CITY OF LARAMIE
WATER SUPPLY MASTER PLAN
LEVEL II
## TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0 INTRODUCTION</td>
<td>1-1</td>
</tr>
<tr>
<td>1.1 PREVIOUS REPORT</td>
<td>1-1</td>
</tr>
<tr>
<td>1.2 SCOPE OF SERVICES</td>
<td>1-1</td>
</tr>
<tr>
<td>1.3 PROJECT LOCATION</td>
<td>1-3</td>
</tr>
<tr>
<td>1.4 PROJECT AUTHORIZATION</td>
<td>1-3</td>
</tr>
<tr>
<td>2.0 CORROSION AND FLOW STUDY OF TRANSMISSION LINES</td>
<td>2-1</td>
</tr>
<tr>
<td>2.1 RESTRICTED FLOWS</td>
<td>2-1</td>
</tr>
<tr>
<td>2.1.1 INTRODUCTION</td>
<td>2-1</td>
</tr>
<tr>
<td>2.1.2 PIPELINE LOCATION</td>
<td>2-1</td>
</tr>
<tr>
<td>2.1.3 INLET STRUCTURE AND RESERVOIR INLET PIPELINE</td>
<td>2-2</td>
</tr>
<tr>
<td>2.1.4 PLANT PIPING ARRANGEMENTS</td>
<td>2-4</td>
</tr>
<tr>
<td>2.1.5 AIR RELEASE VALVESPROFILE OF PIPELINE</td>
<td>2-4</td>
</tr>
<tr>
<td>2.1.6 HYDRAULIC RESTRICTIONS</td>
<td>2-6</td>
</tr>
<tr>
<td>2.1.7 DETERMINATION OF “C” FACTORS ON 20” AND 24” TRANSMISSION MAINS</td>
<td>2-7</td>
</tr>
<tr>
<td>2.1.8 CONCLUSIONS</td>
<td>2-8</td>
</tr>
<tr>
<td>2.2 CORROSION PREVENTION</td>
<td>2-8</td>
</tr>
<tr>
<td>2.2.1 INTRODUCTION</td>
<td>2-8</td>
</tr>
<tr>
<td>2.2.2 APPROACH</td>
<td>2-9</td>
</tr>
<tr>
<td>2.2.3 TEST PROCEDURES AND RESULTS</td>
<td>2-10</td>
</tr>
<tr>
<td>2.2.4 CONCLUSIONS</td>
<td>2-21</td>
</tr>
<tr>
<td>2.3 APPURTENANCES</td>
<td>2-22</td>
</tr>
<tr>
<td>2.3.1 CONCLUSIONS</td>
<td>2-23</td>
</tr>
<tr>
<td>3.0 SYSTEM OPERATIONAL ANALYSIS</td>
<td>3-1</td>
</tr>
<tr>
<td>3.1 EXISTING CITY DISTRIBUTION AND STORAGE SYSTEM</td>
<td>3-1</td>
</tr>
<tr>
<td>3.1.1 WATER SUPPLY</td>
<td>3-1</td>
</tr>
<tr>
<td>3.1.2 PRESSURE ZONES</td>
<td>3-1</td>
</tr>
<tr>
<td>3.1.3 STORAGE RESERVORIES</td>
<td>3-2</td>
</tr>
<tr>
<td>3.1.4 DISTRIBUTION SYSTEM PIPING</td>
<td>3-5</td>
</tr>
<tr>
<td>3.1.5 TOPOGRAPHY</td>
<td>3-5</td>
</tr>
<tr>
<td>3.1.6 CONCLUSIONS</td>
<td>3-5</td>
</tr>
<tr>
<td>3.2 FUTURE SERVICE PLAN</td>
<td>3-5</td>
</tr>
<tr>
<td>3.2.1 CONCLUSIONS</td>
<td>3-8</td>
</tr>
<tr>
<td>3.3 HYDRAULIC MODEL</td>
<td>3-8</td>
</tr>
<tr>
<td>3.3.1 CYBERNET HYDRAULIC MODELING SOFTWARE</td>
<td>3-8</td>
</tr>
<tr>
<td>3.3.2 PARAMETERS AND ASSUMPTIONS USED</td>
<td>3-10</td>
</tr>
<tr>
<td>3.3.3 RESULTS OF SYSTEM ANALYSIS</td>
<td>3-11</td>
</tr>
</tbody>
</table>
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.3.4 CONCLUSIONS</td>
<td>3-14</td>
</tr>
<tr>
<td>4.0 FLOW TEST OF PIONEER CANAL</td>
<td>4-1</td>
</tr>
<tr>
<td>4.1 INTRODUCTION</td>
<td>4-1</td>
</tr>
<tr>
<td>4.2 AUTUMN FLOW STUDY</td>
<td>4-1</td>
</tr>
<tr>
<td>4.3 WINTER FLOW STUDY</td>
<td>4-4</td>
</tr>
<tr>
<td>4.4 CONCLUSIONS</td>
<td>4-7</td>
</tr>
<tr>
<td>5.0 RATE STUDY</td>
<td>5-1</td>
</tr>
<tr>
<td>5.1 INTRODUCTION</td>
<td>5-1</td>
</tr>
<tr>
<td>5.2 1995-96 FINANCIAL BASELINE CONDITIONS</td>
<td>5-1</td>
</tr>
<tr>
<td>5.3 10-YEAR FINANCIAL PLAN</td>
<td>5-1</td>
</tr>
<tr>
<td>5.3.1 CAPITAL IMPROVEMENT PROGRAM</td>
<td>5-5</td>
</tr>
<tr>
<td>5.3.1.1 WELLFIELDS</td>
<td>5-5</td>
</tr>
<tr>
<td>5.3.1.2 SOLDIER SPRINGS AND LOW RESERVOIRS</td>
<td>5-5</td>
</tr>
<tr>
<td>5.3.1.3 36-INCH RAW WATER PIPELINE REPLACEMENT</td>
<td>5-6</td>
</tr>
<tr>
<td>5.3.1.4 SPUR WELLFIELD</td>
<td>5-6</td>
</tr>
<tr>
<td>5.3.1.5 24-INCH PIPELINE</td>
<td>5-6</td>
</tr>
<tr>
<td>5.3.1.6 20-INCH LAGOON PIPELINE</td>
<td>5-7</td>
</tr>
<tr>
<td>5.3.1.7 ZONE 3 MODIFICATIONS</td>
<td>5-7</td>
</tr>
<tr>
<td>5.3.1.8 CORROSION CONTROL PROGRAM</td>
<td>5-8</td>
</tr>
<tr>
<td>5.3.1.9 INTERNALLY-FUNDED PROJECTS</td>
<td>5-8</td>
</tr>
<tr>
<td>5.4 10-YEAR WATER RATE PROJECTION</td>
<td>5-8</td>
</tr>
<tr>
<td>5.4.1 OPERATING REVENUES</td>
<td>5-11</td>
</tr>
<tr>
<td>5.4.2 OPERATING EXPENSES</td>
<td>5-11</td>
</tr>
<tr>
<td>5.4.3 NON-OPERATING REVENUES AND EXPENSES</td>
<td>5-11</td>
</tr>
<tr>
<td>5.4.4 DEBT SERVICE</td>
<td>5-11</td>
</tr>
<tr>
<td>5.4.5 CAPITAL PROJECTS FUNDED FROM RATES</td>
<td>5-11</td>
</tr>
<tr>
<td>5.5 CONCLUSIONS AND RECOMMENDATIONS</td>
<td>5-12</td>
</tr>
<tr>
<td>6.0 CONCEPTUAL LEVEL PLANS AND COST ESTIMATES</td>
<td>6-1</td>
</tr>
<tr>
<td>6.1 ALTERNATIVE ANALYSIS</td>
<td>6-1</td>
</tr>
<tr>
<td>6.1.1 36&quot; RAW WATER LINE</td>
<td>6-1</td>
</tr>
<tr>
<td>6.1.1.1 INLET STRUCTURE</td>
<td>6-1</td>
</tr>
<tr>
<td>6.1.1.2 36&quot; RAW WATER LINE</td>
<td>6-2</td>
</tr>
<tr>
<td>6.1.1.3 CONCLUSIONS</td>
<td>6-7</td>
</tr>
<tr>
<td>6.1.2 24 INCH TRANSMISSION LINE</td>
<td>6-8</td>
</tr>
<tr>
<td>6.1.2.1 CORROSION</td>
<td>6-8</td>
</tr>
<tr>
<td>6.1.2.2 CONCLUSIONS</td>
<td>6-8</td>
</tr>
<tr>
<td>6.1.3 20&quot; TRANSMISSION LINE</td>
<td>6-9</td>
</tr>
</tbody>
</table>
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.1.4 CITY DISTRIBUTION SYSTEM</td>
<td>6-9</td>
</tr>
<tr>
<td>6.1.4.1 INTRODUCTION</td>
<td>6-9</td>
</tr>
<tr>
<td>6.1.4.2 PRESSURE ZONES</td>
<td>6-10</td>
</tr>
<tr>
<td>6.1.4.3 ADDITIONAL STORAGE</td>
<td>6-12</td>
</tr>
<tr>
<td>6.1.4.4 ADDITIONAL CONVEYANCES</td>
<td>6-15</td>
</tr>
<tr>
<td>6.1.4.5 PRESSURE ZONES 3,4, AND 5</td>
<td>6-16</td>
</tr>
<tr>
<td>6.1.4.6 PUMP STATION MODIFICATIONS</td>
<td>6-16</td>
</tr>
<tr>
<td>6.1.4.7 CORTHELL HILL</td>
<td>6-17</td>
</tr>
<tr>
<td>6.1.4.8 THE SPUR WELL FIELD</td>
<td>6-18</td>
</tr>
<tr>
<td>6.1.4.9 WATER TREATMENT PLANT PUMP STATION</td>
<td>6-19</td>
</tr>
<tr>
<td>6.1.4.10 CONCLUSIONS</td>
<td>6-21</td>
</tr>
<tr>
<td>6.1.5 PIONEER CANAL</td>
<td>6-21</td>
</tr>
<tr>
<td>6.1.5.1 CANAL LINING OPTIONS</td>
<td>6-21</td>
</tr>
<tr>
<td>6.1.5.2 INFLUENCES OF CANAL LINING ON GROUNDWATER</td>
<td>6-23</td>
</tr>
<tr>
<td>6.1.6 APPURTENANCES</td>
<td>6-23</td>
</tr>
<tr>
<td>6.2 RECOMMENDED CAPITAL IMPROVEMENTS</td>
<td>6-24</td>
</tr>
<tr>
<td>6.2.1 36&quot; RAW WATER LINE</td>
<td>6-24</td>
</tr>
<tr>
<td>6.2.1.1 INLET STRUCTURE</td>
<td>6-24</td>
</tr>
<tr>
<td>6.2.1.2 NEW RAW WATER PIPELINE TO THE WTP</td>
<td>6-24</td>
</tr>
<tr>
<td>6.2.1.3 COST ESTIMATES</td>
<td>6-26</td>
</tr>
<tr>
<td>6.2.2 24 INCH TRANSMISSION LINE</td>
<td>6-27</td>
</tr>
<tr>
<td>6.2.2.1 CORROSION</td>
<td>6-27</td>
</tr>
<tr>
<td>6.2.2.2 COST ESTIMATES</td>
<td>6-29</td>
</tr>
<tr>
<td>6.2.3 20&quot; TRANSMISSION LINE</td>
<td>6-31</td>
</tr>
<tr>
<td>6.2.4 CITY DISTRIBUTION SYSTEM</td>
<td>6-31</td>
</tr>
<tr>
<td>6.2.4.1 INTRODUCTION</td>
<td>6-31</td>
</tr>
<tr>
<td>6.2.4.2 PRESSURE ZONES</td>
<td>6-31</td>
</tr>
<tr>
<td>6.2.4.3 ADDITIONAL STORAGE</td>
<td>6-33</td>
</tr>
<tr>
<td>6.2.4.4 ADDITIONAL CONVEYANCES</td>
<td>6-34</td>
</tr>
<tr>
<td>6.2.4.5 PRESSURE ZONE 3</td>
<td>6-34</td>
</tr>
<tr>
<td>6.2.4.6 PUMP STATION MODIFICATIONS</td>
<td>6-36</td>
</tr>
<tr>
<td>6.2.4.7 CORTHELL HILL</td>
<td>6-37</td>
</tr>
<tr>
<td>6.2.4.8 THE SPUR WELL FIELD</td>
<td>6-37</td>
</tr>
<tr>
<td>6.2.4.9 PRIORITIES</td>
<td>6-39</td>
</tr>
<tr>
<td>6.2.4.10 COST ESTIMATES</td>
<td>6-40</td>
</tr>
<tr>
<td>6.2.5 PIONEER CANAL</td>
<td>6-43</td>
</tr>
<tr>
<td>6.2.5.1 WINTER FLOW STUDY</td>
<td>6-43</td>
</tr>
<tr>
<td>6.2.5.2 CANAL LINING</td>
<td>6-45</td>
</tr>
<tr>
<td>6.2.5.3 INFLUENCES OF CANAL LINING ON GROUNDWATER</td>
<td>6-45</td>
</tr>
</tbody>
</table>
TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>6-45</td>
</tr>
<tr>
<td>6.2.5.4 COST ESTIMATES</td>
<td>6-46</td>
</tr>
<tr>
<td>6.2.6 APPURTENANCES</td>
<td>6-48</td>
</tr>
</tbody>
</table>
# LIST OF FIGURES

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Figure 2-1 Existing Pipeline Hydraulics</td>
<td>Back Pocket</td>
</tr>
<tr>
<td>Figure 3-1 Water Supply System Existing Configuration</td>
<td>Back Pocket</td>
</tr>
<tr>
<td>Figure 3-2 Schematic Diagram of Piping in Vicinity of High/Low Booster Station</td>
<td>3-3</td>
</tr>
<tr>
<td>Figure 3-3 Existing Water Supply System Configuration</td>
<td>3-4</td>
</tr>
<tr>
<td>Figure 3-4 Projected Development by 2045</td>
<td>3-7</td>
</tr>
<tr>
<td>Figure 3-5 Existing Water Transmission, Distribution and Storage System Pressure Zone Map with Pressure Contours under Peak Hour Conditions</td>
<td>3-12</td>
</tr>
<tr>
<td>Figure 3-6 Existing Water Transmission, Distribution and Storage System Available Fire Flow Map under Peak Hour Conditions</td>
<td>3-13</td>
</tr>
<tr>
<td>Figure 4-1 Canal Lining Exhibit - Pioneer Canal</td>
<td>4-2</td>
</tr>
<tr>
<td>Figure 6-1 36&quot; Raw Water Line Alternatives</td>
<td>6-3</td>
</tr>
<tr>
<td>Figure 6-2 Alternative Points of Connection for Spur Well Pipeline</td>
<td>6-20</td>
</tr>
<tr>
<td>Figure 6-3 Typical Cross Sections - Pioneer Canal</td>
<td>6-22</td>
</tr>
<tr>
<td>Figure 6-4 Alternative No. 4</td>
<td>Back Pocket</td>
</tr>
<tr>
<td>Figure 6-5 24&quot; Cathodic Protection Plan</td>
<td>6-28</td>
</tr>
<tr>
<td>Figure 6-6 Proposed Pressure Zone Configuration</td>
<td>6-32</td>
</tr>
<tr>
<td>Figure 6-7 Possible Locations of Proposed Zone 3 Tank &amp; Pipelines</td>
<td>6-35</td>
</tr>
<tr>
<td>Figure 6-8 Corthell Hill Pipeline</td>
<td>6-38</td>
</tr>
<tr>
<td>Figure 6-9 20 Foot Parshall Flume Modifications</td>
<td>6-44</td>
</tr>
</tbody>
</table>
# LIST OF TABLES

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Table 2-1</td>
<td>Soil Resistivity Frequency Distribution</td>
</tr>
<tr>
<td>Table 2-2</td>
<td>Soil Resistivity Frequency Distribution</td>
</tr>
<tr>
<td>Table 2-3</td>
<td>Cathodic Protection Current Requirement 36 Inch Steel Raw Water Line</td>
</tr>
<tr>
<td>Table 4-1</td>
<td>Summary of 1995 Autumn Flow Measurements</td>
</tr>
<tr>
<td>Table 4-2</td>
<td>Summary of 1995-1996 Winter Flow Date</td>
</tr>
<tr>
<td>Table 5-1</td>
<td>City of Laramie Water Fund 1995-96 Baseline Revenue Requirements</td>
</tr>
<tr>
<td>Table 5-2</td>
<td>City of Laramie Water Fund Capital Improvement Plan</td>
</tr>
<tr>
<td>Table 5-3</td>
<td>City of Laramie Water Fund Forecasted Debt Service Schedule</td>
</tr>
<tr>
<td>Table 5-4</td>
<td>City of Laramie Water Fund Forecasted Cash Flows</td>
</tr>
<tr>
<td>Table 5-5</td>
<td>City of Laramie Water Fund Capital Financing Plan</td>
</tr>
<tr>
<td>Table 6-1</td>
<td>Alternative Matrix for a 50 Year Design Life Pipe system to Supply Raw Water From Sodergreen Reservoir to the Laramie WTP</td>
</tr>
<tr>
<td>Table 6-2</td>
<td>Conceptual Design Cost Estimate for 42-inch Pipeline</td>
</tr>
<tr>
<td>Table 6-3</td>
<td>Conceptual Design Cost Estimate for Replacement of Approximately 3,400 Linear Feet of 24 inch DIP Transmission Line</td>
</tr>
<tr>
<td>Table 6-4</td>
<td>Conceptual Design Cost Estimate for “Hot Spot” Cathodic Protection on the 24 inch DIP Transmission Line</td>
</tr>
</tbody>
</table>
## LIST OF TABLES

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Table 6-5 Conceptual Design Cost Estimate for Zone 3 Modification</td>
<td>6-40</td>
</tr>
<tr>
<td>Table 6-6 Conceptual Design Cost Estimate for Spur Monocline Well Field</td>
<td>6-41</td>
</tr>
<tr>
<td>Table 6-7 Conceptual Design Cost Estimate for Corthell Hill</td>
<td>6-42</td>
</tr>
<tr>
<td>Table 6-8 Conceptual Design Cost Estimate for Winter Flows Measurement at the 20 foot Parshall Flume</td>
<td>6-46</td>
</tr>
<tr>
<td>Table 6-9 Conceptual Design Cost Estimate for Lining the Pioneer Canal</td>
<td>6-47</td>
</tr>
<tr>
<td>Table 6-10 Conceptual Design Cost Estimate for Winter Flow Study and Groundwater Study</td>
<td>6-48</td>
</tr>
</tbody>
</table>
PROJECT PERSONNEL
CITY OF LARAMIE WATER SUPPLY MASTER PLAN LEVEL II

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1.0 INTRODUCTION

1.1 PREVIOUS REPORT

In 1983 the “Laramie Water Master Plan Final Report for the City of Laramie”, by Banner Associates, was prepared. The City is rapidly approaching the limits of its existing developed water supply and thus, the City felt a need to examine its water supply sources and alternatives for expansion. In an effort to keep this master plan consistent with present development trends and to develop a plan that would endeavor to anticipate future development trends and their related impacts to the water supply, treatment, transmission, and distribution system, the City of Laramie, in conjunction with the Wyoming Water Development Commission (WWDC) commissioned a City of Laramie Water Supply Master Plan Level I study in 1994. This study was completed by Western Water Consultants, Inc. and submitted to WWDC in January 1995.

The Level I study addressed four primary areas of the water system. These were supply, treatment, transmission and distribution. Section 10.0 of the Level I study summarized the recommendations for further study and improvements to extend the capability of the existing system to meet future needs. In summary, this section recommended: 1) further analysis of the cathodic protection system and pipeline condition of the 20" and 24" transmission lines from the water treatment plant to the 8 million gallon (MG) storage reservoir, 2) study of the ability of the Pioneer Canal to deliver water during the winter months, 3) study of Laramie’s rate structure, 4) further study to optimize the joint use of surface and ground water resources, operational analysis, and pump and backup power needs for existing booster pump stations for Zones 2 through 5.

1.2 SCOPE OF SERVICES

The purpose of the City of Laramie Water Supply Master Plan Level II is to perform additional analysis to determine necessary improvements to the supply, transmission, distribution and storage components of Laramie’s raw and potable water system. The following is the Scope of Services for the City of Laramie Water Supply Master Plan Level II study as developed by the WWDC.

Corrosion and Flow Study of Transmission Lines

a. **Restricted flows.** The supply pipe lines from Sodergreen Reservoir to the Water Treatment Plant (WTP) operate at less than 50% of calculated efficiency. Using flow tests, visual inspection or other appropriate methods, the Consultant shall identify the constraints preventing full capacity flows. The Consultant shall identify the structural integrity and hydraulic capacity of the pipeline.
b. **Corrosion prevention.** The Consultant shall continue and expand the corrosion evaluation begun in the Level I study. Specifically, the Consultant shall develop plans and cost estimates to upgrade the existing cathodic protection system, and evaluate the condition and protection needs of the 20 and 24 inch transmission lines between the water treatment plant and the 8 million gallon storage reservoir (approximately 13,500 feet). The Consultant shall evaluate the pipelines for corrosion, the condition of the protective exterior coating, the adequacy of the cathodic protection and electrical continuity. The Consultant should give special consideration to reaches where maintenance has been high, leaks reported, or where corrosive soils exists.

c. **Appurtenances.** All appurtenant structures above ground (manholes, diversion structures, etc.) on pipelines subject to evaluation shall be inventoried and condition evaluated. The need for additional corrosion protection shall be determined.

**System Operational Analysis (KYPIPE)**

The Consultant shall use the existing KYPIPE model to evaluate changes in flow conditions resulting from alterations in pipeline capacities and/or system configuration in the transmission and distribution system. Booster stations and/or additional storage shall be evaluated as methods of maintaining uniform delivery pressures. The Consultant shall recommend physical or operational modifications or improvements to maximize operational efficiency. The Consultant shall, as a part of this analysis, insure that recommended system alterations or enlargements account for the potential of alternative future sources of water to Laramie. Specifically, consideration shall be given to the integration of increased surface water supplies and additional groundwater development.

**Winter Flow Test of Pioneer Canal**

The Pioneer Canal delivers Laramie water from the diversion structure on the Laramie River to Sodergreen Reservoir. A pipeline from the reservoir conveys water to the WTP. Winter operation of the WTP, which will be necessary when future demands require expanded use of surface sources, will be affected by the winter flow characteristics of the Pioneer Canal. Freezing, conveyance losses, and other factors affecting the efficiency of water delivery shall be evaluated by the Consultant. The Consultant shall provide the WWDC and the sponsor with a letter summary of the technical issues that should be included in an access agreement with the Lake Hattie/Pioneer Canal board. The access agreement will include provisions and conditions for future winter use of the canal if the flow test results indicate that winter use of the canal is feasible at the flow rates and under the conditions required for expanded use of surface water. The Consultant shall insure to the extent possible that recommendations for future winter use of the canal are in compliance with Wyoming water law and that the long term best interests of the City of Laramie are protected.
Rate Study

Based on priorities established by the sponsor, the Consultant shall develop and recommend a rate schedule which will provide the cash flow necessary to meet the City’s share of proposed system capital improvements, debt service, and operational costs for the construction and rehabilitation plan as outlined in the Level I study. The rate schedule shall include consideration of alternative sources of funding, population projection, new areas added to the system, and other factors affecting the financial viability of the water utility. The rate study will include a limited review of consumer classes and historical water usage. A complete bill frequency analysis will not be performed and a forecast of customer water usage by rate class will not be developed. Such analyses are normally required to confirm the equity of existing rates among customer classes. Revenue projections from existing rates will be developed using existing estimates of future growth in the customer base and other available information.

