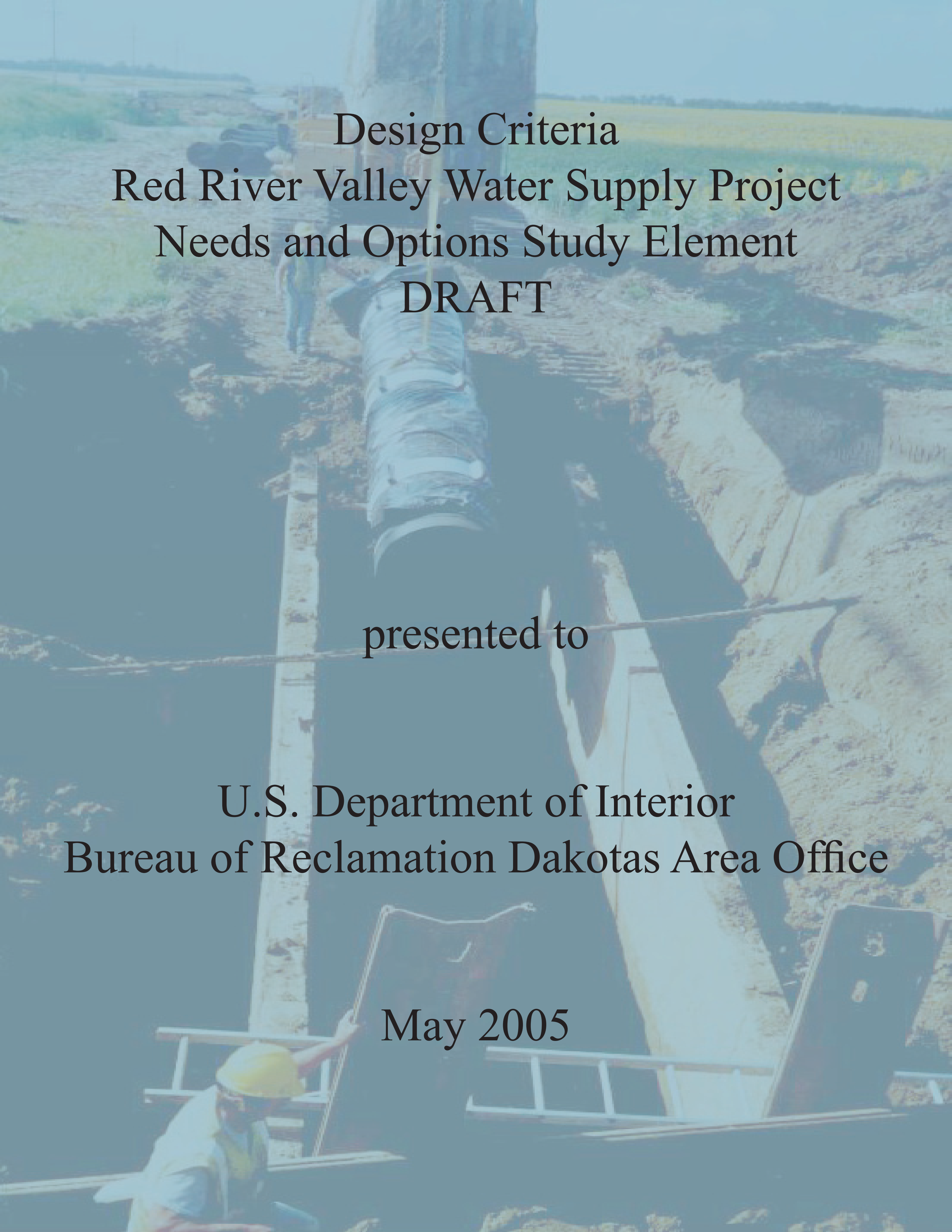

Design Criteria
Red River Valley Water Supply Project
Needs and Options Study Element
DRAFT

presented to

U.S. Department of Interior
Bureau of Reclamation Dakotas Area Office

May 2005

This report was prepared by Houston Engineering Inc. (HE) and Montgomery Watson Harza Americas (MWH) for the U.S. Department of the Interior Bureau of Reclamation under contract with the Garrison Diversion Conservancy District.

MWH

DESIGN CRITERIA
RED RIVER VALLEY WATER SUPPLY PROJECT
NEEDS AND OPTIONS STUDY ELEMENT
DRAFT

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U.S. DEPARTMENT OF INTERIOR
BUREAU OF RECLAMATION DAKOTAS AREA OFFICE

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SECTION 1

SECTION 1

INTRODUCTION

1.1 BACKGROUND

This report outlines the criteria to be used for the water source and supply engineering options for the Red River Valley Water Supply Project (Project) facilities for the Report on Red River Valley Water Needs and Options (Needs and Options Report) design. Modifications and refinements may occur during the study and design process and could result in changes to the criteria presented in Section 2. The intent of the document is for it to be flexible and for changes to be adapted during the progress of the study. However, the continuing planning, predesign and development of design criteria for the Project should remain within the framework established through this process. The chosen criteria will emphasize cost effectiveness, environmental compatibility and long-term flexible operation and construction biddability with reduced maintenance costs.

The objectives of this report are to establish the criteria necessary for a realistic design of the Project water supply and storage facilities and also to document the reasons behind their selection. This document is presented to satisfy the requirements in the contract scope of services between the Garrison Diversion Conservation District (Garrison Diversion) and the U.S. Department of Interior Bureau of Reclamation (Reclamation).

1.2 SCOPE OF REPORT

This report lists the necessary design criteria, describes system component evaluation and selection, and provides sufficient detail to allow the design and study of the Project element to proceed. The design criteria for the following items are included:

- Pipelines
- Pump stations
- Water storage facilities
- Raw water intake/diversion
- System instrumentation and control
- Wellfield

1.3 COMMON ACRONYMS AND SHORTHAND TERMS

CEC	Categorical Exclusion Checklist
CEQ	Council on Environmental Quality
CERC	USGS Columbia Environmental Research Center
COE	U.S. Army Corps of Engineers
CWA	Clean Water Act

DEIS	Draft Environmental Impact Statement
DWRA	Dakota Water Resources Act
EA	Environmental Assessment
EIS	Environmental Impact Statement
EPA	Environmental Protection Agency
ESA	Endangered Species Act
ESWTR	Enhanced Surface Water Treatment Rule
FEIS	Final Environmental Impact Statement
FONSI	Finding of No Significant Impact
FWCA	Fish and Wildlife Coordination Act
Garrison Diversion	Garrison Diversion Conservancy District
Groundwater	Ground water
GDU	Garrison Diversion Unit
GIS	Geographic Information System
HEP	Habitat Evaluation Procedures
HYDROSS	Hydrologic River Operation Study System (modeling)
Master POS	Master Plan of Study
MNDNR	Minnesota Department of Natural Resources
MOU	Memorandum of Understanding
MPCA	Minnesota Pollution Control Agency
MR&I	Municipal, Rural, and Industrial
NDCC	North Dakota Century Code
NDDH	North Dakota Department of Health
NDSU	North Dakota State University, Fargo
Needs and Options Report	Report on Red River Valley Water and Options
NHPA	National Historic Preservation Act
NOI	Notice of Intent
NPDES	National Pollutant Discharge Elimination System
NPDWR	National Primary Drinking Water Regulations
NRDA	Natural Resource Damage Assessment
NSDWR	National Secondary Drinking Water Regulations
NVWA	North Valley Water Association
O&M	Operation and Maintenance
Red River Valley Water Supply Project (no acronym)	
Reclamation	Bureau of Reclamation
RRBC	Red River Basin Commission
RSWU	Ransom Sargent Water Users
SCADA	Supervisory Control and Data Acquisition
SDWA	Safe Drinking Water Act
Service	U.S. Fish and Wildlife Service
SHPO	State Historic Preservation Office

SPOS	Specific Plan of Study
SWC	North Dakota State Water Commission
SWTR	Surface Water Treatment Rule
Technical Service Center	Reclamation Denver Technical Service Center
UND	University of North Dakota, Grand Forks
USGS	U.S. Geological Survey
HUCs	hydrological unit codes
WCPA	Water Conservation Potential Assessment
WUA	weighted usable area

COMMON TECHNICAL ACRONYMS

ac-ft	acre feet
BAT	best available technology
cfs (ft^3/s)	cubic feet per second
$CHBR_2CI$	Dibromochloromethane
$CHBrCL_2$	Bromodichloromethane
$CHCL_3$	Chloroform
D/DBPR	disinfectants/disinfection byproducts rule
DBP	disinfectants/disinfection byproducts
ESWTR	enhanced surface water treatment rule
ft/mi	feet per mile
ft^3	cubic foot
FY	fiscal year
GAC	granular activated carbon
gpcd	gallons per capita per day
gpm	gallons per minute
HH	household
WUA	Weighted usable area

SECTION 2

SECTION 2

DESIGN CRITERIA

2.1 INTRODUCTION

This section describes the water demands and the criteria needed for conceptual (planning level) design of the: pipelines, pump stations, water storage facilities, raw water intake, and system instrumentation and control systems for the Project. Where necessary, component selection precedes the design criteria selection and discussion. The evaluation was based, in part, upon the results of a workshop held on July 30 and 31, 2003 with Reclamation engineering staff, the Garrison Diversion staff and the consultant team of Houston Engineering and MWH (consultant).

The workshop was organized to identify and discuss the individual components of the Project that would involve alternatives and their engineering elements. This information was used in developing the design criteria specifically for the Project water supply and handling alternatives.

The intent of this work will be to better define the engineering issues that will need to be fully discussed in the Project Needs and Options Report Task Order (Engineering 4 and 5 partial). The objective is to establish the initial design criteria that can be used to complete the conceptual design information and resolve (at the beginning of the study to the extent possible) technical/engineering concerns and reduce the potential for having to modify future work products.

Future municipal, rural and industrial (MR&I) water demand estimates are being developed by Reclamation in the Needs and Options Report. The municipal and rural water demands are based on the population studies completed to date by Reclamation. The future water demands will be used to develop design flow rates for MR&I water systems. All MR&I water systems within the project area have been contacted and surveyed to determine current and future service area, facilities and water needs. Reclamation is also developing estimated future industrial water demands for new industrial development.

Water demand will be a significant criterion in determining system size and the selection of reliable options for providing a long-term MR&I water supply. This section of the evaluation will discuss basic design criteria regarding per capita use factors, maximum rates, storage and other water supply criteria and will become the basis of alternative description and criteria selection.

The results of the Needs and Options Report will be used to determine the design year (2050) water demands. These water demands will be calculated using the projected populations of all the involved communities and rural water systems multiplied by the per capita water demands and the appropriate peaking factors. The projected industrial and large commercial use will be estimated separately and added to the other project water requirements. All municipal and rural water demands were calculated based on historical water demands. The needs assessment sections in the Needs and Options Report provide detailed estimates of future MR&I water demands for use in sizing the Project alternative features.

Individual users were assumed to be eventually incorporated into some type of organizational structure similar to an existing North Dakota RWA.

2.2 PIPELINES

2.2.1 General

This section presents pipeline and appurtenances design criteria to be used in the conceptual design of the Project facilities. The following items will need to be considered in the development of the pipeline 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

2.2.2 Alignment

An optimum pipeline alignment connects the water users to the water sources with the most direct routes possible at the lowest capital and operating expense. Route selection for the Project options will vary with each of the selected alternatives. The routing and alignment should carefully consider the following siting constraints:

- Conflicts with surface or subsurface facilities and structures
- Hydraulic constraints of the system
- Property values
- Paralleling existing roadways or rights-of-way
- Year-round accessibility
- Route geology, topography and soil chemistry
- Environmental sensitive areas

2.2.3 Excavation

Excavation must be conducted 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. Due to the depth of bury for this pipe (excess of 7.5 ft to crown), the required shoring will add to the installation cost. The selection for a protective system (shoring), and/or angle of excavated slopes shall be determined after considering applicable local, state and federal (OSHA) safety standards and regulations and the geotechnical analysis recommendation.

Pipeline construction efficiency is typically optimized where available space allows stringing pipe and stock piling 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 public 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 pipe cover, size of the pipe, encountered soils and resultant trench section required under current safety regulations, and size of excavation and pipe handling equipment used by the contractor.

Easements from private parties and public entities will be needed in most locations along the pipeline transmission route except for options that use the existing GDU facilities. 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. It is critical to allow sufficient area for bulk material storage, equipment maintenance and storage and mobilization when establishing the work areas.

Where easements are to be acquired, they are typically comprised of some minimum width of permanent easement (pipe trench) 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 and material storage adjacent to the pipe, to the bottom of the pipe, and trench side slopes per current safety regulations. The permanent easement may also need to include space for a future parallel pipeline or other facilities (cleanout, waste ponds, etc.). In general, it is anticipated a 60-80 foot (ft) permanent easement and a 40 to 60 ft. temporary work/access easement will be required due to large pipe diameters required in this project.

The maximum length of allowable open trench (safety concern issues) will generally be limited to less than 2,000 feet. This requirement may be more stringent in urbanized areas to ensure public safety, under certain site conditions (high groundwater), 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 ample working room (minimum 3 to 4 feet plus OD of pipe) of clearance all around the pipe and related structures during installation. Figure 1 provides a typical pipe in trench installation detail.

