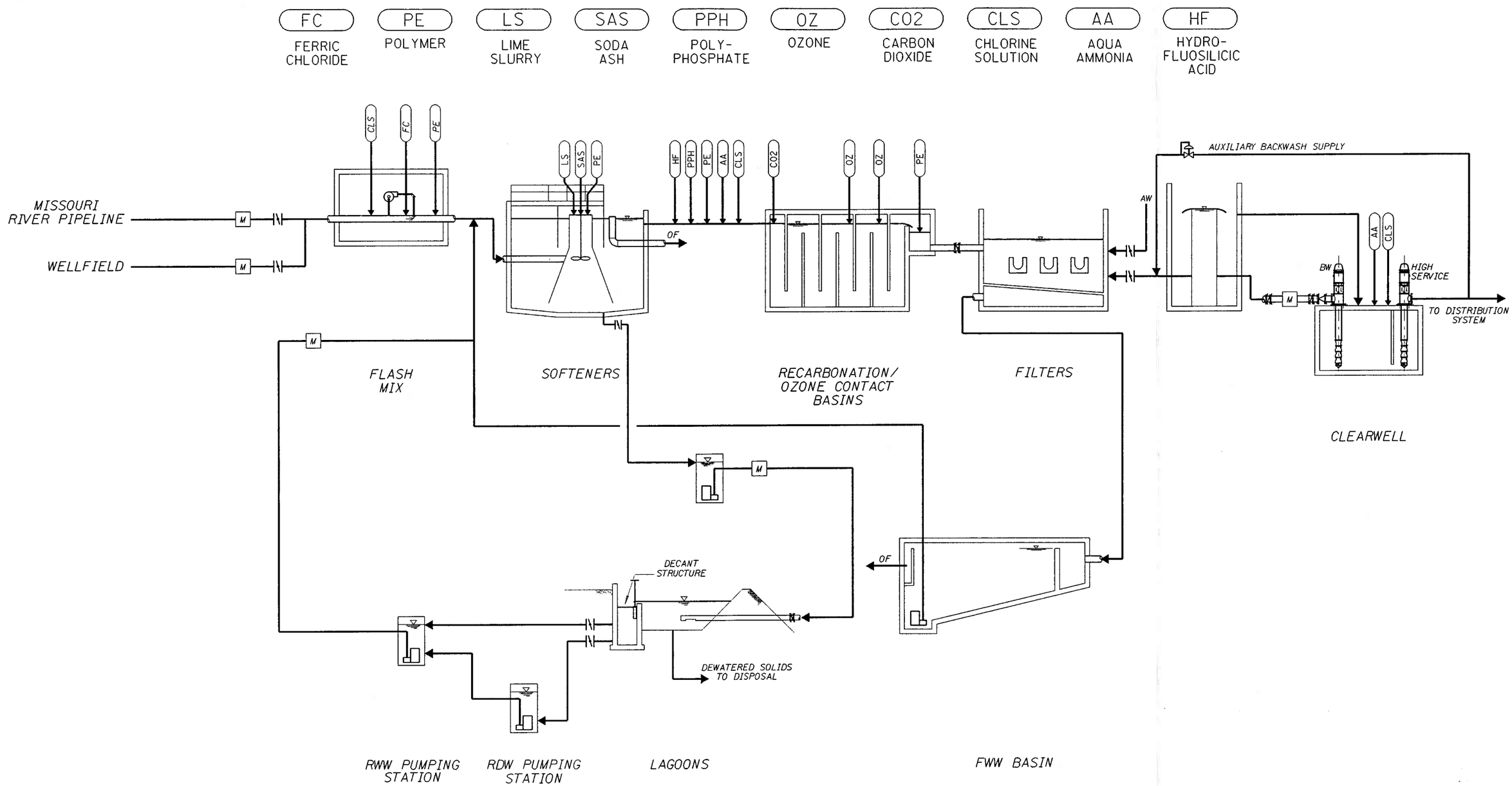
Raw Water Intake. The City's water intake structure is located near the east shore of Lake Sakakawea, near the intersection of Shell Creek Bay and the Van Hook Arm. Water is withdrawn from the lake intake structure through a raw water lift station (located on the shore approximately 630 ft from the intake structure) and is pumped to a transfer station (located approximately 50 ft landward from the raw water lift station). From the transfer station, the water is repumped through approximately 11 miles of pipeline to the City of Parshall's water treatment plant.

The existing intake structure consists of 21-inch tall by 21-inch diameter drum screen mounted at the inlet to the suction piping. The bottom of the intake screen is located approximately 1 ft above the lake floor at elevation 1810. The suction piping consists of approximately 630 ft of 8-inch buried polyethylene pipe. The raw water lift station consists of two self-priming centrifugal pumps and associated appurtenances. This equipment is located within a 10-ft-diameter, 32-ft-deep, precast concrete structure. A sump pump is located at the bottom of the structure to keep the inside dry. The pumps are rated for 400 gpm with one pump operating.

A report prepared by the U.S. Army Corps of Engineers (COE) in January 1993 discussed several methods to address problems of ensuring an adequate raw water supply at low lake levels. For the purposes of the present report, it is assumed a modified version of Alternative 1 discussed in the COE report will be constructed consisting of the extension of a new 10-inch pipe to lake floor elevation 1800. This extension would allow operation to an estimated lake elevation of 1808.5 or roughly 8.5 ft below the minimum lake elevation at which the existing intake has operated with a bubbler system.

The proposed intake system (Figure 6-14) would provide three 15 hp submersible pumps in a submerged concrete structure and approximately 3400 ft of buried 10-inch diameter polyethylene intake pipe which would bypass the existing raw water lift station and discharge directly into the wetwell of the existing raw water transfer station. The existing raw water lift station should be maintained as a backup water supply intake.

Raw Water Pumping Station. The existing raw water transfer station serves as a temporary holding facility for pretreatment and transfer pumping. Beneath the transfer station is a 40,000 gallon concrete storage facility that provides detention time for pretreatment as well as equalization storage. Two electric motor-driven vertical turbine pumps are also housed within the facility to transfer water from the station to the City of Parshall's water treatment facility, approximately 11 miles away. The total pumping capacity of this facility is 400 gpm with both pumps in operation. The controls within the transfer station are automatic, and the transfer pumps can be operated from the City's water treatment plant.



PARSHALL WATER TREATMENT PLANT
PROCESS FLOW DIAGRAM
FIGURE 6-11

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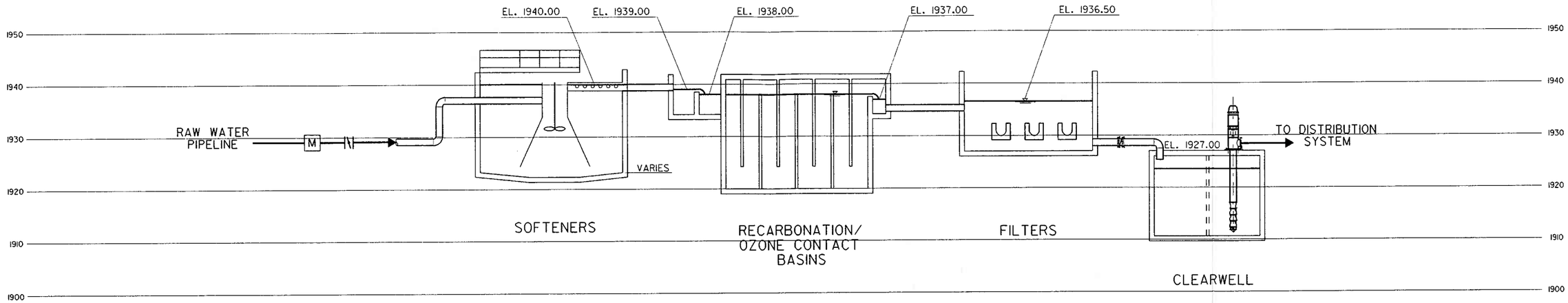
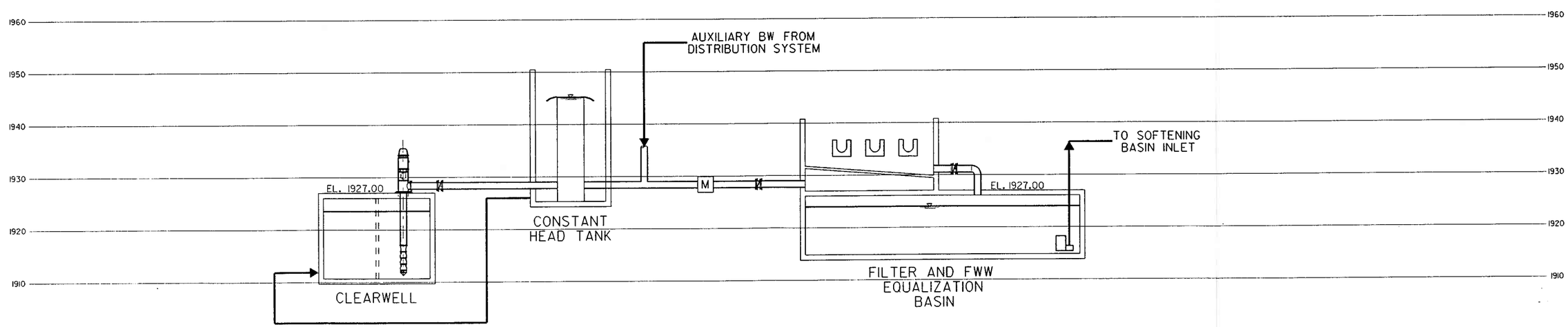


MAIN PROCESS FLOW HYDRAULIC PROFILE (1.5 MGD)



FILTER BACKWASH HYDRAULIC PROFILE (2.8 MGD)

SCALE IN FEET
0 10' 20'
SCALE VERTICAL
SCALE HORIZONTAL : NONE

PARSHALL WATER TREATMENT PLANT
HYDRAULIC PROFILE
FIGURE 6-12

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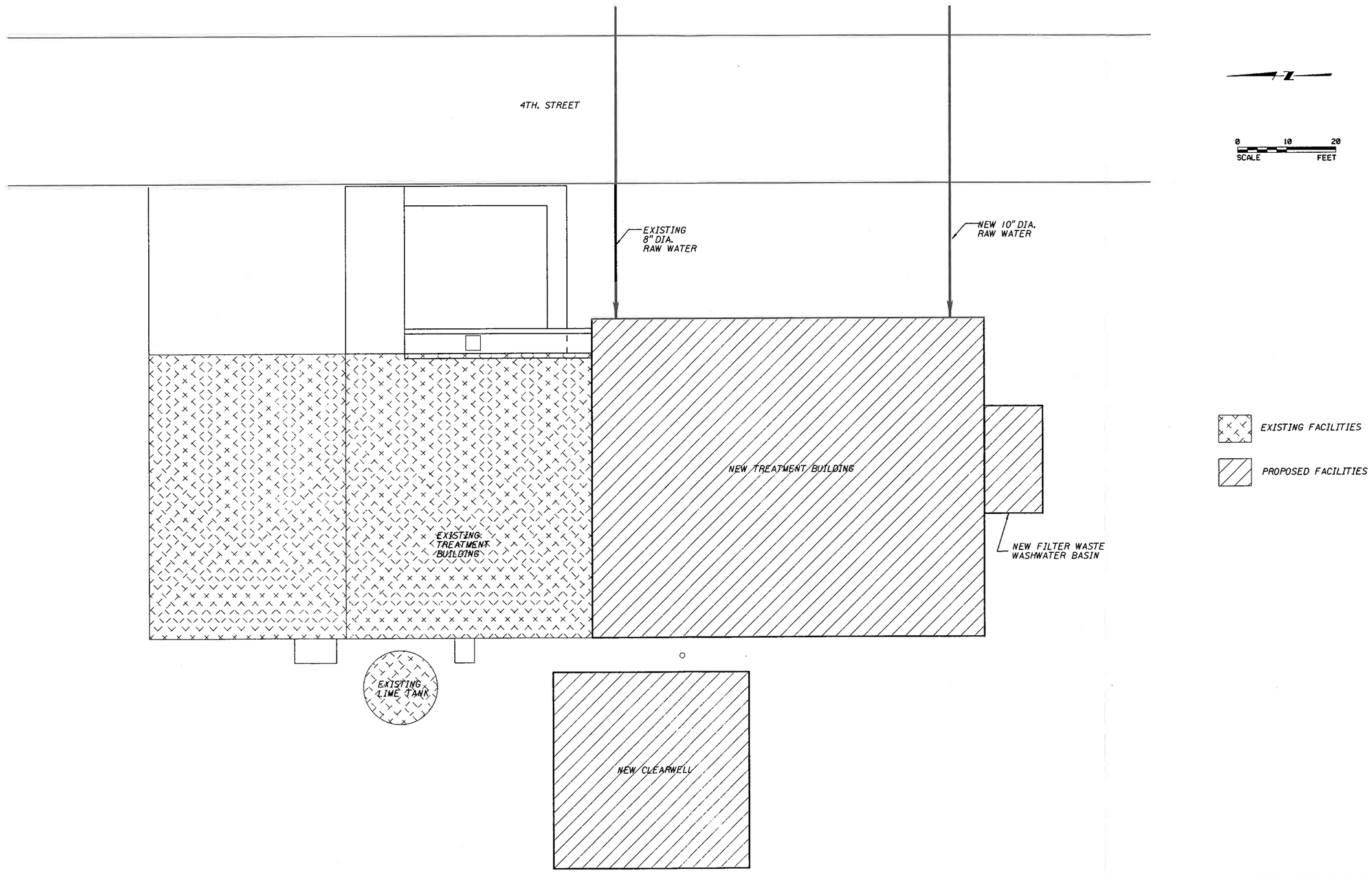

PARSHALL WATER TREATMENT PLANT
SITE PLAN
FIGURE 6-13

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TABLE 6-3

PARSHALL WTP
DESIGN CRITERIA

Description	Units	Current	Proposed
Plant Capacity			
Design Capacity	mgd	0.58	1.5
	gpm	400	1,000
Raw Water Intake			
Type: Screened Intake Drum (Existing)	-	-	-
Intake Crib (Proposed)	-	-	-
Intake Depth	MSL	1812.5	1803.0
Pipe Diameter	in	8	10
Raw Water Pumps			
Horizontal Centrifugal (Existing)	No.	2	2
Capacity	mgd	0.6, 0.6	0.6, 0.6
Submersible	No.		3
Capacity	mgd		0.8, 0.8, 0.8
Raw Water Pumping Station			
Raw Water Pumps			
Vertical Turbine	No.	2	4
Capacity	mg	0.3, 0.3	0.3, 0.3, 1.0, 1.0
Transfer Pipeline			
Length	ft	55,700	55,700
Diameter	in	8"	8", 10"
Flash Mix (Coagulation)			
Type: Pumped Diffusion			
Location: Pre-softening			
Number	No.	-	1
Mixing Energy	sec^{-1}	-	750
Pump Power	hp	-	0.75
Softening Basins			
Type: Solids Contact Units (IDI Accelators, IS Type)			
Number	No.	1	2
Dimensions (Diameter)			
Basin 1 (Existing)	ft	20	20
Basin 2 (New)	ft	-	30
Water Depth (Sidewall)			
Basin 1	ft	12	12
Basin 2	ft	-	13.5
Available Area			
Basin 1	sq ft	260	260
Basin 2	sq ft	-	590