Conceptual Level Plans and Cost Estimates

The Consultant shall prepare conceptual level plans and cost estimates for the recommended system improvements. Maximum use shall be made of plans and cost estimates generated in the Level I study. The Consultant’s work in developing cost estimates for WWDC Level III construction requests shall be based on WWDC’s direction following Laramie’s decision regarding the priorities for development of surface and groundwater sources.

1.3 PROJECT LOCATION

The City of Laramie Water Supply Master Plan Level II Study covers the following areas: the City of Laramie, Wyoming; an area between Sodergreen Reservoir and the City of Laramie, within the right-of-way of Highway 230; and an area between the Laramie River and Sodergreen Reservoir within the Pioneer Canal.

1.4 PROJECT AUTHORIZATION

The City of Laramie Water Supply Master Plan Level II Study was authorized by the Wyoming Water Development Commission in a contract with RBD, Inc. Engineering Consultants dated June 2, 1995, under Service Agreement Number 05SC0290696.
2.0 CORROSION AND FLOW STUDY OF TRANSMISSION LINES

2.1 RESTRICTED FLOWS

2.1.1 INTRODUCTION

The Level I study reported an estimated cost of $215,000 for rehabilitation and Cathodic protection of the 36" diameter raw water pipeline from Sodergreen Reservoir to the Water Treatment Plant (WTP). These costs were based on “The 36-inch steel pipe appeared to be in good condition at the one location excavated. Only two leaks have been reported on the pipeline, although several other leak locations are suspected because of green vegetation along the pipeline route.” The Level I report also stated that “An evaluation of the internal condition of the 36-inch . . . steel pipelines should be conducted at several locations to confirm that the assumptions based on the coupons obtained at the taps are correct . . . This should be verified by inspecting . . . the pipelines to assure that the coatings were applied correctly at the joints and that the internal pipeline and coating conditions are as expected.”

One of the key components of the Level II study was to verify the assumptions of the Level I study. In addition, the City of Laramie also expressed concerns regarding recent leaks and the ability of the pipeline to meet future needs. The City concerns focused on the capital costs of recent repairs, the reliability of the pipeline and its hydraulic capacity. Therefore, the hydraulic capabilities, remaining service life and value of the 36" diameter raw water pipeline was investigated. The following items were determined during our field investigation and office analysis:

2.1.2 PIPELINE LOCATION

Due to the presence of surface water at recent breaks of the 36" pipeline near the WTP and the 90-degree bend (see Figure 2-1 in the back pocket of this report), the location of the pipeline in these areas was well defined. However, as further explained later in this section of the report, surface water due to possible pipeline breaks at other locations along the 36" pipeline is not visible. This phenomenon makes locating the 36" pipeline, and addressing the integrity of the pipeline, difficult. In fact, prior to beginning any type of analysis on the 36" pipeline, a fundamental question needed to be addressed; where, near Sodergreen Reservoir, was the 36" pipeline located? Groundwater table elevations didn’t afford the City any means to determine if pipeline leaks exist near Sodergreen Reservoir and, since no leak repairs were required within this stretch of the 36" pipeline, the location of the pipeline near Sodergreen Reservoir became obscured over time. Fortunately, the City of Laramie had design drawings (dated 1945 by R. J. Tipton and Associates) for the 36" pipeline. These drawings were tied to section lines/corners and, ultimately, the layout of the pipeline was retraced (a site visit to the Wyoming Department of Transportation (WDOT) was required to obtain documentation of the date when Hwy. 230 was relocated). Based on the documentation available from the WDOT, the pipeline design drawings and a field survey, it was determined that the majority of the pipeline was in the Hwy. 230 Right-Of-Way (R.O.W.)
Following our field survey, an attempt was made to field locate the pipe near Sodergreen Reservoir to provide an internal inspection access site. The site selected was approximately 50 feet north of the Hwy. R.O.W., but accessing the pipe proved unachievable at this location. Essentially, once the contractor encountered groundwater, the permeability of the surrounding soils turned the exercise into the draining of Sodergreen Reservoir. Following this experience, future attempts at accessing the pipeline were made as far from Sodergreen Reservoir as practical.

Field locates were completed on the entire pipeline by tracing an induced electromagnetic field imparted to the steel pipeline as discussed further within Task 2.2. The actual pipeline location was then surveyed and plotted on Figure 2-1.

2.1.3 INLET STRUCTURE AND RESERVOIR INLET PIPELINE

The existing inlet structure was constructed around 1946 to house a bar screen and the headgates for the 36" pipeline. Although a structural analysis of the inlet structure was not included in the Scope of Services, some of the concrete on the outlet side of the headgates was visible. Upon effecting a seal on the downstream side of the headgates (see below) a limited visual inspection of the concrete on the inside of the structure was possible. This inspection showed that the concrete had surface spalling and, when the water surface in the structure was lowered to an elevation below the surrounding groundwater/water surface elevation (WSE) in Sodergreen Reservoir, minor leaks were visible through the concrete wall. The exposed concrete on the outside of the structure was in good condition. Repair of the interior surface spalling can be readily made through a number of construction techniques. However, the real concerns are the quantity of steel rebar reinforcement within the walls of the structure that has been exposed to the environment and for how long? Small sections of the wall(s) can be rehabilitated by removing and replacing the steel that has been exposed. However, large sections that have been rusted out will jeopardize the structural integrity and may call for more substantial improvements. Unfortunately, the only way to investigate the entire structure will require lowering the water surface elevation in Sodergreen Reservoir, thus allowing inspection of the entire interior face.

Based on our limited field observations, we believe the structural integrity of the inlet structure can be restored. It should be noted that the costs for repair of the structure will be highly dependant on the ability to lower the water surface elevation (WSE) in Sodergreen Reservoir. If the WSE in Sodergreen cannot be lowered to permit the necessary improvements to the inlet structure, a complex system of sheet piling and/or an earthen embankment will need to be constructed. We feel considerations should be made for rehabilitation of this structure, as discussed further under Task 6.0.

During the early stages for the corrosion and hydraulic analysis of the existing 36" raw water pipeline from Sodergreen Reservoir to the WTP, RBD, Inc. became aware that the sealing ability of headgates at the Sodergreen reservoir inlet structure was marginal. The inability of the headgates to effect a seal was apparently due to a lack of guides on the gates which allowed the gates to meander across the concrete sealing face. Due to these headgates,
isolation of the 36" pipeline proved to be one of the most difficult aspects of the entire Level II project. Although in the end the 36" pipeline was isolated, the work that went into this project task took a specially fabricated steel plate, supplemental steel plates, screw jacks and more than two days to complete. The initial plan was based on installing a steel plate against the concrete and across the upstream face of the 36" pipeline as it exited the inlet structure. Unfortunately, large debris (primarily broken concrete rip-rap and rocks) was found in the bottom of the inlet structure across the pipe opening which prohibited a full seal of the steel plate. Ultimately, a seal was completed by bracing across the steel plate to smaller steel plates at the downstream face of the headgate openings and by installing a plastic "diaper" across the larger steel plate. This arrangement was effective, but the video inspection work on the project which followed revealed that there was still a minor amount of flow entering the pipeline.

If improvements are not made at the inlet structure to allow for shutting off flow to the 36" pipeline, the potential damages that could occur during a failure of the 36" pipeline, if it takes days to complete a shutdown, could be significant. The ability of the existing slide gates to complete a seal will only degrade with time, thus increasing the flow through the pipeline and potential damages. We feel it is extremely critical that these headgates be considered for replacement, as discussed further under Task 6.0.

The City had also expressed concerns regarding the potential ability for silt in Sodergreen Reservoir to be carried through the pipeline and eventually enter the WTP. This silt load increases the cost of treatment, may deposit in the pipeline thus affecting the hydraulic capacity and must be periodically removed from the flocculation basins. Presently, the operators periodically "flush" this load, in addition to the WTP sludge accumulation, into the backwash ponds. In fact, backwash pond #1 is almost totally full and it is anticipated that removal of this material will be required in the not-to-distant future. A visual inspection of the tailings left from cleaning the Pioneer Canal at Hwy. 230 is also an indicator of the potential amount of silt deposition into Sodergreen Reservoir. An analysis of the silt accumulations within Sodergreen Reservoir was not included in the Scope of Services.

During our work associated with isolating the 36" pipeline, a rowboat was borrowed from the City and attempts were made to find the end of the 36" pipe in Sodergreen Reservoir. It was hoped to document the conditions at the pipe entrance for the City. Unfortunately, the long length of the pipeline, the windy conditions in the reservoir and the bottom growth in the reservoir prohibited finding the end of the pipeline. However, we were able to locate the top of the pipe out to approximately 180' away from the inlet structure. Upon locating the top of the pipe, it was noted that considerable silting was apparent during probing across the centerline of the pipe. In addition, the sides of the pipe could not be located. This suggested that the pipeline was at least partially covered with earthen materials-indicating that sediments carried into Sodergreen Reservoir from the Pioneer Canal are finding their way to the intake of the 36" pipeline and ultimately into the inlet structure and the WTP. Suggested improvements to reduce the silt load are discussed further under Task 6.0.
2.1.4 PLANT PIPING ARRANGEMENTS

A key element which needed to be addressed prior to isolating the 36" pipeline was how to drain the water in the 36" pipeline and how to supply feed water to the WTP while the 36" pipeline was off-line. The City solved this problem by cutting in an isolation valve on the raw water pipeline in the WTP (a small amount of flow was retained in the low spot of the pipeline at the “Fish Trap Vault”). From this valve, a drain line was connected to a drain sump which discharged into backwash pond no.1. Then, just prior to shutting down the 36" pipeline, backwash pond no.1 was filled to its maximum level through this new piping arrangement. The water stored in backwash pond no.1 was then backfed into the WTP through a temporary pipeline and pump station. Should the time required to complete the testing of the pipeline deplete the storage available, additional water could be supplemented to backwash pond no.1 through the 36" pipeline. This could be accomplished by opening a bypass valve installed on the steel isolation plate at the inlet structure. This arrangement provided the City with a reliable source of treatable water during the evaluation of the 36" pipeline.

2.1.5 AIR RELEASE VALVES/PROFILE OF PIPELINE

Upon isolation of the raw water pipeline, a video inspection was undertaken to provide a safe and economical means of assessing the internal condition of the pipeline. The internal inspections were accomplished through a series of three excavation sites and one in-plant inspection port. The placement of the internal inspection ports was selected to optimize the data collection process. Each internal inspection port was completed by excavating to and accessing the top of the pipe and cutting out an access hole of sufficient size to permit entry of the video camera. The camera is capable of videotaping up to approximately 1800' of pipeline, depending on the internal condition of the pipe, number of bends and whether the camera was being pulled upstream or downstream. During the video inspections, a corrosion assessment of the interior and exterior was also made on the pipeline. In addition, six external inspection sites along the pipeline were used to further assess the structural integrity of and corrosion on the outside of the pipeline. A more complete description of this work is provided in Section 2.2-(Corrosion Prevention).

The external inspections were completed by digging down to the top of the pipe. The external inspection locations were selected, in conjunction with the internal inspection sites, to provide additional corrosion assessments on the exterior of the pipeline. Upon excavating to the top of the pipeline, a corrosion assessment of the exterior of the pipeline was completed. A more complete description of this work is provided in Section 2.2-(Corrosion Prevention). A survey was also completed to obtain the horizontal locations and elevations of the top of the pipe for both the internal and external inspections. Upon completion of the inspections, each access port was welded shut and the site restored to its preconstruction condition.

This work resulted in video footage of almost the entire pipeline. Unfortunately, there were two locations where video was not obtainable. The first location was at the “Fish Trap
Vault.” Coming from internal pit no.1 (see Figure 2-1), the video camera could not pass through a 36" by 20" abrupt vertical plate reducer and, coming from inside the WTP, the video camera could not get past the 20" side outlet tee in the “Fish Trap Vault.” The second location was approximately the first 500' of pipe from Sodergreen Reservoir toward the WTP. This data was unobtainable due to apparent silting in the pipeline which the camera could not track past when video taping from internal inspection site number 3 towards the inlet structure. Prior to installing the steel plate at the inlet structure, an attempt was made to obtain this video footage from the inlet structure working downstream. Unfortunately, large chunks of concrete and other debris prevented the video camera from tracking down the pipeline from this direction.

The video inspection revealed information that would have not been achievable through other means. The inspection revealed corrosion sites (discussed in Section 2.2-Corrosion Prevention), “stick dams,” failures of the internal felt tar paper lining, elliptical pipe sections, low spots and miscellaneous obstructions. The video inspection, accompanied with the field survey, also provided critical information related to the vertical profile of the pipeline. A pipe profile (see Figure 2.1) was obtained by plotting the top of pipe elevation from the internal and external inspection sites on a plan and profile sheet. Then, realizing that if the pipeline was constructed with a continuous downhill gradient, per the design drawings, and given that there was a minor amount of flow within the pipeline, the depth of water in the pipeline should have remained relatively constant. Unfortunately, this was not the case. For example, if the eye of the video camera (set at approximately the midpoint of the pipe) was to dip below the water surface, it could only mean an accompanying dip in the pipe itself.

Figure 2-1 shows that there are numerous dips and high spots in the existing pipeline. This is of concern hydraulically since each high spot has the potential to trap air, thus reducing the pipeline capacity. This becomes even more critical on a low head system since the low water pressure cannot reduce the extent of the trapped air. In fact, this reduction in capacity was experienced by the plant operators when the pipeline had significantly less capacity for months after the inspections as compared to prior to the inspections. This reduction in capacity also affected the project work plan. As an integral part of the hydraulic investigation, a corporation stop was installed on the pipeline at all of the internal inspection sites. The work plan called for installing clear plastic tubing on the corporation stops. Once installed, the water surface in the tubes could be measured when the pipeline was flowing at varying rates, and this data could be added to the previously surveyed top of pipe elevations. The intended end result was a map of the hydraulic grade line.

Unfortunately, adequate results were never obtained due to the reduction of capacity associated with the air entrainment problem. The potential hydraulic gains, costs and value of the hydraulic grade line map associated with removing the trapped air was discussed with the City. The discussions focused on the need to install Air Release Valves (ARV’s) at every highpoint in the system in order for the existing pipeline to achieve its full design capacity. Based on these discussions, the City decided not to install the ARV’s until the entire system had been evaluated hydraulically and the corrosion assessment was complete. The City did however request installing access manholes on the pipeline to allow for future hydraulic
grade line mapping. There are two existing air release valve vaults on the pipeline; neither ARV was functional.

The profile of the pipeline also revealed another interesting condition. The City commented that they only experienced breaks at the location of the 90-degree bends and near the WTP. A review of the ground profile at the existing pipeline alignment shows that these are the only areas along the pipeline where the elevation of the ground is lower than the WSE in Sodergreen Reservoir. Since the ground surface in all other locations is higher than the WSE in Sodergreen Reservoir, the pipeline could have numerous breaks and there would not be sufficient head to drive the water to the surface and thus the break would not be apparent, regardless of its size.

2.1.6 HYDRAULIC RESTRICTIONS

The video inspections also revealed hydraulic restrictions in addition to those associated with trapped air at high points in the pipeline. In particular, locations were noted where the pipe had an elliptical (i.e., squashed) cross section. This condition may have occurred when the pipe was driven on with minimal cover. Although it probably does not affect the system hydraulics considerably, it is another indicator which must be weighed in the assessment of the pipeline. The video tape also noted locations where the interior felt tar paper had become dislodged from the pipe at select pipe joints. At these locations the paper acts as a dam collecting silt and debris. The video tape also revealed one location where sticks had collected into a “stick dam”.

In addition, a considerable body of knowledge was gained regarding construction of the pipeline at the “Fish Trap Vault.” The documentation related to the field construction at the vault was questioned by the City of Laramie since Drawings of Record could not be found. The video inspection revealed a considerable low spot in the pipeline at the “Fish Trap Vault,” a drastic reduction in the pipe diameter (36” to 20”) and inefficient flow conditions (i.e., side outlet tee) through the vault. The low spot at the “Fish Trap Vault” does not have a drain. Therefore, since this is the low spot in the pipeline and could not be videotaped, it is quite conceivable that this area also is encumbered with debris.

A plot of the hydraulic profile of the existing pipeline (see Figure 2-1), at the ultimate design flow rate of 15.75 MGD (9.25 MGD under current water right plus future 6.5 MGD water right transfer from the Laramie River), shows that the head loss from the “Fish Trap Vault” to the WTP is approximately equal to the headloss in the 8000 feet of pipe from Sodergreen Reservoir to the “Fish Trap Vault.” This is primarily attributable to the sudden reduction from 36" pipe to a 20" pipe, the length of 20" pipe, the 20" side outlet tee and the length of 24" pipe. Spreadsheets showing the hydraulic capacity of the existing pipeline are included in the technical Appendix. Notice is drawn to the significant losses from the “Fish Trap Vault” to the WTP.

In addition to calculating the theoretical pipeline capacity, field tests were undertaken to determine the maximum pipeline capacity. These tests were completed prior to beginning

Wyoming Water Development Commission 2-6 RBD, INC.
the flow analysis on the existing pipeline by measuring the time required to fill the north side floculation/sedimentation basins through the existing 36" steel pipeline while flow was being sent directly to the filters. In this manner, when the filter flow is added to the flow entering the floculation/sedimentation basin, the total capacity of the pipeline was estimated (since the filter flow was measured on the outlet pipe and does not account for volumetric changes in the filter basins, the filter flow rates may be slightly askew). A plot of the depth in the basin versus the time required to fill the basin is included in the technical Appendix. This plot shows that the fill rate of the floculation/sedimentation basin, with a water surface elevation in Sodergreen at 7364.18' (obtained through a field survey), is 7.8 MGD. By adding the maximum flow of 3.7 MGD through the filters during this test to the 7.8 MGD going to the floculation/sedimentation basins, the data shows that the maximum pipeline capacity is approximately 11.5 MGD at a Sodergreen WSE of 7364.18. This correlates with the information supplied by the City of Laramie (Lytle 1995).

It should also be noted that the City of Laramie specified that the hydraulic evaluation of the 36" pipe line be based on the elevation of the bottom of the discharge gates to the Pioneer Canal at the outlet of Sodergreen Reservoir. Paul Rechard of Western Water Consultants indicated the floor of the Pioneer Canal flume out of Sodergreen Reservoir is at elevation 7360.50. With this elevation, RBD Inc. based the alternative analysis under task 6.0 on an elevation slightly above this elevation and determined that elevation 7361.00 is the lowest elevation in Sodergreen Reservoir at which the future combined flow of 15.75 MGD could be transported to the Laramie WTP. It should also be noted that since our analysis was conducted when the WSE in Sodergreen was higher than the controlling elevation, the capacity of the 36", per the evaluation criteria, is less than 11.5 MGD. In fact, since there is less than 9 feet of head differential between the WTP (7352.25) and the governing WSE in Sodergreen Reservoir (7361), the head loss in this pipeline, per unit of flow, is extremely critical.

2.1.7 DETERMINATION OF “C” FACTORS ON 20” AND 24” TRANSMISSION MAINS

An integral part of this project was determining the roughness coefficients or “C” factors for the 20" and 24" pipelines from the WTP into the City of Laramie. The long range plan for these pipelines calls for constructing a pump station at the WTP that utilizes the existing 20" and 24" pipelines, thus increasing their existing capacity to convey the future WTP capacity of 15.75 MGD to the City of Laramie.

To assist in determining the future requirements of this pump station, the roughness coefficient of each pipeline was determined. This was accomplished by conducting a flow test on each pipeline during a period of minimal demand, i.e. early in the morning, at approximately 3:00 A.M. to 5:00 A.M.. During this time, the use on the system is minimal. The test was accomplished by passing as much water through the pipeline as possible and recording the metered flow through the pipeline at the WTP and then measuring the hydraulic gradeline or HGL (the elevation to which water would rise to if in a vertical pipe open to the atmosphere) of the pipeline at various locations between the WTP and the City. As a check on the accuracy of the flow meters on the 20" and 24" pipelines, the
corresponding drop in WSE at the WTP wetwell was measured and the flow rate was calculated by measuring the change of volume in the wetwell over a set period of time. To achieve the maximum obtainable flow through the pipeline, City Staff opened up existing “blowoffs” on each respective pipeline. These “blowoffs” are nothing more than a low spot in the pipeline by which the pipeline can be drained. By opening the blowoffs, the quantity of flow through the pipeline was maximized.

The HGL was measured by surveying the elevation of the locations where the HGL was to be determined. Then, the pressure in the pipeline was measured before, during and after the flow test. By adding in the pressure, converted to feet, to the previously surveyed elevation of the pipeline, the HGL during the test was determined. Determination of the roughness coefficient could readily be obtained once the overall length between HGL stations and the overall length of the pipeline was known. The lengths were obtained by locating the structures used to measure the HGL on the Wyoming Department of Transportation plan sets for Hwy. 230 and by field survey in the City. Upon completion of the data collection, the determination of the roughness coefficient or “C” factor was readily accomplished by solving the Hazen-Williams equation for flow in closed pipes for the one unknown quantity, the “C” factor. The results of this analysis are included in the technical appendices. The analysis demonstrated that the existing “C” factors on the 20" and 24" pipelines are approximately 120.

2.1.8 CONCLUSIONS

The existing 36" pipeline does not have the capacity to meet the long term needs of the City of Laramie. The pipeline has significant hydraulic restrictions, in the form of trapped air, squashed pipe, silt and stick dams, and inadequate pipe size. The most significant losses are from the “Fish Trap” vault into the WTP. The isolation gates on the inlet structure are not performing adequately and need to be replaced. In addition, the next time the water level in Sodergreen Reservoir is lowered, an evaluation of the interior of the inlet structure should be performed. Consideration should be given to modifying the pipeline in Sodergreen Reservoir to reduce the silt load into the WTP.

The existing 20" and 24" pipelines from the WTP to the City have a “C” factor of 120. This is indicative of a good clean pipe, although not new.

2.2 CORROSION PREVENTION

2.2.1 INTRODUCTION

The corrosion inspection and testing program was designed to provide a condition assessment of four facilities:

1. The 36 inch steel raw water pipeline from Sodergreen Reservoir to the Water Treatment Plant.
2. The 24 inch ductile iron pipe (DIP) treated water pipeline from the westerly City limits to the 8 MG Storage Reservoir (approximately 25,500 feet). Please note that the Scope of Services identified the 13,500 feet as being from the 8 MG Storage Reservoir to the WTP when in reality the 13,500 feet is from the 8 MG Storage Reservoir to the Laramie River.