If during construction, the water table is encountered and is above the trench bottom. The engineer shall be notified, and appropriate dewatering must be implemented to lower the water level below the trench bottom. Trench water will have to be disposed of in a manner that protects both ground and surface water resources.

2.2.4 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 12 inches above (or as otherwise noted in design). This material must be compacted so that pipe walls are well 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, for ductile iron and PVC pipe, is specified to be 75 percent of the maximum deflection angle recommended by the manufacturers (this is typically in a range of 5°-3° for pipes 12 inches to 64 inches).

Steel pipe after installation shall not show deflection greater than 1.5 percent for mortar-lined and mortar-coated pipe, specials, and fittings; 2.25 percent for mortar-lined and flexible-coated pipe, specials, and fittings; and 3.0 percent for flexible-lined and flexible-coated or bare pipe, specials, and fittings. The allowable deflection shall be based on the design inside diameter.

For steel pipe, moderate deflections and long radius curves may be made by means of beveled joint rings, by pulling standard joints, by using short lengths of pipe, or a combination of these methods; provided that pulled joints shall not be used in combination with bevels. The maximum total allowable angle for beveled joints shall be 5 degrees per pipe joint. Unstable trench bottom or sidewalls 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 a mixture of small aggregate and sand particles. Pipe zone backfill for flexible coated pipe (tape-coated steel pipe or polyolefin-coated) shall consist of clean sand, 100% passing or 3/8 mesh screen and at least 90% passing or No. 4 sieve, with a minimum sand equivalent of 30 unless otherwise required or recommended by the manufacturer to maintain warranty. Pipe zone backfill for mortar-coated steel pipe or pretensioned steel cylinder concrete pipe (SCCP) shall be sand or 1/2-inch minus 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

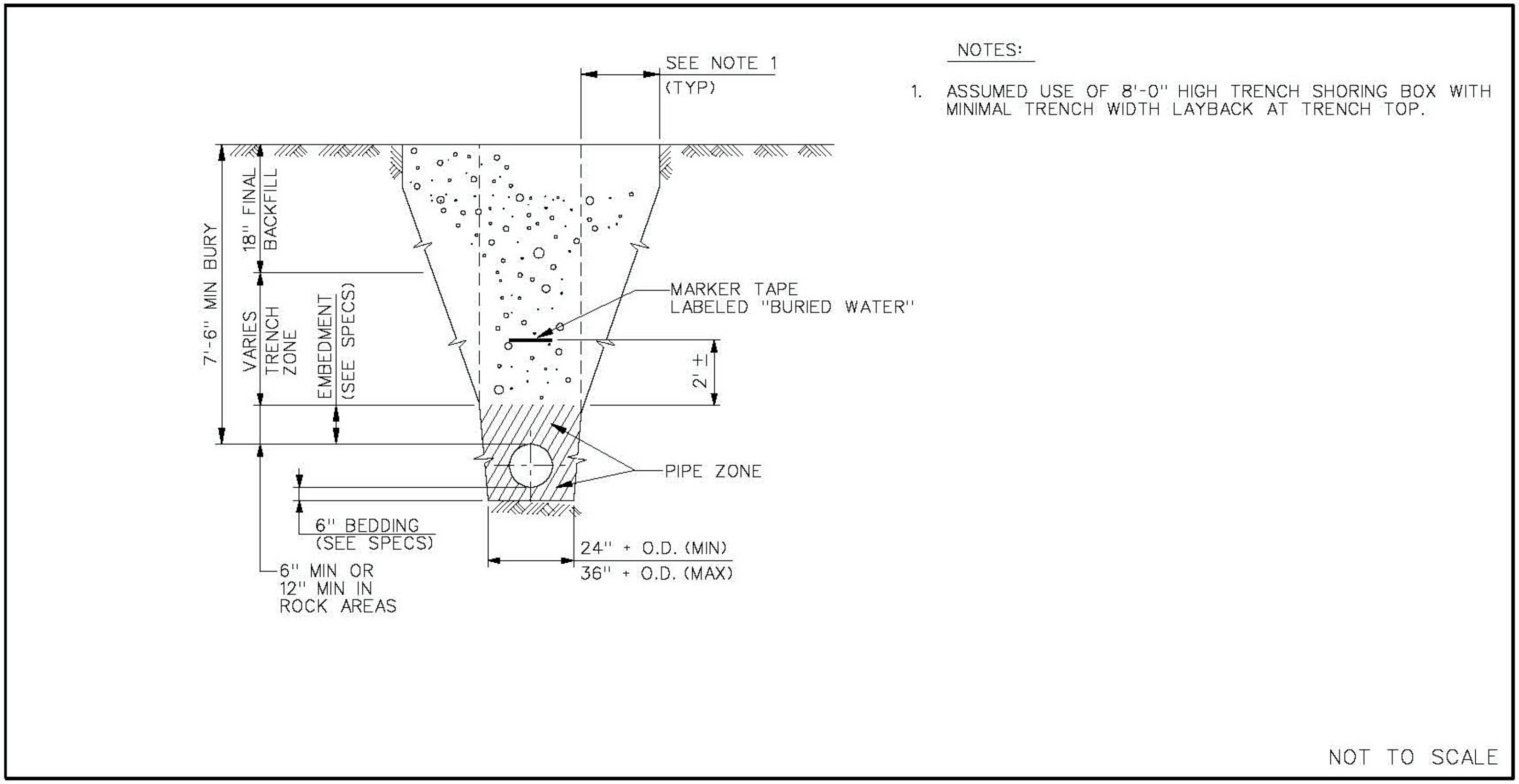


Figure 1
Typical Trench Detail Section

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. Ductile and steel pipe shall be layed directly in bedding material. Bedding shall be such that it forms a full-length solid supporting contact for the pipe. No blocking or supports will be allowed. Bedding will be dry and shall not have any frost penetration. Pipe ends will be protected with polyethylene sheeting until such time as the pipes are jointed to avoid lining damage and drying of lining material.

2.2.5 Backfill

Trench and structure backfill shall be compacted to a minimum of 90 percent maximum compaction. Where the pipeline is constructed under roadways, the backfill shall be compacted to a minimum of 95 percent maximum density (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. In areas where backfill is composed of only larger material or mortar protective rock shield overcast will need to be considered in order to protect the pipe coating if a flexible coating system is selected (see Figure 1).

2.2.6 Depth of Cover

The design depth of cover for the Project pipeline will generally 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 (as possible)
- Minimum depths of cover as required by the department of health.
- Freeze protection (frost depth) applied

Freeze protection is the primary justification for depth of cover. This criteria has been accepted generally in the State of North Dakota and confirmed by our independent analysis.

External Loadings. The pipe structure must be designed to handle the imposed earth loads and superimposed loads aboveground without damage to the pipe, its lining, or its protective surface coatings. Except in the case of very deep or relatively wide trenches, the internal pressures have typically controlled pipe design. The current OSHA standards will typically result in wider trenches and increased earth loads on the pipe. This needs to be considered as part of final design in order to optimize cost-effectiveness.

Utility Conflicts. When other utilities are encountered on the pipeline grade, they must either be avoided or protected. Generally, this is accomplished by installing the proposed pipeline deeper, or relocating the other utility. Utilities, such as sewers, which must be maintained on grade, are difficult to relocate. In some instances, the water supply pipeline may have to be installed deeper with adequate separation (for avoiding contamination) or the alignment changed 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 typical excavations. Pipeline locations will need to be identified, documented and signed where appropriate.

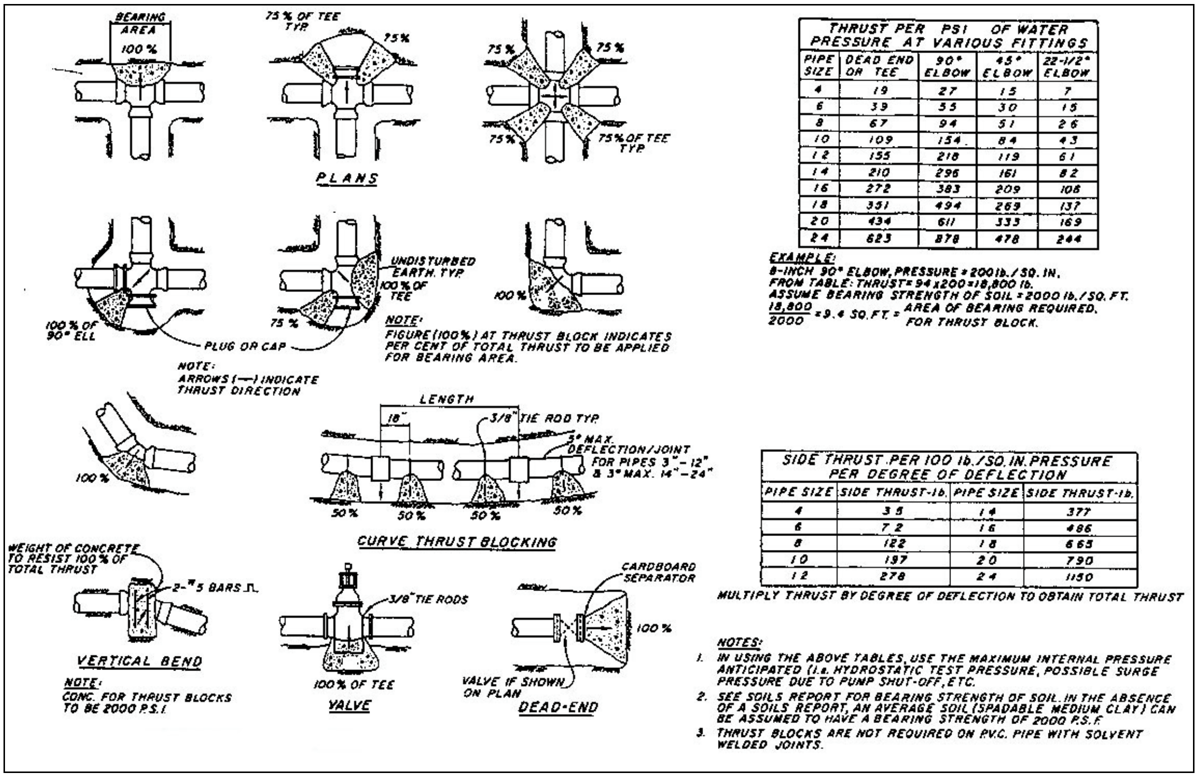
Pipeline Grading. Pipeline grading to avoid installation of air-vacuum and air-release valves when small elevation changes are encountered (hummocks) must be traversed and may result in shallower or deeper cover in specific instances. Where necessary, special measures to accommodate shallow cover and provide adequate physical and cold weather protection (insulation and encasement) may have to be considered.

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

2.2.7 Thrust Restraint

Changes in pipe direction (normally 3 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 Project transmission pipelines would typically be by use of welded or mechanically restrained joints or poured in place concrete 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 realistic safety factor (typically 1.5). Restraint also would be used on steep slopes or where future excavation may allow the pipeline to separate. For the large diameter pipe expected on this project, either welded (steel pipe) or fully restrained (ductile iron) thrust restraint is recommended. The following example (Figure 2) provides a concrete thrust block detail for smaller diameter piping where necessary.

PVC pipe does not provide for a joint restraint system. The manufacturers of PVC pipe have not developed a joint restraint system, only third party systems are available. These third party systems restrain the pipe joint by clamping around the pipe and in various forms biting into the pipe or imparting point loads on the pipe. They are highly dependent upon installation techniques and conditions as well as storage conditions prior to installation. The use of these third party systems is not recommended and caution should be considered using them regarding their potential down sides. Where changes in direction are required in PVC pressure pipe alternate materials (DIP) should be considered.



THRUST PER PSI OF WATER PRESSURE AT VARIOUS FITTINGS				
PIPE SIZE	DEAD END OR TEE	90° ELBOW	45° ELBOW	22-1/2° ELBOW
4	19	27	15	7
6	39	55	30	15
8	67	94	51	26
10	109	154	84	43
12	155	218	119	61
14	210	296	161	82
16	272	383	209	108
18	351	494	269	137
20	434	611	333	169
24	623	878	478	244

EXAMPLE:
8-INCH 90° ELBOW, PRESSURE = 200 lb./SQ. IN.
FROM TABLE: THRUST = 94 x 200 = 18,800 lb.
ASSUME BEARING STRENGTH OF SOIL = 2000 lb./SQ. FT.
$\frac{18,800}{2000}$ = 9.4 SQ. FT. = AREA OF BEARING REQUIRED. FOR THRUST BLOCK.