TABLE 6-3
(Continued)

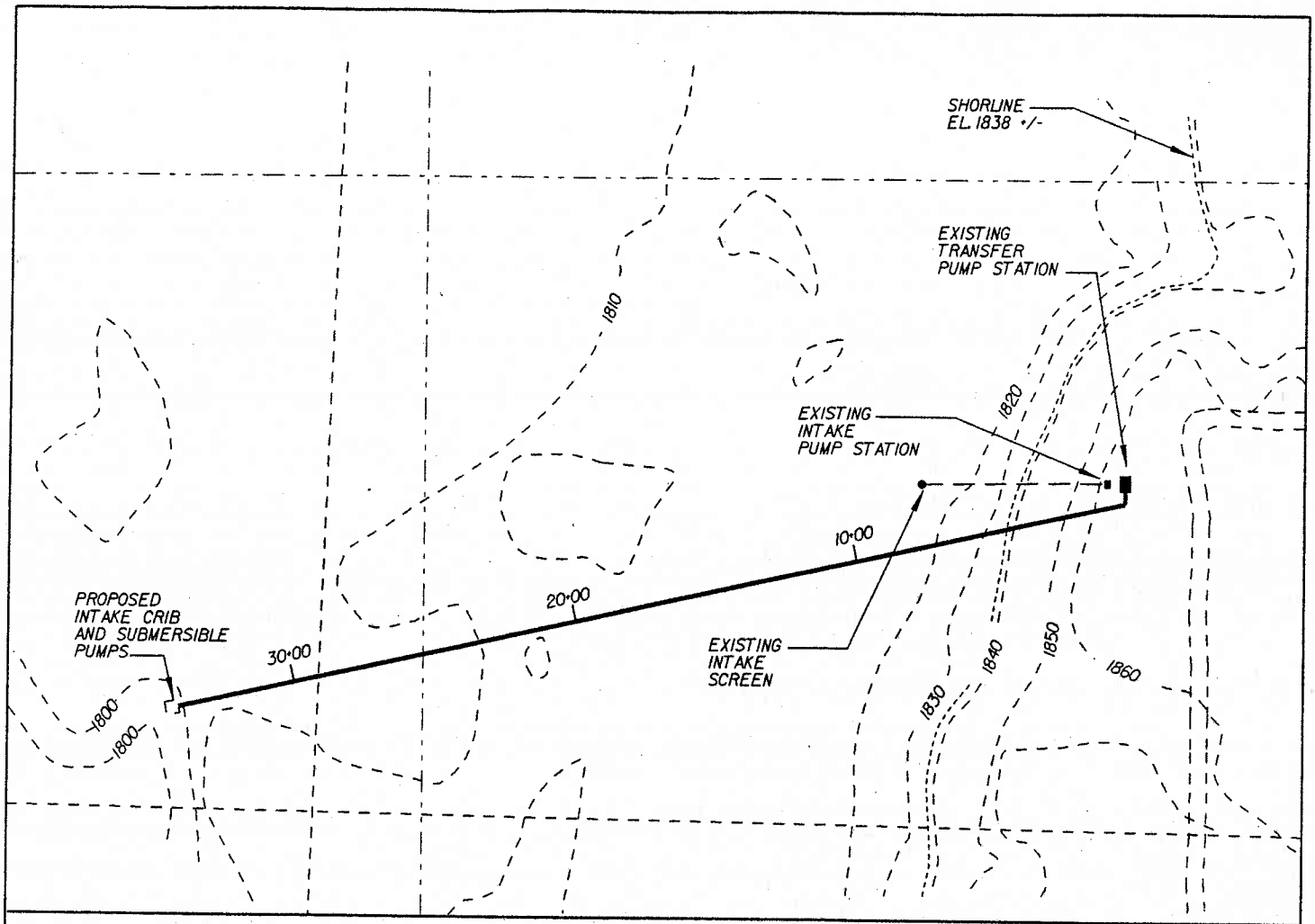
Description	Units	Current	Proposed
Design Flow			
Basin 1	mgd	0.5	0.5
Basin 2	mgd	-	1.0
Surface Loading Rate			
Basin 1	gpm/sq ft	1.3	1.3
Basin 2	gpm/sq ft	-	1.2
Recarbonation Basins			
Type: Rectangular, baffled concrete basins			
Number of Basins	No.	-	2
Basin Dimensions (w x l)	ft x ft	-	6 x 12
Water Depth	ft	-	18
Volume (each)	gal	-	10,400
Detention Time	min.	-	20
Ozone Contact Basins			
Type: Two-stage, rectangular, baffled concrete basins			
Number of Basins	No.	-	2
Basin Dimensions (w x l)	ft x ft	-	6 x 12
Water Depth	ft	-	18
Volume (each)	gal.	-	10,400
Detention Time	min.	-	20
Filters			
Type: Packaged, gravity, granular media (IDI)			
Filter No. 1 (Existing)			
Bays Per Filter	No.	3	3
Bay Dimensions (w x l)	ft x ft	6 x 8	6 x 8
Bay Area (each)	sq ft	48	48
Design Rate Per Bay	mgd	0.2	0.2
Filtration Rate at Design	gpm/sq ft	2.8	2.8
Filter No. 2 (New)			
Bays Per Filter	No.	-	3
Bay Dimensions (w x l)	ft x ft	-	8 x 12
Bay Area (each)	sq ft	-	96
Design Rate Per Bay	mgd	-	0.3
Filtration Rate at Design	gpm/sq ft	-	2.4
Total Filter Area	sq ft	144	432
Filtration Rate			
All Filters In Service	gpm/sq ft	2.4	2.4
One Small Bay in Backwash	gpm/sq ft	3.6	2.7
One Large Bay in Backwash	gpm/sq ft	-	3.1

TABLE 6-3
(Continued)

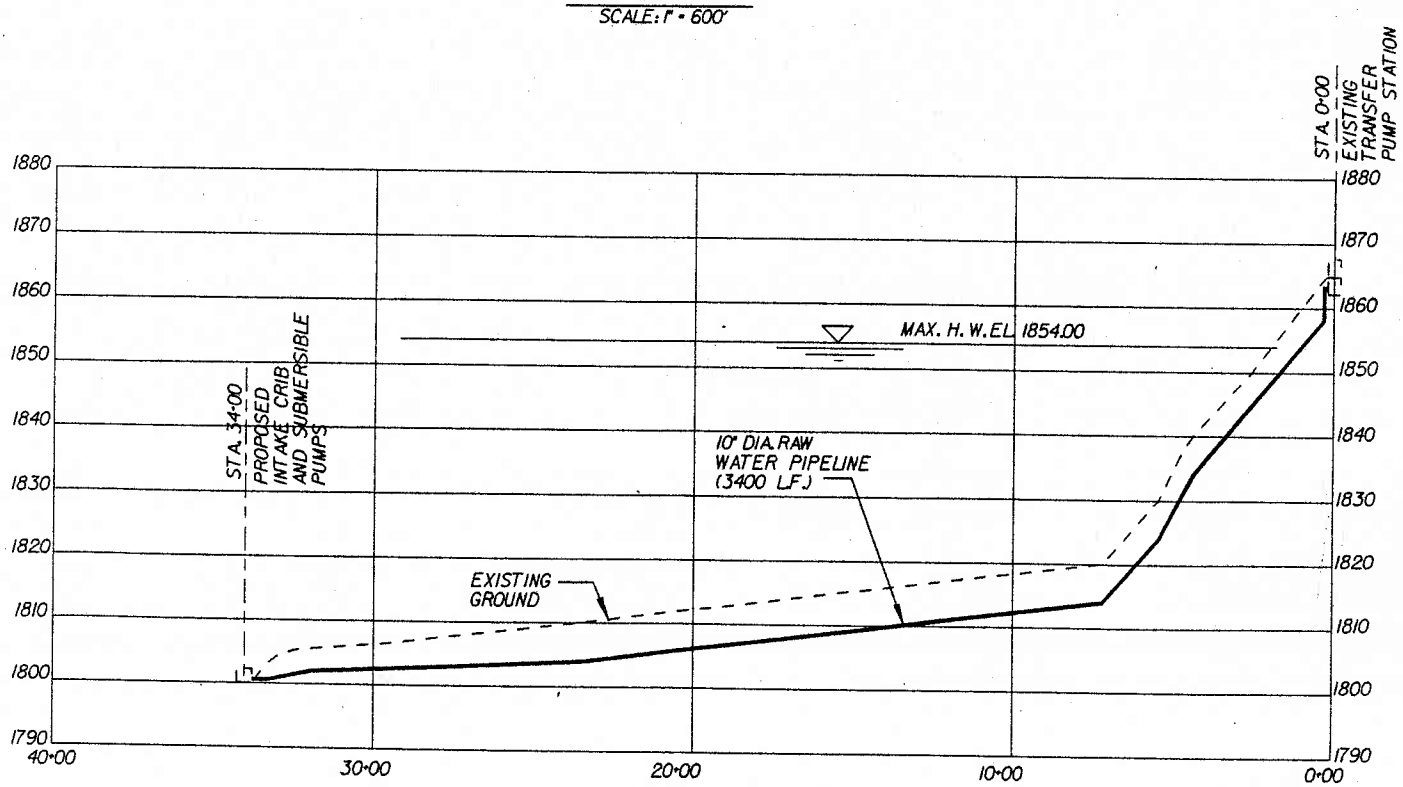
Description	Units	Current	Proposed
Filter Media			
Anthracite Coal			
Depth	in	-	20
Effective Size	mm	-	1.0
L/d		-	508
Sand			
Depth	in	-	10
Effective Size	mm	-	0.5
L/d		-	508
Total			
Depth	in	-	30
L/d		-	1016
Filter Backwash			
Type: Water wash through constant head tank			
Backwash Pumps	No.	1	1
Auxiliary Supply: From High Service Pumps to Constant Head Tank			
Pump Capacity	gpm		1,920
Backwash Rate	gpm/sq ft		20
Type: Air Scour			
Air Blower	No.	1	1
Blower Capacity	scfm		384
Air Scour Rate (max)	scfm/sq ft		4
Backwash Volume			
Total (8 min. wash)	gal	8,000	16,000
Filter Waste Washwater (FWW) Handling			
Equalization Basin (Existing)			
Type: Rectangular concrete			
Number	No.	1	2
Dimensions			
Existing	ft x ft	11 x 21.5	11 x 21.5
New	ft x ft	11 x 21.5	11 x 21.5
Water Depth (sidewall)	ft	9	9
Volume			
Existing	gal	16,000	16,000
New	gal	-	16,000
Total	gal	16,000	32,000
Backwashes Held	No.	2	2
Clearwell			
Type: Concrete basins below building floor slab			
Number of Basins	No.	2	
Dimensions			

TABLE 6-3
(Continued)

Description	Units	Current	Proposed
Existing Filtered Water Storage	ft x ft	11 x 38	11 x 38
Existing Clearwell	ft x ft	9.5 x 38	9.5 x 38
New Clearwell	ft x ft		38 x 38
Water Depth		9	9
Volume			
Existing Filtered Water Storage	gal	28,500	28,500
Existing Clearwell	gal	26,000	26,000
New Clearwell	gal	-	100,000
Total	gal	54,500	154,500
Detention Time	hr	2.6	2.5
Percent of Max. Daily Production	%	11	10
Finished Water Pumps			
Type: Vertical Turbine	No.	2	4
Capacity	mgd	0.4, 0.4	0.4, 0.4, 0.7, 0.7



PLAN
SCALE: 1" = 600'



PROFILE
SCALE: HORIZ. - 1" = 600'
VERT. - 1" = 30'

PARSHALL RAW WATER
INTAKE MODIFICATIONS
FIGURE 6-14

Modifications to the existing raw water pumping station would include the installation of two additional 60 hp vertical turbine pumps. A 10-inch PVC line would be constructed parallel to the existing 8-inch raw water transmission line to convey the increased flow to the WTP.

Flash Mix. A pump diffusion flash mix system will be provided to allow efficient coagulation and formation of microfloc prior to the softening process. Pumped diffusion flash mix is recommended because it has the advantages of lower energy requirement, higher efficiency, non-proprietary parts, and lower maintenance costs than alternative mechanical systems. A mixing energy value (G) of 750 sec^{-1} is proposed.

Chemical Feed. The following chemical feed systems are proposed for the Parshall WTP. Chemicals which are currently used at the plant are shown underlined:

- Ferric chloride - used as a primary coagulant.
- Polymer (liquid and dry) - used as a coagulant aid, flocculant aid, filter aid, and sludge conditioner.
- Lime (quick lime) - used in softening for removal of carbonate hardness.
- Soda Ash (sodium carbonate) - used in softening for removal of noncarbonate hardness.
- Polyphosphate - used to prevent precipitation of carbonate hardness.
- Chlorine - used as a disinfectant.
- Aqua Ammonia - used with chlorine to form a chloramine distribution system disinfectant.
- Ozone - used as a primary disinfectant, for control of disinfection by-products, and for control of tastes and odors.
- Potassium Permanganate - currently used for taste and odor control.
- Carbon Dioxide - used for recarbonation (pH adjustment and stabilization of precipitation potential).

The use of sulfuric acid for stabilization of softened water will be discontinued with the plant expansion. At a 1.5 mgd capacity and with the need to construct contact basins for ozonation, it will be more efficient to convert to carbon dioxide for stabilization (recarbonation). Carbon dioxide will be more convenient to feed, provide reduced maintenance for the operators, is less hazardous, and will not increase the total dissolved solids and sulfate levels as does sulfuric acid. For reasons of process efficiency ferric

chloride will be substituted for alum as a primary coagulant, and a polymer will be substituted for sodium aluminate as a coagulant aid, flocculant aid, and filter aid.

All chemical feed systems will need to be upgraded to handle the delivery requirements of the increased plant capacity. Where possible, existing chemical feed locations will be used for upgraded systems; however, additional chemical storage and feed area will be needed in the new water treatment facilities building for new chemical feed systems (e.g., ferric chloride, aqua ammonia, and ozone). In addition, all chemical feed systems will be upgraded to comply with current code requirements (e.g., Uniform Fire Code, Article 80 - Hazardous Materials). As an example, a chlorine scrubber will be required to prevent release of gas from the chlorine storage and feed facilities.

Softening. The existing softening basin at the Parshall WTP is a 20 foot diameter steel tank solids contact unit (IDI Accelator, IS Type). In order to increase the plant capacity by one mgd an additional 30 foot diameter solids contact unit of a similar design will be installed. At design flows of 0.5 mgd through the existing basin (No. 1) and 1.0 mgd through the new basin (No. 2) the surface loading rate will be 1.3 gpm/sq ft and 1.2 gpm/sq ft, respectively. The existing basin will need to be raised to a higher elevation, as discussed below.