3. Approximately 1,180 feet of 20 inch cast iron treated water line from just west of the intersection of Park & Pine Streets north to the intersection of Custer & Pine Streets. Please note that the Scope of Services was misleading as noted in item 2 above.

4. Pipeline appurtenances including the 36 inch line intake structure at Sodergreen Reservoir; valve vault at Hwy 230 (sta 977+00) on the 24 inch line; and the tunnel below I-80 carrying the 24 inch line and a 20 inch pipeline.

The Level I Study, conducted in 1994, provided a preliminary investigation of the 36 inch line and a portion of the 20 inch and 24 inch lines as well as other facilities not included in this study. The Level I Study concluded the 36 inch line, because it had experienced only two reported leaks over 47 years, was a potential candidate for corrosion rehabilitation using cathodic protection and the 24 inch DIP line West of the Laramie River was a likely candidate for replacement within 5 to 10 years based on its leak history and soil corrosivity measurements. No recommendations were made for the 20 inch line as it had not experienced any reported corrosion problems.

The purpose of this Level II testing program was to more fully evaluate and document the existing condition of the above facilities and determine if rehabilitation is feasible or whether replacement is actually necessary.

2.2.2 APPROACH

The field testing program consisted of four major components: surface soil resistivity measurements, pipeline excavation and testing, internal pipeline video inspection, and visual inspection (appurtenances).

Surface soil resistivity measurements are a corrosion industry benchmark test method used to evaluate the corrosiveness of the soil along the pipeline alignment. A sufficient amount of data enables the development of a soil corrosivity profile which can identify segments of "hot" or corrosive soils. Soil resistivity measurements were obtained at approximately 250 to 300 foot intervals along the entire 36 inch pipeline and the surveyed section of the 24 inch pipeline.

Pipeline excavation allows for direct visual inspection and testing of the pipelines, external coating condition assessment, and soil sample gathering and testing. Visual inspection and testing consists of cleaning the pipe to bare metal and classifying the type of corrosion attack (pitting, general, etc.) and measuring corrosion pit size and depth. External coating assessment consists of determining the type of coating material and application method,
material thickness, adhesion to the metal substrate, pliability, and odor (where applicable). Soil samples from excavations are analyzed for soil chemistry variables including pH, chloride, sulfide, moisture content, resistivity, and oxidation-reduction potential (ORP). This information combined with all other test data and observations provides the technical basis for the conclusions regarding the condition of the pipelines. Eight, sixteen, and two excavations were planned for the 36 inch, 24 inch and 20 inch lines respectively.

External corrosion is but one potential threat to the integrity of water pipelines. Internal corrosion must also be evaluated to properly assess pipeline condition and projected service life. The 36 inch raw water line was suspected to have a greater potential for internal corrosion because it transports untreated water as opposed to the other pipelines which transport treated water. With this in mind, a video camera was utilized to visually record the condition of the inside of the entire 36 inch line (except the first 500 feet which was not accessible with the camera and at the “fish trap” vault) and of approximately 2,700 feet of the 24 inch line. This internal inspection allows documentation of the condition of the interior lining as well as the pipe itself. Also, as side benefits, video inspection documents all high and low spots in the pipe profile and locates accumulations of sediment and other debris.

Finally, inspections were performed on three previously identified pipeline appurtenances. Each of these appurtenances are located in concrete vault structures with access manholes. The inspections consisted of visual observations of the concrete structure and of the visible piping within for the purpose of determining rehabilitation requirements.

2.2.3 TEST PROCEDURES AND RESULTS

Soil Resistivity Measurements - 36 Inch Steel Raw Water Line

Soil resistivity is a measure of a soil's ability to conduct electrical current flow which is a fundamental component of the electrochemical process of corrosion. A low soil resistivity value, which translates into a greater ability to conduct electric current, is considered to be more corrosive than a high soil resistivity value. The correlation between soil resistivity and corrosion of a buried metal is, unfortunately, not statistically high enough to predict corrosion rates at any specific location on a pipeline. However, it serves as an excellent indicator of potentially corrosive conditions and the measurements may be inexpensively obtained at the soil surface without excavation.

The criteria used to classify soil resistivity measurements is the same one utilized in the Level I Study for consistency and is shown below:
<table>
<thead>
<tr>
<th>Corrosivity Rating Index</th>
<th>Resistivity Range (ohm-cm)</th>
<th>Anticipated Corrosion Activity</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>0-1,000</td>
<td>Extremely Corrosive</td>
</tr>
<tr>
<td>2</td>
<td>1,001-1,500</td>
<td>Very Corrosive</td>
</tr>
<tr>
<td>3</td>
<td>1,501-5,000</td>
<td>Corrosive</td>
</tr>
<tr>
<td>4</td>
<td>5,001-10,000</td>
<td>Moderately Corrosive</td>
</tr>
<tr>
<td>5</td>
<td>Over 10,001</td>
<td>Mildly Corrosive</td>
</tr>
</tbody>
</table>

Two soil resistivity measurements were obtained at each of twenty seven locations (54 total readings) along the 36 inch pipeline alignment (refer to Technical Appendix A - Table 1). All measurements were conducted using the Wenner 4-Pin Method (ASTM G57) with the pin spacing varied to most closely represent the actual pipeline depth in the field.

Table 2-1 below summarizes the frequency distribution of the 36 inch line soil resistivity data.

<table>
<thead>
<tr>
<th>Corrosivity Index</th>
<th>36 Inch Steel Raw Water Line</th>
<th>% of Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4</td>
<td>7.4</td>
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<td>2</td>
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<tr>
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<tr>
<td>Total Readings</td>
<td>54</td>
<td>100</td>
</tr>
</tbody>
</table>

As Table 2-1 indicates, 87% of the total readings are classified 3, 4, or 5 which is considered corrosive to mildly corrosive. These classifications suggest the soils are not particularly aggressive which is buttressed by the fact that only two leaks on the 36 inch line were reported through 1994. However, caution must be exercised when interpreting this type of data. Because soil is not homogeneous, localized variations in resistivity exist which can affect corrosion activity. Industry experience show that unprotected steel pipelines, even in soils with moderate corrosion classifications, will eventually incur significant corrosion damage and leakage.

Soil Resistivity Measurements – 24 Inch DIP Line

Two soil resistivity measurements were obtained at each of eighty three locations (166 total readings) along the 24 inch pipeline alignment (refer to Technical Appendix A - Table 2). All measurements were conducted using the Wenner 4-Pin Method (ASTM G57) with pin spacing varied to most closely represent the actual pipeline depth in the field.
spacings of 5 and 10 feet. Table 2-2 below summarizes the frequency distribution of the 24 inch line soil resistivity data.

Table 2-2
Soil Resistivity Frequency Distribution

<table>
<thead>
<tr>
<th>Corrosivity Index</th>
<th>24 Inch DIP</th>
<th>% of Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>15</td>
<td>9.0</td>
</tr>
<tr>
<td>2</td>
<td>19</td>
<td>11.5</td>
</tr>
<tr>
<td>3</td>
<td>95</td>
<td>57.2</td>
</tr>
<tr>
<td>4</td>
<td>26</td>
<td>15.7</td>
</tr>
<tr>
<td>5</td>
<td>11</td>
<td>6.6</td>
</tr>
<tr>
<td>Total Readings</td>
<td>166</td>
<td>100</td>
</tr>
</tbody>
</table>

As Table 2-2 shows, the data follows a normal distribution pattern in that approximately 20% of the readings were above and below the "normal" range of corrosive soils while approximately 60% of the readings fell into the corrosive or "normal" range. With this type of data distribution, industry experience suggests that occasional external corrosion failures for ductile iron pipe (DIP) will occur in the more aggressive soils at typical pipe ages between 10 and 30 years. Certain "hot spot" areas may experience multiple leak occurrences in a shorter period of time while other areas may not incur any leaks for 50 to 60 years.

Pipeline Excavation & Testing - 36 Inch Steel Raw Water Line

Eight inspection excavations were completed on the 36 inch raw water line. The excavation locations were determined by first locating three sites for internal inspection access ports that would allow the video camera to travel approximately 1,500 feet (max. camera reach) each direction from the access port. This would allow video coverage of the entire pipe interior. The remaining five external excavations were evenly distributed along the pipe alignment to allow a representative portrayal of pipe and coating condition for different sections of the line.

The inspections consisted of visual observations of the condition of the external coating and the underlying pipe, measurement of corrosion pit depths, soil classification, groundwater depth, depth of pipe burial, and collection of one soil sample from each site for laboratory analysis. Refer to Technical Appendix B for individual pipeline inspection report forms. Also, refer to Technical Appendix C for photographs of typical coating condition as well as samples of severe corrosion damage.

The inspections revealed that the external coating was a field applied hot coal tar enamel with a variable thickness from a few mils to approximately one-quarter inch. Coatings of this type and age were generally machine applied onto the pipe as it was suspended over the pipeline ditch after joint welding but prior to laying in the ditch. Because these kinds of
coatings stayed soft until it cooled and cured, they typically show signs of cold flow movement from marks left by the coating application machine or rock impingement if the pipe was backfilled too quickly.

The coating on the 36 inch line had significant amounts of cold flow movement which resulted in numerous small areas where the coating coverage was very thin and afforded little corrosion protection. The soils in this area contain may small stones, cobbles and pieces of shale which have pushed through and breached the coating, particularly on the pipe bottom and lower sides where the weight of the pipe and soil overburden adds more pressure. In future installations a cold flow coating could be thicker. All eight excavations showed numerous rock indentations in the coating and subsequent corrosion of the pipe itself where the coating had actually been breached by rock penetration. The extent of the resulting corrosion pitting that was observed varied widely from negligible to very severe where the pipe wall was perforated and leaking.

In those areas where the coating had not been damaged or breached, it was in good condition as was the underlying pipe. It had excellent adhesion to the pipe surface such that a hammer was required to chip the coating off to inspect the pipe. Although the material had become somewhat brittle with age, there was still a slight solvent odor remaining which often indicates the coating is still alive.

As previously mentioned, the pipe was in good condition except at flaws in the coating where corrosion damage varied significantly from one location to another. Unfortunately, there are a great many such flaws caused by rock pressure. Holes in the pipe were discovered and repaired at three locations among these eight excavations and an additional leak near the water treatment plant was repaired by the City in December 1995. Other leaks are suspected to exist at several other locations where the growth of surface vegetation suggests a nearby water source below.

In addition, it is believed that a large number of leaks (10 to 50 and possibly more) may exist but have been undetected due to the high water table and very low operating pressure of the line. In these areas, pipeline leakage moves into the groundwater flow rather than rising to the ground surface and is virtually undetectable. Further, many more areas likely exist where there is little remaining metal wall thickness and future leakage is imminent.

The soils observed in the excavations were mostly brown and gray clayey sands from the surface to a depth of three to five feet with layers of sands, gravels, and shales intermixed with the clayey sands at greater depths. Groundwater was observed in all excavations and was typically above the top of the pipe in those locations west of the midpoint of the line.

Soil samples were obtained from near the springline of the pipe from each excavation. These samples were lab analyzed for pH, oxidation-reduction potential (ORP), chloride, sulfide, resistivity (saturated), and moisture content (refer to Technical Appendix D for the soil sample data). These tests were conducted to determine if any special or unusual soil chemistry characteristics were affecting external corrosion of the pipeline.
The pH determines whether the soil is acidic or alkaline. Generally, acidic soils (pH < 7.0) are of greater concern as acids promote higher corrosion rates on buried ferrous metals. The data shows the pH of all samples to be slightly alkaline and non-corrosive.

The ORP of a soil is significant as it is a measure of whether the soil is anaerobic and can support the existence of sulfate reducing bacteria (SRB). Certain types of these bacteria secrete a type of sulfur based acid that attacks ferrous metals such as carbon steel and ductile iron. The ORP combined with sulfide tests, moisture contents, and soil classification data provides an indicator of whether SRB's may be a problem. An ORP greater than 100 mv indicates aerobic conditions. An ORP between 0 and 100 mv may or may not indicate anaerobic conditions and a negative ORP indicates anaerobic conditions. The data shows aerobic conditions at all locations except one which is believed to be an error in the test data or a contaminated sample. Given this information, it appears SRB's are not a problem on the 36 inch pipeline.

Sulfide testing also provides an indicator regarding the presence of SRB's. Only trace amounts were detected and combined with the ORP data suggests that SRB corrosion is not occurring.

Chloride contents above 150 ppm combined with moisture contents above 12% to 15% are often associated with corrosive soils. Chloride ions react easily with ferrous metal ions in a suitably wet environment which results in accelerated corrosion rates. No significant amounts of chlorides were present in any of the samples.

Soil resistivity tests were conducted to provide data about the soil adjacent to the pipe versus the previously discussed surface resistivities which measure the average soil resistivity from pipe depth to surface. Resistivities at both in-situ and saturated conditions were compared to evaluate the effect of soil moisture. For the 36 inch line, the soil sample resistivities were much lower (more corrosive) than the surface resistivities, and because the soil moisture contents were so high, are more likely to represent the actual environment surrounding the steel pipe.

It is possible the surface resistivity data may have been skewed by the top 5 feet of dry soil that exists all along the pipe alignment.

One final test was conducted on the 36 inch line. A cathodic protection current requirement test was performed to estimate how much current might be required to protect the line and to provide further indications as to the overall effectiveness of the existing external coating. For this industry standard test, a portable test rectifier was connected to a nearby existing buried metal culvert near the midpoint of the line to provide temporary cathodic protection current to the pipe.

It should be noted that this test requires electrical continuity of the pipe joints to produce fully accurate results. Pipe joint electrical continuity is assured at all welded joints which were the type of joint observed in the eight excavations. However, the Level I study
indicated “Dresser” type mechanical couplings were installed at approximately 500 foot intervals during pipe construction. These mechanical couplings do not assure electrical continuity since they employ a thin rubber boot between the coupling and the pipe to provide a water tight seal. Frequently though, electrical continuity does exist across the coupling as a small misalignment between the pipe and coupling causes a physical contact between the follower rings on the coupling and the pipe. This metal-to-metal contact creates an electrically continuous path as does the commonly found rust buildup between the coupling and the pipe. These paths usually have a higher electrical resistance than what is provided by a welded pipe joint and cannot be relied upon over time to maintain continuity due to pipe movement or earth settlement.

Dresser couplings were not observed in any of the eight excavations but the impact of their possible presence at other locations was considered during the evaluation of the test results.

Table 2-3 below shows the test results.

**Table 2-3**
*Cathodic Protection Current Requirement*
*36 Inch Steel Raw Water Line*

Rectifier Output: 11.5V @ 7.5A DC

<table>
<thead>
<tr>
<th>Test Location</th>
<th>Current On</th>
<th>Current Off</th>
</tr>
</thead>
<tbody>
<tr>
<td>TS 1 (East end)</td>
<td>-0.653</td>
<td>-0.636</td>
</tr>
<tr>
<td>TS 2</td>
<td>-0.652</td>
<td>-0.630</td>
</tr>
<tr>
<td>TS 3</td>
<td>-0.735</td>
<td>-0.669</td>
</tr>
<tr>
<td>TS 4</td>
<td>-0.823</td>
<td>-0.668</td>
</tr>
<tr>
<td>TS 5 (rect. location)</td>
<td>-1.087</td>
<td>-0.721</td>
</tr>
<tr>
<td>TS 6</td>
<td>-0.763</td>
<td>-0.699</td>
</tr>
<tr>
<td>TS 7</td>
<td>-0.721</td>
<td>-0.683</td>
</tr>
<tr>
<td>TS 8 (West End)</td>
<td>-0.687</td>
<td>-0.654</td>
</tr>
</tbody>
</table>

Rectifier Output: 22V @ 14A DC

<table>
<thead>
<tr>
<th>Test Location</th>
<th>Current On</th>
<th>Current Off</th>
</tr>
</thead>
<tbody>
<tr>
<td>TS 1 (East end)</td>
<td>-0.683</td>
<td>-0.648</td>
</tr>
<tr>
<td>TS 2</td>
<td>-0.691</td>
<td>-0.652</td>
</tr>
<tr>
<td>TS 3</td>
<td>-0.793</td>
<td>-0.693</td>
</tr>
<tr>
<td>TS 4</td>
<td>-0.898</td>
<td>-0.668</td>
</tr>
<tr>
<td>TS 5 (rect. location)</td>
<td>-1.649</td>
<td>-0.921</td>
</tr>
<tr>
<td>TS 6</td>
<td>-0.822</td>
<td>-0.705</td>
</tr>
<tr>
<td>TS 7</td>
<td>-0.773</td>
<td>-0.696</td>
</tr>
<tr>
<td>TS 8 (West End)</td>
<td>-0.753</td>
<td>-0.681</td>
</tr>
</tbody>
</table>
The pipe-to-soil (P/S) measurements reveal it was not possible to fully protect the pipe (P/S more negative than \(-0.850\) volts) at all locations given the limits of the test equipment. However, cathodic protection may be feasible using either a larger rectifier or perhaps using several smaller systems spread out along the pipe alignment.

Taking into consideration that the line is electrically grounded at the water treatment plant (East end), the data suggests that the external coating on the pipe is in relatively poor condition as the amount of current needed to protect a similarly grounded but well coated line is less than five amperes. This supports the excavation inspection observations that the external coating is breached in many locations and that significant corrosion damage has likely occurred on the pipe.

These conclusions assume the line had electrical continuity during the test despite the potential presence of the previously discussed “Dresser” couplings. This assumption is believed to be valid for two reasons. First, prior to performing the testing, a Tinker & Rasor Model PD Short Locator was used to trace the pipeline locations. This device imparts a signal onto the pipe similar to standard pipe locating equipment. However, pipe locators used a high frequency (> 5,000 cycles/sec) signal which can sometimes “jump” across electrical discontinuities in buried piping whereas the Model PD uses a low frequency (750 cycles/sec) signal which does not “jump” across electrical discontinuities. The entire pipeline alignment was traced using the Model PD which strongly indicates the pipeline had electrical continuity during the test.

The second reason that pipe joint electrical continuity was assumed to exist is the test data itself. Significant pipe-to-soil potential shifts (75 millivolts or greater) between “Current On” and “Current Off” at each test location are usually an indicator of some degree of electrical continuity in the pipeline. It should be noted that lesser sized potential shifts were observed at TS 1 and TS 2 which could be the result of “Dresser” couplings or the impact of these being located furthest away from the temporary rectifier. In either event, there is sufficient evidence to believe the majority of the pipeline had electrical continuity during the test and that the resulting conclusions are valid.

The significance of a poor external coating goes beyond the cost of more expensive cathodic protection systems. If sufficient coating disbondment (loss of adhesion between coating and pipe) has occurred at the many breaches in the coating, it may not be possible to fully control corrosion using cathodic protection. Water can migrate under broken coatings for many inches but cathodic protection current cannot. Thus corrosion underneath the coating may occur despite the use of cathodic protection.

The visual inspections of the raw water line revealed typical localized coating disbondment at the breaches in the coating but no large disbonded areas were discovered. It is not known if coating disbondment has progressed in other areas or will progress to the extent of preventing effective cathodic protection, but given the type and age of the coating material, industry experience suggests it is a potentially serious problem.
With the extent of the existing corrosion damage and poor coating effectiveness and the potential problems with coating disbondment, it may not be cost effective to salvage and rehabilitate the pipeline to extend its service life another 30 to 50 years.

**Pipeline Excavation & Testing - 24 Inch Treated Water Line**

Sixteen inspection excavations were completed on the 24 inch ductile iron water line. The excavation locations were determined from three sources of information: previous leak history, cell-to-cell potential survey, and soil resistivity data. In addition, one more location was inspected as the pipeline had been excavated by a local developer making a tap on the line.

Only two leaks had been reported on the 24 inch line. One was located near the east side of the Laramie River crossing and the other approximately 2,000 feet east of Hwy 230. The cell-to-cell potential survey, which measures direct current (DC) flowing in the earth between two closely spaced reference electrodes was conducted over the pipeline along its entire alignment from HWY 230 to the 8 MG storage reservoir. This survey attempts to locate areas of high current flow in the soil that might be an indicator of corrosion activity on the pipeline. It is the only surface electrical survey method that can be used on uncoated, electrically discontinuous pipe (i.e., the pipe joints are not electrically continuous) such as the unbonded, rubber gasketed ductile iron pipe of the 24 inch line. It is, unfortunately, difficult to interpret the survey data as DC in the soil can be caused by sources other than corrosion of the subject pipe. Other buried metal objects like culverts, cans, fence posts, etc. can exhibit characteristics that will appear similar to corrosion on the pipeline and stray current caused by cathodic protection systems in the vicinity. In addition to this limitation, the data can be affected by the junction potential between the reference cells and the soil. A potential of several millivolts can often be generated between the reference cells and the soil in which they are placed. This is most often a problem when the soil surface is dry. Water was carried along to wet the surface to reduce this junction potential problem. Refer to Technical Appendix E for the Cell-to-Cell Potential Survey data.

The Cell-To-Cell Potential Survey revealed a number of locations of potential corrosion activity. These locations along with the surface soil resistivity data provided the basis for the excavation sites.

The inspections, like those previously performed on the 36 inch steel line, consisted of visual observations of the external condition of the pipe, measurement of corrosion pit depths, soil classification, groundwater depth, depth of pipe burial, and collection of one soil sample from each site for laboratory analysis. Refer to Technical Appendix F for individual pipeline inspection report forms, Technical Appendix G for photographs of typical pipe condition, and Technical Appendix H for soil sample results.

The pipeline inspections revealed a wide range of external corrosion activity from negligible to severe. In certain localized areas the pipe had experienced significant corrosion while in others, the pipe appeared to be brand new. For example, the pipe was experiencing serious...
corrosion attack approximately 2,000 feet east of Hwy 230 (Western Segment - Dig No.1) with numerous pits exceeding 0.10 inches deep and several deeper than 0.15 inches yet was in excellent condition 2,000 feet further East (Western Segment - Dig No.2).

Overall, the 24 inch pipe appeared to be in good condition. The majority of inspections revealed only minor external corrosion activity. The western segment of the line, which runs from HWY 230 to the Union Pacific railroad tracks and includes the Laramie River crossing, had evidence of random, localized pitting attack which in some cases exceeded 0.25 inches deep but this occurred in only a few excavations.

In fact, it appears that in many areas the composition of the surrounding soil has served to provide a partial barrier to corrosion. In virtually all of the excavations east of the Laramie River, a very hard, black cement like coating was bonded to the outside of the pipe. Significant effort with a hammer was required to chip and dislodge this material. Lab inspection and analysis revealed it to be a calcareous cemented sand which bonded to the pipe surface. It is believed that the native clay and sandy soils contain dissolved calcareous minerals that became cemented to the pipe after repeated groundwater level fluctuations (wet/dry cycles).

Unfortunately, as with all coatings, it is not a perfect barrier and corrosion can occur at breaks or flaws. This was observed in several excavations where there were breaches due to rocks or other natural causes.

Soil samples were obtained from near the springline of the pipe from each excavation (see Technical Appendix H). Except for a few locations, the soil chemistry characteristics do not indicate unusually aggressive soil conditions. The resistivity data does suggest a potentially corrosive environment may exist in certain areas.