SIDE THRUST PER 100 lb./SQ. IN. PRESSURE PER DEGREE OF DEFLECTION			
PIPE SIZE	SIDE THRUST-lb.	PIPE SIZE	SIDE THRUST-lb.
4	35	14	377
6	72	16	486
8	122	18	665
10	197	20	790
12	278	24	1150

MULTIPLY THRUST BY DEGREE OF DEFLECTION TO OBTAIN TOTAL THRUST

NOTES:
1. IN USING THE ABOVE TABLES, USE THE MAXIMUM INTERNAL PRESSURE ANTICIPATED (i.e. HYDROSTATIC TEST PRESSURE, POSSIBLE SURGE PRESSURE DUE TO PUMP SHUT-OFF, ETC.
2. SEE SOILS REPORT FOR BEARING STRENGTH OF SOIL. IN THE ABSENCE OF A SOILS REPORT, AN AVERAGE SOIL (SPADABLE MEDIUM CLAY) CAN BE ASSUMED TO HAVE A BEARING STRENGTH OF 2000 P.S.F.
3. THRUST BLOCKS ARE NOT REQUIRED ON P.V.C. PIPE WITH SOLVENT WELDED JOINTS.

Figure 2

Concrete Thrust Block Detail

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

2.2.8 Pipe Materials

Pipe materials that are appropriate alternatives for the various Project transmission pipeline size and pressure requirements include: steel pipe, pretensioned steel cylinder concrete pipe (SCCP), ductile iron pipe (DIP), and polyvinyl chloride (PVC) pipe (small size to local points of use). Based upon the discussions with the Reclamation and Garrison Diversion, at the Project workshop HDPE, FPR and other pipe materials will not be considered. All pipeline material options will be designed according to the latest national standards (AWWA).

Steel pipe (AWWA C200) is manufactured to a wide variety of sizes and wall thicknesses. A pipe interior cement mortar lining (AWWA C205) is recommended for all transmission lines above 6-inches in diameter. Several exterior coating alternatives can be used depending on field soil corrosivity. Alternative exterior coatings are an 80-mil cold-applied plastic tape (AWWA C214), a 1-inch thick reinforced cement mortar coating (AWWA C205) or extruded polyolefin coating. 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. The availability of this pipe is limited and presents special and difficult corrosion protection issues. Its use as a specialty item (i.e., crossings) would be limited in the Project application.

Ductile iron pipe (DIP) (AWWA C151) is standard in sizes to 64-inch and up to a 350 psi working pressure. A cement mortar lining conforming to AWWA C104 and appropriate corrosion protection would be recommended. Push-on rubber-gaskets (AWWA C111) and proprietary restrained joints are available.

Polyvinyl chloride (PVC) pipe (AWWA C900) is standard in sizes up to 12-inch and 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 12 inches. Fittings for PVC pipe in excess of 12 inches shall be ductile iron and shall conform to the requirements of AWWA C110.

In general, pipelines less than 24-inch diameter will be PVC pipe meeting AWWA Standard C900. Transmission pipelines with diameters greater than 12-inches will be mortar-lined, and either mortar-coated or tape-coated steel with cathodic protection; or mortar-lined and polyethylene sleeved ductile iron pipe with cathodic protection. SCCP would only be considered under special conditions (i.e., low-pressure gravity flow situations where cathodic protection is not an issue) and/or at the Owner’s specific direction. No further discussion of SCCP will be provided. Both types of ferrous pipes were assumed to have bonded joints.

2.2.9 Pipe Pressure Classes

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

- Handling
- Internal Pressure
- External Loads
- Deflection
- Buckling

The minimum thickness for handling of welded steep pipe shall be the nominal diameter divided by 240. In no case shall the steel cylinder thickness be less than No. 12 gage.

The design minimum thickness of ductile iron pipe for internal pressure shall be as specified by the AWWA C150.

For steel pipe except as otherwise indicated, materials, fabrication and shop testing of specials and fittings shall conform to the requirements stated above for pipe and shall conform to the dimensions of AWWA C208.

2.2.10 Operating Pressure

Generally, operating pressures in the Project 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 design target, but may only be increased to possibly eliminate the need for reservoirs and optimize pump station design and thus reduce construction costs.

2.2.11 Fittings and Specials

Tape-coated steel pipe and mortar-coated steel pipe will have fabricated steel specials and fittings conforming to AWWA C208. Reinforcement of outlets will be in accordance with AWWA Manual M-11. Fittings for DIP shall conform to AWWA C110. PVC pipe fittings 12 inches and larger shall be mechanical joint. Fittings shall be mortar lined ductile iron (Class 250) and shall conform to the requirements of AWWA C110.

2.2.12 Pipe Joints

Pipe being considered for the Project transmission pipelines under 60-inches in diameter can 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 large steel pipe will be restrained by welding. Ductile iron pipe joints are restrained by use of ductile iron restrained coupling adapters.

The use of restraint of any PVC pipe will need to be evaluated on a site specific basis. Depending on the application, thrust restraint may be accomplished with concrete blocks or by using sections of D.I. pipe in critical locations.

Steel pipe with welded joints will have zero tested leakage. Non-welded joints at operating pressure should have zero leakage, but will have a test leakage maximum allowance (10 gpd/in-diameter-mile/24 hours). Test standards include:

- Ductile iron and PVC – AWWA Manual M41 and C600 (joint)
- Welded Steel – AWWA Manual M11 (welded steel)

2.2.13 Corrosion Protection

Corrosion evaluation and mitigation measures will be considered during the design and development of the pipeline delivery system once the final pipeline alignment is selected. As described earlier, various corrosion protection methods involve the use of: mortar linings; tape or mortar coatings; polyethylene sleeves; epoxy; and fusion bonding. All iron and steel pipe joints will be electrically bonded. The use of various cathodic protection methods (impressed current) will depend on the results and recommendations of the onsite specific corrosion evaluation but use of an impressed current system is anticipated. Insulation flange kits will be provided at connections to structures and other pipelines and other pipe materials where applicable. The cost for impressed current corrosion protection should be included in all pipeline estimates (Table 1).

TABLE 1

ESTIMATED COST OF CORROSION PROTECTION FOR PROJECT ESTIMATING

Pipe Type	Cost $/Mile	Notes
Ductile Iron (24”-66”)	$20,000	Reclifiers and deep well anodes, bonded joints, test stations
Small Diameter Steel with gasket joints (<66”)	$15,000	Reclifiers and deep well anodes, bonded joints, test stations
Large Diameter Steel with welded joints	$10,000	Reclifiers and deep well anodes, test station

* Based on local North Dakota project experience assuming the same mildly corrosive soils

2.2.14 Mainline Valves

Mainline isolation valves will be provided to limit the length of pipeline that must be repaired, cleaned or maintained 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 interval spacing of 5 miles and will be incorporated at other sites (crossings, siphons, etc) as necessary. Butterfly valves or other slow closing valves are suitable for use as isolation valves. The mainline isolation valves will be designed for installation in a manner which will prevent freezing, designed to minimize hydraulic transients (surge), and will include a bypass valve leg. All isolation valves on the main and secondary large diameter valves will be in accessible vaults and be provided with necessary accessories and freeze protection.

2.2.15 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 (Figures 3 and 4). Screened standpipe(s) located above the ground also will be provided to allow for venting. 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. A PVC standpipe pipe may be acceptable in other locations that are of low risk to mechanical damage. UV (sunlight) resistant PVC piping should be used for all exposed PVC piping.

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 maximum 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.

2.2.16 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 isolation valves (Figure 5). Blowoff size will allow a reasonable “time-to-drain” for tributary pipe segments; typical diameters for blowoff piping and valves will be approximately 20 percent of carrier pipe diameter.

Where reasonable, blowoffs will be drained to daylight. An energy dissipater (riprap or soil cement) will be provided where pipeline drainage may cause erosion. However, in most cases, will be below grade and it will be necessary to provide a standpipe so that a pump can be inserted to drain 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 depending on drain application. Chlorination/dechlorination facilities (permanent or portable) may be required to address regulatory concerns.

2.2.17 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 24-inch diameter side outlet and blind flange and will be marked with a steel post. Manhole covers will be secured and insulated (Figure 6) for cold weather application.