Recarbonation Basins. As discussed above, carbon dioxide will be substituted for sulfuric acid to provide stabilization of softened water (recarbonation) in a rectangular, baffled concrete basin. The recarbonation basins will be constructed as part of the ozone contact basins with similar water depths and dimensions. Flow splitting weirs on the inlet to the recarbonation basins will maintain an equal distribution of flow between both the recarbonation and ozone contact basins. Detention time at design flow will be 20 minutes for recarbonation.

Ozone Contactor. The ozone contact basins will be constructed at a location downstream of the recarbonation process and upstream of the filters. This location allows prior reduction in ozone demand through the clarification process with subsequent removal of some ozonation by-products (e.g. aldehydes) across the filter prior to distribution. However, because of the limited head available between the existing softener and the existing filters (1.5 ft) it will be necessary to increase the water surface elevation in the existing softener to allow the installation of recarbonation and ozone contactors between softening and filtration. This can be accomplished by jacking the steel softening basin (No. 1) to the appropriate elevation and pouring a new base support of reinforced concrete.

The proposed ozone contact basins will be two stage, rectangular, baffled concrete basins. Two basins will be provided, each with a hydraulic capacity of 0.75 mgd. A detention time of 20 minutes will be provided for disinfection, and taste and odor control. The ozone generation equipment will be located directly over the contact basins. Subject to additional analysis during design, it is proposed that the most cost-effective system will be medium frequency generation of ozone from gaseous oxygen (GOX) produced

on-site by the pressure swing adsorption (PSA) method with supplemental capacity and redundancy provided by on-site storage of liquid oxygen (LOX).

Filters. The existing filters at the Parshall WTP are packaged, gravity, granular media filters (IDI) with three separate modules (bays) which are individually controlled and backwashed. The filtration rate at the design capacity (0.5 mgd) is 2.8 gpm/sq ft. It is proposed to install an additional filter unit, with three separately controlled and backwashed filter bays, to allow the expansion of the plant capacity to 1.5 mgd. At the design capacity for the new filter unit (1.0 mgd), the filtration rate will be 2.4 gpm/sq ft.

Each of the new filter bays will have twice the area (96 sq ft) as each of the existing filter bays (48 sq ft). The alternative to installing the larger filter bays with the plant expansion is to install identical units to the existing filter bays which would result in 9 independently controlled filter bays, an excessive number for a 1.5 mgd capacity plant. In order to backwash the different sizes of filter bays at the appropriate rates, it is proposed to use a single capacity of backwash pump sized for the higher rate required, with a constant head tank to allow reduction of backwash supply by overflowing the difference in pump capacity back to the clearwell when backwashing the smaller filters. The use of a constant head tank will also allow a supplemental backwash connection directly to the high service distribution system as an alternative to installing a second backwash pump.

Filter Waste Washwater Handling and Disposal. Currently, filter waste washwater (FWW) is stored in an equalization basin prior to being pumped to the head of the plant for retreatment. The FWW volume generated by a single backwash is approximately 8,000 gallons. The single existing FWW equalization basin has a capacity of 16,000 gallons, or two complete backwashes. In order to provide storage capacity for two backwashes for the larger filters proposed, it will be necessary to construct a new FWW equalization basin with a volume equal to the existing basin. Otherwise, the inability to backwash filters would be a limiting factor in plant capacity.

Clearwell. The existing 54,400 gallon reinforced concrete clearwell is located directly beneath the building slab. The existing clearwell provides a operating storage volume of 11 percent of maximum day flow at the current plant capacity of 0.5 mgd. In order to provide a reasonable amount of operating storage for the expanded plant, it will be necessary to construct a new clearwell. Construction of a new 100,000 gallon clearwell (154,000 gallons total) will provide approximately 10 percent of maximum day capacity in operating storage at a flow rate of 1.5 mgd. Although 10 percent of maximum day capacity for clearwell storage is larger than the 4 percent proposed at the Williston WTP and the 6 percent proposed at the Minot WTP, the higher percentage of storage is justified for the Parshall WTP because the plant is smaller and operates in batch mode more frequently.

Finished Water Pumping Station. The Parshall WTP currently has two vertical turbine high service pumps taking suction directly from the buried clearwell. Each pump has a

capacity of about 0.36 mgd. In order to reach the proposed plant capacity of 1.5 mgd, it is proposed to add two additional pumps with a capacity of 0.8 mgd each.

Sludge Handling and Disposal. Currently sludge from the softening basins is routed to a sludge pumping vault, and then is pumped to lagoons for disposal. Given that the City owns 28 acres of land surrounding the Parshall WTP, the use of on-site lagoons for sludge dewatering and ultimate disposal is an appropriate solution; however, the sludge lagoons should be constructed with decant structures to allow a return of decant water to the treatment plant for retreatment which will extend the useful life of the lagoons.

Instrumentation and Electrical. A new electrical service and distribution system to handle the new load at the expanded plant (especially ozone generation facilities) will be necessary. An evaluation of existing electrical systems will be conducted during design. Significant instrumentation will be added with the new process equipment. All new facilities and key existing facilities will be controlled through new PLC based distributed control systems (DCS). The plant DCS system can interface with the distribution SCADA system which will be provided in the Parshall WTP control center; however, it is not proposed that the SCADA system will control the WTP.

COST ESTIMATES

Cost estimates for the three WTP projects were prepared using information from similar facility designs, equipment suppliers, and recent bids from projects of similar complexity. The construction costs presented herein have been broken down to indicate costs attributable to the following categories:

- Expansion
- Upgrades
- New Regulation Needs
- Biota Transfer Control

Estimated construction costs for the Minot WTP, Williston WTP, and Parshall WTP are presented in Tables 6-4, 6-5, and 6-6, respectively.

TABLE 6-4

MINOT WTP COST ESTIMATE

Description	Cost
Inlet Structure	$ 500,000
Softeners	1,700,000
Recarbonation	300,000
Ozone	3,500,000
Filters	1,200,000
Chemical Feed	800,000
Operations Building	500,000
Finished Water Pumping Station	400,000
Electrical/Instrumentation	1,500,000
Yard Piping	300,000
Clearwell	500,000
Overflow Containment	1,000,000
Total Construction Cost	$12,200,000

Category	Cost	Percent
Expansion	$ 4,900,000	40
Upgrades	2,000,000	16
New Regulation Needs	4,300,000	36
Biota Transfer Control	1,000,000	8
Total	$12,200,000	100

TABLE 6-5

WILLISTON WTP COST ESTIMATE

Description	Cost
Raw Water Screening and Pumping Station	$ 250,000
Pretreatment	700,000
Softeners	1,300,000
Recarbonation/Ozone	2,200,000
Filters	1,150,000
Chemical Feed	850,000
Operations Building	1,500,000
Lagoons	400,000
Finished Water Pumping Station	100,000
Electrical/Instrumentation	1,500,000
Total Construction Cost	$9,950,000

Category	Cost	Percent
Expansion	$ 3,450,000	35
Upgrades	2,850,000	28
New Regulation Needs	3,650,000	37
Biota Transfer Control	0	0
Total	$9,950,000	100

TABLE 6-6

PARSHALL WTP COST ESTIMATE

Description	Cost
Intake	$ 250,000
Raw Water Pumping Station Modifications	150,000
Softener and Filter	400,000
Chemical Feed	350,000
Ozone	300,000
Lagoons	150,000
Finished Water Pumps	50,000
Building	100,000
Clearwell	200,000
Electrical/Instrumentation	250,000
New Town/Makoti Finished Water Pumps	200,000
Total Construction Cost	$2,400,000

Category	Cost	Percent
Expansion	$1,200,000	50
Upgrades	400,000	17
New Regulation Needs	800,000	33
Biota Transfer Control	0	0
Total	$2,400,000	100

Section 7

HOUSTON ENGINEERING
AMERICAN ENGINEERING
MONTGOMERY WATSON

SECTION 7

ESTIMATES OF CONSTRUCTION AND O&M COSTS

INTRODUCTION

This section summarizes the estimates of capital construction costs of the NAWS Pre-Final Design, selected plan facilities. The construction costs of the raw water treatment and the treated water distribution are presented by major subsystem segment for each of three NAWS regional systems (East, Parshall, and West Systems) as shown in Figures 7-1 through 7-3. Operation and maintenance (O&M) costs for each system are also described and presented. Estimates of financing costs describing the funding of the local share of the capital improvements are developed with input from the SWC. The capital construction costs, O&M costs and financing costs form the basis to estimate average user costs per 1,000 gallons of treated project water.

CAPITAL CONSTRUCTION COSTS

Estimates of construction cost for the various components of the selected plan for the East, Parshall and West Systems were developed in Sections 4, 5, and 6. These facilities include:

- Pipeline systems
  - Raw water pipelines
  - Finished (treated) water pipelines
- Hydraulic/surge controls
- Pump stations
  - Raw water pump stations
  - Treated water pump stations at the WTPs
  - Booster pump stations in the treated water delivery system
- Telemetry/SCADA system
- Storage reservoirs
- Raw water intakes/pump stations
- Water treatment plants

Construction cost estimates for the pipelines, hydraulic controls, pump stations, telemetry/SCADA systems, raw water intakes, and reservoirs were prepared from a

combination of manufacturers' quotes, cost estimates using preliminary quantity take-offs, and comparative data from actual bids from the Southwest Pipeline Project. Cost estimates for the three water treatment plant expansions were prepared using information from designs of similar facilities, equipment suppliers, and recent bids from projects of similar complexity. Construction costs for the water treatment plant expansions have been separated to indicate costs attributable to:

- Expansion
- Upgrades
- New regulations needs

Estimated construction costs (current dollars) for all of the system components are presented in Table 7-1. An allowance of 30 percent of construction costs has been included in project costs to cover costs for legal, engineering, land and easements, and contingencies. Land and easement costs are assumed to be three percent of construction costs. The total estimated cost of the NAWS project is $163,900,000.

NAWS SYSTEM SEGMENT COSTS

A breakdown of the East, Parshall and West Systems into segments provides a method to determine construction costs of major elements of the system by function and user area. Major segments include costs for pipelines, pump stations, intakes, hydraulic controls, and storage reservoirs for the three systems are shown in Figures 7-1, 7-2, and 7-3. The water treatment plants and the telemetry/SCADA systems are described as individual segments for the East, Parshall and West systems.

East System

The East system has a total of 10 segments shown on Figure 7-1. Segment 1 is the raw water supply system and Segment 2 is the Minot Water Treatment Plant. Segments 3 through 9 are the treated water distribution systems serving the users in the East system. Segment 10 is the telemetry/SCADA system. The total construction of the East system is $90,466,000. The individual segment costs are shown in Table 7-2.

Parshall System

The Parshall system has four segments as shown on Figure 7-2. The total construction cost for the Parshall system is $5,805,000. The construction costs, by segment, are presented in Table 7-3.

West System

The West system has five segments as shown on Figure 7-3. The water treatment plant, Segment 1, also includes the upgrades of the raw water pump station. The total construction cost of the West system is $29,829,000. The construction costs, by segment, are presented in Table 7-4.

TABLE 7-1

PROJECT COST ESTIMATE

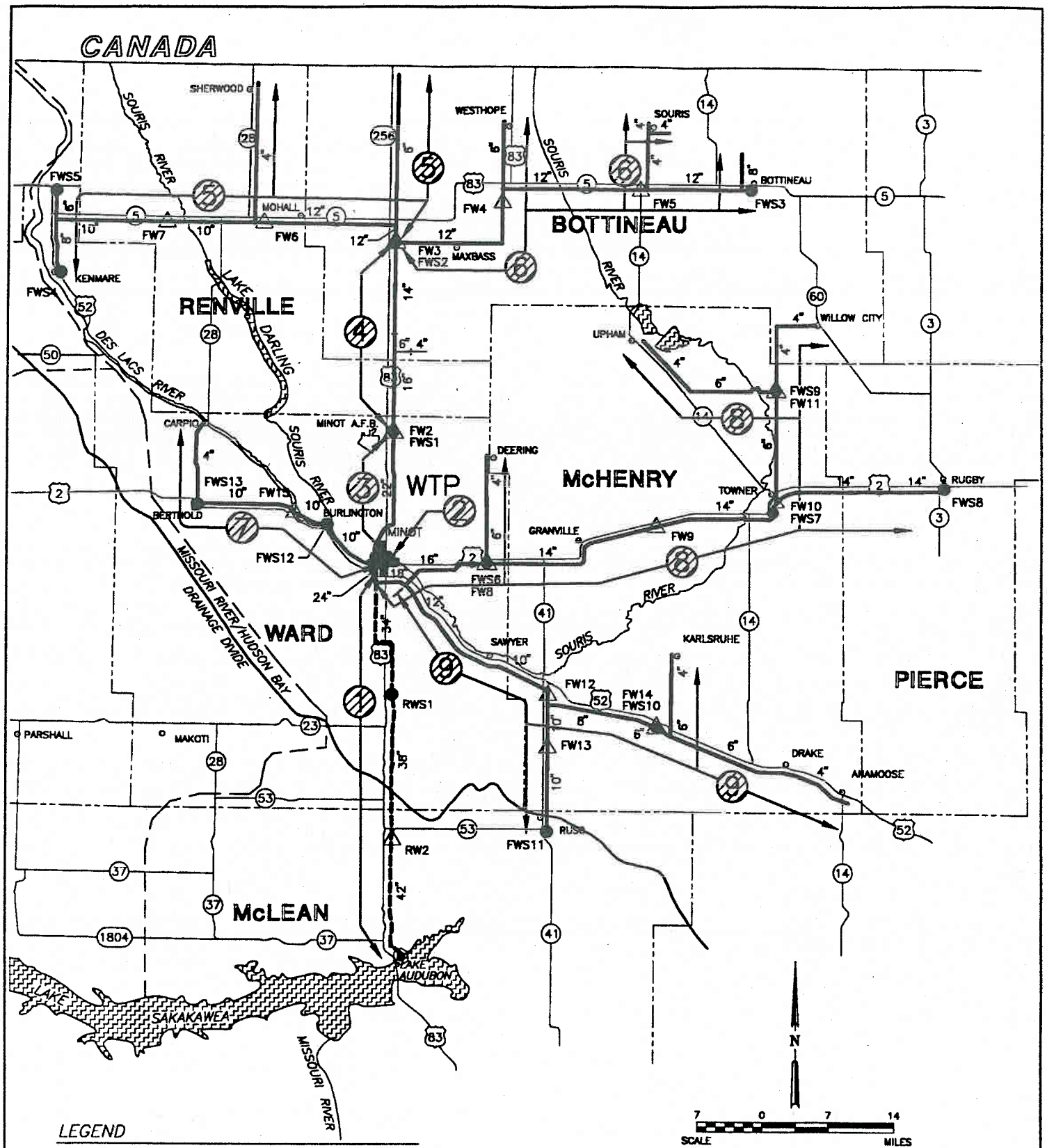
NORTHWEST AREA WATER SUPPLY PROJECT
PRE-FINAL DESIGN