Based on all the test and excavation inspection information collected, it is believed the overall external integrity of the 24 inch line is satisfactory. The majority of the line can be expected to provide an additional 20 to 30 years of service life. Two areas where leaks have already occurred are located in aggressive soils and the pipe condition warrants replacement. A manageable amount of corrosion leakage (estimated at 1 to 3 leaks/year) on the remainder of the line can be expected to occur in the future as random localized pitting eventually causes failures. The worst areas for likely corrosion problems have been identified as follows:

1. From the east side of the mobile home park located east of the valve vault at HWY 230 (Station 977+00) to a point approximately 1,000 feet further east of the mobile home park.

2. From the west side of the Laramie River east to approximately 2nd & Sanders Street.

3. From a point approximately 250 feet west of Adams Street to a point approximately 1,750 feet west of Adams Street.
4. From a point midway between 8th & Springcreek and 9th & Russell to a point midway between 10th & Russell and 11th & Russell (approximately 1,000 feet).

5. From a point approximately 50 feet east of 15th & Spring Creek to the east side of the intersection of Park Ave. & Spring Creek (approximately 2,250 feet).

Refer to Task 6.2.2 (Recommended Capital Improvements) for a vicinity map which incorporates the above five areas.

**Pipeline Excavation and Testing - 20 Inch Cast Iron Line**

This 20 inch treated water cast iron pipeline runs under street pavement from a point approximately 150 feet west of the intersection of Park & Pine Streets to a point just north of the intersection of Custer & Pine Streets where it ties into the distribution system piping.

Because of its relatively short length and the higher cost to excavate and repair the pavement, only two excavations were completed. Refer to Technical Appendix I for individual pipeline inspection report forms and Technical Appendix J for soil sample results. The pipe excavations revealed no corrosion problems and the soil sample data does not indicate particularly aggressive soil. No leaks have been reported by the City on this section of pipe and an excavation inspection performed during the Level I study revealed no evidence of external corrosion. This line can be expected to provide 20 to 30 years of additional service life although an occasional localized leak may occur in the future.

**Internal Pipeline Inspection - 36 Inch Steel Raw Water Line**

To fully evaluate the integrity of the 36 inch line, an internal inspection using a video camera was conducted. Four inspection access ports were cut into the pipe to allow camera insertion. The camera, commonly used for sewer line inspection, was capable of tracking approximately 1,500 feet in each direction from each access port. The access ports were spaced so that the entire line could be inspected. The excavations for three of these access ports were combined with the external inspection excavation locations to optimize cost (refer to Technical Appendix K for video inspection report forms).

V.S.R. Corporation of Lafayette, Colorado provided the camera and support services and performed the inspections on September 27, 1995 through September 29, 1995.

In general, the interior of the pipeline appears to be in good condition. The majority of the coal tar enamel lining is intact with many small areas of delamination particularly adjacent to the welded pipe joints where heat from the welding burned the lining. A relatively thin scale of corrosion product covered the exposed steel areas but very little tuberculation was observed. It was not possible to determine the extent or depth of corrosion pitting, if any, beneath the surface scale deposit.
The pipe was manually inspected within a few feet of each of the access ports. There were several small areas of approximately one inch in size where the lining was not intact at two of the ports. The surface rust scale was scraped away which revealed little significant corrosion damage and no pitting was observed. The coal tar lining was approximately 100 to 150 mils thick and was in excellent condition. It was still very pliable and had a strong solvent odor.

Nearly all of the pipeline was traversed with the camera. The only portion not completed was approximately the first 500 feet east from the intake structure at Sodergreen Reservoir. The camera was in the pipe approximately 1,050 feet traveling west from Access Port #3 toward the intake structure when it encountered some debris that it could not pass. An attempt had previously been made to lower the camera into the pipe through the intake structure for travel eastward but this also was unsuccessful as debris blocked the path.

It should be noted the camera went under water at various locations for a total distance of approximately 1,200 feet indicating low spots in the line. Several of these low spots exceeded 300 feet in length. The camera was unable to provide a discernible video quality while its lens was underwater.

Based on the video camera inspection results and the manual inspections completed at each of the various access ports, it is believed the interior of the 36 inch line is in good condition and its integrity is not threatened by significant internal corrosion.

**Internal Pipeline Inspection - 24 Inch DIP Treated Water Line**

As with the 36 inch line, to better evaluate the integrity of the 24 inch treated water line, an internal inspection using a video camera was conducted. Initially, two access ports were planned each of which would allow inspection of approximately 3,000 feet (1,500 feet each direction) of pipe for a total coverage of 6,000 feet.

After completing the inspection of the first section of pipe and manually inspecting a piece of the 24 inch line that had been previously removed by the City (for other operational reasons) near the planned site for the second inspection section, it was decided that internally inspecting the second section of pipe was not necessary. The excavation for the access port for the first section was combined with the external inspection excavation locations to optimize cost.

V.S.R. Corporation of Lafayette, Colorado provided the camera and support services and performed the inspection on September 20, 1995.

In general, the interior of the pipeline appeared to be in excellent condition. The majority of the cement mortar lining was intact although there were numerous small areas of delamination or carbonation particularly adjacent to the pipe joints. A relatively thin scale of corrosion product covered the exposed pipe areas but very little tuberculation was observed. It was not possible to determine the extent or depth of corrosion pitting, if any,
beneath the surface scale deposit. The pipe joints were not mortared but showed no evidence of significant corrosion activity.

The pipe was manually inspected within a few feet of the access port. There were several small areas of approximately one-half inch to one inch in size where the mortar lining was not intact. The surface rust scale was scraped away which revealed little significant corrosion damage and no pitting was observed. The cement mortar lining was approximately 0.125 inches thick and appeared to be in sound condition with strong adhesion to the pipe wall.

A piece of the 24 inch pipe approximately 10 feet long that had previously been removed by the City was available for manual inspection. This was completed on September 29, 1995 at the field yard of Bird & O'Donnell Construction, Inc. where the pipe segment had been stored outside. The cement mortar lining was in excellent condition even after being exposed to outdoor weather for months and its condition is believed to be typical of the lining in the remainder of the line.

Based on the evidence collected during the internal inspection work, internal corrosion does not appear to be a threat to the integrity of the 24 inch DIP treated water pipeline.

2.2.4 CONCLUSIONS

36 Inch Steel Raw Water Pipeline

This 8,500 foot long pipeline from Sodergreen Reservoir to the Water Treatment Plant has experienced six external corrosion leaks between the Fall of 1993 and December 1995 and other existing leakage is suspected. The external coating was observed to be breached in many locations upon excavation inspection due to rock penetration. Corrosion damage in these damaged coating areas was severe. Cathodic protection current requirement testing also suggests the coating effectiveness is poor and that mitigation of further corrosion using cathodic protection may be feasible but difficult particularly if coating disbondment has occurred.

Soil tests indicate the pipe environment is not unusually aggressive which supports the fact that corrosion leakage has been low for a 48 year old pipeline. However, even in these soils, unprotected steel piping will, if given enough time, ultimately develop leaks.

The internal integrity of this pipeline appears to be intact. While a few localized areas may have more damage than was revealed during the video inspection, there does not appear to be a significant problem.

In summary, the 36 inch raw water pipeline has been severely damaged by external corrosion. It is believed that numerous holes (10 to 50) already exist but have been undetected due to the high water table and low operating pressure of the line. It can be expected that corrosion leakage will increase at a rapidly increasing rate if left unmitigated.
This line should be considered for replacement or extensive rehabilitation.

24 Inch DIP Treated Water Line

This line consists of approximately 4.9 miles of 24 inch ductile iron pipe from HWY 230 across town to the 8 MG storage reservoir. Only two corrosion leaks have been reported from the installation date in 1964 through December 1995.

Although this line was installed without an external coating, corrosion appears to have been controlled partly due to the formation of a calcareous cement like deposit from the surrounding soils that has bonded itself to the pipe and formed a barrier. This "coating" material occurs most predominantly east of the Laramie River.

External corrosion pitting, severe in certain small localized spots, was observed in several excavations where the "coating" was breached but it is the exception rather than the rule.

Soil conditions are not unusually corrosive but certain "hot spot" areas are likely to develop greater corrosion rates.

Internal corrosion has not been detected in any inspections to date and does not appear to be a threat to the structural integrity of the pipe.

Based on all information available to date, it appears that external corrosion problems on this line are not severe enough to warrant replacement of the entire line and that the majority of it can be expected to provide an additional service life of 20 to 30 years. Replacement of two segments of pipe that have already experienced leakage may be warranted. Corrosion leakage for the remainder of the line can be expected to occur in localized areas each year (estimated 1 to 3 leaks/year). Because the leak rate is expected to be manageable, this line is a good candidate for "hot spot" cathodic protection wherein sacrificial anodes are installed in specific locations to provide very localized corrosion protection.

2.3 APPURTENANCES

Three main pipeline appurtenances were inspected for deterioration: the inlet structure on the 36 inch raw water line at the outlet of Sodergreen Reservoir; the valve vault on the 24 inch line adjacent to HWY 230 (sta 977+00); and the vaults at each end of the tunnel under I-80 which contain the 24 inch treated water line and a 20 inch water line.

The 36 inch raw water line inlet structure is a reinforced concrete structure approximately 7 feet wide by 12 feet long by 20 feet deep with gear operated sliding metal gates designed to control water flow into the pipeline. Due to the fact that this structure is in operation at all times and cannot be taken out of service for routine inspection, it was not possible to complete a full corrosion inspection. However, the steel components that were visible during shut down of the 36 inch line for the internal video inspection did not show evidence of significant corrosion attack or deterioration.
The valve vault on the 24 inch DIP treated water line located on the east side of HWY 230 (Sta 977+00) commonly referred to as the “Roach Valve” is a small reinforced concrete structure containing a mainline isolation valve. A visual inspection was performed on December 14, 1995. The vault floor was wet but there was no standing water. The piping and valve exteriors were in good condition with only surface rusting. The concrete structure was also in good condition and not in need of any specific rehabilitation efforts.

The vaults on each end of the tunnel beneath I-80 contain both the 24 inch DIP treated water line and a parallel 20 inch steel water line. A visual inspection was performed on December 14, 1995. There was several inches of standing water on the floor of each vault. The piping and piping support exteriors were in good condition with only surface rusting. The concrete structures were also in good condition and not in need of any specific rehabilitation efforts. No inspection was performed on the tunnel wall nor on the interior pipe support saddles.

2.3.1 CONCLUSIONS

No significant deterioration was observed on the following three main pipeline appurtenances: the inlet structure on the 36 inch raw water line at the outlet of Sodergreen Reservoir; the valve vault on the 24 inch line adjacent to HWY 230 (sta 977+00); and the vaults at each end of the tunnel under I-80 which contain the 24 inch treated water line and a 20 inch steel water line.
3.0 SYSTEM OPERATIONAL ANALYSIS

3.1 EXISTING CITY DISTRIBUTION AND STORAGE SYSTEM

3.1.1 WATER SUPPLY

The City of Laramie receives its water supply from both surface and groundwater sources, with surface water providing approximately 38% of the total supply, and groundwater sources providing the remaining 62%. Surface water from the Laramie River is diverted to Sodergreen Lake via the Pioneer Canal. An intake structure at Sodergreen Lake delivers water into a 36-inch steel pipeline, which in turn brings the water to the WTP, located approximately 22 miles west of Laramie. During the summer months, treated water flows from the WTP through two steel transmission lines, a 20-inch pipeline and a 24-inch pipeline, to the City’s distribution system. During the colder months, when system demands for water are reduced, the WTP has only been run to serve users between the plant and the City. The 20-inch pipeline terminates at its connection to the distribution system near Optimist Park on the west side of Laramie, while the 24-inch line continues along Spring Creek Drive to its terminal connection to the 8.0 million gallon (MG) Lo-Level storage reservoir on the east side of the City.

The City’s groundwater sources include three well fields; the Turner Well Field, the Pope Well Field, and Soldier Springs. The Turner Wells deliver water directly into the Lo-Level storage reservoir. Water from the Pope Wells and Soldier Springs, which are located south of the City, enter the Zone 1 distribution system through a 16-inch pipeline. The water from the Pope Wells requires pumping, whereas water from Soldier Springs flows by gravity under typical conditions. However, when the high capacity wells in the Pope Well field are on line, the water from Soldier Springs requires pumping to overcome back pressure in the pipeline. An additional groundwater source, the Spur Well Field, is expected to be added to Zone 2 in 1996. All water sources presently enter Zone 1. A detailed evaluation of the City’s existing and proposed groundwater sources is included in the City of Laramie Water Supply Master Plan, Level I (Western Water Consultants, January, 1995).

3.1.2 PRESSURE ZONES

The City of Laramie’s distribution system is divided into five pressure zones. See Figure 3-1, in the back pocket of this report. Zone 1 is the lowest zone in terms of elevation, and includes West Laramie and most of the older parts of the City. Zone 2 includes the University of Wyoming, and all areas bounded by 30th Street to the east and 9th, 11th and 13th Streets to the west. The division of Zones 1 and 2 is effected by the closure of isolation valves between the two zones. Zones 3 through 5 are much smaller, and are located above Zone 2. Zone 3 is defined as the Alta Vista subdivision, Zone 4 is defined as the Indian Hills subdivision, and Zone 5 is defined as the Imperial Heights subdivision. In terms of elevations served, the pressure zones are defined as follows:
Zones 2 through 5 are served by a system of pump stations which pump water to progressively higher elevations. The pump station with the largest capacity is the Hi-Lo Pump Station, which actually houses two batteries of pumps to serve two different pressure zones. The Lo-Level pumps pump water from Zone 1 to the Zone 2 distribution system, which is served by the Hi-Level storage tanks. The Hi-Level pumps pump from Zone 2 to Zones 3, 4 and 5. Figure 3-2 is a schematic representation of the Hi-Lo Pump Station piping, and Figure 3-3 is a diagram showing the overall water system configuration. In addition to the Hi-Lo Pump Station, the City operates three other pump stations. The Alta Vista Pump Station, located near the intersection of Willet and 30th Street, pumps water to the Alta Vista subdivision (Zone 3) and Indian Hills subdivision (Zone 4) during periods of high demand. The Indian Hills Pump Station, located on Grays Gable Road just east of the Alta Vista subdivision, acts to boost pressures in the Indian Hills subdivision. The Imperial Heights Pump Station, located east of the Hi-Lo Pump Station on Highway 30, boosts pressures in the Imperial Heights subdivision. The entire water distribution system is designed to lift water in a ladder-like progression from Zone 1 to Zone 2, and thence to the other zones. Under the current system, during periods of high demand, water delivered to customers in the Indian Hills subdivision may pass through four different pump stations on its way to the point of demand. Though this is a rare occurrence, water may be pumped to Zone 2 through the Lo-Side pumps, then to Zone 3 through the Hi-Side pumps. Under certain conditions, water destined for Zone 4 may also pass through the Alta Vista Pump Station. The Alta Vista Pump Station acts to boost system pressures during periods of moderate to high demand. A detailed description of the equipment and operation of the pump stations is included in the City of Laramie Water Supply Master Plan, Level I. However, further evaluation of and recommendations for the pump stations are included later in this report.

In addition to the five interconnected pressure zones which form the City’s distribution system, there is a separate zone which encompasses the Laramie Regional Airport. The airport is served by a 10-inch pipeline approximately 11,000 feet in length, and pressure is supplied by a booster pump station which houses two pumps.

### 3.1.3 STORAGE RESERVOIRS

The City of Laramie currently maintains three storage reservoirs, all located on the east edge of the City. The Lo-Level reservoir, which serves Zone 1, has a capacity of 8.0 MG, with
TO REYNOLDS STREET (ZONE 2)

1.5 MG RESERVOIR

2.0 MG RESERVOIR

TO LEWIS AVENUE (ZONE 2)

TO GRAND AVENUE (ZONE 2)

TO WTP AND ZONE 1

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FLO
CITY OF LARAMIE
EXISTING WATER SUPPLY SYSTEM CONFIGURATION

ZONE 4
MAX DAY DEMAND (1995) = 0.32 MGD (2.0%)

ZONE 5
MAX DAY DEMAND (1995) = 0.13 MGD (0.8%)

ZONE 3
MAX DAY DEMAND (1995) = 0.68 MGD (4.2%)

ZONE 2
MAX DAY DEMAND (1995) = 8.5 MGD (53%)

ZONE 1
MAX DAY DEMAND (1995) = 6.4 MGD (40%)

LOW SIDE PUMPS

HIGH SIDE PUMPS AND ALTA VISTA PUMPS

LOW LEVEL RESERVOIR (8.0 MG)

HIGH LEVEL RESERVOIRS (3.5 MG)

POPE WELLS AND SOLDIER SPRINGS (5.4 MGD)

LARAMIE WATER TREATMENT PLANT (7.0 MGD)

INDIAN HILLS PUMPS

IMPERIAL HEIGHTS PUMPS

FIGURE 3-3
an overflow elevation of 7260 feet. There are two water storage tanks, the Hi-Level tanks, serving Zone 2; a 1.5 MG tank and a 2.0 MG tank. The 1.5 MG tank has an overflow elevation of 7350 feet, and the 2.0 MG tank has an overflow elevation of 7352 feet. There is no existing storage for Zones 3, 4 and 5, and consequently all system demands and fire flows must be pumped into these zones.

3.1.4 DISTRIBUTION SYSTEM PIPING

The City’s distribution system is composed primarily of cast iron and ductile iron pipe, with PVC pipe seeing greater use in recent years. However, some of the oldest water mains were constructed of steel. The vast majority of distribution lines in the City are 6 inches in diameter or larger, and are supplied by an extensive system of transmission mains. The City’s primary transmission mains are a 12-inch line in Reynolds, east of 15th Street; a 14-inch line in Lewis, which becomes a 16-inch line east of 15th Street; and a system of two mains in Grand Avenue, which range in size from 12 inches to 16 inches.

3.1.5 TOPOGRAPHY

The predominate characteristic of the Laramie system is the flat topography. The water treatment plant is located approximately 20 miles west of Laramie, at an elevation of 7345 feet. The lowest elevation found within the City is approximately 7130 feet along the Laramie River, a difference of 215 feet (which is equivalent to approximately 93 psi of static head). This extremely flat grade limits the amount of flow that the 20-inch and 24-inch pipelines can carry from the water treatment plant to the City to approximately 7.5 MGD.

3.1.6 CONCLUSIONS

The City of Laramie’s demands for water are supplied by both surface and groundwater sources, with approximately 62% coming from its water wells and the rest from its water treatment plant. The distribution system is divided into five pressure zones, with the majority of the population currently living in Zones 1 and 2. All pressure zones, with the exception of Zone 1, are served by pump stations. There are five pump stations; the Hi and Lo sides of the Hi-Lo Pump Station, the Alta Vista Pump Station, the Indian Hills Pump Station, and the Imperial Heights Pump Station. The pump stations are staged so that water is lifted from zone to zone until it reaches the point of demand. The distribution system is served by three storage reservoirs, with a total capacity of 11.5 million gallons.

3.2 FUTURE SERVICE PLAN

Section 3 of the City of Laramie Water Supply Master Plan, Level I, is an analysis of the anticipated future water supply needs of the City, and includes projections of future growth in and around the City, along with mapping information showing the anticipated locations of growth. According to the Level I study, much of the anticipated growth will occur to the north and south of Zone 2. Significant growth is also expected on the east side, as well as in West Laramie. The area expected to experience the least growth is the portion of Zone
1 located east of the Laramie River, which includes most of the older portions of the City. Figure 3-4 is a map of the City’s distribution system showing the magnitudes and locations of growth, based on the planning numbers developed in the Level I study.

The planning information presented in the Level I study was used to determine the anticipated demands on the water transmission, distribution and storage (TD&S) system over a 50-year period extending to the year 2045. Demands were calculated 25 years into the future to aid the City of Laramie in planning capital improvements. This planning information was used to evaluate the effects of future development on the existing system, which were modeled in order to identify deficiencies in the TD&S system. The TD&S system must be capable of meeting Peak Hour plus fire flow demands.

Based on the Level I projections, the following parameters were used in this Level II report:

- **Peak Day Demand (1995)**: 16.1 MGD (million gallons per day)
- **Peak Day Demand (2020)**: 20.9 MGD
- **Peak Day Demand (2045)**: 25.3 MGD
- **Peak Hour Demand (1995)**: 22.8 MGD
- **Peak Hour Demand (2020)**: 29.4 MGD
- **Peak Hour Demand (2045)**: 35.7 MGD
- **Peak Day Demand Rate**: 554 gpcd (gallons per capita per day)
- **Peak Hour Demand Rate**: 781 gpcd

One of the essential elements of any water distribution system is its ability to provide fire flows to all hydrants within the system. The Uniform Fire Code and Laramie Fire Department officials were consulted to determine the required fire flows for the various building zones. Three different rates of fire flow were identified, depending on the building zone, and are given below:

<table>
<thead>
<tr>
<th>Fire Flow Rate</th>
<th>Building Zone</th>
</tr>
</thead>
<tbody>
<tr>
<td>1,500 gpm for 2 hours</td>
<td>R1, R2, R2M, LR, RR</td>
</tr>
<tr>
<td>3,000 gpm for 3 hours</td>
<td>B1, B1R, O, R3, LM</td>
</tr>
<tr>
<td>4,000 gpm for 4 hours</td>
<td>I1, I2, IP, C2, B2</td>
</tr>
</tbody>
</table>

All fire flow rates given above must be delivered to the hydrant or hydrants at a minimum pressure of 20 psi. The Uniform Fire Code allows multiple hydrants to be used to deliver the mandated fire flow to a site. The fire flow rates given above are considered minimum flows for each zone. For purposes of determining the amount of water storage required for each pressure zone, the maximum fire flow rate of 4,000 gpm for 4 hours was used. Zones 1 and 2 have several areas zoned for the maximum fire flow. Zone 3 has some existing commercial development requiring the maximum fire flow, along with other undeveloped areas zoned for industrial and commercial development. It is anticipated that as development
CITY OF LARAMIE
PROJECTED DEVELOPMENT BY 2045

TO WATER TREATMENT PLANT

ZONE 1

ZONE 2

ZONE 3

ZONE 4

ZONE 5

770 ADD'L RESIDENTS

100 ADD'L RESIDENTS

1,470 ADD'L RESIDENTS

1,500 ADD'L RESIDENTS

1,135 ADD'L RESIDENTS

425 ADD'L RESIDENTS

1,005 ADD'L RESIDENTS

1,400 ADD'L RESIDENTS

775 ADD'L RESIDENTS

THESE PIPES NOT TO SCALE

1,300 ADD'L RESIDENTS

CORTHELL HILL

1,200 ADD'L RESIDENTS

TO POPE WELL FIELD AND SOLDIER SPRINGS

1,195 ADD'L RESIDENTS

975 ADD'L RESIDENTS

250 ADD'L RESIDENTS

865 ADD'L RESIDENTS

770 ADD'L RESIDENTS

590 ADD'L RESIDENTS

770 ADD'L RESIDENTS

1,470 ADD'L RESIDENTS

1,500 ADD'L RESIDENTS

ZONE 2

ZONE 3

ZONE 4

ZONE 5

FIGURE 3-4

Engineering Consultants
occurs on the east side of the City, it will include commercial and retail development
typically associated with new growth, again requiring the maximum fire flow rate.