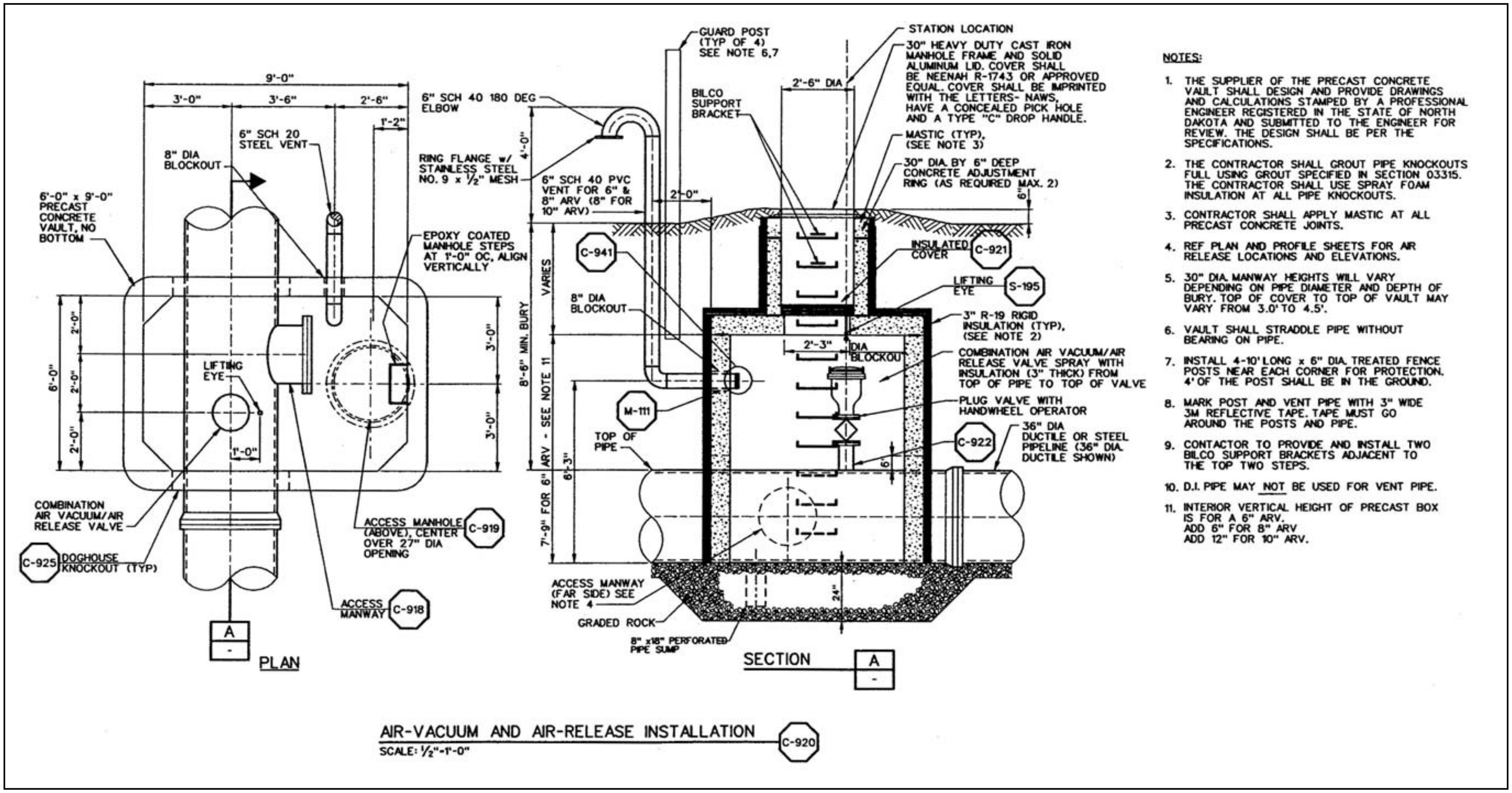


Figure 3
Example Cold Weather AV/AR Valve Installation

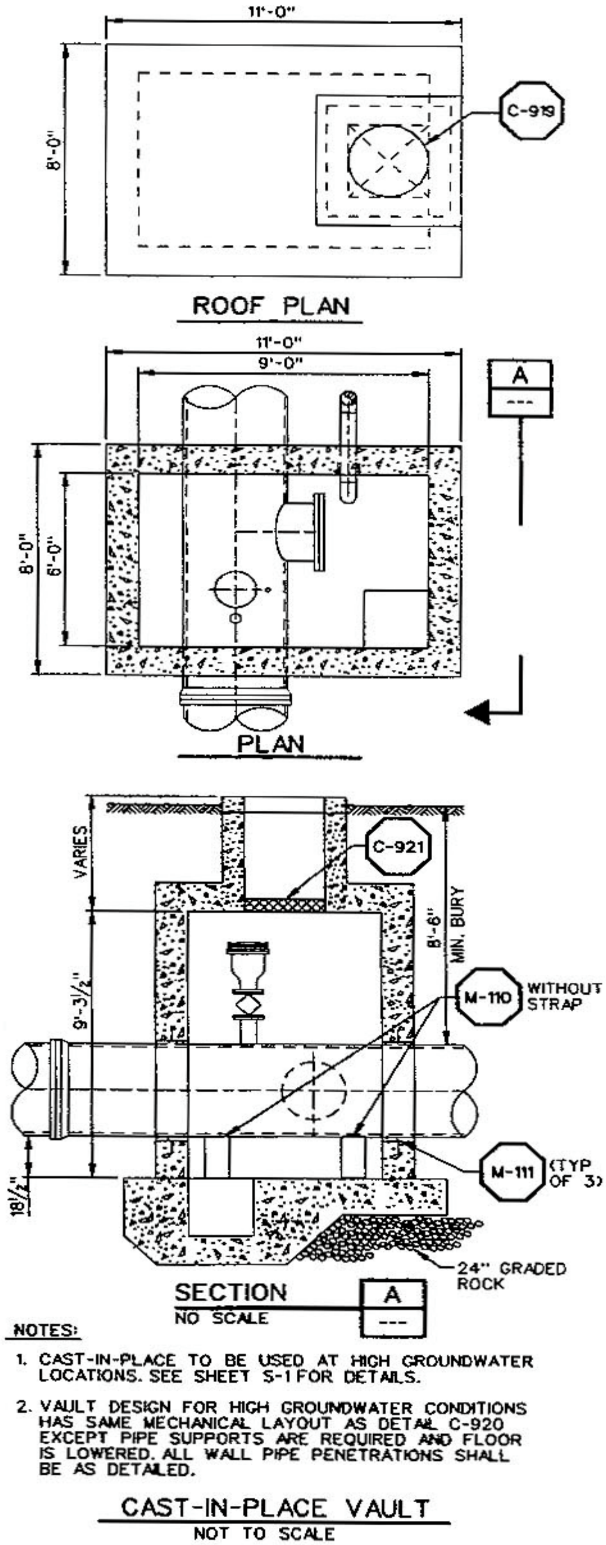


Figure 4
Example Cast-in-Place Vault A/V Valve

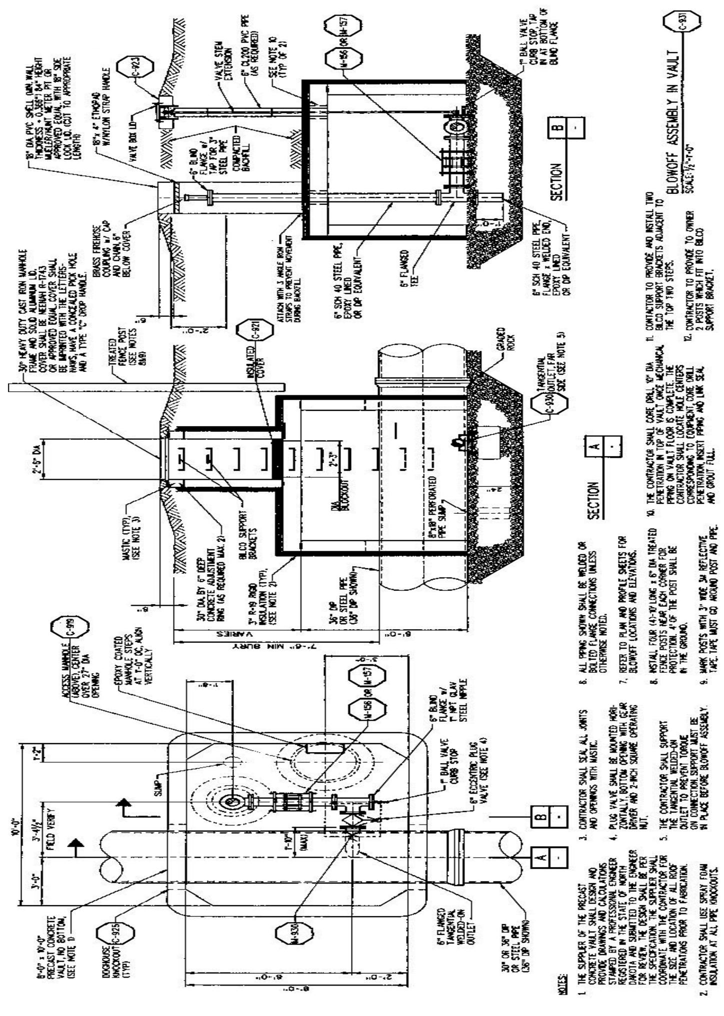


Figure 5
Example Cold Weather Blowoff Valve Installation

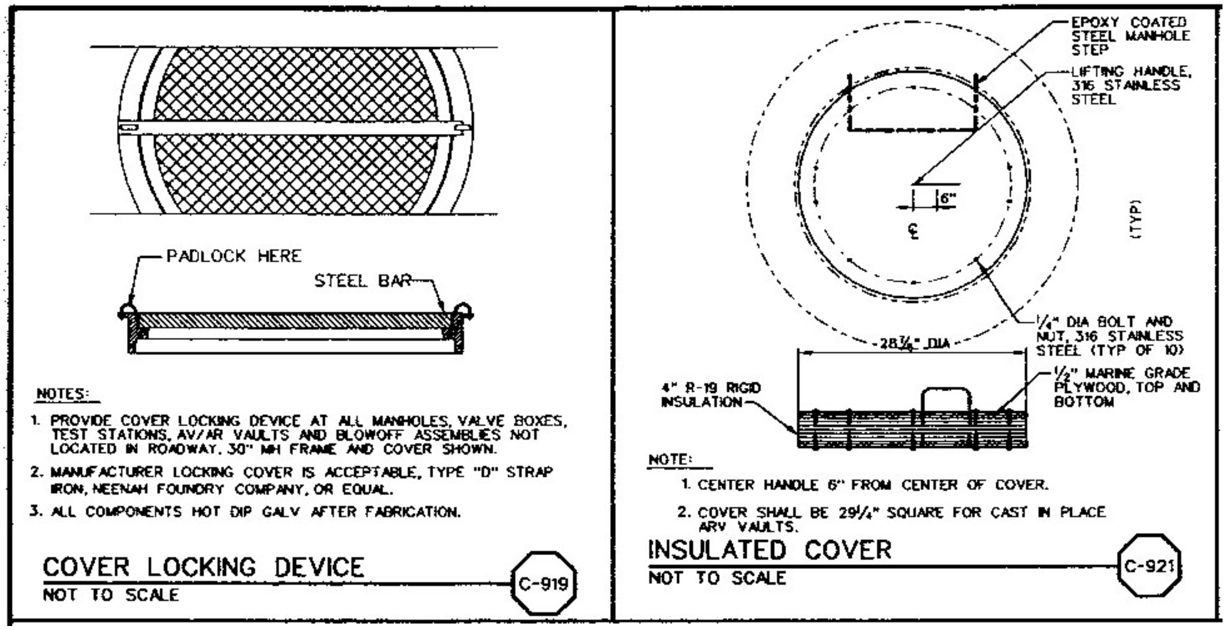


Figure 6
Example Cold Weather and Secured Manhole

2.2.18 Turnout Service Connections

Turnout structures will consist of: isolation valves, flow meter to all service users, flow control valve, and a maintenance bypass. Flow deliveries will be controlled by an automatic control system (automated valve and meter or by an operator at the remote 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.

2.2.19 Vaults

Vaults will be used to house various project amenities discussed above. 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 (or drains), and unit space heaters;
- Communications, intrusion and entry alarms;
- Drainways in floors; and
- A battery-powered emergency power supply for the SCADA and local control system for use in the event of line power failure (24 hour emergency operation).

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 very unlikely instance where vaults would have to be placed in roadways, the top of vaults shall be flush with the finished surface with highway rated openings. Outside roadways, vaults shall extend at least one foot above grade or what is required to avoid surface flooding. Manway covers shall be of cast iron or fabricated aluminum, at least 24 inches in diameter. Manway covers will be solid, no openings, and shall be sealed to prevent water from gaining access. Access to vault with large equipment and appurtenances will be provided by hatchways confined space safety requirements need to be considered.

2.2.20 Roadway Crossings

The Project 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 appropriate tunneling methods. For other non-paved and low traffic 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 need to be accommodated.

2.2.21 Utility Crossings

At roadway crossings, utilities may be encountered which generally parallel the road. These would typically be above the profile of the Project transmission pipeline, given the greater depth of the pipeline at roadway crossings. One exception may be any local existing sewer lines, which may have been installed deeper.

Clearances between sewer lines and the water 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. Wherever possible the water line should be installed above the sewer pipeline. Where adequate separation is not possible, steel pipes would have welded joints and non-steel pipes would be encased in concrete to prevent contamination of the finished water due to exfiltration of the sewer line. 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 installing a concrete cradle between the two utilities. We do not expect that the Project pipelines and existing sewer line interferences will be a significant problem, but it would need to be considered in selecting pipeline alignment through populated areas.

2.2.22 Wetlands and Creek Crossings

When establishing the Project pipeline alignments all efforts should be made to attempt to avoid major wetlands. However, we can expect that several water courses will be encountered. When crossing streams the pipe will be installed below scour depth and adequate cover must be maintained under stream beds. 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 banks 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. If possible, stream crossings should be undertaken during low flow periods. Various lining and piping materials can be used to divert flow and minimize the erosion that takes place in the diversion channels. It will be important to initiate the Corps of Engineers Dredge and Fill Wetland Permit (404) process early enough to avoid potential delays.

2.2.23 Metering Facilities

Metering facilities proposed for the Project pipeline will include a “master” meter at the diversion end of the system (Missouri River or principal supply works), meters at any pump stations, and meters on the distribution 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 summarized (multiple) totalized meter outflow will never exactly equal metered inflow to the pipeline because of inherent 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 (5%) over the full range of flows contemplated;
- Accurate repeatability;
- Field calibration capabilities;
- Minimum error from SCADA system components;
- Low life cycle costs; and
- Minimal meter drift and need for calibration.

Several types of meters exist that can be used on the smaller pipe segments. Venturi meters and/or magnetic meters are suitable for use in the large Project transmission pipeline system.

2.2.24 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 (noted as constructed drawings) 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 and located wire should 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 use of a global positioning system to aid in the location of non-metallic pipe to aid in location documentation.

2.2.25 Hydraulic Design

Depending on the alternative selected, the total length of major transmission pipeline required for the Project alternative may extend over 300 miles. Hydraulic design must not only account for the hydraulic construction issues involved in construction and operation, 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 capital construction costs for the various pipe diameters throughout the system.

Initial pipe sizes will be chosen to keep the velocities in the 12-inch and smaller pipes below 5 fps, at design flows; pipe velocities in major trunk lines should not exceed 8 fps under average day design conditions. Peak flow pipeline velocity (limited duration) should not exceed 10 fps. The initially investigated design flow will be the combined flows developed by delivering, over a 24-hour period, each customer's maximum day peak water demand. However, opportunities to add storage capacity to alternatives will need to be evaluated to potentially reduce pipe size requirements. The pipe diameter which meets the hydraulic constraints and also yields the lowest life-cycle costs will be chosen for service. For planning and conceptual design purposes, the minimum delivery pressure (turnout, reservoir, tank or other customer discharge) will be 20 psi. The minimum hydraulic grade will be +20 feet.

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. An average (50 years) Hazen-Williams friction factor of 140 was assumed for all pipe materials for planning purposes.

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 for bonding purposes will be 4.0 percent, the assumed annual power inflation rate will be 1.0 percent (based on conversations with staff from the Western Area

Power Administration), and the chosen pipeline life span (nominal) equals 50 years. The initial and design flows and years required to achieve the flows vary with reach. An initial average power cost of $0.045 per kWh (including demand and wheeling charges) was used in the analysis, which is the cost anticipated to be effective January 1, 2006 (start of final design anticipated).

2.3 BOOSTER PUMP STATIONS

2.3.1 General

The primary pump and booster pump stations combine with the pipelines and storage reservoirs to form a complete system for transporting water to the Project water users. The development of the pipeline delivery system 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 for each unit. Since the project can have two distinctly different objectives – augmentation of existing supplies during drought conditions and out of basin (Missouri River water) replacement – the delivery system will vary. This section establishes the various parameters necessary for all pump station design. Where practical, pump station design criteria and detailed design should remain as consistent as possible between the various locations in order to facilitate operation and maintenance.

2.3.2 Siting

System or pipe segment 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 and other institutional requirements
- Environmental acceptability;
- Relative property values;
- Year-round accessibility; and
- Availability and accessibility of adequate power supplies and communication options.

2.3.3 Intake and Primary Pump Station Structures

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

Intake. A water system intake must provide a reliable supply that will ensure uninterrupted services under all conditions. The intake should be sited in a location that can be easily accessed and maintained. The intake should not interfere with other functions (navigation, utilities, etc.) or create environmental concerns (fish takes, recreation, etc.). Electrical service should be available with a reasonable distance to the intake site.

The intake should be physically sized to provide the ultimate maximum daily capacity of the system over the 50-year planning period. The essential factor in siting and design of a surface water intake is reliability. River or canal intakes in North Dakota will present unique problems with regard to winter operation (icing and freezing), protection of fish and other species, potential season fluctuation in flow rates and issues regarding water levels and water rights and water withdrawals. It is critical that the required site studies and investigation be conducted prior to selecting an intake location. Since many intake structures combine water diversion, grit collection and removal, screening (course to fine), and intake pumping, it is critical that there be a thorough understanding of the potential problems and issues that can arise in implementing and operating an intake facility.

Primary Lift Pump Station. Once an out of basin water source reaches the Red River basin the supply water (raw or treated depending on option selected) will need to be conveyed to a variety of locations for either further treatment or, in the case of an out of basin treated supply, final distribution to a customer.

For an augmented in basin source (i.e., wells, storage, etc.), pump distribution would likewise be necessary in most instances.