System Components	Systems East	Parshall	West
Pipelines			
Large Diameter (14 to 42 inch)	$46,991,000	0	$4,447,000
Small Diameter (4 to 12 inch)	16,618,000	$2,842,000	10,872,000
	$63,609,000	**$2,842,000**	**$15,319,000**
Pumping Stations	**3,900,000**	**0**	**1,800,000**
Reservoirs	**6,500,000**	**400,000**	**1,600,000**
Intakes	**2,700,000**	**400,000**	**0**
Telemetry/SCADA	**1,035,000**	**163,000**	**845,000**
Hydraulic Controls	**522,000**	**0**	**265,000**
Water Treatment Plants			
Expansion	5,900,000	800,000	3,500,000
Upgrades	2,000,000	400,000	2,900,000
New Regulation Needs	4,300,000	800,000	3,600,000
	$12,200,000	**$2,000,000**	**$10,000,000**
Construction Cost Totals	**$90,466,000**	**$5,805,000**	**$29,829,000**
Engineering and Contingencies	27,100,000	1,700,000	9,000,000
PROJECT COST TOTALS	**$117,566,000**	**$7,505,000**	**$38,829,000**

TABLE 7-2

NAWS SEGMENT ANALYSIS
EAST SYSTEM

Segment No.	Location	Facility Description	Quantity	Construction Cost
E1	Lake Audubon to Minot WTP	• Raw Water Intake and Pump Station	1	$2,700,000
		• 42" Ø Pipe	64,000 LF	7,420,000
		• 38" Ø Pipe	83,000 LF	8,400,000
		• 34" Ø Pipe	79,000 LF	8,120,000
		• 24" Ø Pipe	9,500 LF	604,000
		• Booster Pump Station (RW2)	1	1,000,000
		• Storage (RWS1)	1	900,000
		• Hydraulic Controls	4	395,000
		Subtotal E1		**$29,539,000**
E2	Minot WTP	• WTP Expansion and Upgrade	1	$12,200,000
		• Hydraulic Control	1	40,000
		Subtotal E2		**$12,240,000**
E3	Minot WTP to Minot AFB	• 24" Ø Pipe	7,200 LF	$446,000
		• 22" Ø Pipe	84,600 LF	4,650,000
		• Booster Pump Station (FW 2)	1	400,000
		• Storage (FWS 1)	1	1,825,000
		Subtotal E3		**$7,321,000**
E4	Minot AFB to Pump Station West of Maxbass (FW 3), includes Upper Souris II	• 16" Ø Pipe	63,200 LF	$2,591,000
		• 14" Ø Pipe	41,260 LF	1,465,000
		• 6" Ø Pipe	5,000 LF	30,000
		• 4" Ø Pipe	11,000 LF	44,000
		• Booster Pump Station (FW 3)	1	325,000
		• Storage (FWS 2)	1	400,000
		• Hydraulic Control	1	4,000
		Subtotal E4		**$4,859,000**
E5	FW 3 to Kenmare, includes Sherwood, Upper Souris I, Renville Co., and All Seasons III	• 12" Ø Pipe	89,340 LF	$1,608,000
		• 10" Ø Pipe	119,000 LF	1,607,000
		• 8" Ø Pipe	36,374 LF	364,000
		• 6" Ø Pipe	98,226 LF	589,000
		• 4" Ø Pipe	76,340 LF	305,000
		• Booster Pump Station (FW 6)	1	150,000
		• Booster Pump Station (FW 7)	1	150,000
		• Storage (FWS 4)	1	250,000
		• Storage (FWS 5)	1	250,000
		• Hydraulic Controls	2	8,000
		Subtotal E5		**$5,281,000**
E6	FW3 to Botteneau, including Westhope, Souris, All Seasons I, and All Seasons II	• 12" Ø Pipe	237,743 LF	$4,280,000
		• 8" Ø Pipe	23,086 LF	231,000
		• 6" Ø Pipe	37,500 LF	225,000
		• 4" Ø Pipe	45,750 LF	183,000
		• Booster Pump Station (FW 4)	1	170,000



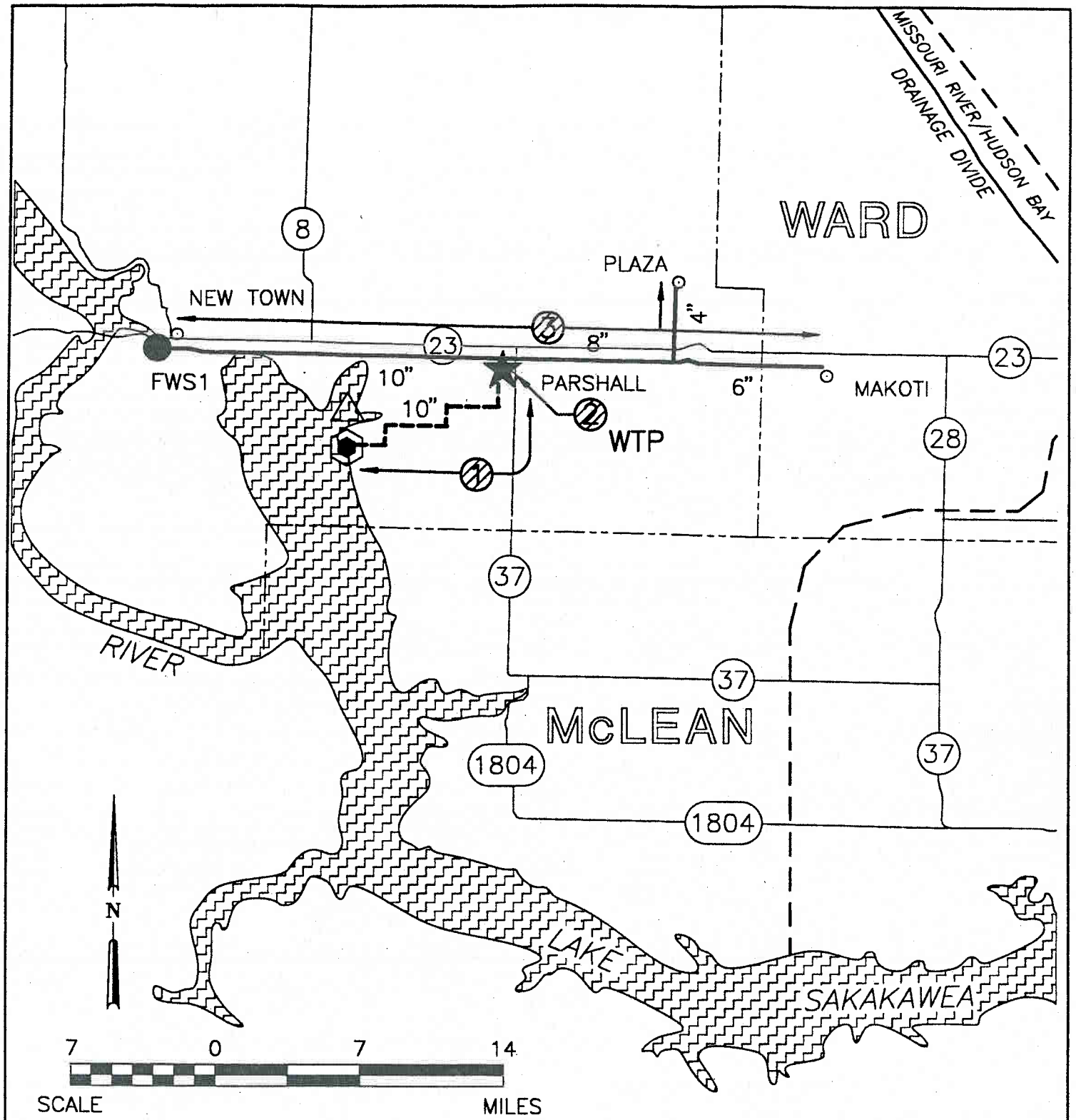
NAWS EAST SYSTEM SEGMENTS

NORTHWEST AREA WATER SUPPLY PROJECT
PRE - FINAL DESIGN

NORTH DAKOTA STATE WATER COMMISSION
AND
GARRISON DIVERSION CONSERVANCY DISTRICT
PROJECT NO. 237-4

MARCH 1, 1995

FIGURE 7-1



LEGEND

PIPELINE	
RAW WATER LINE	
CITIES	MOHALL
COUNTY LINES	
STATE HIGHWAYS	23
FEDERAL HIGHWAYS	2
SYSTEM BOUNDARIES	
WATER TREATMENT PLANTS	★
INTAKES	⬢
STORAGE RESERVOIRS	●
PUMPING STATIONS	△
STORAGE RES. W/PUMPING STA.	▲

TELEMETRY AND SCADA

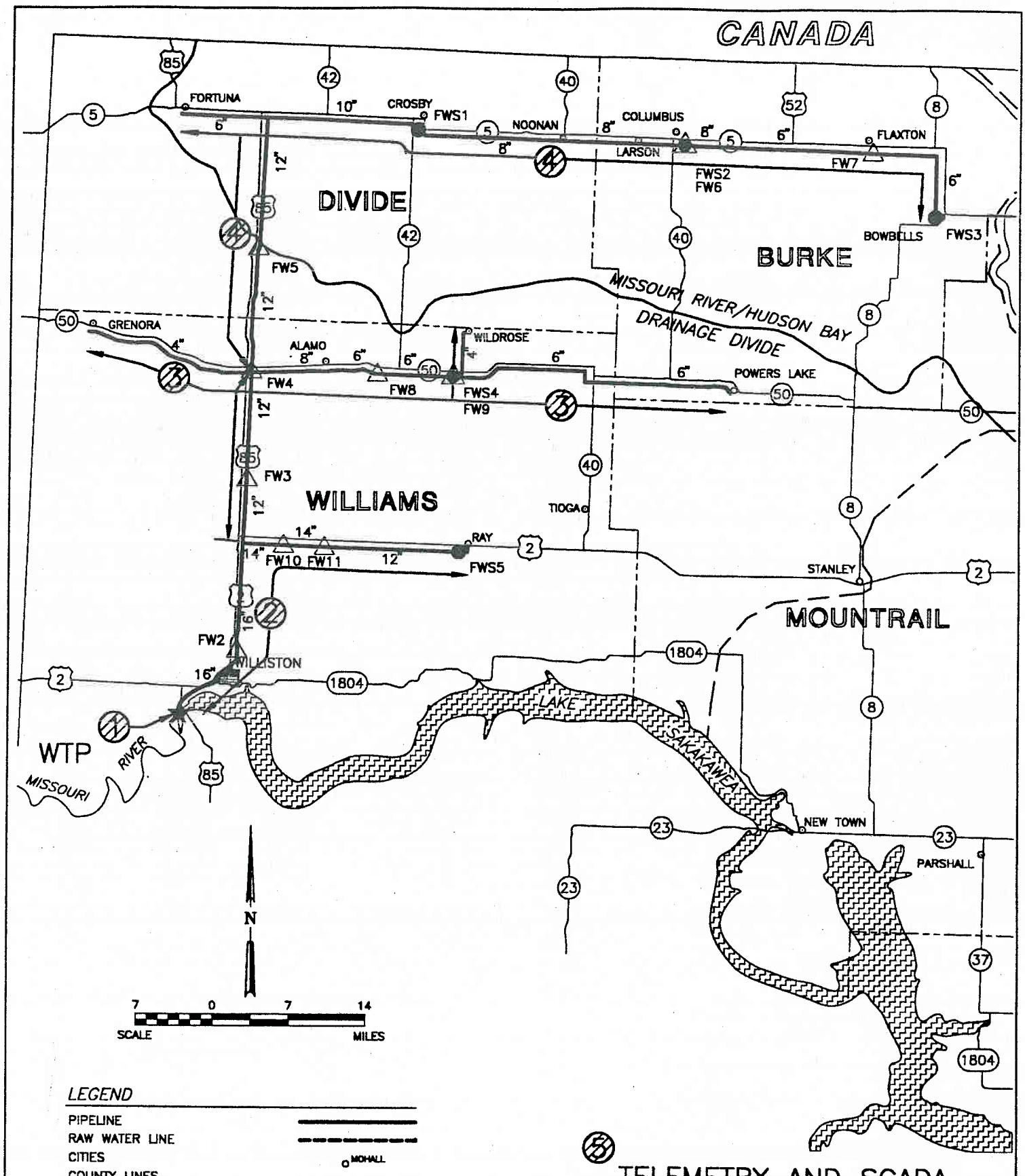
NAWS PARSHALL SYSTEM SEGMENTS

NORTHWEST AREA WATER SUPPLY PROJECT
PRE - FINAL DESIGN

NORTH DAKOTA STATE WATER COMMISSION
AND
GARRISON DIVERSION CONSERVANCY DISTRICT
PROJECT NO. 237-4

MARCH 1, 1995

FIGURE 7-2



NAWS WEST SYSTEM SEGMENTS

NORTHWEST AREA WATER SUPPLY PROJECT
PRE - FINAL DESIGN

NORTH DAKOTA STATE WATER COMMISSION
AND
GARRISON DIVERSION CONSERVANCY DISTRICT
PROJECT NO. 237-4

MARCH 1, 1995

FIGURE 7-3

TABLE 7-2 (continued)

NAWS SEGMENT ANALYSIS
EAST SYSTEM

Segment No.	Location	Facility Description	Quantity	Construction Cost
		• Booster Pump Station (FW 5)	1	250,000
		• Storage (FWS 3)	1	550,000
		• Hydraulic Controls	2	8,000
		Subtotal E6		**$5,897,000**
E7	Minot to Berthold, including Burlington and Carpio	• 10" Ø Pipe	110, 713LF	$1,495,000
		• 4" Ø Pipe	47,500 LF	190,000
		• Booster Pump Station (FW 15)	1	130,000
		• Storage (FWS 12)	1	250,000
		• Storage (FWS 13)	1	250,000
		• Hydraulic Controls	2	24,000
		Subtotal E7		**$2,339,000**
E8	Minot to Rugby, including Deering, Granville, Towner, Upham and Willow City	• 18" Ø Pipe	23,627 LF	$1,205,000
		• 16" Ø Pipe	50,979 LF	2,090,000
		• 14" Ø Pipe	281,688 LF	10,000,000
		• 6" Ø Pipe	143,589 LF	862,000
		• 4" Ø Pipe	132,711 LF	531,000
		• Booster Pump Station (FW 8)	1	325,000
		• Booster Pump Station (FW 9)	1	325,000
		• Booster Pump Station (FW 10)	1	225,000
		• Booster Pump Station (FW 11)	1	50,000
		• Storage (FWS 6)	1	225,000
		• Storage (FWS 7)	1	250,000
		• Storage (FWS 8)	1	700,000
		• Storage (FWS 9)	1	150,000
		• Hydraulic Controls	4	35,000
		Subtotal E8		**$16,973,000**
E9	Minot (intersection of pipe to Granville) to Ruso and Anamoose, includes Sawyer, Karlsruhe and Drake	• 12" Ø Pipe	66,643 LF	$1,199,000
		• 10" Ø Pipe	113,112 LF	1,527,000
		• 8" Ø Pipe	36,000 LF	360,000
		• 6" Ø Pipe	117,000 LF	702,000
		• 4" Ø Pipe	71,500 LF	286,000
		• Booster Pump Station (FW 12)	1	170,000
		• Booster Pump Station (FW 13)	1	130,000
		• Booster Pump Station (FW 14)	1	100,000
		• Storage (FWS 11)	1	250,000
		• Storage (FWS 10)	1	250,000
		• Hydraulic Controls	2	8,000
		Subtotal E9		**$4,982,000**
E10	East System Telemetry and SCADA	• RTUs, MTU/Computer and Radio System	1	**$1,035,000**
		East System Total Construction Cost		**$90,466,000**