3.2.1 CONCLUSIONS

Much of the anticipated growth in the Laramie area is expected to be around West Laramie,
to the north and south of Zone 2, and on the east side. Using information contained in the
Level I study, demands were calculated for 25 years and 50 years in the future. However,
only the 25-years demands were used to model recommended improvements to the water
distribution system per direction of the City. In addition, the system’s ability to deliver fire
flows was examined, based on the requirements given in the Uniform Fire Code.

3.3 HYDRAULIC MODEL

3.3.1 CYBERNET HYDRAULIC MODELING SOFTWARE

Our contract with the Wyoming Water Development Commission called for use of the City’s
existing KYPIPE model to investigate the operation of the water TD&S system. However,
for the reasons discussed in the following paragraph, it became clear that the existing model
could not be relied upon to give accurate results. Moreover, in our opinion the KYPIPE
model was not capable of performing the kind of sophisticated analysis which was required,
and the City would be ill-served by using the existing model for this analysis. A decision
was made to construct a new model, using only that information from the KYPIPE model
that could be independently verified. Using Cybernet network analysis software, a complete
integrated model of the City’s TD&S system was constructed. Cybernet is an AutoCAD­
based network analysis program. Its numerical analysis “engine” uses the same algorithms
as KYPIPE. However, Cybernet has numerous features not found in KYPIPE, including
sophisticated graphical capabilities such as the ability to produce pressure contour maps, fire
flow maps, and the means to identify overburdened links in the distribution system.
Although the distribution system was modeled with CYBERNET, RBD, Inc. will deliver
KYPIPE files to WWDC upon completion of the project.

The City supplied data input files and documentation for a hydraulic model that had
previously been written using the University of Kentucky’s KYPIPE program. However, the
existing model was actually split into two modules; one for Zone 1, and one for Zones 2
through 5. This meant that the flows between Zones 1 and 2 (through the Hi-Lo Pump
Station) were not modeled, and had to be assumed or calculated by other means. When
constructing the Cybernet model, information on pipe lengths and diameters was taken from
the input files for the KYPIPE model and cross-checked against the City’s system maps.
Information on system demands was also obtained from the KYPIPE model. In the course
of verifying the information from the KYPIPE model, numerous errors were discovered,
particularly in the way that demands were modeled. For example, the KYPIPE model
contained no demands for the University of Wyoming campus, which in actuality uses a
significant amount of water. It was further found that future demands were poorly modeled,
and did not reflect the growth patterns laid out in the Level I study. Seven different input
files, modeling several different demand patterns, were provided by the City. However, there was no documentation indicating what scenarios the various input files were intended to model, and the author of the KYPipe model was unable to provide this information. Another problem in the existing KYPipe model was the method by which it was calibrated. By standard engineering practice, a model is calibrated by comparing results of hydrant tests with the available fire flows generated by the model, and then adjusting the friction factor of the pipes in the model until a good correlation between the model results and actual hydrant tests is obtained. In the KYPipe model supplied by the City, the characteristics of each hydrant in the model were adjusted on a case by case basis to try and match the test results. The result of this method is that select hydrants in the model were calibrated, rather than the model as a whole. Because each of the hydrants calibrated in the model had a different head loss factor, two hydrants might be of identical manufacture and age, and yet be modeled as having radically different hydraulic characteristics. This meant that the model could not reliably predict fire flows at hydrants that had not been calibrated by this method, such as new hydrants added to the system. In essence, the model couldn't predict fire flows for a hydrant unless the hydrant had been flow tested, and the results entered into the KYPipe model. Another significant deficiency of the existing KYPipe model was that no elevations had been included in the input data files. Without this information, system pressures cannot be modeled because the available pressure at any point in the system is dependent, in part, on the elevation of that point. Using two-foot interval topographic contour maps provided by the City, the elevations of each node was obtained and then entered into the Cybernet model.

A master model of the entire TD&S system was constructed, with demand patterns input into the model to simulate Peak Day and Peak Hour conditions for the current TD&S system, and the system at buildout. Demands were obtained by analysis of the KYPipe data files, and from billing records provided by the City. Calibration of the model was performed by selecting from the City's hydrant test records a set of hydrant flow tests which met specific criteria. These criteria were that hydrants chosen were all tested by the Laramie Fire Department (LFD) within the last seven years, were all manufactured by Mueller, and were all served by a water main at least ten inches in diameter. Both static pressures and pressures measured during the flow tests were compared to the pressures predicted by the hydraulic model. No information was available to indicate what demands the system was experiencing during any of the flow tests. Therefore, calibration of the model was performed under existing Peak Day demands. This yielded fire flows from the model that were generally less than those measured by the LFD, but sufficiently representative for purposes of statistical comparison. It was found that there was significant variability between hydrants in terms of the pressure losses at similar flow rates. This is due to two things: 1) the pressure loss through a hydrant varies considerably with age (the design of the foot valves used in the bases of hydrants has changed over the years); and 2) hydrant manufacturers only publish pressure-loss curves for flows up to 1,000 gpm, and their behavior at higher flows is not documented.
3.3.2 PARAMETERS AND ASSUMPTIONS USED

In general, we minimized the number of assumptions used in this analysis by making use of the extensive information provided by the City. The City of Laramie employs a sophisticated SCADA (supervisory control and data acquisition) system to perform real-time monitoring of tanks, pumps, pipe flows, treatment plant flows, and pressures within the system. The City provided AutoCAD-based maps of the water system, along with quarter-section system maps based on aerial photography performed in 1985. Through our meetings with Laramie Water Department staff, the functioning of the entire TD&S system was described, along with the operational regime of the pump stations. The City provided billing records for the University of Wyoming so that demands within the campus could be accurately added to the system and modeled. The input files from the KYPIPE model were used to assign demands on a geographical basis, and the magnitudes of those demands were analyzed and adjusted to more accurately represent total demands within the system. Discussions with Laramie Water Department staff provided direction on the modeling of the pump stations. The operation of the pump stations is centrally controlled using a complex program. The City identified the underlying scheme of the pump stations’ operation, which allowed us to model the pumps under specific conditions. As explained earlier in this report, the water system is modeled under stressed conditions (peak demands, fire flows, etc.) in order to identify weaknesses in the system. Therefore the pumps were modeled under those specific scenarios.

Numerous iterations of the Cybernet hydraulic model were run under a variety of scenarios. Peak Day and Peak Hour scenarios were run for the existing system using current demands estimated from information contained in the Level I study. Available fire flows were also calculated under existing Peak Hour conditions, which represents one worst-case scenario. The fire flows generated under these conditions are generally less than what is indicated in the City’s hydrant test records. The reason for this discrepancy is that the actual hydrant tests were conducted over a period of many years, by various personnel with varying levels of training, performed at different times of day and of the year. It is highly unlikely that more than a small percentage of the tests were performed under conditions near to Peak Hour demands. This makes our results conservative, providing a reasonably accurate prediction of the minimum fire flows that would be available under the worst conditions. Another consideration is the variation in hydraulic performance between fire hydrants of different manufacture and age. Even two hydrants built by the same company may exhibit significantly different hydraulic characteristics if one hydrant is several years older than the other. In our experience in modeling fire flows, we have found that a good rule of thumb is to assume 10 psi of head loss through a hydrant at a flow of 1,000 gpm. Therefore our analysis of available fire flows calculated fire flows at a residual pressure of 30 psi. It was assumed that where required fire flows are in excess of 1,000 gpm, multiple hydrants would be used as allowed by the Laramie Fire Department, and that 30 psi of available pressure in the main serving a hydrant would translate into 20 psi of available pressure at the hydrant as required by the Uniform Fire Code.
As discussed earlier in this report, a Peak Day demand of 16.1 MGD was used for existing conditions. This number is based on a Peak Day per capita consumption of 554 gpcd, and a population estimate in 1995 of 29,137. The Peak Hour demand of 22.8 MGD was arrived at using a per capita consumption of 781 gpcd. The source of these figures is the City of Laramie Water Supply Master Plan Level I. Demands for Peak Day and Peak Hour conditions at buildout were calculated by similar means. It was assumed that future growth in and around the City of Laramie would proceed as described in the Level I study, and that growth would account for increased water usage. Therefore, in the buildout version of the hydraulic model, large point demands were assigned to locations in the existing model where the Level I study indicated growth would occur. In most cases growth is expected to occur where no existing distribution system elements yet exist. Another assumption made was that per capita water consumption would not change. The logical result is that, in the hydraulic model, all additional water demands representing future conditions were accounted for by these point demands. The magnitude of these demands was calculated using the figures for population and peak per capita consumption given in Section 5 of the Level I study.

3.3.3 RESULTS OF SYSTEM ANALYSIS

Figure 3-5 shows system pressures under Peak Hour conditions. The analysis indicates that system pressures are generally maintained at or above 30 psi. One exception is the Corthell Hill area, which may experience Peak Hour pressures less than 28 psi. In addition, reports from the City indicate that low pressures are occasionally experienced around Grand Avenue just west of 13th Street. This location is in the upper elevations of Zone 1. Our analysis indicates that pressures may drop to 30 psi or lower in this area, depending on the specific demand conditions. Another area subject to low pressures is near the north end of 9th Street. This is primarily due to its relatively high elevation within Zone 1.

Figure 3-6, which shows the available fire flows under existing Peak Hour conditions, indicates numerous deficiencies. The portion of Zone 2 which is east of 15th Street and south of Grand Avenue also lacks adequate hydraulic conveyance, as shown by low fire flows. Of great concern is the lack of available fire flow to the Corthell Hill area, which in addition to being served by inadequate conveyance, also is located at a higher elevation than surrounding areas. Our analysis indicates that under Peak Hour conditions, hydrants in the area may be unable to provide any significant fire flow at the required pressure of 20 psi at the hydrant. In existing Zones 3 and 4, the existing pump stations appear to be able to deliver fire flows sufficient for residential development. However, available fire flows may not be sufficient for commercial and industrial developments. There is no commercial development in Zone 4 at this time, and so the fire flows sufficient for commercial development are not currently required. The Imperial Heights Pump Station does not have fire pumps, and our analysis shows that the existing pumps cannot deliver adequate fire flows to this subdivision. Another area which appears to have difficulty receiving adequate fire flows is the northwest portion of Zone 2. This area is hydraulically remote from the tanks and pump station supplying it. Also this area shares a characteristic common to most of the portions of the system which have trouble receiving adequate fire flows; it is on the periphery of the system, which means that there are fewer feeds and more dead-end lines.
CITY OF LARAMIE
EXISTING WATER TRANSMISSION, DISTRIBUTION AND STORAGE SYSTEM
PRESSURE ZONE MAP WITH PRESSURE CONTOURS UNDER PEAK HOUR CONDITIONS

LEGEND
--- JUNCTION NODE AND PIPE (TYPICAL)
- PRESSURE CONTOUR (PSI)
  (CONTOUR INTERVAL 4 PSI)

ZONE BOUNDARY

ZONE 1
ZONE 2
ZONE 3
ZONE 4
ZONE 5

To Water Treatment Plant Gearwell

To South Laramie

From Pope Well Field and Soldier Springs

FIGURE 3-5

RBD, Inc.
209 South Meldrum
Fort Collins, Colorado 80521
(970) 482-5922
CITY OF LARAMIE
EXISTING WATER TRANSMISSION, DISTRIBUTION AND STORAGE SYSTEM
AVAILABLE FIRE FLOW MAP UNDER PEAK HOUR CONDITIONS

ZONE 1

ZONE 2

ZONE 3

ZONE 4

ZONE 5

NOTE: AVAILABLE FIRE FLOWS ARE SHOWN AT A RESIDUAL PRESSURE OF 30 PSI IN THE DISTRIBUTION SYSTEM. PRESSURE DROP THROUGH A HYDRANT FLOWING AT 1000 GPM IS APPROXIMATELY 10 PSI.

LEGEND

JUNCTION NODE AND PIPE (TYPICAL)

AVAILABLE FIRE FLOW (GPM)

FIGURE 3-6

RBD, Inc.
209 South Meldrum
Fort Collins, Colorado 80521
(970) 482-5922
3.3.4 CONCLUSIONS

Modeling was performed by constructing a new distribution system model using Cybernet. This was done due to the inadequacies of the City’s existing KYPIPE model, and the superior capabilities of Cybernet. The Cybernet model was constructed using data files from the City’s existing KYPIPE model, system maps provided by the City, and other information provided by City staff. The Cybernet model was then calibrated using fire hydrant test records provided by the City. Assumptions were minimized by incorporating data from the City’s SCADA (supervisory control and data acquisition) system, and from an understanding of system function gained through discussions with City staff. The computer model was used to model system function under Peak Day and Peak Hour conditions. Available fire flows were modeled under Peak Hour conditions, and compared with hydrant test data. The model indicated that low pressures and inadequate fire flows could occur in several areas, including Corthell Hill and the northern portions of Zone 2. In addition, it was found that the pump station serving the Imperial Heights subdivision could not deliver adequate fire flows.
4.0 FLOW TEST OF PIONEER CANAL

4.1 INTRODUCTION

The Pioneer Canal Flow study was divided into two parts, an autumn flow study and a winter flow study. The autumn flow study was completed during September and October 1995 with the winter flow study completed from November 1, 1995 to March 31, 1996. The purpose of the autumn flow study was to estimate the flow at predetermined cross-section stations along the length of the canal. The purpose of the winter flow study was to estimate the flow at a predetermined cross-section station close to Sogergreen Reservoir. Each of the studies will be discussed in turn.

After signing the contract, the original scope was modified at the suggestion of the City of Laramie. The scope change was outlined in the Pioneer Canal Flow Study, Action Plan" dated September 13, 1995. The Action Plan stated that velocity measurements will begin September 1995 and be completed by March 31, 1996. The scope of Task 4 was further modified, by a letter from the City of Laramie, to include researching water well information for the vicinity around the canal. The Action Plan and letter are provided in the technical appendix, for reference.

4.2 AUTUMN FLOW STUDY

Prior to the study six cross-section stations, at approximate 0.5-mile intervals, were field surveyed in the Pioneer Canal. For purposes of the study the canal was stationed from the diversion structure (station 0+00) to the most downstream cross-section station as shown on Figure 4-1. During the study velocity measurements were taken at each cross-section station on four different days (two days each calendar month).

As stated above, measuring from the diversion structure the cross-section stations used during the autumn flow study were as follows:

A) 26+00  
B) 52+80  
C) 79+20  
D) 105+60  
E) 135+34  
G) 158+40

It should be noted that station 158+40 is the location agreed to by RBD, Inc. and Western Water Consultants as the location where backwater effects from the Sodergreen Reservoir high pool elevation minimally influence flow and it was agreed that this location should be the assumed “end” of the Pioneer Canal.

The method used for collecting cross-section velocity measurements was consistent with the United States Bureau of Reclamation (USBR) and United States Geographic Survey (USGS)
NOT TO SCALE

NOTES
1. STATION 158+40 IS THE LOCATION OF THE END OF THE PIONEER CANAL AS AGREED BY RBD INC. AND WESTERN WATER CONSULTANTS.
2. THE REACHES ARE IDENTIFIED BETWEEN AUTUMN FLOW MEASUREMENT STATIONS.
3. REACHES ARE REFERENCED TO ATTACHED SPREADSHEET.

LEGEND

EXISTING CONTOURS
EXISTING INDEX CONTOURS
EXISTING DITCHES, CANALS
EXISTING PAVED ROADS
EXISTING DIRT ROADS

CANAL LINING EXHIBIT -- PIONEER CANAL
LARAMIE WATER SUPPLY PROJECT -- LEVEL II STUDY

FIGURE 4-1
methods as outlined in the USBR Water Measurement Manual. Briefly, velocity measurements were taken as body-centered subsections throughout the cross-section. Typically, velocity measurements were taken at the 0.6 depth of flow elevation for a given subsection. Subsections varied in width from less than 0.5-feet to 2.0-feet. Velocity measurement subsections were narrower in horizontal length where it was judged that boundary effects were causing large velocity gradients (i.e. close to the banks and at places where there was a relatively large difference in flow depth). In the center of the canal where the velocity gradient was smaller, the subsection widths were generally somewhat wider.

At approximately station 4+00 there is a 20-foot parshall flume and recording gage maintained by the Pioneer Canal-Lake Hattie Irrigation District. Western Water Consultants was called upon to provide flow records for the days of velocity measurements. The provided flow record was used as the assumed flow entering the canal (100-percent of flow) and was used to prorate the flow at the downstream cross-section stations.

The findings of the autumn flow study are reported in Table 4-1.

### TABLE 4-1
Summary of 1995 Autumn Flow Measurements

<table>
<thead>
<tr>
<th>Description</th>
<th>Station</th>
<th>Sept. 19th</th>
<th>Sept. 28th</th>
<th>Oct. 12th</th>
<th>Oct. 31st</th>
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<tbody>
<tr>
<td>Staff Gage at Parshall Flume *</td>
<td>4+00</td>
<td>32 cfs</td>
<td>31.5 cfs</td>
<td>30.7 cfs</td>
<td>29 cfs</td>
</tr>
<tr>
<td>Reported Diversion at Parshall Flume **</td>
<td>4+00</td>
<td>36.5 cfs</td>
<td>36.5 cfs</td>
<td>34.5 cfs</td>
<td>34.1 cfs</td>
</tr>
<tr>
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<td>26+40</td>
<td>29.7 cfs</td>
<td>32.4 cfs</td>
<td>29 cfs</td>
<td>28.6 cfs</td>
</tr>
<tr>
<td>Cross-section Station</td>
<td>52+80</td>
<td>27.3 cfs</td>
<td>30.8 cfs</td>
<td>27.6 cfs</td>
<td>27.2 cfs</td>
</tr>
<tr>
<td>Cross-section Station</td>
<td>79+20</td>
<td>25.8 cfs</td>
<td>29.3 cfs</td>
<td>25.5</td>
<td>25.2 cfs</td>
</tr>
<tr>
<td>Cross-section Station</td>
<td>105+60</td>
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<td>30.1 cfs</td>
<td>24.8</td>
<td>26.1 cfs</td>
</tr>
<tr>
<td>Cross-section Station</td>
<td>135+34</td>
<td>23.7 cfs</td>
<td>26.3 cfs</td>
<td>22.9</td>
<td>22.5 cfs</td>
</tr>
<tr>
<td>Cross-section Station</td>
<td>158+40</td>
<td>22.2 cfs</td>
<td>24.2 cfs</td>
<td>21.5</td>
<td>21.2 cfs</td>
</tr>
</tbody>
</table>

* Staff Gage reading is at 20-foot Parshall Flume. Staff Gage reading was used in conjunction with Table 20 in the Bureau of Reclamation, Water Measurement Manual to estimate flow.

** Reported Diversion readings as reported by Western Water Consultants.

Station 158+40 is the end of the Pioneer Canal as agreed to by RBD, Inc. And Western Water Consultants. Flow measurements from station 26+40 to station
158+40 are reported as part of this contract. RBD, Inc. has no responsibility for the reported staff gage flow at station 4+00. Graphs of the data summarized above is presented on the graph below.

**Autumn Flow Data**
**Pioneer Canal**

![Autumn Flow Data Graph](image)

4.3 **WINTER FLOW STUDY**

As stated in the Task 4 introduction, the purpose of the winter flow study was to estimate the flow at a predetermined cross-section station in the Pioneer Canal close to Sogergreen Reservoir. For purposes of this study, winter was from November 1, 1995 through March 31, 1996.

The winter flow study was to collect velocity measurements, twice a month, at station 135+34. Station 135+34 was chosen by Western Water Consultants because at this location there is a relatively large amount of longitudinal slope in the canal and the cross-section narrows. The thought was that because of the relatively increased slope and narrower cross-section that the velocity would be higher than in other reaches of the canal. Therefore, because of the relatively high velocity (energy) the icing conditions would be minimized throughout the winter.
The method used for collecting cross-section velocity measurements was consistent with the United States Bureau of Reclamation (USBR) and United States Geographic Survey (USGS) methods as outlined in the USBR Water Measurement Manual. Briefly, velocity measurements were taken as body-centered subsections throughout the cross-section. Typically, velocity measurements were taken at the 0.6 depth of flow elevation for a given subsection. Subsections varied in width from less than 0.5-feet to no greater than 2.0-feet. Velocity measurement subsections were narrower in horizontal length where it was judged that boundary effects were causing large velocity gradients (i.e. close to the banks and at places where there was a relatively large difference in flow depth). In the center of the canal where the velocity gradient was smaller, the subsection widths were somewhat wider.

The estimated flow at the winter cross-section station was compared to staff gage readings and its subsequent estimated flow through a 2-foot flume that was temporarily installed by the City on the throat of the 20-foot parshall flume, at station 4+00.

Throughout the study period a variety of weather conditions presented themselves. On field days temperature ranged from approximately 40 degrees, F to 10 degrees F and a west wind was generally present from 5 mph to in excess of 40 mph. These temperatures presented a challenging environment in which to work and collect data.

Interestingly, a variety of ice conditions were also present throughout the study period. During field days ice condition varied from needing to break through two or three inches of ice in November to cutting through an estimated 12 or 13 inches of ice in early January to rotten ice and slush in February and March. Then again at the end of March there was solid ice. Early in February, flow was riding on top of the ice. Early in March, frazil ice was observed as a sub layer to the more solid ice above it (about three inches of solid ice).

It was expected that because of velocities through the flume it would be generally ice free. However, on at least two occasions approximately 6 inches of ice needed to be broken out of the flume before staff gage readings could be taken. Also on at least two occasions anchor ice needed to be broken out of the flume. An interesting observation was that immediately downstream of the flume beginning about 100-feet below the flume for a reach of up to 200 yards there was open water throughout the winter.

Techniques used to get to open water through the ice were hand chopping, melting and sawing through the ice. Tools used to get through the ice included: a weed burner, an axe, a steel rod chipping bar and a chain saw. The period of time required to get through the ice and to get the cross-section station in condition to take velocity measurements varied from an estimated 20 minutes for one person to longer that two hours for two persons.

The technique used to "clear" the cross-section of ice so velocity measurements could be taken was to "clear" of ice an approximate two to three foot strip of ice then measure water depth, ice thickness and the distance from the top of the ice to the water surface. This was done because pressure flow was the flow regime generally found. On two occasions some flow had "broken-out" from under the ice above the winter cross-section and was partially
flowing on top of the ice and partially flowing under the ice. On one occasion, there was an air space between the bottom of the ice and the water surface.

For determining the total flow during the occasions when pressure flow was observed, a literature search was undertaken to research existing techniques for ice covered pressure flow in natural channels. The method used to determine the pressure flow was an averaged roughness value for an irregular cross-section. This method was chosen because of its applicability to the Pioneer Canal conditions. The flow equations used to determine the total flow and other technical information are described in the Technical Appendix.

In general, the estimated flow at the winter cross-section station was about 75-percent to 80-percent of the estimated flow at the Parshall Flume. The findings of the winter flow study are reported in Table 4-2 on the following page. However, in one case, March 28th, there was more flow at the cross-section than at the flume. There may be a couple of explanations for this as summarized below:

1. The day was relatively warm (above freezing) thus there may have been some ice melt or snow melt runoff that reached the cross-section.