The location and type of pump station and its related appurtenances will vary significantly by location and source. A Missouri River intake will involve locating a reliable intake and pump station on the main stem of the river or in an existing GDU facility. This will need to include the appurtenances to protect the pumps from debris and freezing conditions. A GDU canal based lift station will require conforming to the site limitations and require ongoing canal maintenance.

The basic structure for a Missouri River main pump station could include a variety of options. The typical pump station would involve a concrete structure on the river with course and medium screening on the intake, some method to clean and maintain the screens, a vault to act as a pump wet well and a pump array discharge pump header and pipeline and infrastructure (electrical, access, communication, etc.). In addition, consideration for sediment removal and the potential for icing conditions would need to be addressed.

Other options that could be considered involve an intake pipeline with a submerged river screen from the river to a shore based (remote) pump station, and infiltration gallery or Raney type collector under the river. Each of these options have site specific limitations and advantageous and disadvantages that need to be carefully reviewed to determine the most cost-effective solution. This analysis is beyond the intent of this discussion. The balance between pump size and horsepower (operating cost), pipe size and class and the advantage of using booster pumping in lieu of sizing a single life alternative all need to be analyzed.

Until the intake options are more fully defined as part of the ongoing study, discussion of this component will be differed. A separate study will be required to analyze the selected options. Additional discussion will be required to determine what studies are necessary and to better define intake options and design criteria.

Booster Stations Along Major Trunk Lines. Booster pump stations located along major trunk lines and the in basin distribution system would be housed in buildings designed to protect all pumps, controls, and necessary appurtenances from adverse weather conditions and potential vandalism. On the basis of economics and aesthetics, a concrete masonry (split face or similar) building is preferred. Building design features will include adequate insulation and heaters capable of maintaining an internal temperature sufficient to prevent freezing, as well as ventilation for limiting summer time temperatures inside to meet manufacturer's specifications for the electrical equipment involved, and dehumidification equipment (e.g. wall mounted air conditioning units as feasible). The security measures will include the installation of steel doors and/or hatches with cylinder locks, steel louvers, perimeter fencing, exterior lighting, and possibly alarms. The buildings should not have accessible windows. A standby power system sufficient to power necessary for the building support systems (e.g. electric heaters, lights, communications, 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. Drainage needs to be considered (drainaway or sump with pump and discharge location). Adequate space will be provided for access and repair as well as for removing or adding future pumps if or when necessary. Depending upon the size of the pump station, skylights or hatches generally will be located above each pump to facilitate future pump removal and repair. In some instances an overhead crane and hoist may be a preferable option.

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

Distribution and Turnout Pump Stations. Smaller capacity pump stations to service individual users will be located in suitable similar 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. Buildings will be stand alone secured and security fencing will be used unless called for due to specific circumstances. Positive drainage away from the pump station will be provided. The general siting provisions listed for the larger mainline pump stations will generally be applicable to all units.

2.3.4 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 that will be required, the vertical turbine configuration is more efficient than a horizontal centrifugal pump. Depending on the flow and discharge pressure required axial or mixed flow pumps may prove to be efficient for low lift applications. The typical 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 in wet wells than horizontal centrifugal pumps. However, horizontal centrifugal pumps are generally easier to maintain in place. They can be disassembled in place without having to remove them; while the motor, discharge header, and pump body (tube, shaft, bowls) 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 necessary pump 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 relief 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 arrestor 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). Surge analysis will have to be performed as part of detailed design to determine the level of surge protection required.

It will be important to consider the number of pumping units to be installed for each station. The more pumps, the greater the flexibility in delivering various flow rates, but at a higher capital cost. However, multiple pumps may have the opportunity of reducing the operating cost without having to utilize expensive variable speed drive units. 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 duty and standby pumps are cycled evenly. The appropriate number of pumps to provide both flexibility in operation and economy will be evaluated in final design for each application. In the case of a 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, have higher reliability, and less equipment service requirements than pumps with variable speed drive units. However, this will need detailed evaluation. It should be noted that while power costs in the area are currently relatively low, this is a long-term project and energy conservation needs to be carefully considered in pump selection

2.3.5 Electrical Service

Each pumping station's electrical supporting system typically will include: additional transformers; motor control centers; pump motors and corresponding wiring and conduits; lighting, heating, and ventilation components; lighting arrestor protection; and other items associated with the instrumentation and control system. Line voltages of 480 volts will supply power to pumps up to a maximum of 600 hp, while 4,160 volts will supply power to all bigger pumps (if necessary). The motor control centers will need to 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 all larger (25 hp and greater) 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.

2.3.6 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 local signals and be responsive to the Supervisory Control and Data Acquisition (SCADA) system.

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 (this should not be a significant issue in most applications). In the event of a low-low level being reached in a pump's respective upstream reservoir or wet well, the pump will cycle off to prevent the reservoir from being “pumped dry”. Local manual operation and/or remote operation from the system control center will be designed for each pump. Large pumps will include a starter interlock/timer to prevent starting more than one pump at a time in order to reduce electrical service demand charges.

2.3.7 Supervisory Control and Data Acquisition (SCADA) Systems

Pumping plant operation is required to be monitored and/or controlled remotely by a SCADA system. 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 either a radio based system or fiber optic communication lines (to be determined).

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. The locations of the Central Computer System could be in a number of locations (read only could be provided over an Internet line) but the treatment plant site (biotreatment or full treatment) would be the most likely conveniently manned location.

2.3.8 Emergency Power

Several factors enter into an analysis of the potential risk of interrupting (vulnerability) pumping capability. Unfortunately, it is seldom possible to compute an accurate or precise risk factor at its implementation. Historical data are generally not representative of future interruptions because the causes of past interruptions may have been partially corrected (improvements to electrical power distribution) and potential future problems 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 for the system to have local storage and gravity or local standby power supply for distribution pumping. Large water pumping stations, therefore, do not include full standby power provisions. The Project out of basin alternatives being considered will include large reservoir storage and a piped gravity supply of treated (biota or higher) water once the drainage basin divide is crossed. Alternatives considering a Sheyenne River delivery would include long-term storage in Lake Ashtabula. This will provide a supply that is not dependent on pumping to the local distribution point for a period of time (8-24 hours – to be determined). Given these provisions, the out of basin supply alternatives would not warrant full standby power to meet a short-term power outage. Small distribution and treated water pump stations may require standby power generation for pump distribution if a gravity storage supply is not available. This will need to be evaluated on a case by case basis.

All pump stations will have on-site, back-up power generators to power key building systems and smaller units may be equipped with circuit connections for portable generators in the event of a long-term or planned interruption in power supply.

2.3.9 Design Parameters

Design parameters for the various pump stations including: design discharge (flow rate), total dynamic head, horsepower requirements, specific speed, net positive suction head, and number of pumping units will be developed using information developed as the planning progresses. 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. Generally, the selection of pumps that will operate in the 75-85 percent efficiency range should be an important objective.

Flow Rate. Preliminary design flow rates for the out of basin Project systems were developed from information contained in the Needs and Options Report

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.

$TDH = z_2 - z_1 + v^2/2g$ + 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 system (reservoirs, pipelines, etc.) is termed the static head. Velocity head (the kinetic energy per pound of the fluid) 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 modeled using MWH Soft, Inc. hydraulics software (InfoWater). The model will use the Hazen-Williams formula to estimate head losses.

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 conceptual design purposes.

Number of Pumping Units. As described before, multiple pumping units provide reliability and more flexibility for pumping a variety of flow rates. 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 will occur with aging of the system. Generally, it is necessary to select a pump that is somewhat larger than the minimum calculated capacity for peak flow due to standard motor size limitations. Limiting the number of pumping units does not provide sufficient flexibility for pumping lower flows. A single large pump may be required to pump at a rate far below its most efficient operating point (high power cost per unit volume) to supply a reduced flow. On the other hand, unnecessary complexity and costs can be added to a pump station by providing too many units. The number of pumping units for each application will be evaluated and determined during completion of detailed design.

Horsepower. Input horsepower to the pumping units is determined by the following equation. Pump efficiency for most applications will be targeted to be in the range of 75 to 85 percent.

$hp = Q62.4h/550e$, where

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

2.4 WATER STORAGE FACILITIES

2.4.1 General

Storage of treated water and raw water 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 and meeting the water supply required. This is particularly true when pumping stations are in series. Storage should be provided in this situation to prevent unstable variation of flow rates as the pumps try to match each other. Having an adequate volume serves as a buffer for pump operation. Additional operational shortage may be incorporated into alternatives to reduce peak day water demand requirements and the associated pipeline/pumping conveyance and treatment systems. The existing available MR&I storage within the planning area will be evaluated to determine its adequacy and whether there is any excess storage available to meet future peak day water demands.

Emergency storage is that storage which is provided to meet demands in the case of the loss of production or upstream transmission capabilities. The most obvious example is the loss of power to either the treatment plant or to any of the pumping systems feeding a storage unit. Other examples of shutdown include maintenance and repair operations in the event of pipeline damage. Because the water from the Project out of basin alternative will be partially pumped to the various users (as opposed to complete gravity flow), the most reliable alternative would be to provide emergency storage at a selected location(s) along the transmission line(s). In our workshop, it was generally agreed that a large treated water (complete SDWA or biota transfer treatment) reservoir across the drainage divide may be most useful for providing gravity flow to the Fargo, Moorhead and related north valley service areas. This structure could provide both emergency and operational storage. The amount of emergency storage required is a function of: availability of an alternative source of supply, and production facilities, expected outage and risk, availability of standby facilities, other local storage, and the potential duration of an outage (difficult to assess).

The Project treated water delivery system should be designed (in phases for ultimate 50 year) to deliver, over a 24-hour period, the combined flow resulting from each identified customer's (domestic and industry) maximum day water demand. Actual daily water demands are subject to both hourly and seasonal fluctuations and will therefore vary from this maximum design flow. Winter months would generally see lower demands, while summer peak hourly rates will easily exceed the design average summer day 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 Project system water supply will depend on a constant, uninterrupted water supply, any reservoirs must be able to provide the entire downstream water demand during instantaneous peaking periods and sustain the system(s) during any emergencies. For large raw water supply projects, the total volume of reservoir storage is recommended to be sufficient to meet the peak flow one-hour water demand. For smaller (>2 mgd) raw water supply systems this volume should be increased to cover the operational range of any pump delivery requirements. The following paragraphs discuss the reservoir siting and operations, construction, and appurtenances. Obviously, this is a significant amount of storage and would need to be divided between centralized storage facility and local reservoirs and tanks. These facilities would be implemented in stages over time by the users of each system.

2.4.2 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 or gravity feed sources; 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, gravity flow and hydraulic considerations, and applicability for reservoir overflows and other discharges.

Location of centralized storage facility(s) for the replacement water supply option are currently being investigated. We would anticipate that that unit would be located within the Red River Valley (RRV) drainage upstream of any major diversion point. Depending upon the treatment scenario considered, this facility could accommodate either biota treated water or finished

SDWA compliant water. A raw (unfiltered) water reservoir would need to be designed to provide for frequent cleaning of settled sediment and disposal of cleaning wastewater.

2.4.3 Operations

Reservoir operations must 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 operation of the reservoir will control the volume of water pumped from the source.

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

2.4.4 Construction

All reservoirs (biota or full treated water) 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 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, functional use of the reservoir, 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 may have winter ice concerns which will be lessened for buried or partially buried reservoirs. Dual cells and/or a flow bypass will facilitate cleaning and 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, possibly making them more acceptable from an aesthetic viewpoint.

Concrete reservoirs for large volumes are preferred due to cost and longevity. As discussed when using raw water (unfiltered) transmission systems, reservoir cleaning issues need to be carefully considered. Obstruction in the reservoir (columns, footings, etc.) need to be minimized and designed not to trap solids. Wash down cleaning water systems need to be considered for remote location (dedicated pumps and water source). A washdown water collection and holding/evaporation/infiltration (pond) system will be necessary to contain and process washdown water and settled solids.

From past experience, above ground circular steel structures are generally the most economical choice for treated water storage volumes below three million gallons but can be subject to problems with cold weather operation. Thus in cold climates, concrete storage reservoirs for large and small storage volumes alike are the preferred option to address climatic concerns with regards to ice formation inside reservoirs during cold weather. Construction of concrete reservoir needs to be phased to the end of a project when a constant water supply will be available to protect the structure. Leaving a concrete reservoir empty for long periods under changing climatic conditions in not recommended. 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.

2.4.5 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. Holding (evaporation/infiltration) pond(s) will be required for flushing reservoir cleaning water for disposal. Reservoirs are considered enclosed spaces so safety concerns for entering the structure must be carefully considered. Sufficient ventilation (12 air charges per hour) should be provided or supplied air must be used.

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.

2.5 RING DIKE STORAGE RESERVOIRS

2.5.1 General Appurtenances

Ring dikes (Figure 7)are open raw water storage reservoirs that can be included in a number of comprehensive alternatives being considered. Generally, ring dikes are being considered for two separate functions depending on the alternative being considered. In case one, a ring dike would be filled from either the Sheyenne or Red River in times of abundant flow (spring) and stored until user demand requires its withdrawal. In the second case, ring dikes would be filled from a Missouri River or Minnesota import pipeline and would be used as a flow regulating reservoir. The design criteria for the ring dikes would be the same although the sizes would vary for each alternative and function. The volume of storage needed for each case is currently under investigation and would need to adjusted for evaporation and potential seepage losses.