TABLE 7-3

NAWS SEGMENT ANALYSIS
PARSHALL SYSTEM

Segment No.	Location	Facility Description	Quantity	Construction Cost
P1	Raw Water Intake to Parshall WTP	• Raw Water Intake and Pump Station	1	$400,000
		• 10" Ø Pipe	55,700 LF	752,000
		Subtotal P1		**$1,152,000**
P2	Parshall WTP	• WTP Expansion and Upgrade	1	**$2,000,000**
P3	Parshall WTP to New Town, Makoti and Plaza	• 10" Ø Pipe	100,207 LF	1,353,000
		• 8" Ø Pipe	41,752 LF	418,000
		• 6" Ø Pipe	42,241 LF	253,000
		• 4" Ø Pipe	16,500 LF	66,000
		• Storage (FWS 1)	1	400,000
		Subtotal P3		**$2,490,000**
P4	Parshall System Telemetry and SCADA	• RTUs, MTU/Computer and Radio System	1	**$163,000**
		Parshall System Total Construction Cost		**$5,805,000**

TABLE 7-4

NAWS SEGMENT ANALYSIS
WEST SYSTEM

Segment No.	Location	Facility Description	Quantity	Construction Cost
W1	Williston WTP	• Raw Water Pump Station and WTP Expansion and Upgrade	1	**$10,000,000**
W2	Williston to intersection of Highways 2 and 85, including Ray	• 16" Ø Pipe	72,544 LF	$2,974,,000
		• 14" Ø Pipe	41,500 LF	1,473,000
		• 12" Ø Pipe	84,700 LF	1,525,000
		• Booster Pump Station (FW 2)	1	400,000
		• Booster Pump Station (FW 10)	1	250,000
		• Booster Pump Station (FW 11)	1	325,000
		• Storage (FWS 5)	1	700,000
		• Hydraulic Controls	6	175,000
		Subtotal W2		**$7,822,000**
W3	Intersection of Highways 2 and 85 to intersection of Highways 85 and 50. Includes Grenora, Alamo, Wildrose and Powers Lake	• 12" Ø Pipe	83,777 LF	$1,507,000
		• 8" Ø Pipe	34,500 LF	345,000
		• 6" Ø Pipe	213,500 LF	1,281,000
		• 4" Ø Pipe	99,600 LF	398,000
		• Booster Pump Station (FW 3)	1	225,000
		• Booster Pump Station (FW 4)	1	170,000
		• Booster Pump Station (FW 8)	1	70,000
		• Booster Pump Station (FW 9)	1	50,000
		• Storage (FWS 4)	1	250,000
		• Hydraulic Controls	4	14,000
		Subtotal W3		**$4,310,000**
W4	Intersection of Highways 85 and 50 to Bowbells, including Flaxton, Columbus, Larson, Crosby, Noonan and Fortuna	• 12" Ø Pipe	124,979 LF	$2,250,000
		• 10" Ø Pipe	78,381 LF	1,058,000
		• 8" Ø Pipe	137,100 LF	1,371,000
		• 6" Ø Pipe	189,519 LF	1,137,000
		• Booster Pump Station (FW 5)	1	170,000
		• Booster Pump Station (FW 6)	1	70,000
		• Booster Pump Station (FW 7)	1	70,000
		• Storage (FWS 1)	1	250,000
		• Storage (FWS 2)	1	250,000
		• Storage (FWS 3)	1	150,000
		• Hydraulic Controls	4	76,000
		Subtotal W4		**$6,852,000**
W5	West System Telemetry and SCADA	• RTUs, MTU/Computer and Radio System	1	**$845,000**
		West System Total Constructon Cost		**$29,829,000**

OPERATION AND MAINTENANCE COSTS

Introduction

Annual costs for the operation and maintenance (O&M) of the NAWS project were developed for each of the three regional systems. It was assumed that there is no cost for the raw water for each system. All O&M costs were based on the average flow as follows:

- East System 10.225 mgd
- Parshall System 0.610 mgd
- West System 4.078 mgd

Operation costs include pump energy, heating ventilating and air conditioning (HVAC), equipment, labor, chemicals, power, and miscellaneous costs. Operation cost for chloramination at the Lake Audubon intake/pump station was also developed for the East System.

Maintenance costs on the equipment and serviceable facilities were developed for the raw water intake/pump stations, pump stations, reservoirs, telemetry/SCADA systems, pipelines, cathodic protection systems, and the water treatment plants. In general, maintenance costs were developed as a percentage of the estimated equipment costs. However, where actual maintenance cost estimates were available for certain types of facilities, these estimates were used and adjusted to meet the configuration of the recommended plan. O&M costs per 1,000 gallons were developed for each of the three regional systems and for the entire NAWS project.

BASIS OF O&M COSTS

The assumptions and unit costs used to develop the O&M cost estimates are presented in the following paragraphs.

Power Costs

A cost of $0.025/kwh was used for all power consumption in the project.

Depending on the location of the individual project facility, actual power rates may be higher or lower than the average rate assumed above. For example, the Lake Audubon pump station would likely be able to use power from Western Area Power Administration (WAPA) at a rate of $0.0125/kwh. If power rate contracts are negotiated for the facilities prior to final facility siting, preferential power rates may be realized.

An evaluation was performed to determine the applicability of using an average power costs for pumping on the NAWS system. The comparison was made using various rates based on WAPA rates and expected rates over the NAWS system. The following power rates were used:

(1) Lake Audubon power at $0.0126/kwh
(2) Max Booster Pumping Plant at $0.026/kwh
(3) Minot WTP at $0.028/kwh
(4) All other pumping facilities at $0.030/kwh

Using these rates for the pump stations in the NAWS system resulted in a total O&M cost of $0.60/1,000 gallons. The total O&M cost using an average rate of $0.025/kwh, is also $0.60/1,000 gallons. A sensitivity evaluation for average power cost rates indicated that a $0.030/kwh rate would yield an O&M cost of $0.62/1,000 gallons.

Pumping Energy

Pumping energy was determined using the average flow rate at each pump station, the pumping head and the pump and motor efficiency. The average flow at each pump station was assumed to be 40 percent of the peak design flow. Pump and motor efficiencies were assumed at 80 percent and 90 percent, respectively. The estimated horsepower was then converted to kwh per year.

HVAC Energy

HVAC energy requirements were estimated assuming 25 kwh/sq ft per year of floor plan area. This estimate assumes the buildings are well insulated. Heating will be provided during the winter to maintain the interior space above freezing and ventilation will be provided during the summer. With the exception of the Lake Audubon Intake/Pump Station and the Max booster pump station on the East System raw water line, all pump stations were assumed to have an area of 1,270 sq ft of area.

Equipment

Annual costs for equipment were estimated using monthly lease rates over a one year period. The following lease rates on equipment were used:

- Pickup trucks - $350/mo
- Backhoe loader - $2,150/mo
- Wheel loader - $3,750/mo

Labor

Labor rates were based on three personnel classifications for staffing the pump stations and delivery systems. Labor for operation of the water treatment plants was based on present labor costs at the existing plants in Minot, Parshall, and Williston and were adjusted to reflect the additional staffing requirements resulting from plant upgrades and expansions. Labor classes and labor costs, including 30 percent for benefits are presented below:

- Supervisor/Manager - $26.00/hr
- Maintenance Laborer - $19.50/hr
- Secretarial - $11.70/hr

Full-time employee costs were based on 2,080 payable hours per year.

Chemical Costs

Chemical costs are based on the annual use of chemicals at the dose or concentration required for the average flow over 365 days per year.

Miscellaneous Costs

An allowance for miscellaneous costs was used for the rental of special equipment (i.e., cranes, barges, pumps, compressors, etc.), for repairs and unusual circumstances.

Cathodic Protection

Operation costs for cathodic protection systems on metallic pipelines were developed from O&M costs experienced in existing systems. Operation costs from the Southwest Pipeline Project were used as a base and adjusted for conditions, lengths and sizes of the metallic pipelines in the NAWS project.

ANNUAL OPERATIONAL COSTS

Annual operation costs were developed for raw water intakes/pump stations, distribution system pump stations and reservoirs, pipelines and cathodic protection systems, and water treatment plants for the East, Parshall and West Systems.

Raw Water Intakes

Operation costs for the raw water intakes on the East, Parshall, and West Systems includes pumping energy, HVAC, and miscellaneous costs. The East System raw water intake at Lake Audubon includes chloramination facilities. The raw water booster station located near the town of Max was also included in the raw water intake system.

Pumping Energy. Pumping energy and annual pumping costs for the raw water pump stations are summarized in Table 7-5.

HVAC. The Lake Audubon intake and raw water pump station includes a screening room and chemical addition facility. The estimated floor area is approximately 4,000 sq ft. The pump station at Max would be equivalent to the area represented by the pump room of the intake facility at about 1,600 sq ft. Using 25 kwh/sq ft/year, the East System HVAC annual costs for the raw water supply are as follows:

- RW1 - $2,500
- RW2 - $1,000

  Total - $3,500

The raw water pump station HVAC annual cost for the Parshall System (RW2) is estimated at $795, based on 1,270 sq ft of area. The West System raw water pump station is an existing facility and is included in the existing operation of the Williston WTP.

TABLE 7-5

NAWS RAW WATER PUMPING ENERGY COSTS

System	Pumping Station Location	Average Capacity (gpm)	Pumping Head (ft)	kwh/year	Annual Pumping Costs ($)
East	RW1	8,340	368	7,034,430	175,900
	RW2	8,340	283	5,409,630	135,200
				Total	311,100
Parshall	RW1	427	71	69,450	1,700
	RW2	427	271	265,100	6,600
				Total	8,300
West	RW1	3,998	63	577,238	14,400

Miscellaneous Equipment. An annual cost allocation for rental of equipment to maintain the intakes and raw water pump stations was estimated based on the requirements to rent cranes for pulling pumps, barges, portable pumps, etc. The following miscellaneous annual costs were provided for each of the three regional raw water intake pump stations:

- East - $5,000
- Parshall - $1,200
- West - $1,000

The miscellaneous cost for the East System is based on the mechanical equipment and complexity of the pump station and intake facility. This facility may require periodic dredging of the intake channel and sediment removal from the intake bay and screening chamber wet wells. The Parshall intake system includes a submersible pump station that will likely require a barge or other equipment to maintain and operate the system.

Lake Audubon Intake Chloramination. The chloramination facility for the Lake Audubon intake/pump station on the East System is designed to provide 4 mg/l of chloramines. At chemical dosages of 4 mg/l of chlorine and 1.2 mg/l of ammonia, about 67.2 tons of chlorine and about 20.2 tons of aqua ammonia would be used per year. At about $520 per ton for chlorine and $250 per ton for ammonia, the annual chemical cost is

about $40,000 per year. Annual operation cost for the chemical feed equipment is assumed at $5,000 per year. The total annual operation cost for chloramination is estimated at $45,000.

No chloramination is provided for the Parshall and Williston raw water supplies.

Pump Stations and Reservoirs

Annual operation costs for the pump stations and reservoirs include pumping energy, HVAC, equipment, and labor. Since these facilities are new construction, the operation and maintenance activities will require new staff and equipment assigned to each regional system.

Pumping Energy. Pumping energy costs for the finished water pump stations at the water treatment plants and the finished water pump stations on the pipeline delivery system were developed for each of the three regional systems. Table 7-6 summarizes the pumping energy costs for the East, Parshall, and West Systems.

HVAC. The HVAC annual costs for the pump stations are based on an assumed area of 1,270 sq ft for each pump station on the distribution system. The finished water pump stations at each treatment plant are included in the treatment plant costs. At 25 kwh/sq ft/year, the annual cost for each station is estimated at $795. The East System has 14 pump stations on the distribution pipelines, for HVAC annual cost of $11,130.

The Parshall System does not have pump stations on the distribution pipelines.

The West System has 10 pump stations on the distribution pipelines, for a HVAC annual cost of $7,950.

Equipment. The equipment needed to perform operation and maintenance on the pump stations and reservoirs on the distribution systems are described in Table 7-7.

Labor Costs. O&M staff required to maintain the pump stations and reservoirs on the distribution systems is summarized in Table 7-8.

Cathodic Protection on Pipelines

Annual operation costs for cathodic protection on the metallic pipelines for the East System and the West System were estimated at $5,000 and $2,000, respectively. Cathodic protection can be applied by impressed current cathodic protection or by galvanic anode cathodic protection. Impressed current gives the operators more flexibility in meeting various soils corrosion properties and for potential existence of stray or foreign currents. Actual corrosion studies during design will determine the number, type, and length of pipeline to be served by cathodic protection systems.

TABLE 7-6

NAWS PUMP STATION ENERGY COSTS

System	Pump Station Location	Average Capacity (gpm)	Pumping Head (ft)	kwh/year	Annual Pumping Cost ($)
East	FW1	7,932	346	6,290,339	$157,200
	FW2	1,010	331	766,240	19,200
	FW3	944	281	607,986	15,200
	FW4	513	224	263,379	6,600
	FW5	418	406	388,971	9,700
	FW6	255	257	150,206	3,800
	FW7	235	317	170,743	4,300
	FW8	878	236	474,921	11,900
	FW9	763	315	550,872	13,800
	FW10	620	223	316,892	7,900
	FW11	66	92	13,917	300
	FW12	316	285	206,418	5,200
	FW13	170	342	133,257	3,300
	FW14	121	315	87,360	2,200
	FW15	156	336	120,138	3,000
				East FW Pumping Total	$263,600
Parshall	FW1	427	346	338,625	$8,500
West	FW1	3,544	185	1,502,729	$37,600
	FW2	1,101	316	797,425	19,900
	FW3	510	300	350,676	8,800
	FW4	369	248	209,746	5,200
	FW5	369	247	208,900	5,200
	FW6	86	296	58,345	1,500
	FW7	77	215	37,944	900
	FW8	84	295	56,796	1,400
	FW9	84	112	21,563	500
	FW10	592	315	427,413	10,700
	FW11	592	345	468,119	11,700
				West FW Pumping Total	$103,400

TABLE 7-7

NAWS EQUIPMENT ANNUAL COST

Equipment Item	East System No.	East System Cost	Parshall System No.	Parshall System Cost	West System No.	West System Cost
Pickup Trucks	4	$16,800	1	$4,200	3	$12,600
Backhoe Loader	1	25,800	0[a]	0	1	25,800
Wheel Loader	1	45,000	0[a]	0	1	45,000
Miscellaneous[b]	--	12,400	--	5,800		6,600
Totals		$100,000		$10,000		$90,000

[a] No major equipment provided. If needed, equipment could be provided from contractors, from other systems, or from cities. Costs for this equipment is included under miscellaneous.