2. On March 28th an anomaly was observed at the flume. It was observed that there was a small hydraulic jump (a couple tenths of a foot) in the converging approach section of the flume.

3. From the graph on page 4-4, from the reach above station 105+60 it appears that groundwater is seeping into the Pioneer Canal.
### TABLE 4-2
Summary of 1995-1996 Winter Flow Data

<table>
<thead>
<tr>
<th>Date of Measurement</th>
<th>Estimated flow at Station 135+34</th>
<th>Estimated Flow at Station 135+34, after adjustment for ice covered pressure flow</th>
<th>Estimated Flow at Station 4+00 (two-foot Parshall Flume)</th>
</tr>
</thead>
<tbody>
<tr>
<td>November 16, 1995</td>
<td>7.7 cfs</td>
<td>* N/A</td>
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</tr>
<tr>
<td>November 30, 1995</td>
<td>7.2 cfs</td>
<td>N/A</td>
<td>8.5 cfs</td>
</tr>
<tr>
<td>December 5, 1995</td>
<td>4.9 cfs</td>
<td>N/A</td>
<td>5.7 cfs</td>
</tr>
<tr>
<td>December 31, 1995</td>
<td>4.6 cfs</td>
<td>N/A</td>
<td>5.4 cfs</td>
</tr>
<tr>
<td>January 8, 1996</td>
<td>4.5 cfs</td>
<td>N/A</td>
<td>5.4 cfs</td>
</tr>
<tr>
<td>January 23, 1996</td>
<td>2.9 cfs</td>
<td>3.4 cfs</td>
<td>5.1 cfs</td>
</tr>
<tr>
<td>February 8, 1996</td>
<td>10.3 cfs</td>
<td>10.8 cfs</td>
<td>14.5 cfs</td>
</tr>
<tr>
<td>February 23, 1996</td>
<td>3.5 cfs</td>
<td>3.8 cfs</td>
<td>4.8 cfs</td>
</tr>
<tr>
<td>March 7, 1996</td>
<td>4.3 cfs</td>
<td>4.6 cfs</td>
<td>5.7 cfs</td>
</tr>
<tr>
<td>March 28, 1996</td>
<td>5.9 cfs</td>
<td>N/A</td>
<td>1.5 cfs</td>
</tr>
</tbody>
</table>

* N/A means that adjustment for pressure flow was not done because pressure flow was not observed.

4.4 CONCLUSIONS

During the Autumn 1995 and Winter 1995/1996 a successful flow monitoring program was completed on the Pioneer Canal. The purpose of the late season flow was to demonstrate that water can flow from the Laramie River down the Pioneer Canal to Sogergreen Reservoir during the late autumn and winter. For purposes of the study the definition of autumn and winter had more to due with anticipated weather conditions than calendar seasons. Autumn was considered September through October 31, 1995 with winter being November 1, 1995 through March 31, 1996.

The autumn flow study estimated flow at predetermined cross-section stations along the length of the canal. In general, the estimated flow at the winter cross-section station was about 75-percent to 80-percent of the estimated flow at the Parshall Flume.
The winter flow study estimated the flow at one cross-section close to Sodergreen Reservoir. Throughout the winter it was possible to flow water the length of the Pioneer Canal to Sodergreen Reservoir.
5.0 RATE STUDY

5.1 INTRODUCTION

This section of the Level II Study addresses the need to forecast user charges necessary to support: 1) debt service payments on Wyoming Water Development Commission ("WWDC") loans; 2) increases in operating expenses due to recommended and internally-funded capital improvements; and 3) inflation. To that end, Section 5.2 below addresses 1995-96 baseline financial conditions of the City of Laramie ("City") Water Fund based on audited financial statements and 1995-96 budget documents. Section 5.3 presents a 10-year financial plan for Level II capital improvements plus other internally-funded capital improvements identified by City staff. Section 5.4 presents 10-year cash flow projections, including trial monthly user charge increases necessary to support forecasted future expenses. Section 5.5 presents the conclusions and recommendations of this rate study.

5.2 1995-96 FINANCIAL BASELINE CONDITIONS

Between 1991-92 and 1993-94, the City’s Water Fund averaged $1,534,624 in operating revenues, based on audited financial statements. Operating income averaged $187,564 over the same period. Net income averaged $170,815 over those three years and suggested a positive trend. Over the same period, retained earnings of the Water Fund increased from $1,666,020 to $2,046,482 and municipal equity increased from $4,925,958 to $5,306,420. This suggests a debt-to-equity ratio of about 0.40. Audited financial statements also report that an allowance for depreciation in the amount of $254,561 was funded from operating revenues in 1993-94. The Water Fund exhibited positive cash flows from operations over the same period. These results do not take into consideration any deferred capital projects or maintenance that may have occurred over these years.

Table 5-1 presents operating revenues and expenses for the 1995-96 budget year. This budget information suggests that the City should maintain positive cash flows and positive net income in that year. However, these baseline conditions are not expected to be maintained due to significant capital needs over the next five years that will require some increases to monthly user charges.

5.3 10-YEAR FINANCIAL PLAN

A 10-year financial plan has been prepared for the City’s Water Fund. A capital improvement program ("CIP"), including both WWDC and internally funded projects, is presented in Table 5-2. A capital financing plan, including a schedule of grant and loan proceeds and debt service payments is included in Table 5-3.
TABLE 5-1
CITY OF LARAMIE WATER FUND
1995-96 BASELINE REVENUE REQUIREMENTS

<table>
<thead>
<tr>
<th>Operating Expenses</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Administration</td>
<td>$297,298</td>
</tr>
<tr>
<td>Pumping &amp; Wells</td>
<td>181,152</td>
</tr>
<tr>
<td>Filter Treatment Plant</td>
<td>190,001</td>
</tr>
<tr>
<td>Transmission &amp; Distribution</td>
<td>220,406</td>
</tr>
<tr>
<td>Meters</td>
<td>125,404</td>
</tr>
<tr>
<td>Buildings &amp; Grounds</td>
<td>68,725</td>
</tr>
<tr>
<td>Water Rights</td>
<td>152,825</td>
</tr>
<tr>
<td>Other</td>
<td>0</td>
</tr>
<tr>
<td>Total Operating Expense</td>
<td>$1,235,811</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Non-Operating Expenses</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Transfer Out to General Fund</td>
<td>56,970</td>
</tr>
<tr>
<td>Debt Service</td>
<td>$335,000</td>
</tr>
<tr>
<td>Total Non-Operating Expense</td>
<td>$391,970</td>
</tr>
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</table>

<table>
<thead>
<tr>
<th>Internally-Funded Projects(^b)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$302,822</td>
</tr>
</tbody>
</table>

**TOTAL REVENUE REQUIREMENTS**

$1,930,603

**WWDC-Funded Projects\(^c\)**

$600,000

\(^a\) Based on 1995-96 budget documents, subject to change.

\(^b\) Net of a $950,000 grant/loan package from the WWDC for the LaPrelle & Turner wellfields and Soldier Springs Reservoir. It is assumed that only $600,000 will be expended from that appropriation.

\(^c\) These include the LaPrelle Wellfield and Soldier & 8MG Reservoir projects.
## TABLE 5-2
### CITY OF LARAMIE WATER FUND
### CAPITAL IMPROVEMENT PLAN

<table>
<thead>
<tr>
<th></th>
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<tr>
<td>Internally-Funded CIP*</td>
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<td>Pumping &amp; Wells</td>
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<td>Transmission &amp; Distribution</td>
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<td>305,000</td>
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<td>72,822</td>
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<td>TOTAL INTERNALLY-FUNDED PROJECTS</td>
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<td>$407,822</td>
<td>$486,822</td>
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<td>Wellfields*</td>
<td>67%</td>
<td>33%</td>
<td>$200,000</td>
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<td>Construct Spur Well and Pipeline*</td>
<td>67%</td>
<td>33%</td>
<td>4,200,000</td>
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<td>Modify Zone 3*</td>
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<td>33%</td>
<td></td>
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<td></td>
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<td>2,334,000</td>
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<tr>
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<td>50%</td>
<td>50%</td>
<td>400,000</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Replace 36-inch Pipeline*</td>
<td>50%</td>
<td>50%</td>
<td>888,000</td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1,799,000</td>
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<tr>
<td>Replace 20-inch Lagoon Pipeline*</td>
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<td>50%</td>
<td>150,000</td>
<td></td>
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<td></td>
<td></td>
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<td></td>
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<tr>
<td>24-inch Pipeline*</td>
<td>50%</td>
<td>50%</td>
<td>312,000</td>
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<td>Corrosion Control Program*</td>
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<td>$0</td>
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<td>$4,716,822</td>
<td>$453,822</td>
<td>$2,252,822</td>
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*Based on budget documents prepared by the City of Laramie. Costs were escalated at 3.5% per annum to the mid-points of construction.

**Based on cost estimates prepared by the City of Laramie. Costs were escalated at 3.5% per annum to the mid-points of construction.

---

*a* Based on budget documents prepared by the City of Laramie. CIP items were escalated at 3.5% per annum to the mid-points of construction.

*b* Based on cost estimates prepared by the City of Laramie. Costs of WTP improvements scheduled for 2002-03 are based on a $600,000 capital cost (1996$), escalated at 3.5% per annum to the mid-point of construction assumed to be January, 2002.

*c* Based on a 1995 $400,000 WWDC grant/loan package, only $200,000 of which was expended in 1995-96. The grant/loan split of the expended funds is 67% / 33%.

*d* Appropriated based on a $3,132,000 (March 1993) cost estimate (Level II), escalated at 3.5% per annum to the mid-point of construction (assumed to be June, 1997) to $3,625,000.

*e* Based on a $2,179,000 (1996$) cost estimate (Level II), escalated at 3.5% per annum to the mid-point of construction assumed to be June, 1998.

*f* Based on a 1995 $550,000 WWDC grant/loan package, only $400,000 of which was expended in 1995-96 per City staff. The grant/loan split of the expended funds is 50% / 50%.

*g* Based on a $2,243,000 (1996$) cost estimate (Level II). It is assumed that $888,000 will be financed in 1996 and the balance funded in 2004 and its cost escalated at 3.5% per annum to the mid-point of construction assumed to be June, 2004.

*h* Based on a $150,000 (1996$) cost estimate (Level II).

*i* Based on $312,000 (1996$) for cathodic protection plus replacement of certain portions of the pipeline (Level II).

The project is expected to be partially funded from amounts left over from the Soldier and 8MG Reservoir projects totaling $550,000.

*j* Based on a $2,750,000 (1995$) cost estimate (Level I), escalated at 3.5% per annum to the mid-point of construction assumed to be June, 2002.
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<td>$1,563,780</td>
<td>$0</td>
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<td>4% / 20 Years</td>
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<tr>
<td>4% / 20 Years</td>
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<td>4% / 20 Years</td>
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<tr>
<td>4% / 20 Years</td>
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<td>0</td>
<td>0</td>
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<td>Total Debt Service Payments</td>
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<td>$225,397</td>
<td>$225,397</td>
<td>$354,165</td>
<td>$354,165</td>
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</tbody>
</table>

a For financial planning purposes only. This does not constitute a commitment of WWDC funds for future projects.
b This debt is expected to be retired in 2000-01.
The WWDC operates on a calendar year basis. Appropriations requests must be prepared by WWDC up to 12 months in advance of project planning. It is assumed for this analysis that total WWDC funding to Laramie through June, 1998 will be $6,150,000. The City, on the other hand, operates on a fiscal year beginning July 1. It is assumed that grant and loan funds obtained through WWDC would be available to the City shortly after the beginning of its fiscal year. Once funds are available for a particular project, design and construction normally proceeds through the next construction season, with most projects being completed within a single season. Therefore, for projects included in this Level II Study it is assumed that debt service payments will begin in December two years following the year the funds were originally appropriated.

5.3.1 CAPITAL IMPROVEMENT PROGRAM

Portions of the CIP were developed for this Level II Study. The CIP also includes projects expected to be funded internally by the City through monthly user charges and plant investment fees as well as projects expected to be funded through grants and loans received from the WWDC. Some of the items included in the CIP were developed during the Level I Study, others were revised during the Level II Study. Some of the items were developed by City staff and are included herein with little additional analysis. This section describes the CIP items, expected sources of funding and opinion of construction cost reported in both 1996 dollars and in real terms. For each item, the expected impact to future operating expenses is discussed, including timing of debt service payments if debt financing is anticipated.

5.3.1.1 WELLFIELDS

Investigations in the LaPrelle and Turner wellfields comprised $400,000 of a $950,000 funding request to the WWDC in 1995. To date, only $200,000 of the budgeted funds have been expended. Funding for the project consisted of a 67 percent grant and a 33 percent loan from the WWDC. It is assumed for this analysis that debt service payments on the expended loan portion will begin in December, 1998 (fiscal year 1998-99). It is assumed that a pro rata share of the expended funds are loan funds that will be repaid. Debt service is assumed to include 20 equal annual payments due in December of each year. The interest rate is assumed to be 4 percent. Debt service is assumed to be paid for from monthly user charges to on-line customers and other sources of revenue available to the City.

5.3.1.2 SOLDIER SPRINGS AND LOW RESERVOIRS

Rehabilitation of the Soldier Springs and Low Reservoirs comprised $550,000 of the $950,000 funding request to the WWDC in 1995. To date, only $400,000 of the budgeted funds have been expended. Funding for the project consisted of a 50 percent grant and a 50 percent loan from the WWDC. It is assumed for this analysis that debt service payments on the expended loan portion will begin in December, 1997 (fiscal year 1997-98). It is assumed that a pro rata share of the expended funds are loan funds that will be repaid. Debt service is assumed to include 20 equal annual payments due in December of each year. The interest
rate is assumed to be 4 percent. Debt service is assumed to be paid for from monthly user charges to on-line customers and other sources of revenue available to the City.

### 5.3.1.3 36-INCH RAW WATER PIPELINE REPLACEMENT

Replacement of portions of the 36-inch raw water pipeline is included in the 1996 funding request to the WWDC. It is expected to be constructed during the 1996-97 fiscal year. It is estimated to cost $888,000 in 1996 dollars based on a Level II Study estimate included in this report.

It is expected that funding for the project will consist of a 50 percent grant and a 50 percent loan from the WWDC. It is expected that funding will be available by July, 1996 prior to beginning the project. Since construction is expected to be completed by October, 1997, it is assumed for this analysis that debt service payments will begin in December, 1998. Debt service is assumed to include 20 equal annual payments due in December of each year. The interest rate is assumed to be 4 percent. Debt service is assumed to be paid for from monthly user charges to on-line customers and other sources of revenue available to the City.

### 5.3.1.4 SPUR WELLFIELD

The Spur Wellfield project is included in the 1996 funding request to the WWDC. It is expected to be constructed during the 1996-97 fiscal year. It is estimated to cost $3,132,000 in 1993 dollars based on a Level I Study estimate, adjusted to include only one chlorine feed system per City staff. This is equivalent to $3,625,000 in 1997 dollars, escalating the original estimate by 3.5 percent per year to June, 1997, the assumed mid-point of construction. (Costs were escalated from March, 1993 to June, 1997, equivalent to 3.5 percent per year over 51 months, or 15.74 percent.)

It is expected that funding for the project will consist of a 67 percent grant and a 33 percent loan from the WWDC. It is expected that funding will be available by July, 1996 prior to beginning design activities. Since construction is expected to be completed by October, 1997, it is assumed for this analysis that debt service payments will begin in December, 1998. Debt service is assumed to include 20 equal annual payments due in December of each year. The interest rate is assumed to be 4 percent. Debt service is assumed to be paid for from monthly user charges to on-line customers.

### 5.3.1.5 24-INCH PIPELINE

Cathodic protection of strategic spots along the 24-inch treated water pipeline is included in the 1996 funding request to the WWDC. The project also includes replacement of portions of the pipeline. It is expected to be constructed during the 1996-97 fiscal year and is estimated to cost $312,000 in 1996 dollars based on a Level II Study estimate included in this report.
It is expected that funding for the project will consist of a 50 percent grant and a 50 percent loan from the WWDC. A portion of the project ($550,000) will be funded by money left over from a previous appropriation that funded the Wellfields and Soldier & Low Reservoir projects described above. It is expected that funding will be available by July, 1996 prior to beginning the project. Since construction is expected to be completed by October, 1997, it is assumed for this analysis that debt service payments will begin in December, 1998. Debt service is assumed to include 20 equal annual payments due in December of each year. The interest rate is assumed to be 4 percent. Debt service is assumed to be paid for from monthly user charges to on-line customers and other sources of revenue available to the City.

5.3.1.6 20-INCH LAGOON PIPELINE

Relocation of a 20-inch pipeline located beneath a lagoon at the country club is included in the 1996 funding request to the WWDC. It is expected to be constructed during the 1996-97 fiscal year. It is expected to cost $150,000 in 1996 dollars based on a Level II Study estimate included in this report.

It is expected that funding for the project will consist of a 50 percent grant and a 50 percent loan from the WWDC. It is expected that funding will be available by July, 1996 prior to beginning the project. Since construction is expected to be completed by October, 1997, it is assumed for this analysis that debt service payments will begin in December, 1998. Debt service is assumed to include 20 equal annual payments due in December of each year. The interest rate is assumed to be 4 percent. Debt service is assumed to be paid for from monthly user charges to on-line customers and other sources of revenue available to the City.

5.3.1.7 ZONE 3 MODIFICATIONS

Funding for modifications to zone 3 of the transmission and distribution system is assumed to be requested in a 1997 funding request to the WWDC. It is expected to be constructed during the 1998-99 fiscal year. It is expected to cost $2,179,000 in 1996 dollars based on a Level II Study estimate included in this report. This is equivalent to $2,334,000 in 1998 dollars, escalating the original estimate by 3.5 percent per year to June, 1998, the assumed mid-point of construction.

It is expected that a funding request to the WWDC for the project would consist of a 67 percent grant and a 33 percent loan. Since construction is expected to be completed by October, 1999, it is assumed for this analysis that debt service payments would begin in December, 2000. Debt service is assumed to include 20 equal annual payments due in December of each year. The interest rate is assumed to be 4 percent. Debt service is assumed to be paid for from monthly user charges to on-line customers and other sources of revenue available to the City. WWDC funding of this project was assumed for financial planning purposes only and does not constitute a commitment on the part of WWDC to fund this project in the future.
5.3.1.8 CORROSION CONTROL PROGRAM

Funding for a corrosion control program is assumed to be requested in a 2001 funding request to the WWDC. It is expected to be constructed during the 2002-03 fiscal year. It is expected to cost $2,750,000 in 1996 dollars based on a Level II Study estimate included in this report. This is equivalent to $3,500,000 in 2002 dollars, escalating the original estimate by 3.5 percent per year to June, 2002, the assumed mid-point of construction.

It is expected that a funding request to the WWDC for the project would consist of a 50 percent grant and a 50 percent loan. Since construction is expected to be completed by October, 2000, it is assumed for this analysis that debt service payments would begin in December, 2001. Debt service is assumed to include 20 equal annual payments due in December of each year. The interest rate is assumed to be 4 percent. Debt service is assumed to be paid for from monthly user charges to on-line customers and other sources of revenue available to the City. WWDC funding of this project was assumed for financial planning purposes only and does not constitute a commitment on the part of WWDC to fund this project in the future.

5.3.1.9 INTERNALLY-FUNDED PROJECTS

Based on 1995-96 budget documents prepared by City staff, a number of projects were identified that are expected to be funded by monthly user charges, plant investment fees, and other sources of revenue that may be available to the City. These projects are included in Table 5-1. Internally-funded projects include water treatment plant improvements as well as smaller projects to support administration, pumping & wells, transmission & distribution, meters, buildings & grounds, water rights, and equipment purchases. Cost estimates for these projects have also been prepared by City staff in current dollars. These costs have been escalated to the midpoint of construction for each project at 3.5 percent per year.

5.4 10-YEAR WATER RATE PROJECTION

Table 5-4 illustrates forecasted cash flows for the City’s Water Fund for budget year 1995-96 and 1996-97 through 2005-06. This section describes the assumptions and methodologies used to prepare these projections.

Table 5-5 presents a capital financing plan which supports the cash flow projections in Table 5-4. Table 5-4 presents net income that may be available for capital and carries it forward to Table 5-5. Trial rate increases have been developed so that revenues from operations are sufficient to recover operating expenses as well as capital projects planned to be funded all or in part by monthly user charges and other revenues.
### TABLE 5-4

**CITY OF LARAMIE WATER FUND**

**FORECASTED CASH FLOWS**

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<td>Monthly User Charges&lt;sup&gt;c,d&lt;/sup&gt;</td>
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<td>Other&lt;sup&gt;e&lt;/sup&gt;</td>
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<td>10,000</td>
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<td><strong>Total Operating Revenues</strong></td>
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<td>154,300</td>
<td>158,900</td>
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<td>$647,425</td>
<td>$496,257</td>
<td>$623,335</td>
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*a* Based on 1995-96 Budget documents.

*b* Projected.

*c* Annual Customer Base Growth 1.0%

*d* Trial Rate Increases 0.0% 20.0% 0.0% 6.0% 0.0% 6.0% 0.0% 6.0% 0.0% 6.0%

*e* Includes hydrant charges and construction reimbursements

*f* Annual Budget Increases 3.0% 3.0% 3.0% 3.0% 3.0% 3.0% 3.0% 3.0% 3.0% 3.0%

*g* Includes ranch lease, wellhead protection, ranch refunds, sewer service charges, and gain on security sale.
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<td>899,500</td>
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<td>Replace 42-inch Pipeline</td>
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<td>$453,822</td>
<td>$2,252,822</td>
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</tbody>
</table>
5.4.1 OPERATING REVENUES

Operating revenues are assumed to include monthly user charges, meter sales and other operating revenues. Other revenues, in turn, are assumed to include hydrant charges and construction reimbursements. Monthly user charges are assumed to increase as a result of growth in the number of on-line customers and any rate increases that are implemented. The delinquency rate among on-line customers is assumed to be constant over the forecast period. Meter sales are assumed to be $20,000 per year and other revenues are assumed to be $10,000 per year over the forecast period.

5.4.2 OPERATING EXPENSES

Operating expenses are assumed to include: administration, pumping & wells, filter treatment plant, transmission & distribution, meters, buildings & grounds, water rights and other expenses. Operating expenses have increased an average of 1.5 percent from 1991-92 through 1993-94. It is expected, however, that over the long-term, operating expenses will increase with the consumer price index. This is assumed to be 3.0 percent per year during the forecast period. This higher annual increase is also prudent since the City may expect somewhat higher operating expenses related to ozonation treatment facilities, operation of the Spur Wellfield, and other new facilities.

5.4.3 NON-OPERATING REVENUES AND EXPENSES

Non-operating revenues and expenses are assumed to continue to include primarily earnings on investments (revenue), transfers to the general fund (expense), and other miscellaneous items. Miscellaneous items are assumed to include ranch lease revenues, wellhead protection funds, ranch refunds, sewer service charges, and any realized gains on security sales. These items are assumed to be constant over the forecast period.