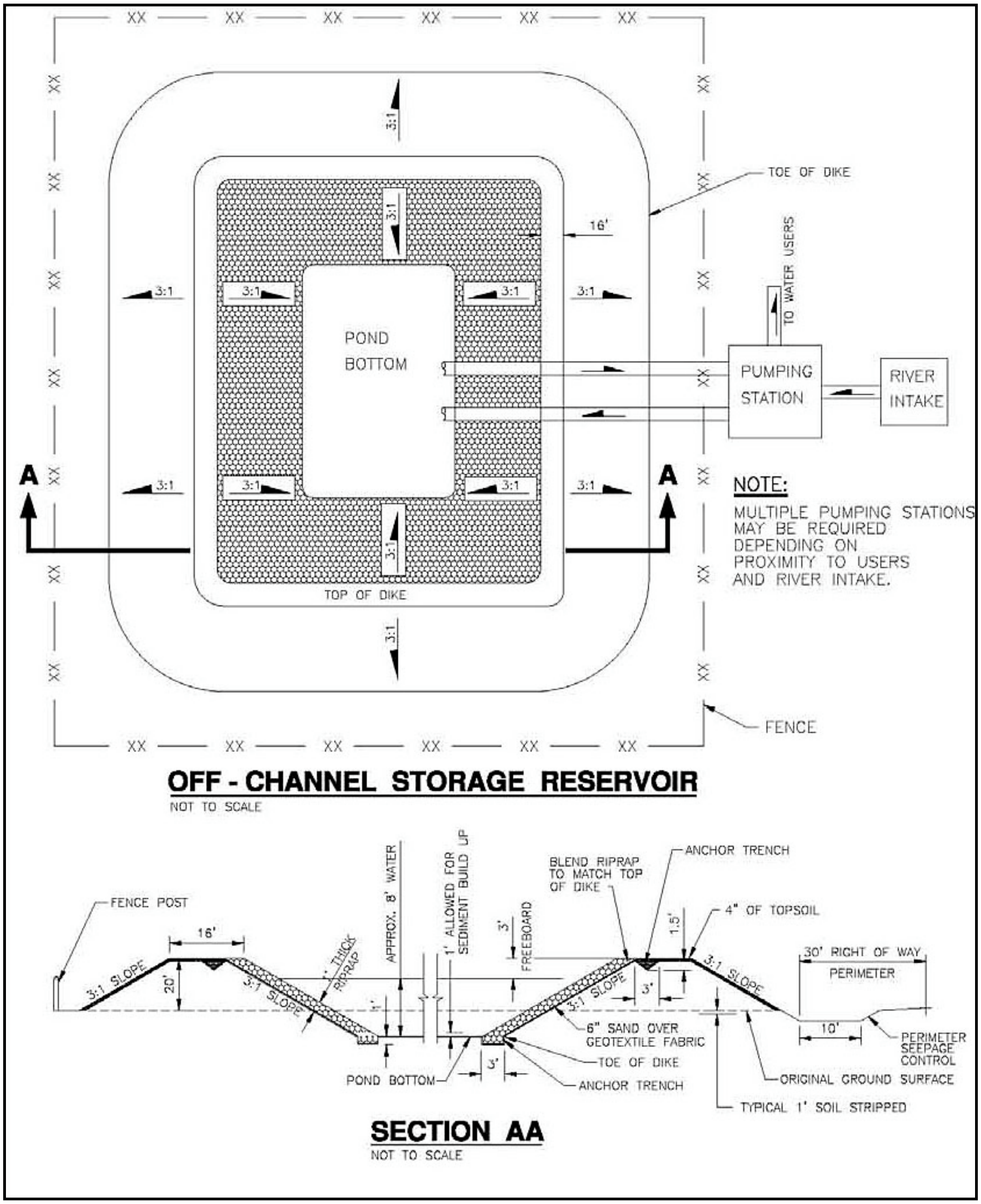


Figure 7
Typical Ring Dike

2.5.2 Siting

Factors that should be considered in locating ring dikes are:

- Proximity to source, demand areas and infrastructure
- Operational parameters of the water delivery system
- Access to electrical service
- Topography of general area
- Land values
- Soil conditions and geology
- Water table
- Flooding potential
- Susceptibility to hazardous spills
- Home, business and utility relocations needed
- Potential depredation in the area by waterfowl
- Potential nuisances caused by mosquitoes, odors etc.

Generally, the ring dikes would be located near their potential water source and in close proximity to the areas with high demand. For this project, the likely location would be between the Red and Sheyenne Rivers near the Fargo/Moorhead/West Fargo metropolitan area. In the case of a Minnesota supplemental supply, the dike would best be located on the Minnesota side of the Red River.

Reservoirs may be located: on the highest elevation land available along the pipeline route between successive pump stations or gravity supplied water sources; 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, gravity flow and hydraulic considerations, aesthetics, and the feasibility for providing reservoir overflows and other discharge appurtenances.

2.5.3 Operations

In the case of a flow-regulating ring dike reservoir, the operations would be similar to traditional water storage units. Reservoir operations must account for both normal and emergency conditions. During normal operations, each reservoir’s level will rise if the fill pumping rate exceeds demand (discharge). Conversely, the reservoir level will fall if downstream demand exceeds the supply pumping rate. In general, the system’s pressure switches will be set to allow pumping to closely approximate 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 allow inflow from the upstream pipe. The operation of the reservoir will control the volume of water pumped from the source.

When the ring dike reservoir is being used to capture water during abundant river flows, the operation would be quite different. River flows would be pumped to the ring dike(s) in relatively short period of time (30 days) during high water periods (flows in excess of existing water rights, water use, and streamflow augmentation needs). This would typically occur during spring snowmelt. The water in the reservoir would then be discharged to conveyance facilities or pumped to treatment plants when demands can not be met from the existing sources.

Reservoirs would be maintained at a full level and would be depleted to empty if conditions dictate. It is preferable to maintain the bottom of the ponds in a wet condition to prevent cracking or burrowing into the clay liner by small animals.

2.5.4 Construction

Reservoirs will be constructed from native soils for economic reasons. At this time it is not anticipated that a geomembrane (geomembrane lining was used in the Phase II appraisal-level design of ring dikes) would be required in the bottom of the reservoir to control seepage or contamination from an outside source. The clay soils in the general area have low permeability. Special consideration for geomembranes will need to given if soils in the selected sites are not clays. Stripping of organic soils up to 1 foot would be required. Geotextile filter materials and a 6-inch layer of sand would be placed on the slopes of the dikes where riprap protection is used. Riprap would be provided around the perimeter of the ponds from an elevation of 3 feet above the highest operating level to the bottom of the pond. Slopes of the dike embankment and excavation would be 3:1. Top width of the embankments would be 16 feet. Native clay materials taken from the excavated pond area would be used in the embankments and would be compacted to 95% of the maximum dry density as determined by AASHTO Designation T99 (Method A). The moisture content shall be within the limits of 1 percentage point below to 4 percentage points above the optimum moisture content. To reduce the percolation rate of the soil, the bottom surface of the reservoir including that area which will be below the dikes, shall be scarified to a minimum depth of 6”, and compacted to 95% of maximum dry density as determined by AASHTO T-99. The bottom of the pond would be graded to a common collection point. The reservoirs would be fenced using 6-foot chain link and access gates would be provided. Perimeter drainage ditches would be necessary to convey external runoff or seepage around and away from the site.

Construction of the ponds could be phased to coincide with the existing and future demands. To limit the amount of potential wave erosion, the fetch would be limited by building the ponds in a multi-cell arrangement. The largest cells would be limited to 1320 feet by 1320 feet (160 acres). This would also provide some operational flexibility in case of contamination or required cell repairs.

2.5.5 Appurtenances

Ring dikes will be equipped with the following devices: a river intake, mechanical screens, pumping facilities, piping and valves, devices to indicate water levels; an emergency overflow; and appropriate security devices (e.g. fencing, locked manholes, doors and gates.

Access to the ring dikes will be by gravel or paved roads depending on the location. Power and communications and/or remote monitoring equipment also will be provided.

2.6 AQUIFER STORAGE AND RECOVERY SYSTEM

2.6.1 General

Aquifer storage and recovery (ASR) systems are gaining popularity as a method of collecting and storing water when supply exceeds demand and then retrieving it when needed. Under suitable conditions, it allows for storage and recovery of water in a natural reservoir for much less cost and with much less surface space than would be required for comparable surface reservoirs.

The basic principles of ASR systems are as follows: An aquifer is determined to have suitable hydraulic and hydrogeochemical properties for ASR use (these properties are discussed hereafter). A water source with water chemistry properties compatible with groundwater chemistry and geochemistry is injected into the aquifer via one or more wells. Typically this injection occurs when the source water is in abundance and demand is relatively low; for example, disinfected river water or treated wastewater effluent. When demand rises to exceed available surface water supplies, water is pumped back out of the aquifer for surface use. The pumped water will typically be a blend of the surface source water and groundwater; the blend will be of variable proportions, depending on the quantity of source water injected into the aquifer and on aquifer hydraulic conditions. Pumping may occur from the same well or wells used for injection, or it may occur from separate wells. Pumping of water is regulated by system demand as well as well water levels and system pressure, and typically requires programmable logic control (PLC) systems and, for multiple well systems, centralized management. Normally the control is most effectively managed by means of SCADA or similar systems.

2.6.2 Aquifer Compatibility

For any ASR system to work effectively, aquifer conditions must be suitable for the proposed use and must be understood for proper system design. Hydraulically, the aquifer must be of suitable permeability to allow both injection and retrieval of water over sustained periods without excessive injection pressures or large drawdowns. The chemistry of the injected water must not react with groundwater to produce undesirable results, typically involving precipitation of minerals that reduce the aquifer permeability or clog screens over time. Injected water should be disinfected or should originate from a source free of contaminants and excessive bacteria such as fecal coliforms.

Aquifer Hydraulic Conditions. Aquifer water level and pumping test data, from previously-constructed wells or from test wells, are needed to evaluate aquifer hydraulic conditions. Usually this involves step-rate and constant-rate pumping tests as well as a recovery test to evaluate aquifer hydraulic conductivity (K), a constant for the tested medium that is equivalent to the rate of flow of water through a cross section of unit width and height under a unit slope. The transmissivity (T) is a related hydraulic parameter that is defined as the rate of flow through a unit width of an aquifer under a unit hydraulic gradient. The relationship between K and T is defined as

$$T = Kb$$

where b is the aquifer thickness. Another useful parameter is the coefficient of storage (S), defined as the volume of water an aquifer releases from or takes into storage per unit surface area of the aquifer per unit change in head. For example, if a cubic foot of aquifer is drained such that the water level drops by one foot, the storage coefficient is the ratio of the volume of water drained to the aquifer to the volume (1 cubic foot) of aquifer drained.

A practical application of aquifer hydraulic parameters is the concept of specific capacity. This is defined as the well production capacity per unit length of drawdown (for example, gallons per minute of well yield per foot of drawdown), as determined in the ratio Q/s, where Q is the pumped flowrate and s is the drawdown in the well.

The concept of specific capacity is simple, but accurate determination of specific capacity is more difficult because well drawdown changes incrementally over time as the aquifer strives for equilibrium. In a laterally extensive aquifer, drawdown under constant-rate pumping is typically larger during early pumping and gradually becomes less over time. It is common practice to determine specific capacity after 24 hours of constant-rate pumping as a reasonable approximation of a well's production potential. It should be understood, however, that unless an external source of recharge is available, specific capacity will slowly diminish over time under constant pumping conditions.

There are various methods for evaluating T, K, and S. Determination of these aquifer parameters is beyond the scope of this document; however, it is important to emphasize that accurate evaluations of these parameters are necessary for effective ASR system design. It is recommended that construction of each ASR well be performed in conjunction with a test well that is used for performance of aquifer pumping tests to evaluate aquifer hydraulic conditions. It is assumed that existing wells will be close enough to the test wells to use the existing wells for observation during testing. If this is not the case, it may be necessary to install additional monitoring wells.

For the purposes of determining aquifer suitability for ASR, the aquifer should allow sufficient storage to meet the minimum requirements of the water use program, beyond existing and planned future reservoir systems. One consideration in evaluating aquifer storage capacity is the amount of storage available in the unconfined space above the aquifer. For a confined aquifer, the storage volume is approximately equal to the unconfined volume of the aquifer multiplied by the storage coefficient S. In the Fargo-Moorhead areas, the aquifers are generally considered to be confined, although localized areas of unconfined conditions are believed to exist where aquifer drawdown has lowered the piezometric surface to below the confining layer. Estimation of this unconfined storage volume has been attempted but, for various reasons, cannot be accurately estimated without extensive additional study. A more attainable approximation is to assume that, for an aquifer system that is not hydraulically connected to a surface water body, outlet, or another aquifer, the volume of water injected into the ASR wells will be about the same as the water removed by the ASR wells. Therefore, if enough ASR wells are installed and the aquifer is large, it should be possible to install enough wells to accommodate the required system storage and demands.