[b] Tools, portable pumps, generators, and maintenance equipment.

TABLE 7-8

NAWS O&M LABOR COSTS

Labor Classification	East System No.	East System Labor Cost	Parshall System No.	Parshall System Labor Cost	West System No.	West System Labor Cost
Supervisor/ Manager	1	$54,080	0	--	1	$54,080
Maintenance/ Laborer	7	283,920	1/2	$20,280	5	202,800
Secretarial	1	24,340	0	--	1	24,340
Totals		$362,340		$20,280		$281,220

Water Treatment Plants

Annual operation costs for the water treatment plants include chemicals, labor, and power. The existing O&M costs of the water treatment plants at Minot, Parshall, and Williston were reviewed and used as a basis for estimating operation costs for the NAWS project upgrades. Where plant expansions increased capacity or upgraded facilities, such items as chemical use, power cost, and labor were scaled up based on average flow. New systems, such as new chemical feed facilities were also included.

Minot WTP Operation Costs. The annual operation cost for the Minot treatment plant was estimated at $975,000/yr. The chemical cost is estimated at $500,000/yr, which includes increased chemical consumption resulting from an increase in plant capacity from 18 mgd to 28 mgd, and the addition of the following new chemical systems:

- Ferric chloride
- Aqua ammonia
- Ozone

Labor cost for the operation of the treatment plant is assumed to remain near current staff levels. Even though the plant capacity will increase, it is estimated that the current staff level will be sufficient to operate the plant. Based on 1994 budget projections for water supply and treatment, the treatment portion of the personnel budget was estimated at about $325,000/yr. Anticipated staff levels include one superintendent and 10 staff positions.

Energy cost for the treatment plant is estimated at $150,000/yr, and includes both power and natural gas. The plant building area is expected to be expanded by about one-third and the addition of ozone will create an additional demand on power. The existing budget for power and gas covers the water supply wells and the finished water pump station (which will be included under the pump stations in the NAWS facilities). It is estimated that less than half of the current total energy budget is directly related to water treatment plant operations.

Parshall WTP Operation Costs. The estimated operation cost for the Parshall water treatment plant is $140,000/yr. This plant is being expanded and upgraded from 0.58 mgd to about 1.5 mgd. It is expected that the annual chemical, labor and power costs for the expanded facility will increase from current levels.

The chemical cost is estimated at $40,000/yr to meet the increased capacity and to cover the additional chemical feed systems to be added to the plant. Several of the existing chemical feed systems will be eliminated (i.e., sulfuric acid and alum). The additional chemical feed systems include:

- Ferric chloride
- Aqua ammonia
- Ozone
- Carbon dioxide

Labor cost for operating the water treatment plant is estimated at $80,000/yr. This assumes that two full-time operators will be required.

Power cost is estimated at $20,000/yr. This represents an increase in the power usage in the plant (excluding water supply pumping). The primary energy load in the plant will be the ozone system. The finished water pumping costs are included under pump stations and reservoirs.

Williston WTP Operation Costs. The estimated operation cost for the Williston water treatment plant is $420,000/yr. This plant would be roughly doubled in size from about 7 mgd to 13 mgd. It is expected that annual operations costs will increase from the current estimate of $300, 000/yr because of the upgrades and expansion of the plant.

The chemical cost is estimated at $160,000/yr. All existing chemical feed facilities in the plant will need to be expanded to handle the increased capacity. Additional chemical feed systems will need to be added, including:

- Ferric chloride
- Aqua ammonia
- Ozone

Labor cost for operating the proposed water treatment plant is estimated at $180,000/yr. This assumes an operations staff of six individuals. The estimated labor cost was based on the average labor rate of the existing staff.

Energy costs for the Williston plant include power and natural gas for heat. The estimated energy cost for the upgraded and expanded facilities is $80,000/yr. This cost does not include the WTP finished water pump station which is included under pump stations and reservoirs. Ozone represents a major power load addition to the treatment facilities.

MAINTENANCE COSTS

Maintenance costs include the costs to repair and service equipment and periodically replace worn parts on mechanical equipment such as valve seats, bearings, bushings, packing, motor rewinding, instrumentation and control servicing and programming, etc. Generally, maintenance cost estimates were developed using 1 percent of the repairable or serviceable mechanical equipment capital cost per year. Where capital costs for systems include structural components, such as in water treatment plants and other facilities, a portion of the total capital cost, usually 2 to 50 percent of the capital cost, is assumed to be items that can be serviced.

For special systems requiring repair or service, specific costs were assigned based on actual costs.

Intakes

Maintenance costs for East and Parshall System intakes used a 50 percent factor applied to the construction costs to estimate equipment costs. One percent of this value represents the annual maintenance cost allocation. For the Parshall intake, the capital costs included modifications to the existing raw water pump station. The Williston intake pump station does not include a significant modification under the NAWS project, therefore a dollar estimate was developed for this facility. A summary of the maintenance costs for intakes is presented in Table 7-9.

TABLE 7-9

NAWS INTAKE MAINTENANCE COSTS

System	Capital Cost ($)	Estimated Equipment Cost ($)	Estimated Maintenance Cost ($/year)
East	2,700,000	1,350,000	13,500
Parshall	400,000	200,000	2,000
West	--	--	2,000

Pump Stations and Reservoirs

Maintenance costs for the NAWS pump stations and reservoirs were estimated using the same factors identified above. A summary of the estimated maintenance costs for the three regional systems is presented in Table 7-10.

TABLE 7-10

NAWS PUMP STATION AND RESERVOIR MAINTENANCE COSTS

System	Capital Cost ($)	Estimated Equipment Cost ($)	Estimated Maintenance Cost ($/year)
East	10,400,000	5,200,000	52,000
Parshall	400,000	200,000	2,000
West	3,400,000	1,700,000	17,000

Telemetry/SCADA

The telemetry and control system is essentially all mechanical and electronic hardware. The maintenance costs for the three regional systems were estimated at one percent of the total capital cost for each facility, as summarized below:

- East telemetry/SCADA system - $10,350/yr
- Parshall telemetry/SCADA system - $1,630/yr
- West telemetry/SCADA system - $8,450/yr

Pipelines and Cathodic Protection

The pipelines and cathodic protection maintenance costs assume that 1.5 percent of the pipeline capital costs represents repairable or serviceable facilities (i.e., air release/vacuum valves, line valves, etc.). Annual maintenance costs are figured at one percent of this value. Maintenance for the hydraulic controls are estimated at 50 percent of the capital costs and one percent of that value. Maintenance costs for the cathodic protection system is estimated at values experienced in existing systems and ratioed to represent the number of miles of metallic pipe in each regional system. A summary of the maintenance costs for pipelines and cathodic protect is presented in Table 7-11.

Water Treatment Plants

The estimated maintenance costs for the water treatment plants for the three regional systems is based on 50 percent of the capital cost representing equipment and one percent of that cost representing the estimated maintenance cost. The maintenance costs for the water treatment plants are presented in Table 7-12.

SUMMARY OF O&M COSTS

A summary of the O&M costs for the East, Parshall, and West Systems is presented in Table 7-13. The O&M cost is also expressed as a cost per 1,000 gallons for each of the three systems.

The entire NAWS system has an average flow rate of about 14.92 mgd and a total O&M cost of $3,450,000/year. This equates to a cost of $0.63/1,000 gallons. It is recommended that an O&M cost of $0.65/1,000 gallons be used to account for contingencies and presently unknown O&M items.

TABLE 7-11

NAWS PIPELINE AND CATHODIC PROTECTION MAINTENANCE COSTS

System/ Facility Component	Capital Cost ($)	Estimated Equipment Cost ($)	Estimated Maintenance Cost ($/year)
East			
• Pipelines	63,609,000	954,135	9,540
• Hydraulic controls	522,000	261,000	2,610
• Cathodic protection	--	--	10,000
		Total	$22,150/yr
Parshall			
• Pipelines	2,842,000	42,630	1,000/yr[a]
West			
• Pipelines	15,319,000	229,785	2,875[b]
• Hydraulic controls	265,000	132,500	1,325
• Cathodic protection	--	--	2,000
		Total	$6,200/yr

[a] Includes allowance for existing 11 miles of 10-year old, 8-inch pipe.
[b] Includes allowance for existing 30-inch finished water pipeline from Williston WTP to the City of Williston.

TABLE 7-12

NAWS WATER TREATMENT PLANT MAINTENANCE COSTS

System	Capital Cost ($)	Estimated Equipment Cost ($)	Estimated Maintenance Cost ($/year)
Minot WTP	12,200,000	6,100,000	61,000
Parshall WTP	2,000,000	1,000,000	10,000
Williston WTP	10,000,000	5,000,000	50,000

FINANCING COSTS

Financing costs for capital construction assume a grant of 75 percent of the project cost. Twenty-five percent of the project cost would represent the local share to be repaid by the system users.

Several financing alternatives for the local share of the capital construction costs are still being evaluated by the SWC. One of these alternatives involves bonding. A bonding alternative was used to estimate the user costs associated with the capital component of the NAWS project. For this bonding scenario, a 13-year construction schedule was assumed. Under this schedule, water deliveries would begin in or near year 5, with income allocated to repayment beginning in year 6.

The bonding scenario assumes capital would be raised in three bond issues covering 5, 5, and 3 years of construction at an interest rate of 7.85 percent. The bond analysis assumed that a 10 percent reserve fund would be maintained.

The bond analysis for the NAWS Pre-Final Design plan for the repayment of the local share of the capital construction costs resulted in a cost of about $1.21/1,000 gallons being required to finance the project.

ESTIMATED USER COSTS

The total user costs were estimated using the local share of the total construction cost of $163,900,000, the annual O&M cost based on $3,450,000/year, and other cost components associated with the project, which include:

1. The estimated capital cost repayment and the O&M costs to the Garrison Project for the use of Lake Audubon.

TABLE 7-13

NAWS SUMMARY OF O&M COSTS

Item	System East	Parshall	West
Average Flow (mgd)	10.23	0.61	4.08
OPERATION COSTS			
Intakes			
• Pumping Energy	$311,100	$8,360	$14,430
• HVAC	3,500	800	0
• Miscellaneous	5,000	1,200	1,000
• Chloramination	45,000	0	0
Pump Stations/Reservoirs			
• Pumping Energy	263,600	8,500	103,400
• HVAC	11,130	0	7,950
• Equipment/Supplies	100,000	10,000	90,000
• Labor	362,340	20,280	281,220
Pipelines/Cathodic Protection			
• Cathodic Protection	5,000	0	2,000
Water Treatment Plants			
• Chemicals	500,000	40,000	160,000
• Labor	325,000	80,000	180,000
• Power	150,000	20,000	80,000
MAINTENANCE COSTS			
Intakes	13,500	2,000	2,000
Pump Station/Reservoirs	52,000	2,000	17,000
Telemetry System	10,350	1,630	8,450
Pipeline/Cathodic Protection	22,150	1,000	6,200
Water Treatment Plants	61,000	10,000	50,000
TOTALS	$2,240,670	$205,770	$1,003,650
Cost/1,000 Gallons	$0.60	$0.92	$0.67

2. A replacement reserve account to generate 10 percent of the total estimated NAWS project costs within a period of 15 years.

The NAWS project is estimated to cost an average of $2.15/1,000 gallons as shown in Table 7-14. For comparison purposes, this table also includes a separate rate analysis assuming the East System Water Treatment Plant would be built in the Missouri River Basin instead of upgrading and expanding the Minot WTP. In this analysis the total capital project costs are estimated at $173,400,000 and the annual O&M costs are estimated at $3,380,000/year. The same bonding analysis for the local share of the capital project costs was used for the Missouri River Basin site as was used for the upgrade of the Minot WTP.

Based on the estimated $2.15/1,000 gal rate for NAWS water, an estimate of the retail water charge for various usage volumes was prepared for selected cities and rural water systems. These are shown in Table 7-15, which also includes comparative data for current charges.

Table 7-16 presents a comparison data for water bills for other cities in North Dakota for 6,000 gallons of water use.

TABLE 7-14

NAWS ESTIMATED WATER USER RATES

Total NAWS Project Costs	Upgrade and Expansion of Minot WTP	Missouri River Basin Softening WTP
Total Construction Cost (million $)	$163.9	$173.4
Annual O&M Costs (million $)	$3.450	$3.380
Estimated Water Rate Cost Components		
Capital Costs/1,000 gals[1]	$1.21	$1.31
O&M Cost/1,000 gals	$0.65	$0.63
Garrison Project, Capital Repayment and O&M costs/1,000 gals	$0.05	$0.05
Replacement Costs/1,000 gals[2]	$0.24	$0.26
Estimated Water Rate/1,000 gallons	**$2.15**	**$2.25**

[1] Preliminary estimate based on one bonding scenario for comparison purposes only.

[2] Cost to generate ten percent of total project cost within 15 years.

TABLE 7-15

ESTIMATED COST OF WATER (RETAIL)
NAWS PRE-FINAL DESIGN

City[1]	Water Use (Gallons Per Month)							
	Current				Current Plus NAWS			
	2000	4000	6000	8000	2000	4000	6000[3]	8000
Bottineau	5.50	7.90	10.30	12.70	10.00	16.90	23.80	30.70
Bowbells	7.50	10.50	13.50	16.50	12.00	19.50	27.00	34.50
Crosby	8.00	14.00	20.00	26.00	12.50	23.00	33.50	44.00
Grenora	10.50	10.75	11.75	12.75	15.00	19.75	25.25	30.75
Kenmare	20.00	25.00	30.00	35.00	24.50	34.00	43.50	53.00
Makoti	3.00	5.00	7.00	9.00	7.50	14.00	20.50	27.00
Minot[2]	6.66	9.33	12.38	15.41	11.16	18.33	25.88	33.41
Mohall	11.30	13.90	16.50	19.10	15.80	22.90	30.00	37.10
Newtown	12.90	15.70	18.50	21.30	17.40	24.70	32.00	39.30
Parshall	29.00	32.00	35.00	38.00	33.50	41.00	48.50	56.00
Powers Lake	16.00	16.00	16.00	17.30	20.50	25.00	29.50	35.30
Rugby	7.75	11.25	14.75	18.25	12.25	20.25	28.25	36.25
Towner	13.00	15.00	17.00	19.00	17.50	24.00	30.50	37.00
Williston[2]	6.58	10.06	13.53	17.02	11.08	19.06	27.03	35.02
All-Seasons RW	39.00	49.00	59.00	69.00	43.50	58.00	72.50	87.00
North Prairie RW	28.00	36.00	44.00	52.00	32.50	45.00	57.50	70.00
Upper Souris RW	32.80	41.60	50.40	59.20	37.30	50.60	63.90	77.20
Williams RW	35.00	45.00	55.00	65.00	39.50	54.00	68.50	83.00

(1) Cities and Rural Water System (RW) to be served by NAWS.
(2) Rates based on cubic feet have been converted to cost per 1,000 gallons.
(3) Average cost in cities for 6,000 gallons is $29.83. Average cost on Rural Water Systems for 6,000 gallons is $65.60.