5.4.4 DEBT SERVICE

Debt service payments are assumed to be $335,000 during budget year 1995-96. This debt is expected to be retired in budget year 2000-01. It is expected that these requirements will increase in 1996-97 due to a 1995 grant and loan package from WWDC. Additional future debt service payment requirements are scheduled in Table 5-2.

5.4.5 CAPITAL PROJECTS FUNDED FROM RATES

For purposes of this analysis, it is assumed that a portion of the capital projects undertaken in each year will be funded from excess net income from operating activities and plant investment fees in addition to WWDC grant and loan funds. It is assumed that the City will not issue any general obligation bonds or revenue bonds during the forecast period, but that internally-funded projects will be paid for through cash reserves, net income, and plant investment fees. It is assumed that monthly user charges will be increased to fund these projects, rather than rescheduling them for a later date.
It should be noted that the City has not prepared capital improvement plans beyond 2000-01 (5 years). As such, there is more confidence in the forecast for the period from 1996-97 through 2000-01 than for the years beyond that. The forecast from 2001-02 through 2005-06 may underestimate the monthly user charges required to support the Water Fund over that period.

5.5 CONCLUSIONS AND RECOMMENDATIONS

Based on capital plans presented in Tables 5-2, 5-3 and 5-5 as well as forecasted cash flows presented in Table 5-4, current monthly user charges will not support the CIP reflected in City budget documents as well as those included in this Level II Study. Based on trial rate increases, monthly user charges are estimated to have to increase 20.0 percent in 1997-98. The timing of this increase is due primarily to an increase in internally funded capital projects in that year. This result presumes that the City will use some of the remaining principal of a 1963 G.O. Bond issue to fund certain projects related to the water transmission and distribution system over the next few years. Depending on the need for additional capital investment, additional sources of funding for the rehabilitation of the transmission and distribution system may be necessary.

If monthly user charges are increased 20.0 percent in 1997-98 and again by 6.0 percent in 1999-2000 to offset the effects of inflation, sufficient revenue is expected to be generated through 1999-2000 to support both debt service payments for WWDC loans associated with recommended Level II improvements and increases in operating expenses due to inflation and other factors. Additional increases in monthly user charges are expected to be necessary beyond the year 2000 to continue to fund both internal projects and to pay debt service on anticipated future WWDC-funded projects. At this point, the magnitude of those increases cannot be determined.

Since additional improvements are being made to the water treatment plant, it may also be prudent to revise water plant investment fees to generate additional revenue for future capital projects supporting new customers.
6.0 CONCEPTUAL LEVEL PLANS AND COST ESTIMATES

6.1 ALTERNATIVE ANALYSIS

Section 2.0-Corrosion and Flow Study of Transmission Lines of this report identifies and discusses RBD, Inc.'s concerns with the existing 36" pipeline. In summary, the hydraulic capacity of the 36" pipeline does not meet the ultimate needs of the City of Laramie and there is sufficient evidence to indicate the pipe is experiencing severe external corrosion attack. In addition, the inlet structure at Sodergreen Reservoir is completely inadequate in its ability to isolate flows to the 36" pipeline. This section summarizes the work performed during RBD Inc.'s evaluation of alternatives for replacing the pipeline and the inlet structure. During this analysis, one of our primary goals was to rehabilitate existing facilities whenever possible.

6.1.1 36" RAW WATERLINE

6.1.1.1 INLET STRUCTURE

The existing inlet structure at Sodergreen Reservoir is very inadequate in its' ability to isolate the 36" pipeline. The inlet structure's two main components are a bar screen and isolation gates. The bar screen was installed to preclude large debris from entering the pipeline. Although the bar screen was not visible (i.e., it was submerged) during our inspection, it is highly unlikely that the screen has been damaged and cannot continue to be utilized. Unless the screen has been severely corroded, at most it would require cleaning by hand. Therefore, no other consideration has been given regarding replacement of the bar screen.

The isolation gates were installed to close flow off to the 36" pipeline. Somehow, either due to a lack of adequate tracking guides or poor seals, the gates do a poor job, at best, of sealing off the inlet structure. Based on RBD, Inc. work, the gates may not be salvageable although the operators are in good working condition. These gates were installed in (approximately) 1947 and thus have provided almost 50 years of service. In addition, design modifications may have occurred since the gates were installed and, since the gates will be over 50 years old in 1997, it is questionable whether replacement parts will be available. However, RBD, Inc. is concerned with attempting to rebuild these gates with the time available. As previously discussed in Section 2.0-Corrosion and Flow Study of Transmission Lines, it is anticipated that Sodergreen Reservoir will be drained to provide access to the gates. Once Sodergreen is drained, time will be of the essence to get the reservoir back on-line. RBD, Inc. does not believe the critical uses of the reservoir will allow the gates to be pulled, shipped back to the factory, evaluated and, if possible, rebuilt. Accordingly, RBD, Inc. cost estimates are based on installing three new inlet gates.

The inlet structure itself appears to be in good condition. Unfortunately, as with the slide gates and bar screen, it is very difficult to assess the condition since the structure was at least partially submerged during this analysis. The best and most practical way to provide a more...
complete assessment of the condition of the structure will involve accessing the inside of the structure when Sodergreen Reservoir is drained for maintenance. Realizing this may not be practical for quite some time, site observations and experience leads to a conclusion that the likelihood of a major structural problem is low. In addition, City staff has been in the inlet structure on numerous occasions and has not discovered any structural defects. Based on field observations and discussions with the City, RBD, Inc. feels that any improvements to the inlet structure will be essentially “cosmetic” and that major repairs will not be required.

6.1.1.2 36" RAW WATER LINE

Based on the hydraulic capacity constraints and the corrosion assessment work completed in this study, RBD, Inc. recommended replacement of the existing 36" pipeline at one of the project meetings. City staff was reluctant to agree with this approach until they could be convinced that every possible effort had been made to utilize the existing pipeline before abandonment. Accordingly, RBD, Inc. prepared four alternative scenarios to approach the project. The four alternatives are listed below with an explanation of the theory behind each alternative. Please note that since each alternative requires rehabilitation of the inlet structure and the piping from the “fish trap” vault into the WTP, the discussion on those items will be minimal. The corrosion investigation revealed the 36 inch raw water line to be in satisfactory internal condition but has suffered extensive external corrosion damage. From a corrosion perspective, Alternatives No. 1 and 2 require further elaboration whereas Alternatives No. 3 and 4 involve new pipe and the corrosion control issues are straightforward. See Figure 6-1 for a conceptual plan view of each alternative.

Alternative No. 1:

Rehabilitate the existing upstream inlet structure; add new air release valves (ARV’s); install new downstream 42" pipe; smart pig existing 36" pipe and make repairs; add cathodic protection to 36" pipe; future 22" outside diameter (O.D.) parallel pipe.

This alternative focuses on increasing the hydraulic capacity of the existing pipeline by removing the constrictions from the “fish trap” vault into the WTP while utilizing the existing 36" pipeline. Restoring the 36" pipeline capacity could be achieved by removing trapped air at the numerous high spots in the system, installing cathodic protection to minimize further external corrosion, and by replacing any sections of pipe which a “smart pig” found to be structurally inadequate. "Smart Pig" refers to a machine commonly used by the oil and gas industry to evaluate the integrity of large diameter pipelines. The machine is inserted into the pipe and is either pushed or pulled throughout the entire length of pipe. Through magnetic flux scanning technology, the machine measures the wall thickness and can tell whether corrosion is internal or external. From a log of the work, the locations needing repair can be pinpointed. The purpose of using this technology is to quantify the extent of the existing corrosion damage. The leak history and field investigation conducted to date suggests there is a high likelihood that damage is extensive and numerous leaks may exist but are masked by a high groundwater table. It is estimated that the line has at least five to ten existing or imminent leaks and may have on the order of one hundred or more. This
SODERGREEN RESERVOIR (W.S.E. = 7632)

**EXISTING 36" DIAMETER PIPE**
- SMART PIG WITH REPAIRS
- ADD AIR RELEASE VALVES (APPROX. 5)
- CATHODIC PROTECTION

**ALTERNATIVE #1**
- INLET STRUCTURE
- FUTURE 22" O.D. PIPELINE
- TO WALDEN CO.
- HWY 230
- FISH TRAP VAULT
- TO LARAMIE
- NEW 42" DIAMETER PIPELINE
- LARAMIE W.T.P.

**ALTERNATIVE #2**
- INLET STRUCTURE
- FUTURE 22" O.D. PIPELINE
- TO WALDEN CO.
- HWY 230
- FISH TRAP VAULT
- TO LARAMIE
- NEW 42" DIAMETER PIPELINE
- LARAMIE W.T.P.

**ALTERNATIVE #3**
- INLET STRUCTURE
- FUTURE 24" O.D. PIPELINE
- TO WALDEN CO.
- HWY 230
- FISH TRAP VAULT
- TO LARAMIE
- NEW 42" DIAMETER PIPELINE
- LARAMIE W.T.P.

**ALTERNATIVE #4**
- INLET STRUCTURE
- NEW 42" DIAMETER PIPELINE
- TO WALDEN CO.
- HWY 230
- FISH TRAP VAULT
- TO LARAMIE

**FIGURE 6-1**

******

*3D Design Inc.*

**Engineering Consultants**
potential order of magnitude difference in the number of leaks dramatically affects the level
of effort and cost of repair. The structural inadequacies of the pipeline would be addressed
via the smart pig and repaired accordingly. The analysis indicated that even with
replacement of the piping from the “fish trap” vault into the WTP, a future 22" O.D. pipeline
would be required. The technical appendix addresses the hydraulic calculations for this
alternative.

Alternative No. 2:

Rehabilitate existing upstream inlet structure; add new (ARV’s); install new downstream
42" pipe; add cathodic protection to 36" pipe; repair future leaks; future 22" O.D. parallel
pipe.

This alternative focuses on increasing the hydraulic capacity of the existing pipeline by
removing the constrictions from the “fish trap” vault into the WTP and utilizing the
remainder of the existing 36" pipeline. Restoring the 36" pipeline hydraulic capacity could
be achieved by removing trapped air at the numerous high spots in the system. A cathodic
protection system would ensure that future corrosion on the pipeline would be minimized by
installation of cathodic protection immediately to mitigate further corrosion and to repair
leakage as it becomes detected over time. The extent of existing and future leakage is
assumed to be the same under both Alternatives No. 1 & 2 but detection and repair is
deferred in Alternative No. 2. The main focus was on repairing future leaks as they occurred.
The analysis indicated that even with replacement of the piping from the “fish trap” vault
into the WTP, a future 22" O.D. pipeline would be required. A spreadsheet is included in the
technical appendix that addresses the hydraulic calculations for this alternative.

Alternative No. 3:

Rehabilitate existing upstream inlet structure; add new (ARV’s); install new downstream 42"
pipe; slip line existing 36" pipe with 32" O.D. pipe; future 24" O.D. parallel pipe.

This alternative focuses on increasing the hydraulic capacity of the existing pipeline by
removing the constrictions from the “fish trap” vault into the WTP and sliplining the
remaining 36" pipeline with a 32" O.D. plastic liner pipe. A 32" O.D. pipe was selected
because it allowed contractors to pull the liner through the pipe without having to replace the
numerous elliptical sections of pipe. The alternative still required installing ARV’s at high
points in the new line to remove trapped air. Since the new line could be designed to
withstand the imposed loads and would be inert to corrosion, cathodic protection of the
existing 36" pipeline was not required. The analysis indicated that even with the replacement
of the piping from the “fish trap” vault into the WTP, a future 24" O.D. pipeline would be
required. A spreadsheet is included in the technical appendix that addresses the hydraulic
calculations for this alternative.
Alternative No. 4:

Rehabilitate existing upstream inlet structure; abandon existing 36" pipe; install new 42" pipe.

This alternative focuses on increasing the hydraulic capacity by installing a new 42" pipeline and abandoning the existing 36" pipeline. Since this line would be constructed to maintain line and grade, ARV's could be minimized. In addition, a cathodic protection system could be installed with the pipeline, at a small margin of the costs to install it on the existing pipeline. A spreadsheet is included in the technical appendix that addresses the hydraulic calculations for this alternative.

Since there were obvious advantages and disadvantages associated with each of the four alternatives, an alternative matrix was constructed to assist in determining which alternative best met the needs of the City. Table 6-1 shows the four alternatives and the evaluation factors by which each alternative was evaluated. The alternative matrix corrosion cost estimates for Alternatives No.1 and No.2 were developed with the following assumptions:

1. "Smart pigging" cost is estimated at $125,000
2. Corrosion damage/leakage repair cost is estimated at $250,000
3. Cathodic protection cost is estimated at $70,000
## TABLE 6-1
ALTERNATIVE MATRIX FOR A 50 YEAR DESIGN LIFE PIPE SYSTEM TO SUPPLY RAW WATER FROM SODERGREEN RESERVOIR TO THE LARAMIE WTP

1996 Costs in 1000's of Dollars

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<th>36&quot;</th>
<th>32&quot;</th>
<th>22&quot;</th>
<th>14&quot;</th>
<th>0 &quot;</th>
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<td>15 36&quot; / 22&quot;</td>
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<td>676</td>
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<td>0</td>
<td>1185</td>
<td>5 42&quot;</td>
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### Alternative No. 1
Rehabilitate existing upstream intake structure; add new ARV's; install new d/s 42" pipe; smart pig ex. 36" pipe and make repairs; add cathodic protection to 36" pipe; future 22" O.D. parallel pipe.

### Alternative No. 2
Rehabilitate existing upstream intake structure; add new ARV's; install new d/s 42" pipe; add cathodic protection to 36" pipe; repair future leaks; future 22" O.D. parallel pipe.

### Alternative No. 3
Rehabilitate existing upstream intake structure; add new ARV's; install new d/s 42" pipe; slip line existing 36" pipe w/ 32" O.D. pipe; future 24" O.D. parallel pipe.

### Alternative No. 4
Rehabilitate existing upstream intake structure; abandoned existing 36" pipe; install new 42" pipe

**Probability Point System:** 0 - 100 with 0 being the least rating possible and 100 being the highest rating possible, 0 - worst case, 100 - best case.

**Cost Value Note:** Future occurrence items such as C.P in future, future leaks, and future maintenance are shown above in their present worth. The future pipe is assumed installed in 20 years and the cost shown above is its present worth.
Column Definitions:

Column 1: Cost of rehabilitation of the existing intake structure
Column 2: Cost of new downstream 42" pipe
Column 3: Cost of smart pig and repairs
Column 4: Cost of Cathodic Protection
Column 5: Cost of A.R.V.'s
Column 6: Cost to repair future leaks
Column 7: Cost to slip line existing 36" pipe
Column 8: Cost to install new 42" pipe
Column 9: Cost of future parallel pipe
Column 10: Maintenance cost
Column 11: Probability of Success
Column 12: Reliability of System
Column 13: Probability of 50 year life of system

* 1 % for 0 - 5 years = 0.05
  75 % for 5 - 10 years = 3.75
  85 % for 10 - 40 years = 34.00
  Total value = 37.80
  Possible value = 50.00
  Percentage = 76 %

6.1.1.3 CONCLUSIONS

When all rehabilitation costs are totaled, Alternative No. 2 is the least expensive because it does not incur the "smart pig" cost. The disadvantage of this alternative, however, is that the pipe condition is not accurately known and may be considerably worse than presumed. This lack of knowledge increases the overall economic and operating risk of this alternative. However, the evaluation considered many other factors, including the probability of success, reliability of the system and the probability of the alternative lasting another 50 years. Examination of all the alternatives reveals that replacement of the entire 36 inch line with a new 42 inch line (Alternative No.4) offers the greatest value for the cost. While the cost is slightly higher than Alternative No.2, a new pipeline eliminates
the operating uncertainty and risk associated with corrosion of the existing line and also satisfies the projected future hydraulic capacity requirements.

6.1.2 24 INCH TRANSMISSION LINE

6.1.2.1 CORROSION

The field investigation to date indicates the 24 inch transmission line is in satisfactory overall condition but that a number of corrosive or "hot spot" areas exist where damage has occurred. There are two practical alternatives available to the City of Laramie:

1) Replace the pipeline.

2) Replace specific short sections of line and institute a "hot spot" cathodic protection program for other areas of the line where corrosion attack is expected to occur.

Cathodic protection of the entire line is not technically nor economically reasonable due to the lack of an external coating and the lack of electrical continuity at the pipe joints.

Alternative No. 1:

Replacement of approximately 25,000 feet of pipe, (estimated cost $2,500,000 - $3,000,000).

Alternative No. 2:

Replacement of approximately 3,400 linear feet of pipe (two separate sections) with new pipe (estimated cost $480,000) and "hot spot" cathodic protection of approximately 4,850 feet of pipe (estimated cost $230,000), can be expected to extend the service life of the pipeline for another 20 to 30 years.

6.1.2.2 CONCLUSIONS

Examination of these two alternatives reveals that while Alternative No. 1 provides the greatest operating reliability and lowest risk, the field test data and pipe leak history do not warrant this expenditure at this time. The Alternative No. 2 pipe replacement should be limited to those areas where leakage has already occurred. "Hot spot" cathodic protection involves the installation of sacrificial anodes at specific locations where corrosion has been observed or is likely to occur based on soil conditions. In addition, as part of a "hot spot" program, the City should install a sacrificial anode on the 24 inch line any time the pipe is excavated for any reason such as a tap, repair, inspection, or crossing by another utility. It must be noted that some localized corrosion leakage can still be expected to occur on the 24 inch line. However, the leak frequency will be greatly reduced from what would occur if no corrosion control measures were implemented. Refer to section 6.2.2 for a conceptual level plan of corrosion improvements.
6.1.3 20" TRANSMISSION LINE

No improvements are warranted on this pipeline because the pipe excavations revealed no corrosion problems and the soil sample data does not indicate particularly aggressive soil. No leaks have been reported by the City on this section of pipe and an excavation inspection performed during the Level I Study revealed no evidence of external corrosion. This line can be expected to provide 20 to 30 years of additional service life although an occasional localized leak may occur in the future.

6.1.4 CITY DISTRIBUTION SYSTEM

6.1.4.1 INTRODUCTION

Under our Scope of Services (see Section 1.2 of this report), one of the tasks of our analysis was to evaluate the function of the City’s water transmission and distribution system. RBD, Inc.’s analysis focused on maintaining uniform delivery pressures, and maximizing operational efficiency. The addition of future sources of water to the system was to be included in the analysis. In response to preliminary submittals by RBD, Inc., City staff identified additional concerns. These concerns dealt with the effects our recommended improvements would have on the function of the water system. In addition, City staff placed a high priority on ensuring the proposed improvements would make maximum use of existing facilities and infrastructure.

City staff informed RBD, Inc. of several existing problems they had observed within the water system, as well as concerns regarding the system’s ability to serve future needs. The following items are intended to describe the major components of those system elements and concerns that City staff directed RBD, Inc. to investigate:

Transmission Lines:

According to City staff, the existing transmission line in Reynolds Street carries a proportionately greater flow than the lines in Grand and Lewis Avenues. The City expressed the concern that the connection of the Spur Well to the north side of the distribution system, combined with anticipated growth in that area, might cause the Reynolds line to be overburdened.

The City is aware of low pressures in the Corthell Hill area. The City asked RBD, Inc. to investigate additional conveyance as a means of improving pressures in this area.

City staff also directed RBD, Inc. to evaluate the need for additional water mains throughout the system to meet future demands. In particular, RBD, Inc. was asked to identify mains approaching capacity, and to recommend improvements to the distribution system to relieve lines which may become overburdened in the future.
Pump Stations:

The City identified existing inadequacies in their pump stations, including a lack of pumping capacity to Zone 5, a need for added pumping capacity in the future to Zone 4, and the lack of backup power capability for all but one of the pump stations. In addition, RBD, Inc. was directed to evaluate the City’s use of variable speed drives used on some of the pumps. In general, RBD, Inc. evaluated the ability of the pump stations to meet future demands.

Additional Storage:

The City directed RBD, Inc. to investigate the need for storage to serve Pressure Zones 3, 4 and 5. The City requested that the amount of storage needed was to be determined for the present, as well as for the 25-year and 50-year horizons.

The City of Laramie Water Supply Master Plan, Level I, gave population projections for a 50-year period beginning with 1995, and identified the areas where growth is likely to occur. The Level I study further calculated the Peak Day and Peak Hour flow rates in terms of gallons per capita per day (gpcd), based on historical demand patterns. Demands were calculated by RBD, Inc. 25 years into the future, based on the Level I study’s planning projections and per capita flows, to aid the City of Laramie in planning capital improvements. The projected peak day demands were used to determine storage needs and identify what improvements to the water distribution system will be required to meet future demands.

In the course of our analysis, RBD, Inc. has identified several deficiencies in the City’s water transmission and distribution system. In many cases the deficiencies were already known to the City. Section 3.3.3 of this report discusses the results of our analysis, and includes system maps showing areas of inadequate pressures and fire flows under existing conditions. This portion of the report reviews the various solutions that were considered in the course of formulating our final recommendations. The main areas evaluated were as follows:

- Pressure zones
- Additional storage
- Additional conveyance
- The proposed Spur Well field
- Existing pump stations
- The 20-inch and 24-inch transmission lines

6.1.4.2 PRESSURE ZONES

Currently Zones 3 through 5 are without storage, and are entirely dependent on pump stations to meet demands. In addition, the Alta Vista and Indian Hills Pump Stations lack backup power. The Imperial Heights Pump Station has a backup generator, but no fire pumps. The Hi-Lo Pump Station and the Alta Vista Pump Station are operated by means of a complex computer program. The Imperial Heights Pump Station, which serves Zone 5,
does not have a fire pump. However, it is the only pump station with a backup power generator. The high side of the Hi-Lo Pump Station, the Alta Vista Pump Station, and the Indian Hills Pump Station all currently have adequate capacity to meet daily demands and provide fire flows under most conditions. However, the lack of backup power to these stations makes them vulnerable to power outages, which could result in an interruption in service to customers served by the pumps. Because of the sheer number of pumps in the system, it may be possible to maintain continuous service under a variety of conditions in the event of failure of either the Alta Vista Pump Station or the Hi-Side Pump Station. However, this type of system redundancy results in high maintenance and energy costs, and the complexity of the system makes it difficult to manage.

The division of a water utility’s service area into pressure zones, combined with sufficient water storage, is an effective way of providing customers with adequate water supplies at consistent pressures. Typically RBD, Inc. recommends that water utilities minimize their dependence on pump stations in favor of elevated storage. The primary reasons for this are:

- Pumping water to elevated storage is more efficient, and therefore less expensive, than pumping directly to customers. The pumps need only pump at the Peak Day demand rate, rather than to meet constantly changing demands.

- If storage reservoirs are properly sized (i.e. to store one Peak Day’s demand, plus fire flows, plus equalization storage), the utility can provide uninterrupted service to customers in the event of plant or equipment failure.

- A storage tank provides consistent service pressures because system pressures are regulated by the water surface elevation in the tank. One foot of storage in a typical tank will contain tens of thousands of gallons. Even if the tank is draining or filling at a very high rate, the water surface elevation changes very slowly.

Dividing a service area into pressure zones also helps minimize capital expenditures for storage reservoirs and other facilities. Even if water users are spread over a wide area, they can be served by a common tank if they are all located at similar elevations, and adequate conveyance exists to deliver water from the tank.