Most of the aquifers in the Fargo – Moorhead area are confined, which, as noted above, tends to limit the available storage capacity to roughly the volume of available storage in unconfined regions of drawdown. However, these confined aquifers also tend to have limited lateral hydraulic continuity (Ripley, 2000). This can be advantageous for ASR systems because it means that most of the water that is injected into the aquifers for storage can be recovered rather than being dissipated outside the well recovery range or flowing out of the system into adjacent aquifers. Thus, the efficiency of recovery is higher in confined systems, and the water quality is easier to manage because less groundwater mixing is expected as a result of limited groundwater migration and recharge.

Initial pumping level for a given pumping rate (after casing storage volume is removed) in each ASR well can be approximated using specific capacity from the equation

$$\text{Initial Pumping Water Level} = SWL - Qp/(Q/s)$$

where SWL is the static water level in the well and Qp is the pumping rate in the well. For injection, Qp/(Q/s) is added to SWL rather than subtracted.

Long-term storage and production capacity at an individual well may be estimated from the Cooper-Jacob semilog plotting method (Cooper and Jacob, 1946), where drawdown is plotted on a linear axis against time on a logarithmic axis. The applicable equation derived from the original Cooper-Jacob solution is

$$Q = T(\Delta s)/264$$

where Q is the flowrate in gpm, T is in gpd/ft, and Δs is the drawdown per log cycle of time. By this method, after determining the initial water level using specific capacity, the slope Δs can be used to project long-term water level in a well at a given flowrate over a given period of constant pumping at a fixed flowrate. The design well flowrate then can be estimated by extrapolating the well drawdown out to the lowest acceptable well pumping level over the longest amount of time during which the well is expected to be pumped. The lowest acceptable pumping level is determined by the top of the aquifer production zone and the associated top of well screens. Drawdown should never extend below the top of screen (to prevent cascading and screen encrustation from oxygen reactions), and the well pump bowls should be set above the top of the screen. Therefore, the optimum pumping level for maximum production is above the top of screens and pump intake, with additional allowances for net positive suction head and for well inefficiencies. To determine long-term water level for injection, after determining the initial water level from the specific capacity method described above, the change in slope at a given injection flowrate is added to initial water level rather than subtracted.

Note that for initial planning purposes, an assumed flowrate of 500 gpm for each well has been used. This value is based on preliminary estimates of T, on well pumping rates as reported by Ripley (2000), and on drilling contractor experience (Randy Pulkrabek, LTP Drilling, January

20, 2005, personal communication). Actual pumping and injection flowrates will need to be determined for each ASR well on an individual basis, using the methods described above.

Aquatic Chemistry. The chemistry of water injected for storage and recovery in an ASR system must be compatible with aquifer groundwater with which it will mix. Incompatible water chemistries may result in mineral precipitation in the aquifer and/or in the well screen, resulting in reduced production and injection potential. Water quality must be accurately characterized at each ASR well location and in the source water to be injected. Chemical parameters to be characterized should include primary ions (Na, Ca, K, Mg, CO_3, HCO_3, H_2CO_3, Cl, SO_4) as well as iron and manganese. Dissolved oxygen, ammonia, nitrate, nitrite, and pH also should be determined. It is recommended that a modeling code for predicting aquatic chemistry reactions, such as PHREEQC2, be used to verify that the source water is compatible with groundwater. Injected water also should be disinfected or should be from a source that is free of biological contaminants.

2.6.3 Well Locations and Spacing

Well locations should be determined based on aquifer locations and characterizations, well spacing requirements, locations of available property, proximity to existing and planned distribution systems, and vicinity land uses. Well spacing requirements are a function of aquifer hydraulic conditions and pumping rates. Wells should be placed at location where testing has indicated that aquifer hydraulic conditions are favorable for injection and pumping. Favorable aquifer conditions include zones of higher hydraulic conductivity, unconfined head (advisable but not essential), and enough depth from surface to top of the aquifer to allow for both water level drawdown during pumping and water level increases during injection. It is advisable that an aquifer hydraulic computer model utilizing an established code such as MODFLOW (McDonald & Harbaugh, 1988) be used to optimize well spacing. The model should estimate the radius of influence in each well and should identify the optimal spacing of wells to prevent or minimize well interference.

Manual estimates to minimize interference between wells can be made using distance-drawdown semilog plots similar to the Cooper-Jacob method (Driscoll, 1995) where drawdown is plotted against distance from the well rather than time:

$$T = 528Q/\Delta s$$

$$r_0 = (0.3Tt/S)^{1/2}$$

where r_0 is the distance in feet from the well with no change in head when pumped for time t in days.

Ideally, wells will be located as close as possible to existing or planned distribution systems to minimize distribution costs. For planning purposes, a minimum of ¾ acre to 1 acre of land should be available per well to allow for well construction and testing, pumphouse construction, and protection zone buffer around the wellhead. Land uses in the vicinity should allow for source water protection planning; wells should meet minimum setback requirements for sources of contamination and should be at locations away from potential contaminant sources (feedlots, known or suspected chemical or fuel spills, fuel or chemical storage facilities, certain types of industry, etc.). A source water assessment and a source water protection plan should be prepared for the ASR wellfield. At a minimum, the source water assessment and protection plan must comply with the requirements of Article 33-18-01-04/05 of the North Dakota Department of Health well construction regulations.

2.6.4 Well Design

All well construction design must meet the regulatory requirements of Article 33-18-01. Additional requirements are discussed below.

Well Types. There are three basic types of wells that can be used for ASR systems. These are production (or extraction) wells, injection (or recharge) wells, and dual-use wells. Production wells are essentially the same as conventional wells used for pumping groundwater from an aquifer. Injection wells are used for injecting water of suitable water quality into the receiving aquifer for storage. Dual-use wells are capable of both production and injection. The design of each type of well, whether it is used for production, injection, or dual-use, is essentially the same. The mechanical features of each well are different, however.

Production wells are equipped with pumps and associated appurtenances, column pipe, check valves, backflow preventers, cable, pump motors and power feeds, motor controls, and also usually include water level sensors and low-level shutoff probes or similar features to prevent dry pumping. They also usually have pressure regulating features and on-off or speed controls regulated by water system pressure or reservoir levels, as well as pressure relief or regulating valves and, usually, blowoff reservoirs or other mechanisms for safeguarding against excessive system pressure.

Injection wells are equipped with injection column piping and check valves to prevent or minimize cascading and may include pressure sensors, high-level shutoff probes, pressure relief valves, overflows, and/or other features to prevent excessive system pressure or backup. Injection wells do not have pumps for outflow; pressure for injection may come from distribution system pressure or from booster pumps.

Dual-use wells are equipped with the features of both production wells and injection wells. They are also equipped with specially designed two-way valves and system controls that allow pumping both into and out of the wells.

Theoretically, dual-use and injection wells can receive injected water at the same rate as the wells were being used for production pumping. In practice, however, injection rates are usually somewhat less than the production capacities of the wells. This helps to avoid problems with system pressure and tends to promote greater longevity of the wells. A common practice is to use injection rates that are about one-half the production capacity of the wells.

Demand. The well must be large enough to accommodate pumping demand and associated pumps, pump column pipe and couplings, electrical cables, and other equipment. As noted above, it is anticipated that enough wells can be installed to meet the system demands for ASR, up to several thousand gpm. For planning purposes as explained previously, a flowrate of 500 gpm per well has been assumed.

Casing Diameter. Casing must be large enough to accommodate all down-hole equipment, including pumps, electrical cable, ASR valves, and water measurement lines. For a 500 gpm ASR well, the diameter of the two-way valve is approximately 12 inches. For planning purposes, a 12-inch nominal diameter steel casing is projected for most wells.

Aquifer Target Zone. The aquifer production target zone will vary from aquifer to aquifer and from within each aquifer from location to location. The target zones should be the most productive intervals (i.e. coarse-grained, free from fine sand, silt, and clay). This should be determined in each well from drill cutting sieve analysis at selected intervals. For example, the average depth to top of aquifer for ten confined aquifers in the Fargo – Moorhead area is about 149 feet below ground level, and the average depth to bottom of aquifer for these aquifers is about 227 feet, for an average aquifer thickness of 79 feet (including layers of lower permeability within the aquifer that may not be very productive). A list of these aquifers and associated depths are shown in Table 2. For planning purposes, it is assumed that the typical confined aquifer target zone will be from about 160 to 230 ft below ground level. However, the production zones for the various aquifers may vary considerably from this estimate, and the productive intervals may not be contiguous but rather may occur from more than one aquifer layer. Target zones need not be together.

Screened Intervals. ASR well screened intervals should correspond to the aquifer target zones, at determined by sieve analyses. The top of the screened interval(s) should be as deep as possible to maximize available drawdown while allowing reasonably thorough open exposure to aquifer target production zones. It is assumed that the typical ASR well will have 70 feet of screen.

Screen Type and Slot Size. For most alluvial system applications, continuous-slot, wire-wound, V-wire stainless steel screens are the most effective commercially-available screen type. Screen slot sizes will vary from location to location, depending on formation grain size and production requirements. For ASR applications, the risk of precipitation of minerals on screens and in filter packs dictates that screen entrance velocities be kept to a minimum. The design criterion for screen entrance velocity is established at 0.1 ft/sec (Anderson, 1998).

TABLE 2

APPROXIMATE AVERAGE DEPTHS TO SELECTED AQUIFER UPPER AND LOWER BOUNDARIES
SOUTHEASTERN NORTH DAKOTA AND SOUTHWESTERN MINNESOTA

Aquifer	Top	Bottom	Avg. Thickness	Information Source
		Confined Aquifers		
West Fargo North	120	192	72	1
Fargo	141	180	39	1
Nodak-Barnes Township.	231	361	130	1
Nodak-Reed Township.	145	182	37	1
94/10-Raymond Township.	133	167	34	1
94/10-Mapleton Township.	167	250	83	1
Prosper	210	270	60	1
West Pleasant	90	145	55	1
Horace	136	233	97	1
Ponderosa	92	247	155	1
West Fargo South	150	235	85	1
Moorhead	168	266	98	2
Average	**149**	**227**	**79**	
		Unconfined Aquifers		
Sheyenne Delta	0	200	200	3
Hankinson	0	40	40	3
Milnor Channel	0	40	40	3
Elk Valley	0	34	34	4
Average	**0**	**79**	**79**	

Sources:

1) Ripley, D., 2000. Water Resource Characteristics of the West Fargo Aquifer System. North Dakota Ground-Water Studies Report No. 106, Part II.
2) Schlag, Allen, 2003. Aquifer Storage and Recovery Prospects for the Moorhead Aquifer. U.S. Bureau of Reclamation unpublished report, December 2003.
3) USBR, 200X. Groundwater Supply for Wahpeton Industrial Demand. U.S. Bureau of Reclamation unpublished report, XXXX.
4) Bartelson, N., and G. Goven, 1998. North Dakota Groundwater Monitoring Program, 1998 Report. North Dakota Department of Health, Division of Water Quality.

For a flowrate of 500 gpm (1.11 cfs), with an entrance velocity of 0.1 ft/sec, the required screen open area is determined from the following rearrangement of Darcy's equation:

$$A = Q/v$$

$$A = (1.11 \text{ cfs})/0.1 \text{ ft/sec} = 11.1 \text{ ft}^2$$

The open area required per linear foot (A_o)of screen (L) is determined by

$$A_o = A/L$$

For an assumed screen length of 70 feet,

$$A_o = 11.1 \text{ ft}^2/70 \text{ ft} = 0.16 \text{ ft}^2/\text{ft}$$

The maximum allowable specific capacity for maintaining this entrance velocity is derived from the equation (Anderson, 1998)

$$Q/s = (45\text{gpm/ft})\ A_o$$

$$Q/s = (45\text{gpm/ft})\ 0.16 \text{ ft}^2/\text{ft} = 7.2 \text{ gpm/ft}$$

Note that at a pumping rate of 500 gpm, an assumed static water level at top of aquifer at 120 ft, and a maximum pumping water level of 240 ft, the theoretical specific capacity would be

$$Q/s = 500 \text{ gpm}/(240 - 120 \text{ ft}) = 4.2 \text{ gpm/ft}$$

This is less than the specific capacity allowed by the design entrance velocity, so a maximum entrance velocity of 0.1 ft/sec is consistent with other design conditions.

The open area per foot of screen can be used to determine the minimum screen slot size required for the design flowrate and entrance velocity. From Anderson, 1998, pg. 240, for $A_o = 0.16$ ft^2/ft**,** 16-inch diameter screen, any slot size equal to or greater than 0.010 inches will be large enough to achieve entrance velocity requirements. Thus, under the assumptions stated, slot size will be determined by grain size rather than entrance velocity.

Slot size is a function of the filter pack gradation, which is, in turn, a function of aquifer formation material gradation. Screen slots should be sized to prevent more than 10 percent of filter material from passing.