TABLE 7-16

COMPARISON OF WATER BILLS IN NORTH DAKOTA CITIES
NAWS PRE-FINAL DESIGN

City	Minimum Charge	Additional Gallons	Cost for[3] 6,000 Gallons	Date Effective
Bismarck	$6.66 for first 300 cubic feet	$1.14/100 cubic feet	$12.38	1985
Dickinson[1]	$8.00 flat rate	$3.25/1000 gallons	$27.50	1991
Fargo	$5.10/mo. for 2000 gallons	$2.55/1000 gallons $2.30/1001-200,000 gallons $2.05/Over 200,000 gallons	$15.30	1994
Grand Forks	$3.17 fat rate	$1.82/1000-30,000 gallons $1.67/30,001-200,000 gallons $1.52/Over 200,000 gallons	$14.09	1994
Mandan	-0-	$1.25/100 cubic feet	$10.03	1992
West Fargo	$3.00 for 2000 gallons	$1.50/2001-10,000 gallons $1.00/Over 10,000 gallons	$9.00	1987
Williston	$3.10 fat rate	$1.30/100-3,000 cubic feet $0.95/Over 3,000 cubic feet	$13.53	1994
Turtle Lake[2]	$14.25 for first 1000 gallons	$1.75/1000 gallons	$23.00	
New Salem[2]	$13.00 for first 1000 gallons	$3.00/1000 gallons	$28.00	
Lansford[2]	$11.00 for first 2000 gallons	$4.00/1000 gallons	$27.00	
Max[2]	$7.50 for first 1000 gallons	$2.90/1000 gallons	$22.00	
Stanley[2]	$18.53 for first 1000 gallons	$3.53/1000 gallons	$36.18	

(1) Dickinson is supplied by the Southwest Pipeline.
(2) These cities are currently supplied by Rural Water Systems.
(3) The average cost in all the larger cities except Dickinson, is $12.47.
The average cost in Dickinson and cities supplied by Rural Water Systems is $27.28.

Section 8

HOUSTON ENGINEERING
AMERICAN ENGINEERING
MONTGOMERY WATSON

SECTION 8

OPERATION PLAN

INTRODUCTION

This section describes an operation plan that delineates alternatives for various agencies to operate the NAWS system, and delineates the relationships between the NAWS system and the municipal and rural water delivery systems.

The SWC was authorized by the 1991 Legislature to plan, construct, operate, and manage the NAWS project. Although the SWC could operate and manage the NAWS project, it intends to turn over the operation and management of the project after delivery of water begins. The type of organizational structure to manage the project will be the choice of the NAWS water users.

OPERATION ALTERNATIVES

Five alternatives have been developed for the potential organizational structures to operate the NAWS system. A description of each is presented in the following sections.

Northwest Water Authority

This organizational structure would be similar to the Southwest Water Authority, and could be created by the legislature, Chapter 61-24.5 of the North Dakota Century Code. The authority would be similar to the Garrison Diversion Conservancy District (GDCD) but would have a different makeup of the board of directors. This authority could have the capability of levying taxes.

Expansion of the GDCD

NAWS could become a component of the GDCD if Divide, Burke, Williams, and Mountrail counties would petition to enter the GDCD pursuant to §61-24-02 of the North Dakota Century Code. If the county commissions voted to submit petitions to the GDCD board of directors, and the petitions were approved, the board would increase to 30 members, the taxable valuation of the GDCD would increase, and the size of the district would significantly increase.

This option would not require legislative approval. The GDCD is authorized to enter into the necessary contracts for operation and maintenance. The GDCD also has authorization to establish a mill levy.

Cooperative Agreements

The GDCD board of directors could enter a cooperative agreement with the SWC to provide the administrative services including operation and maintenance of the NAWS system. The basic authority for cooperative agreements is Article VII, §10, of the North Dakota Constitution which states, in part: "Agreements, including those for cooperative or joint administration of any powers or functions, may be made by any political subdivision with any other political subdivision, with the state, or with the United States, unless otherwise provided by law or home rule charter." This option would not require legislative action.

Joint Water Board

Water Resource Districts (WRD's) are authorized, pursuant to §61-16.1-09(20) of the North Dakota Century Code, to develop water supply systems, store and transport water, and provide, contract for, and furnish water service for domestic, municipal, and rural water purposes, irrigation, milling, manufacturing, mining, metallurgical, and any and all other beneficial uses, and fix the terms and rates for water supply. The appropriate water resource districts (and possibly the cities to receive water) could establish a joint board pursuant to Article VII, §10, of the North Dakota constitution and §61-16.1-11 of the North Dakota Century Code to construct, operate, and maintain the NAWS system. The GDCD could enter a cooperative agreement with a joint board to provide the administrative services and operation and maintenance services. This option would not require legislative action.

Cooperative

A cooperative could be established which could act like an area-wide rural water district. Under this arrangement no taxation authority would exist. A board of directors would be elected.

Section 9

HOUSTON ENGINEERING
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MONTGOMERY WATSON

SECTION 9

CONSTRUCTION SCHEDULE

INTRODUCTION

A preliminary construction schedule was developed with the objective of supplying the largest water users with high quality treated water at the earliest opportunity. Although definitive dates of construction have yet to be developed, input from the SWC has identified the following priorities for construction to meet the above goal:

1. Lake Audubon to Minot
2. East System service to Upper Souris
3. East System service to Carpio
4. Parshall System
5. West System
6. East System service to Bottineau
7. East System service to Rugby
8. East System service to Anamoose

As a part of each system's scheduling priority, the basic telemetry/SCADA control system will be installed. Remote telemetry units will be installed as piping and facility segments are constructed. The telemetry/SCADA systems are included in total for the Parshall and the West system construction schedules. On the East System, only the portion of the telemetry/SCADA system corresponding to the construction schedule segment would be built. The East System telemetry/SCADA system has a total construction cost of $1,035,000, and is described in detail in Section 4.

A detailed implementation schedule will be prepared once the SWC defines the actual priorities for the individual project elements. The final schedule will be based on a number of issues, including but not limited to economic considerations, water quality regulations, and local interest in being served by the project.

A description and summary of cost for each construction priority element is presented below. Cost estimates are based on the segment analysis presented in Section 7.

AUDUBON TO MINOT

The Audubon to Minot element includes the raw water supply system and the Minot WTP upgrades in the East System. This system will supply a peak day raw water flow to the treatment plant of about 28 mgd. The project includes the raw water intake/pump station, raw water pipeline, raw water booster pump station near the town of Max, the raw water storage basin and the expansion and upgrades at the Minot WTP.

The following construction segments and construction costs describe this system and are detailed in Section 7.

• E1 - Lake Audubon to Minot	$29,539,000
• E2 - Minot WTP	12,240,000
Total	$41,779,000

EAST SYSTEM SERVICE TO UPPER SOURIS

This construction element provides the distribution piping, booster pumps, storage reservoirs, and hydraulic controls from the Minot WTP to the north. This element serves Minot AFB, Mohall, Sherwood, Upper Souris I, Upper Souris II, Renville County, All Seasons III, and Kenmare on the East System.

The following construction segments and construction costs describe this system and are detailed in Section 7.

• E3 - Distribution piping - Minot WTP to Minot AFB	$7,321,000
• E4 - Distribution piping - Minot AFB to FW3	4,859,000
• E5 - Distribution piping - FW3 to Kenmare	5,281,000
Total	$17,461,000

EAST SYSTEM SERVICE TO CARPIO

This construction element provides distribution piping, booster pumps, storage reservoirs, and hydraulic controls from the finished water distribution line from the Minot WTP, west to serve Burlington, Berthold and Carpio on the East System.

The following construction segment and construction cost describe this system and is detailed in Section 7.

• E7 - Distribution piping - Minot to Carpio	$2,339,000

PARSHALL SYSTEM

The Parshall System includes the raw water intake, raw water pipelines, Parshall WTP upgrades, and the finished water distribution system serving New Town, Makoti, and Plaza.

The following construction segments and construction costs describe this system and are detailed in Section 7.

• P1 - Raw Water Intake and Pipeline	$1,152,000
• P2 - Parshall WTP	2,000,000
• P3 - Distribution Pipelines	2,490,000
• P4 - Parshall System Telemetry	163,000
Total	$5,805,000

WEST SYSTEM

The entire West System is identified as a construction schedule element. This system includes the raw water pump station improvements, Williston WTP upgrades, pump stations, storage reservoirs, the finished water distribution pipelines, and hydraulic controls.

The following construction segments and construction costs describe this system and are detailed in Section 7.

• W1 - Raw Water Pump Station and WTP Upgrades	$10,000,000
• W2 - Distribution Piping to Ray	7,822,000
• W3 - Distribution Piping to Grenora and Powers Lake	4,310,000
• W4 - Distribution Piping to Bowbells and Fortuna	6,852,000
• W5 - West System Telemetry	845,000
Total	$29,829,000

EAST SYSTEM SERVICE TO BOTTINEAU

This portion of the East System provides distribution piping, pump stations, storage reservoirs, and hydraulic controls to the cities of Maxbass, Westhope, Souris, and Bottineau, and includes All Seasons I and All Seasons II.

The following construction segment and construction cost describe this system and is detailed in Section 7.

- E6 - Distribution System to Bottineau $5,897,000

EAST SYSTEM SERVICE TO RUGBY

This portion of the East System provides distribution piping, pump stations, storage reservoirs, and hydraulic controls to provide service from Minot to Rugby. Service to the cities of Deering, Granville, Towner, Upham, and Willow City would also be provided.

The following construction segment and construction cost describe this system and is detailed in Section 7.

- E8 - Distribution System to Rugby $16,973,000

EAST SYSTEM SERVICE TO ANAMOOSE

This portion of the East System provides distribution piping, pump stations, storage reservoirs, and hydraulic controls to provide service to Anamoose. Service will also be provided to Sawyer, Ruso, Karlsruhe, and Drake. This distribution system connects to the distribution system providing service form Minot to Rugby (E8) described above.

The following construction segment and construction cost describe this system and is detailed in Section 7.

- E9 - Distribution System to Anamoose $4,982,000

Appendix A

HOUSTON ENGINEERING
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NORTHWEST AREA WATER SUPPLY PROJECT
PRELIMINARY CORROSION EVALUATION

PREPARED
FOR

NORTH DAKOTA STATE WATER COMMISSION
AND
GARRISON DIVERSION CONSERVANCY DISTRICT

MAY 1994

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Intermountain Corrosion Service, Inc.

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EXECUTIVE SUMMARY

This report discusses potential corrosive soil conditions believed to exist along the proposed alignment of steel or iron pipe sections for the Northwest Area Supply Project (NAWS) located in North Dakota. A preliminary evaluation has determined that soils are likely to be corrosive to coated steel and iron pipe. Extreme variation in soil conditions is characteristic of the glacial till found in the subject area. Soils data evaluated for this report were taken from USDA reports and an existing corrosion investigation prepared for the Southwest Pipeline project.

Corrosion control measures are likely to be necessary to increase the useful life of the metallic pipeline system. Components of the corrosion control plan should include high performance coatings, cathodic protection where required, and proper material selection.

Each of the proposed pipeline materials, will require application of corrosion control measures specific to the material and the environment. Appropriate corrosion protection measures can be developed during design to insure equal life for alternative materials.

A pre-design corrosivity investigation is necessary to evaluate conditions specific to the pipeline system, and recommend an effective corrosion control plan.

1.0 INTRODUCTION

This report has been prepared to identify factors which have the greatest probability of promoting corrosion attack on metallic pipeline materials being considered for the Northwest Area Water Supply Project (NAWS) located in northwestern North Dakota.

Environment or soil corrosion is discussed in terms of proposed pipeline materials: tape coated steel, ductile cast iron, and mortar coated steel. Soil information is taken from previously published soil survey reports. Soil chemistry, necessary for a corrosivity analysis, is not included in these reports. This report discusses soil conditions and elements that are expected to occur along the proposed pipeline alignments, and their potential corrosive affect.

Research and investigative data indicate the corrosion rate of carbon steel, grey cast iron, and ductile cast iron used for piping material is essentially equal in a common environment. To date, there is no conclusive evidence to the contrary, therefore, these materials will be assumed identical in their need for corrosion protection.

2.0 SOIL CHEMISTRY

2.1 General

Corrosion of underground pipelines seldom results from a single factor. Several variables are usually involved, and each contributes in some degree to the total corrosion effect. Major causes of corrosion result from environmental differences which exist along the pipe surface. Variations in types of soil traversed by the pipeline and differences in oxygen content, moisture content, and chemical constituents of soil and water lead to the formation of corrosion cells. Extraneous influences created by adjacent structures and electrical fields from stray currents also contribute to corrosion damage.

In the Northern Great Plains, man has disrupted the natural form of the soil to develop conditions known as saline seeps. A saline seep is an area of native soil that collects dissolvable salts that are transported by water in the soil from areas that have been fallow farmed. An unbalance can occur that can increase native soil conductivity to greater than twice that of the farmed soil.

2.2 Soil Resistivity

Corrosion of a metal is electrochemical in nature and is always accompanied by the flow of electric current. Resistivity is a measure of the ability of a soil to conduct an electric current and is, therefore, an important parameter in consideration of corrosion data. Soil resistivity is primarily dependent upon the chemical content and moisture content of the soil mass. The greater the amount of soluble chemical constituents present in the soil, the lower will be the resistivity. As moisture content increases, resistivity decreases until maximum solubility of dissolved chemicals is attained. Beyond this point, an increase in moisture content results in dilution of chemical concentrations and resistivity increases.

The corrosion rate of steel or iron in soil normally increases as resistivity decreases. Therefore, in any group of soils, maximum corrosion will occur in the lowest resistivity areas. The following classification of soil corrosivity, relating absolute resistivity values, is frequently utilized for analysis of soil data:

TABLE A

Corrosion Class	Resistivity ohm-cm
Severely Corrosive	0 - 500
Very Corrosive	500 - 1,000
Corrosive	1,000 - 3,000
Moderately Corrosive	3,000 - 10,000
Slightly Corrosive	Above 10,000

The NAWS pipeline is expected to traverse soil which varies in resistivity. Sections of pipe located in low resistivity areas become anodic to sections of pipe located in high resistivity areas. This results in the formation of corrosion cells, with metallic dissolution occurring at the anodic regions.

The presence of substantial range of resistivity along a pipeline would be conducive to the formation of long line corrosion cells. For this type of cell action, the magnitude of variation in resistivity is of greater significance than the absolute value of resistivity. For example, the amount of corrosion occurring on a pipeline located in a soil environment which ranges in resistivity from 2,000 ohm-cm to 20,000 ohm-cm will frequently be greater than for a pipeline located in soils which range from 1,000 ohm-cm to 2,000 ohm-cm. The greater the range in resistivity values, the greater will be the driving force between anodic and cathodic areas along the pipeline, and the resulting metal loss due to corrosion.

2.3 pH, Acidity, Alkalinity

The concentration of hydrogen ions indicated the degree of acidity. The degree of acidity is expressed as the pH which is the negative logarithm of the reciprocal of the hydrogen ion concentration.

Seldom do we encounter soils with a pH lower than 5 (acid), or greater than 8 (alkaline), where 7.0 is neutral. Soils with a pH 5 and lower can contribute to severe corrosion cell activity resulting in rapid deterioration of exposed metals. Acid soil condition are usually limited to soils containing decomposed acidic plants and needles from coniferous trees.

Soils with a pH range between 6.5 and 7.5 are excellent for sulfate reduction which will result in bacteriological corrosion of metals.

Soils with a pH of 8 and above can contain a high concentration of dissolved salts resulting in a low soil resistivity. Soils of this type are referred to as alkaline or calcareous. Alkaline soils are high in sodium and potassium, while calcareous soils are high in magnesium and calcium. Alkaline earth elements of calcium and magnesium tend to form a protective layer on ferrous metal surfaces recognized as calcareous deposits that are formed through precipitation, which is a favorable environment for metals.

2.4 Chloride

Chloride ions in very small concentrations, or an impurity in a raw material can cause development of active corrosion cells. Increased chloride concentrations result in lowering soil resistivity, thereby facilitating corrosion currents.

2.5 Sulfates

Pipeline materials are sensitive to the presence of soil sulfates. Sulfate levels are frequently associated with high concentrations of organic matter, which leads to bacteriological corrosion. Also, the primary source of soil electrons is from decomposition of organic matter which contributes to corrosion cell activity. Only sulfate reducing bacteria are capable of reducing soil sulfates, therefore, their presence must be known.

3.0 PIPELINE ROUTES

3.1 General

The NAWS pipeline system is separated into the West, East, and Parshall systems. Pipeline route soils data were taken from soil surveys published by the U.S. Department of Agriculture (USDA). While the generality of the soil data does not permit specific design considerations for each route, general observations are possible and are described below.

3.2 East System

The metallic sections of the East System extend north, south, and east from Minot. The proposed alignment of the south reach is through soil ranging from clay loam to sandy loam which is generally well drained. pH is generally neutral, above pH 6.6, but pockets of acid soils are likely. Pockets of clay, peat or acid soils, would increase the corrosivity along the pipeline route. Risk of corrosion in the USDA reports is rated as moderate to high for uncoated steel and low for concrete.

The soil characteristics for the proposed alignment for the north and east reaches are similar with sandy and loamy soils, which are moderately well drained with pockets of poor drainage. pH is generally neutral, greater than pH 6.6, but pockets of acid soils are likely. Pockets of clay, peat or acid soils, would increase the corrosivity along the pipeline route. Risk of corrosion in the USDA reports is rated as moderate to high for uncoated steel and low for concrete.

3.3 Parshall System

Based on proposed pipeline sizes, no metallic construction materials will be used in this section. Soils in this area have a risk of corrosion rated as moderate to high, however, and corrosion protection should be considered for pump stations and tank bottoms.

3.4 West System

Soil data is not available for Williams County, location of the proposed metallic pipeline sections for the West System. The soils along this proposed alignment are believed to be similar to those encountered along the south reach in the East System.

The USDA data indicates the soils in the West System could be quite similar to soils addressed in the report, " Corrosion Control of Southwest Pipeline," prepared in 1982 for the section immediately south of the West System. This survey extended south from Lake Sakakawea to Dickinson, then terminated just south of the Hettinger-Adams County Line. Conclusions presented in the report found extreme variations in soil resistivity; chemical analysis of soil samples rated 90% as moderate to very corrosive to steel, 40% with sulfates content rated as aggressive to concrete, and found 59% contained anaerobic bacteria. Several stray current sources also were located.

4.0 CATHODIC PROTECTION

4.1 General

Today cathodic protection plays an ever increasing role in virtually every industry in our economy. Cathodic protection has been applied to a wide variety of subjects ranging from thumb tacks to offshore drilling platforms. The U.S. Department of Transportation has recognized cathodic protection as an essential element to control corrosion on buried pipelines and has adopted regulations requiring buried gas and petroleum pipelines to be cathodically protected. Also, buried steel gas tanks are required by federal regulations to be equipped with cathodic protection.

The combination of a good coating system and cathodic protection can reduce maintenance costs considerably. All coating systems are subject to defects or coating faults; no coating system has been developed that has permanent life. Any defect in a coating system will result in corrosion at the defect or holiday. When holidays are underground, corrosion can be halted by cathodic protection. Protective coatings, and cathodic protection cannot replace each other, but instead enhance the performance of each.

When a defect occurs in a protective coating, exposing the metal surface to the environment, corrosion will proceed at an accelerated rate causing deep pitting and early failures in the structure. This is because of the relationship that exists between the very small anodic area at the location where the metal is exposed, and the extremely large cathodic area composed of the surface still protected by the coating. This relationship results in all of the potential for corrosion that is inherent in the entire structure being concentrated at the small anodic area, resulting in accelerated corrosion at this location.

Voids in coating may be the result of "holidays" or pin-holes that go undetected and unrepaired during the coating applications, or they may be caused by disbonding and deterioration of the coating that occur with time.

Many variable factors present themselves during the design of a cathodic protection system. It is the Corrosion control Engineer's responsibility to address the variable and unpredictable factors to provide the most practical and cost effective cathodic protection system for the structure involved. Satisfactory design results from: experience, application of scientific formula, and the practical experience of the Engineer.

Cathodic protection systems should operate with the lowest practical power consumption to provide even current distribution along the structure. Cathodic protection is a form of corrosion, an electromechanical process, where the process is controlled to mitigate damage or failure by a natural process. Generally, cathodic protection can be divided into two basic forms, **impressed current** where an external power source is utilized to force current through the cathodic protection circuit, and **galvanic** where the driving force is the difference in potential between two metals. Special consideration must be given to control the effects to foreign structures when impressed current systems are installed.

4.2 Galvanic Anode Cathodic Protection

Galvanic anode systems are desirable due to the absence of outside power and lack of maintenance expense. Galvanic cathodic protection systems use no outside source of power. The current used for protection is derived solely from the anode itself, and is due to the difference in potential between the anode and the cathode. Galvanic anodes are generally distributed along the pipeline and installed in close proximity of the cathode.

The entire system consists of an ingot of anode material connected to the cathode so there is little or no maintenance involved in operations. The galvanic anode corrodes in order to produce the protective current, and is consumed over a period of time. Consumption of the anode material gives it a finite life. When the anode is completely expended, it must be replaced if the system is to continue operation.

Other characteristics often present problems to the Corrosion Engineer. Most troublesome of these is the small current output of the anodes. Often, the system design calls for large protective currents. Installation of multiple anodes can be used, but may not be economically effective.

4.3 Impressed Current Cathodic Protection

As the required current for a system becomes larger, there comes a point at which use of a galvanic system is unacceptable. At this point, the impressed current system comes into play. The differences between the two systems is basically the **source of power**. Whereas the galvanic system derived its power from the difference between two metals, the power for an impressed current cathodic protection system comes from an external AC power source. Thus, the system is not limited by a low driving voltage, and consequently has theoretically unlimited output capacity. As a result, the output of the rectifier (volts) is normally adjustable over a wide range making the system highly versatile and readily adaptable to changes within the projected system.

Rectifier stations can be widely spaced along the pipeline. Concentrating the protective current source means current must be distributed over a larger area requiring the anodes to be located remote from the cathode. As a result, it is necessary to either encroach beyond the pipeline right-of-way, or install the anodes in a anode well located within the right-of-way. The well, referred to as a deep well, is a method which has been successfully utilized for years and is the preferred method for long pipeline cathodic protection. One impressed current station can protect up to 15 miles of pipeline.

Because of the many variables involved, and the possibility of equipment failure, impressed current systems require a certain amount of maintenance and repair. In addition, there is a constant cost for power which must be taken into account as an operating expense.

5.0 FOREIGN STRUCTURES

5.1 Foreign Cathodic Protection

Installation of a pipeline near an impressed current cathodic protection station or a foreign structure that is receiving cathodic protection exposes the proposed pipeline to induced corrosion failures caused by stray direct currents. The proposed pipeline can be either the source of stray current activity or its victim. Therefore, the design should minimize potential stray current effects.

Cathodically protected structures are likely to be located in the vicinity of some of the proposed pipeline alignments. Examples include Department of Defense (DOD) missile silos, natural gas pipelines, petroleum pipelines, other water pipelines, and underground storage tanks. Each of the owners of these structures should be notified of the pipeline project. Representatives should indicate the type and location of any cathodic protection stations located in the vicinity of the proposed pipeline alignments. This information will be required during the design phase of the project. The new pipeline design can incorporate features necessary to control or mitigate stray currents should they occur. Stray currents can be effectively controlled by: protective coatings, electrical isolation of pipeline segments, resistance bonds, cathodic protection, or a combination of these.

The magnitude of stray current corrosion activity expected on the proposed pipeline system is not easily predicted. Interaction between existing and new pipelines will depend substantially on the new pipeline design. There are methods for measuring transient current conducted in the ground at the vicinity of the existing cathodically protected pipeline. The results, however are inconclusive since they do not include an essential constituent of the circuit, the proposed pipeline. Monitoring facilities are therefore required to provide for testing the pipeline after installation.

5.2 Alternating Current (AC) Power Line Interference

Safety features should be incorporated if a section of the proposed alignment parallels overhead transmission lines. The problems associated with induced voltages and currents from overhead AC power transmission lines are not only evident and troublesome after a pipeline or structure is buried in the power line corridor, but are also evident and dangerous during construction of the pipeline or structure.

The AC coupling effect has occurred with individual steel sections of pipe, and joined or welded sections of pipe laid along the pipeline trench in preparation for burial. If not properly grounded, these sections of pipe will react as large capacitors storing induced voltages and current. Discharge of these "capacitors" through construction personnel can result in serious shocks and fatalities. If the pipe sections are placed on sandbags or wood skids, the problem can be even more serious.

6.0 PIPELINE MATERIALS

6.1 General

When analyzing soil and other test data for corrosiveness, it is necessary to consider the particular type of structure involved. Due to basic differences in the type of pipe available for use and resulting differences in the corrosion phenomena associated with each, it is necessary to analyze results of this investigation in terms of each type of pipe under consideration. The pipe materials addressed in this investigation are mortar coated steel or iron, and tape or enamel coated steel or iron.

6.2 Cement Mortar Coated Pipe

Materials considered under this category will be cement mortar lined and coated steel in accordance with AWWA C205. Based upon expected soil conditions along the proposed alignment, soils should be considered corrosive to steel or iron in concrete. The cement mortar coated pipe should be electrically isolated from all side outlets and connections to existing facilities. To allow the application of cathodic protection, when necessary, and to monitor corrosion activity, electrical continuity is required. Continuity can be provided by welded joints, or the installation of continuity bonds where rubber ring joints or mechanical couplings are used.

For future testing and monitoring of corrosion activity and cathodic protection levels, test stations will be required and should be installed along the entire length of the pipeline. Test stations should also be installed at insulating fittings and foreign pipeline crossings.

6.3 Tape or Enamel Coated Pipe

Pipe considered under this category will be steel or iron pipe with a dielectric coating. Based upon available data analysis of soil along the proposed alignment in the West and East Systems, the environment is considered corrosive for steel.

Pipeline tape has been successfully used as an exterior protective coating for steel pipe. The tape system should conform to AWWA Standard C214. Tape dimensions and thickness, cleaning standards and inspection, and test requirements are a few of the special requirements which should be addressed in the specification as recommended in the forward of AWWA C214.

All appurtenances and exterior field joints should be coated with a mastic and a cold applied tape, in accordance with AWWA C209, respectively, to form an all-encompassing protective coating.

Cathodic protection will be required to prevent corrosion damage to the steel pipe due to imperfections or holidays in the pipe coating. The enamel or tape coated pipe should be insulated from all lateral connections and tie-ins to other pipelines and structures. In order to apply cathodic protection to the pipeline, electrical continuity along the entire length of the pipeline is required. Electrical continuity can be obtained by welding the pipe joints or installation of continuity bonds where rubber ring joints are used.

For future testing and monitoring of corrosion activity and cathodic protection levels, test stations will be required and should be installed along the entire pipeline. Test stations should also be installed at insulating fittings and foreign pipeline crossings. The pipe should be electrically isolated from other metallic structures. Isolation is required to prevent galvanic corrosion between dissimilar metals or metals with dissimilar coatings. Isolation is also required for the proper operation of the cathodic protection system.

7.0 RECOMMENDATIONS

Considerable knowledge of the pipeline alignments is necessary to assess contributing factors to the corrosion processes. An in-depth investigation is recommended to provide sufficient information to develop conclusions, and present recommendations for the entire pipeline system. The procedures we are recommending will provide necessary information to assess potential corrosion problems and assist in selecting the most desirable corrosion control procedures.

The field investigation should include site reconnaissance, contact with local utility owners, and soil data collection. Site reconnaissance will provide information such as stray current sources, rectifier locations, and pipeline sections which require special attention. Contact with local utility owners will reveal potentially corrosive conditions which are not obvious, and can reduce some testing efforts. Soil data collected along the alignment will be compared to laboratory analysis of soil samples to assess fully the corrosive conditions specific to each material alternative. Soil samples should be collected at pipeline depth during the geotechnical phase of the pipeline design.

A design report should be prepared presenting an evaluation of the soil corrosivity in terms of the selected pipeline materials. Specific corrosion control measures should be described including coating systems, cathodic protection, and facilities required for future testing. The following procedures summarize the tasks necessary for a complete corrosivity study:

1. Review potentially corrosive conditions with other agencies and utility owners.

2. Conduct soil resistivity measurements along each pipeline route, with condensed intervals where suspected problem soils exist. Obtain preliminary design information.

3. Analyze soil resistivity test and select locations for soil samples to be taken for chemical analysis. Samples would be analyzed for:

   Resistivity
   pH
   Sulfates
   Chlorides
   Total Acidity (when required)

4. Analyze soil resistivity tests and laboratory data from samples taken and evaluate corrosivity of soil conditions in terms of selected material alternatives.

5. Evaluate potential effects of foreign structure cathodic protection systems and other sources of stray ground currents.

6. Recommend specific corrosion control measures for each pipeline segment.