Filter Pack. Filter pack should be clean, rounded, siliceous sand. The gradation of filter pack depends on the aquifer materials being filtered. This depends on the uniformity coefficient (C_u) of the aquifer material:

$$C_u = D_{60}/D_{10}$$

where D_{60} is the grain size for which 60 percent of the aquifer material is smaller and D_{10} is the grain size for which 10 percent of the aquifer material is smaller. From Anderson (1998),

1. for aquifer material with $C_u < 2.5$, use uniform filter material with $C_u < 2.5$ and with the D_{50} of the filter pack 4 to 6 times the D_{50} of the aquifer.
2. For aquifer material with C_u between 2.5 and 5, use uniform filter material with $C_u < 2.5$ and with the D_{50} of the filter pack < 9 times the D_{50} of the aquifer.
3. For aquifer material with $C_u > 5$, use uniform filter material with $C_u < 2.5$ and between 6 to 9 times the D_{30} and the D_{90} of the aquifer.

Depth. Each well should be deep enough to completely penetrate the aquifer and should include a minimum 10-foot blank casing (tailpipe) at the bottom to allow for gradual accumulation of sand during development and pumping. For planning purposes, it is assumed that the average well will be about 250 feet deep. As noted previously, however, actual well depths will vary considerably; 250 feet was selected as a moderately conservative well depth for planning purposes.

Well Casing. Well casing should be mild steel with a minimum wall thickness of 0.375 inches. Depending on actual well depth, a heavier wall thickness may be required; final design should be based on well depth and state well construction regulations. Well casing will be sealed to create a vacuum in the well casing above the water level in the well; this will reduce the potential for oxidation of well casing, screen, and column pipe. A vacuum release will be required to provide emergency and maintenance release of vacuum.

Well Seal. The well seal should be designed in accordance with state regulatory requirements as specified in NDDH Article 33-18-01.

Surface Completion. It is assumed that each well surface will be on a concrete slab, unless geotechnical considerations at each wellsite dictate that foundation footings be used. The well casing will project at least two feet above grade and at least 18 inches above slab. A sanitary well seal will be required.

2.6.5 Pump Selection

Examination of pump curves for submersible pumps for an assumed 500 gpm flowrate with an assumed total dynamic head (TDH) of 200 feet dictates a 40 hp pump:

$$hp = Q(TDH)/3956\eta$$

where η is pump efficiency (typically about 70 percent for submersible pumps).

2.6.6 Inflow and Outflow

ASR well inflow and outflow are typically through separate lines up to the wellhead. Piping into and out of the well may be by a single pipe assembly or by separate piping. When separate piping is used, the well diameter must be increased, sometimes substantially, to accommodate the space requirements of the piping. This could result in a significant increase in drilling costs and complexity. Because it is anticipated that as many as 20 wells will be required for the ASR system, it is recommended that a single pipe assembly be used in the wells. This requires the use of valves specially designed for use in ASR systems. Inflow and outflow piping requirements are discussed below.

Valves. ASR wells require a two-way valve that prevents cascading and air entrainment from injected water while allowing water to be pumped out. The valve is placed below the static water level. The valve will be regulated by solenoid control and by pneumatic pressure using inert (nitrogen) gas to open, close, and constrict the valve as necessary.

A check valve is also placed in the well column pipe within a few feet of the submersible pump. The check valve is required to prevent backflow and reverse spin in the pump. The check valve should be sized to fit the column pipe. Each well also should be equipped with a double-check backflow preventer and with an isolation valve (gate or butterfly) downstream of the backflow preventer.

A pressure relief valve should be installed at each ASR wellhead for blowout protection. This is discussed hereafter. A manual discharge valve for purging and development also should be provided.

Blowout Protection. Each well pumping system should be equipped with a pressure relief valve and overflow discharge to prevent system blowout or damage from excessively high pressure. This may consist of a tee at the wellhead and a valve on the tee that is mechanically set to open for discharge in the event system pressure reaches a predetermined upper limit. Downstream of the pressure relief valve, discharge would be conveyed in a short pipe out of the pump house and into an overflow area. This may consist of a gravel sump, manhole, sanitary sewer, ditch, or other suitable overflow receptacle.

Column Piping. Well column pipe should be sized to accommodate the design flowrate for the pump. Pipe should be large enough to avoid large friction headlosses between the water pumping level and the surface, but no larger than necessary to minimize suspended pipe weight and extension and to allow sufficient room in the well for down-hole accessories such as sounding tubes, pressure transducers, air lines, and power cables. For pump column pipe, a velocity of 8 to 10 ft/sec and a headloss of no more than about 8 feet per 100 feet is a reasonable guide. For steel pipe, from Anderson (1998), pg. 138, a column pipe nominal diameter of 6 inches is acceptable. Pipe should be galvanized steel with threaded couplings and should be centered in the well using centralizers.

Wellhead Piping. All above-ground wellhead piping should be ductile iron with flanged fittings.

Metering. Each ASR well must be metered both for inflow and outflow. This should be accomplished by installing a flow meter with flow totalizer on both the inflow line and the outflow line. The meters should be located at least 10 pipe diameters downstream of pipe bends or fittings and at least five pipe diameters upstream of bends or fittings. Meters should be sized to accommodate the flow requirements of each ASR well. The meters also should provide electronic signal to the PLC.

Water Level Monitoring. Each ASR well should be equipped with at two means of water level monitoring. One of these means must include a down-hole pressure transducer and low-level shutoff to prevent breaking of suction at the pump. The transducer also should transmit water levels to the PLC. The other water level method may be a 1-inch PVC tube and wellhead port for manual measurement by means of a water level sounder, or it may be a ¼-inch diameter air line tube equipped with a fitting for a pressurized air or nitrogen bottle and pressure gage.

Pressure Switch. Each ASR well should be equipped with a pressure switch and pressure gage. The switch should interact with the PLC to regulate pumping and to shut off flow when pressure reaches an upper limit. The pressure switch should be selected based on the anticipated range of pressure that could be expected in the system.

2.6.7 Water Quality Monitoring

Water quality monitoring is necessary for any ASR system to verify that the water quality of both the influent and the effluent is acceptable. As noted previously, if the influent water chemistry is incompatible with groundwater chemistry, precipitation and encrustation can occur, resulting in reduced pumping potential, reduced storage capacity, increased maintenance costs, and reduced system lifespan. A regular sampling program will be required. Samples should be collected from ports on inflow and outflow pipelines for laboratory analysis periodically. In addition, continuous automated monitoring of key field water parameters should also occur to serve as indicators of water chemistry conditions and changes.

Sampling Ports. Sampling ports should be installed near each wellhead for both inflow and outflow lines. Ports should consist of threaded taps in the pipeline and should be needle-nosed, stainless steel taps that allow very low-flow, non-aerated sample collection.

Automated Sampling. Automated samplers should be installed on both the inflow and outflow lines at each well. The automated samplers should monitor for pH, electrical conductivity, total dissolved solids, turbidity, and temperature. The automated samplers should transmit data to the PLC.

2.6.8 Instrumentation and Controls

The process control of the ASR system is critical, assuring efficient operation. The control system should include redundancy, distributed process control, centralized monitoring, supervisor control and data archiving.

System Architecture. The system architecture should allow for redundancy of critical elements, along with distributed process control. The distributed process control allows for the process to continue to operate in a normal condition when a failure, within the system, occurs. This type of control would be facilitated through the use of local PLC controllers at each process site. The individual PLCs would supply the centrally located SCADA system with equipment status, process indications, and alarms. This data would be archived at a central site, allowing for historical trending and report generation.

SCADA System. Operator interfacing of process variables should be through a graphical computer based software package. This software package would provide Supervisor Control and Data Acquisition (SCADA). The SCADA system communications should be by means of secure radio telemetry. SCADA workstations should be capable of receiving data for storage and archiving, as well as real-time monitoring and alarm annunciation. The telemetry system should accommodate centralized troubleshooting of remote PLC programs without interruption of scheduled data transfers with other process controllers.

Programmable Logic Controller. Each ASR well should be equipped with a local PLC system that would be programmed to monitor and regulate flow at the local well based on process parameters, including system pressure and flow. Each PLC also should be capable of receiving and transmitting, via secure radio telemetry, data from the central SCADA system and other remote process controllers. Each well site should be provided with local status and indication through a panel mounted Human Machine Interface (HMI). The HMI should be connected to the local PLC though a serially networked interface. This would allow local set point changes, local alarm acknowledgement, and status and process variable indication.

2.6.9 Pump and Controls Housing

Each ASR wellhead should be enclosed in a pump house. This can most easily be done using prefabricated wood or metal buildings mounted on concrete pad foundations. The structures should be insulated and equipped with heating and ventilation and, if required to protect the PLC, air conditioning. The structures should be well-lit and provide at a minimum 115-V power outlets. One wall of each structure should have a roll-up door large enough to accommodate installation and removal of all equipment in the building. A minimum 4-foot by 4-foot roof hatch should be installed over the wellhead to allow for installation and servicing of down-hole pump equipment. The buildings should meet all state and local codes, including fire, wind, snow load, property boundary setback, etc. Each structure should be secured by locking doors and should be protected by a chain-link fence around the entire building that encloses at least a 100-foot setback radius to protect the wellhead.

2.6.10 Power

Each ASR well should be provided with 460-V, 3-phase electrical power. An on-site pad-mounted or pole-mounted transformer also will be required at each site to allow for 230-V and 110-V power hookups for instruments, equipment, lighting, and outlets.

2.7 SYSTEM INSTRUMENTATION AND CONTROL

2.7.1 General

The Project water delivery system will be large and complex, comprised of numerous pump stations, canal turnout water storage reservoirs, service connections, many miles of pipe and appurtenances, water treatment plants (interfacing with two major existing facilities), and intake structures. 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 water supply system are:

- The continuous production and supply of safe drinking water;
- The automatic execution of corrective measures and automatic response to potentially hazardous 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.
- Record keeping for accounting purposes

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

Water Conveyance System. The primary objective of a water/conveyance control system is to provide a reliable water supply at a minimum cost. Major benefits derived from the system are:

- Data acquisition for process analysis, control, measuring, maintenance functions and report generation;
- Instant observation of the status of any key process equipment flow or water quality parameter;
- Control of complex, multi-variable processes;
- Control of tedious or routine procedures;
- Alarm functions for process equipment malfunctions, including serious conditions such as leaks, overflowing and drain conditions, or exceedance of process and equipment set points or limits; and
- Monitoring water use at turnout point

Control options for a water transmission system 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 pressure control 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 design tasks will evaluate all four levels of the system based on the local conditions, size and complexity of the water supply systems, the management philosophy of the owner, and the anticipated financial constraints. The final recommendation will be based on operator’s preference and on sound engineering judgment.

For the purpose of this technical document, it is envisioned that some level of computer based monitoring and control equipment using digital signals (i.e. on/off electrical pulses) sent along a coaxial cable or fiber optic line for transmission of measurement and control signals. A large number of signals can be sent along a single coaxial cable or fiber optic line 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 transmission 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, a pump use cycle can be controlled by a PLC. The PLC is programmed to rotate duty pump use based on a pre-determined time schedule, high wetwell level, and high demand. Hence, the operator would not normally control this function; he would only receive status signals for the PLC to indicate the progress of status of the pump cycle.

The size of computer system required by a system 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 system 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 system operation, useful for system operations, and a luxury or secondary information system. The essential items are: flow rate control, chemical feed rate control, pump rate control, analysis and recording of in-pipe water quality, control and recording of residual chlorine, and detection of 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 mechanical process, closed-loop automatic control of residual chlorine and the pH of the water being transmitted in the system, and devices that check the possibility of leakage or pressure loss (the latter require extensive custom designed software). All these items are not vital to normal operation. Lastly, are items defined as items that make operation easier without providing compensatory savings in cost. Prior to final design, the needs and tradeoffs of the various systems will be discussed with Garrison Diversion staff.

2.7.2 Data Exchange

At this point in the process, it is envisioned the following operational data would be transmitted to a centralized location 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.

2.7.3 Computer Control Center

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

- A building to house the central computer, system operating area, and two office spaces would be manned or monitored remotely 24 hours per day, 7 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.

2.7.4 System Communications

System communications between the water treatment plant(s) and pumping and transmission pipeline equipment will be by radio telemetry or fiber optic line, depending upon location of the various facilities and available utilities. The communication system configuration will have to be determined in greater detail during design activities.

2.8 REFERENCES

Anderson, K.E., 1998. Ground Water Handbook. National Ground Water Association, Westerville, Ohio, 401 pp.

Cooper, H.H., and C.E. Jacob, 1946. A generalized graphical method for evaluating formation constants and summarizing well field history. *Transactions, American Geophysical Union,* Vol. 27, No. 4.

Driscoll, F.G., 1995. Groundwater and Wells. Johnson Screens, St. Paul, Minnesota, 1089 pp.

McDonald, M.G., and A.W. Harbaugh, 1988. A modular three-dimensional finite-difference ground-water flow model. U.S. Geological Survey Techniques of Water-Resource Investigations, Book 6, 586 pp.

North Dakota Department of Health, Well Construction Regulations, Article 33-18-01.

Pulkrabek, Randy, LTP Drilling, January 20, 2005, personal communication.

Ripley, D.P., 2000. The Water Resource Characteristics of the West Fargo Aquifer System. North Dakota Ground-Water Studies No. 106, Part II.