The City of Laramie is divided into five pressure zones. Three of these, Zones 3, 4 and 5, are served only by pump stations. Zone 2 is served by both the Lo side of the Hi-Lo Pump Station, and by the Hi-Level storage tanks. Zone 1 is served by the Lo-Level storage reservoir. All treated water from the WTP and the City’s groundwater sources flow directly into Zone 1. Those areas in Zones 1 and 2 not receiving adequate flows and/or pressures are discussed later in this report. However, it appears that Zones 1 and 2 function well overall. Zones 3, 4 and 5 are each quite small, and are not served by storage. The potential exists to redefine these zones in order to make the distribution system more efficient.

The Laramie Regional Airport exists as a stand-alone pressure zone, which is served by a booster pump station. According to the Level I study, the recommended fire flow for the
airport is 4,000 gpm for four hours. The existing booster pump station and 10-inch pipeline serving the airport are capable of delivering less than 1,000 gpm. The Level I study discussed two alternatives to increase the water supply to the airport. The first was to construct two additional pipelines to link the airport into the City’s distribution system, along with a storage tank. This approach would allow the recommended fire flows to be delivered to the airport, and would provide infrastructure to serve future growth. However, the cost given in the Level I study of more than $2,000,000 is very high when compared to the immediate benefits. The solution recommended in the Level I study was to build a 16-inch pipeline and booster station to serve the airport. This estimated cost of this approach was $1,200,000. While this solution is considerably less costly, the benefits are even fewer than the previous alternative. The capacity of the 16-inch line would only be needed during fire flow conditions. Under normal conditions, only a fraction of its capacity would be utilized. The combined capacities of the existing 10-inch pipeline and the proposed 16-inch pipeline would be adequate to supply over 8,000 people if a storage tank was constructed at the airport, and nearly 5,500 people without a storage tank. According to the growth projections presented in the Level I study, development of this magnitude in the area around the airport is extremely unlikely. A third alternative would be to construct a 1,000,000 gallon tank to store fire flows, and use the existing 10-inch pipeline and booster pump station to keep the tank full and supply normal demands. The problem with this solution is that the water in the tank would tend to become stagnant unless demands in the area were sufficient to turn it over every few days and therefore a cost was not determined for this alternative. A least cost solution may be to increase the capacity of the existing booster station by adding a fire pump, although the structural integrity of the existing 10" pipeline would need to be examined if this alternative was to be explored further. The Level I study did not look into the cost associated with adding a fire pump to the existing booster station. A 10-inch pipeline is not normally utilized to carry flows of 4,000 gpm because the headloss through the pipe at such flowrates is excessive. However, this would only be necessary during fire flow conditions, and the capacity of the existing pumps could then be dedicated to serving normal demands. RBD, Inc. recommends that a separate Level II study be completed for the Laramie Regional Airport because this effort is outside of the scope of work associated with this Level II study.

6.1.4.3 ADDITIONAL STORAGE

The City of Laramie currently maintains three storage reservoirs, with a combined capacity of 11.5 million gallons (MG). RBD, Inc. recommends that a water utility maintain sufficient storage to meet Peak Day demands with the largest source of treated water out of service. According to the Level I study, the peak production rates for the City’s surface water and groundwater supplies are as follows:

<table>
<thead>
<tr>
<th>Source</th>
<th>Peak Capacity (MGD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water Treatment Plant</td>
<td>7.0</td>
</tr>
<tr>
<td>Pope Wells &amp; Soldier Springs</td>
<td>5.4</td>
</tr>
<tr>
<td>Turner Wells</td>
<td>4.0</td>
</tr>
<tr>
<td>Total</td>
<td>16.4</td>
</tr>
</tbody>
</table>
If the City’s water treatment plant (the largest source of treated water) should need to be shut down during a period of high demand, the City would still be able to produce approximately 9.4 million gallons per day (MGD) from its existing groundwater sources. The 9.4 MGD of groundwater, combined with the 11.5 MG in storage, is more than adequate to supply the City during current Peak Day demands.

Another consideration in determining whether adequate storage exists in a water system is the location of the storage in relation to the customers to be served. One purpose of water storage facilities is to provide an emergency supply in the case of a power outage or plant failure. Water tanks must be located so that water can flow by gravity from the tank to users at adequate pressures and flow rates to meet both peak demands and fire flows. All of the City of Laramie’s existing water supplies, and 70% of its storage, serve Pressure Zone 1. The remaining storage is dedicated to Zone 2, and no storage is available for Zones 3, 4 and 5. The following table shows how existing water demands break down by zone:

<table>
<thead>
<tr>
<th>Zone</th>
<th>Existing Peak Day Demand (MGD)</th>
<th>Percentage of Total System Demand</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6.44</td>
<td>40%</td>
</tr>
<tr>
<td>2</td>
<td>8.53</td>
<td>53%</td>
</tr>
<tr>
<td>3</td>
<td>0.68</td>
<td>4.2%</td>
</tr>
<tr>
<td>4</td>
<td>0.32</td>
<td>2.0%</td>
</tr>
<tr>
<td>5</td>
<td>0.13</td>
<td>0.8%</td>
</tr>
<tr>
<td>Total</td>
<td>16.1</td>
<td>100%</td>
</tr>
</tbody>
</table>

From the table it can be seen that the current combined Peak Day demands of Zones 2 through 5 are approximately 9.66 MG. This is nearly three times the available storage of 3.5 MG. Currently, all of the treated water entering Zones 2 through 5 must pass through the Lo-Side Pump Station. In the event of a major equipment or power failure affecting the Lo-Side pumps, no other source of water is currently available to 60% of the system’s users.

Fortunately, the City plans to develop the Spur Well Field, and to bring the water produced by the Spur Wells into Zone 2. The Level I Study estimated that the peak production rate of the Spur Wells will be 4.0 MGD. This 4.0 MGD, combined with the 3.5 MG available to Zone 3 in the two Hi-Level reservoirs, will provide 7.5 MG available to meet Peak Day demands in the event of failure of the Lo-Side pumps. Though this is still somewhat less than the estimated Peak Day demand for Zone 2, City staff have stated that the maximum likely length of a power outage to the Hi-Lo Pump Station is two hours. Therefore, potential storage facilities for Zones 2 through 5 have been evaluated on the assumption that they need only supply the system demands, plus fire flows, for a maximum of two hours.
While development of the Spur Well Field will provide system reliability for Zone 2, it will not provide any direct benefit to Zones 3, 4 and 5. A water storage tank will provide the storage necessary to provide the residents in the new Zone 3 the same system reliability as the rest of the City.

The total storage required to serve Zones 3, 4 and 5 under current conditions includes two hours of water at the Peak Day demand rate (based on a 2-hour maximum power outage, per City staff); a fire flow volume of 960,000 gallons; and an equalization storage of approximately 12,000 gallons. A fire flow volume of 960,000 gallons results from a fire flow of 4000 gpm for 4 hours, as required by the Uniform Fire Code for areas zoned for higher-density business and commercial developments, such as the Walmart store near Imperial Heights. Equalization storage is the volume of water stored in the tank which is used when demands during the day exceed the average Peak Day demand. To summarize, the existing storage requirement for Zones 3, 4 and 5 is shown below:

<table>
<thead>
<tr>
<th>Storage Type</th>
<th>Volume</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 Hours of Peak Day Demand</td>
<td>94,200</td>
</tr>
<tr>
<td>Fire Flow Storage</td>
<td>960,000</td>
</tr>
<tr>
<td>Equalization Storage</td>
<td>12,000</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>1,066,200</td>
</tr>
</tbody>
</table>

As stated previously, the Peak Day demand in the year 2020 (25-year buildout) is estimated to be 2.1 million gallons. Using the same method used to calculate current storage needs, the expected required storage for Zones 3, 4 and 5 at that time is shown below:

<table>
<thead>
<tr>
<th>Storage Type</th>
<th>Volume</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 Hours of Peak Day Demand</td>
<td>175,000</td>
</tr>
<tr>
<td>Fire Flow Storage</td>
<td>960,000</td>
</tr>
<tr>
<td>Equalization Storage</td>
<td>16,000</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>1,151,000</td>
</tr>
</tbody>
</table>

City staff have also asked what the storage requirement for Zones 3, 4 and 5 would be in the year 2045 (the 50-year buildout). Based on a population of 45,667, given in the Level I study, and a Peak Day demand rate of 554 gpcd, the City’s combined Peak Day demand in 2045 will be approximately 25.6 million gallons. Of this approximately 10%, or 2.5 million gallons, will be by residents of Zones 3, 4 and 5. The required storage for Zones 3, 4 and 5, making the same assumptions as before regarding the Hi-Lo Pump Station, is calculated as:

<table>
<thead>
<tr>
<th>Storage Type</th>
<th>Volume</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 Hours of Peak Day Demand</td>
<td>210,000</td>
</tr>
<tr>
<td>Fire Flow Storage</td>
<td>960,000</td>
</tr>
<tr>
<td>Equalization Storage</td>
<td>20,000</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>1,190,000</td>
</tr>
</tbody>
</table>

The alternative for providing storage to Zones 3, 4 and 5 would require increasing the pumping capacity of the pump stations. A fire pump would need to be added to the Imperial Heights Pump Station, in order to provide adequate fire flows. In addition, the pumps in this pump station are described in the Level I study as being mis-sized, having been chosen to
pump to elevated storage. It should be possible to modify the pumps to operate more efficiently. In addition, to provide the reliability required for a system of the size of Zones 3 through 5 a dual electric feed or emergency generator at each pump station is required.

The Alta Vista and Indian Hills Pump Stations operate on an intermittent basis, depending on system demands in the Alta Vista and Indian Hills subdivisions. During periods of low demands, these areas are supplied by the Hi-side of the Hi-Lo Pump Station. As demands increase in the future, modifications would need to be made to ensure that the Hi-side pumps can adequately supply peak hour demands, plus fire flows. In addition, growth in Zones 3 and 4 may make modifications to the Alta Vista and Indian Hills Pump Stations necessary to ensure that peak demands can be met, assuming that no storage for these zones is constructed.

6.1.4.4 ADDITIONAL CONVEYANCES

The American Water Works Association (AWWA) guidelines for water distribution systems call for velocities in distribution lines to be maintained below 5 feet per second (fps). In RBD, Inc.’s analysis of the City’s distribution system, including the water mains in Reynolds Street, Lewis Avenue and Grand Avenue, this criterion was used to determine what lines were overburdened now, and what lines may become overburdened in the future. Currently, the City’s water mains do not exceed this velocity under Peak Hour demands.

RBD, Inc. has also examined the ability of the distribution system to convey flows without excessive pressure loss. As water flows through a pipeline, the walls of the pipe exert a frictional resistance to the flow which acts to reduce the pressure in the pipeline with distance. The longer a pipe is, the greater the pressure drop through the pipe at a given flow rate. In evaluating alternatives for increasing service pressures and/or flow rates to those areas identified as lacking, the performance of the existing pipelines was examined, and then the benefit of additional conveyance was modeled.

In determining the proper diameter of a pipeline, it is necessary to first determine what the maximum flow rate is that the pipe will carry, and the maximum pressure loss that can be tolerated at that flow rate. Typically, fire flow conditions govern the selection of the diameter of a distribution line. This is because fire flows are generally much greater than flows resulting from normal demands. It follows that if a distribution line is so undersized that excessive pressure losses occur under normal demand conditions (i.e. without fire flows), the pipe will almost certainly be unable to deliver adequate fire flows. The AWWA guidelines stated above are for peak demand conditions without fire flows. When sizing a pipeline to carry fire flows in addition to normal demands, the criteria used are a flow rate specified by the local fire department, delivered to the hydrant at a specified minimum pressure.
6.1.4.5 PRESSURE ZONES 3, 4, AND 5

RBD, Inc. modeled pipelines to serve a potential storage tank for Zones 3, 4 and 5. These pipelines must be able to deliver Peak Hour demands plus fire flows to the three pressure zones in the event of pump station failure. However, because of the configuration of the pump stations, a power outage affecting all of the pump stations is extremely unlikely, according to City staff. Under typical conditions, the existing pump stations would pump to an elevated storage tank, and under fire flow conditions both the tank and the pump stations would supply fire flows.

Several factors had to be considered in sizing pipelines to deliver water from a storage tank serving Zones 3, 4 and 5. The location of such a tank has an effect on the optimum diameter of a pipe connecting the tank to Zone 5. This is because the Imperial Heights Pump Station does not have a fire pump. A pipeline serving this area from a storage tank would have to deliver the difference between the capacity of the pump station, and the required fire flow plus Peak Hour demand. Likewise, because they are part of different power grids, a power failure is assumed to affect either the Hi-Side Pump Station or the Alta Vista Pump Station, but not both. Therefore, some water can be pumped to Zones 3, 4 and 5 under worst case conditions. Several possible scenarios were modeled to determine the worst case that might reasonably occur, and then pipelines were modeled to determine the minimum sizes necessary to provide adequate flows and fire protection under these conditions.

City staff have expressed the concern that if a storage tank is constructed to serve Zones 3, 4 and 5, increased static pressures in portions of the existing Zone 3 may cause an increased incidence of waterline leaks or breaks. Typically water utilities design system components to handle the maximum static and dynamic pressures a pipeline will be subjected to. Although static pressures are a concern, what can be more damaging to the pipes and valves of a water distribution system are pressure transients, or sudden and momentary increases in pressure. These transients are caused by the opening and closing of valves, or by pumps cycling on and off. The potential for pressure transients in the existing Laramie system is considerable because much of the system is served directly by pumps. These pumps constantly cycle on and off to meet changing demands, and create a significant potential for pressure transients well in excess of a properly designed pumping station pumping to a storage tank. The City’s use of hydroconstant pumps has acted to reduce the effects of pressure transients caused by pumping. However, the potential for pressure surges still exists as long as large portions of the distribution system are pressurized by pumps.

6.1.4.6 PUMP STATION MODIFICATIONS

Section 8 of the Level I study discussed in detail the pumps at each of the City’s pump stations. As part of RBD, Inc.’s analysis, the ability of the pumps to pump to a storage tank serving Zones 3, 4 and 5 was evaluated. Based on modeling studies, it appears that the existing pumps can be modified for this purpose. Typically modifications made to pumps involve adding or removing stages or bowls, and/or trimming the impellers. The purpose of these physical modifications is to change the performance characteristics of the pumps so
that they can operate efficiently.

Currently, the City uses variable speed drives (VSD’s), or similar devices, on several of its pumps. The purpose of VSD’s is to allow a pump to provide constant pressures over a range of flow rates. This allows a pump to meet constantly changing demands. VSD’s are typically used in systems where there is no tank to provide flows and regulate pressures. As with any mechanical system, some energy is lost whenever it is transferred from one system to another. Pumps with VSD’s do not operate as efficiently as fixed speed pumps, and consequently have greater power requirements. Should the City construct additional storage to serve Zones 3, 4 and 5, many of the existing VSD’s could be removed, and the pumps modified to pump to the elevation of the storage tank at a fixed flow rate. Currently, some pumps must run 24 hours per day to meet constantly changing demands. If additional storage is constructed by the City, the pumps would need only pump at the average demand rate for the day.

6.1.4.7 CORThELL HILL

RBD, Inc.’s modeling studies indicate that the Corthell Hill area can experience service pressures of less than 28 psi at street level during periods of peak demand. This finding was corroborated by City staff. In addition, modeling studies indicate that portions of Corthell Hill may not be able to receive adequate fire flows at the minimum pressure of 20 psi during periods of peak demand. The pressure problems on Corthell Hill are primarily due to its high elevation relative to the Hi-Level storage tanks, which serve Zone 2. In addition, the lack of adequate hydraulic conveyance serving the area causes significant pressure losses under even moderate flow conditions. The three existing pipelines serving the area all experience significant pressure loss under Peak Hour demands and fire flow demands, according to model studies performed by RBD, Inc. One alternative to alleviate this situation is to construct additional conveyance. In examining this option, RBD, Inc. sized pipelines to solve the existing flow and pressure problems, as well as to provide conveyance to serve future growth in the area. Various pipeline diameters were modeled under peak hour plus fire flow demands. An alignment was assumed which follows section lines and/or existing pipeline routes. An 18-inch diameter pipeline was the minimum size in the model that would provide adequate service pressures and fire flows to Corthell Hill, as well as serve future growth in the area.

As an alternative to constructing a new pipeline, the incorporation of the Corthell Hill area into pressure Zone 3 was investigated. Elevations in the Corthell Hill area range from approximately 7200 feet to 7240 feet. If Corthell Hill is made part of Zone 3, static pressures could range from approximately 110 psi to 126 psi. In order to maintain more normal pressures, pressure reducing valves (PRV’s) would need to be installed in the pipelines serving Corthell Hill. Corthell Hill’s distribution system is currently served by three connections to the Zone 2 distribution system. By closing isolation valves in two of these lines—the 8-inch line to the west, and the 6-inch line to the northwest—and closing valves in lines in 24th and Garfield Streets, it is possible to feed Corthell Hill from Zone 3. However, this would reduce the number of feeds into Corthell Hill from three pipes to one, an 8-inch
pipeline. It would also mean that water destined for Corthell Hill would be pumped up to the Zone 3 distribution system, only to have its pressure reduced back down before entering the Corthell Hill system. This approach may be worth consideration by the City as a short-term solution to provide adequate pressures and fire flows to Corthell Hill. However, it would reduce the reliability of the system, due to the single feed, and provide no infrastructure to serve future growth.

Another alternative would be to pump water into the Corthell Hill distribution system. One of the three pipelines serving Corthell Hill would pass through a pump station. The other two would either be closed using isolation valves, or have check valves (CV’s) installed. The CV’s would allow water to enter Corthell Hill from Zone 2 if the pump station suffered a breakdown and the pressure in Corthell Hill dropped. One drawback to this approach is that maintenance costs would increase with the addition of another pump station, and system reliability would be reduced. Another concern is that this approach would be a short-term fix in that it would not provide any infrastructure to serve future growth.

6.1.4.8 THE SPUR WELL FIELD

The United States Environmental Protection Agency (EPA) is in the process of formulating new standards for municipal water supplies obtained from deep wells. These standards are not anticipated by the EPA to be finalized before the year 2000. However, discussions with EPA staff revealed that the required chlorine contact time (T) required for disinfection of groundwater supplies is likely to be similar to what is required under existing standards for surface water supplies. Currently, T requirements range from 30 minutes to one hour, depending on such factors as water temperature and turbidity. According to the Level I study, the pipeline proposed to bring the Spur Well water into the City’s distribution system is expected to be 18 inches in diameter, and approximately 27,000 feet long. Based on these parameters, and on the maximum 7-day yield of 4.0 MGD estimated in the Level I study, the minimum time for water to travel from the well field to the distribution system is 1.75 hours. The EPA reports that full credit is given for travel time in a conduit between the point of treatment and the first point of demand, when calculating effective T. Therefore, the proposed 18-inch transmission line should fulfill the EPA standards for T when said standards are promulgated.

In response to the request by the City to assess the remaining capacity in the Reynolds Street line, RBD, Inc. modeled flows under existing and future demands. According to the population projections in the Level I study, approximately 43% of the anticipated growth in and around Laramie is expected to occur in Zone 2. City staff expressed concerns that the 12-inch line in Reynolds would be overburdened by these future demands. This concern is quite valid in that the northern portion of Zone 2 is hydraulically remote from its point of supply, the Hi-Lo Pump Station and the Hi-Level reservoirs. However, once the Spur Well field is developed and a pipeline is constructed to deliver the well water into the distribution system, this area will be fed from two points. This will relieve the Reynolds line because it will no longer be the single primary pipeline supplying the northern portion of Zone 2.
The pipeline serving the Spur Wells was modeled for two different points of connection to the distribution system. The first point is at the intersection of Reynolds and 30th Streets, as recommended in the Level I study. The second is at the intersection of Beaufort and 15th Streets. (See Figure 6-2.) This later point was chosen because of the existing 12-inch line in Beaufort, and because water delivered to this point would more rapidly disperse into the distribution system. According to RBD, Inc.'s model, if the point of connection is at Reynolds and 30th Streets, water velocity in the Reynolds line exceeds 5 fps between 23rd and 30th Streets (approximately 2391 feet of pipeline) under Peak Hour conditions in the year 2020. When the alternate point of connection was modeled under the same conditions, the only pipeline experiencing a velocity in excess of 5 fps in the model was 306 feet of 8-inch line in 15th Street.

City staff have expressed concerns that introduction of the Spur Well flows into the distribution system would reverse flow direction, particularly under fire flow conditions. The impact of this is likely to be minimal. According to modeling studies performed by RBD, Inc., the introduction of flows from the Spur Wells into the northern portion of Zone 2 will increase available fire flows in that area, and improve pressures slightly during periods of high demands. In the case of a fire demand in the northern portion of Zone 2, the high service pumps in the Lo-Side Pump Station would come on line to supply the increased demand in the zone. The hydrants supplying the fire flows would receive water from both the pump station and the Spur Well line. Under conditions of normal demands, customers in the area would still receive water from both sources. The source fraction from the two sources at any point of demand in the system would depend on the location of the point relative to the two sources, and the magnitude of the demand.

6.1.4.9 WATER TREATMENT PLANT PUMP STATION

As discussed previously, the potable water from the City’s water treatment plant (WTP) is delivered to the City via two pipelines; a 20-inch diameter line, and a 24-inch diameter line. Water flows by gravity through these pipelines for approximately 20 miles to the City’s distribution system. Because of the extremely flat topography of the Laramie Plains, the elevation difference between the WTP and the City’s 8.0 MG storage reservoir is only 85 feet. This limits the amount of water that can be delivered through the two pipelines to approximately 7.5 MGD. The City plans to ultimately expand its surface water supply and treatment capacity to 15.75 MGD. In order to deliver this increased flow of water to the City’s distribution system, two alternatives exist; adding additional pipeline capacity, or utilizing pumps to push more water through the existing pipelines. To deliver the additional 8.25 MGD to the City via gravity flow, a 30-inch diameter pipeline would be required, which would cost in excess of $10,000,000. However, a review of the hydraulics of the existing pipelines shows that they are underutilized due to the limitations posed by the flat grade of the pipes. A pump station developing approximately 230 feet of head would be able to pump 15.75 MGD through the existing pipelines. The discharge pressure of the pumps would be approximately 100 psi. The pressure within the 20-inch and 24-inch pipelines would be 100 psi near the WTP, and would attenuate as water flowed towards Laramie. According to model studies performed by RBD, Inc., the pressure in the pipelines at their point of
CITY OF LARAMIE
ALTERNATE POINTS OF CONNECTION FOR SPUR WELL PIPELINE

ZONE 1-2 BOUNDARY

LOCATION PROPOSED IN LEVEL II STUDY

ZONE 2

ZONE 3

LOCATION PROPOSED IN LEVEL I STUDY

SCALE 1"=800'

600 1000

FIGURE 6-2

RBD, Inc.
209 South Meldrum
Fort Collins, Colorado 80521
(970) 482-5922
connection to the City distribution system would be between 60 psi and 70 psi. Pressures in this range should not cause any significant increase in waterline breaks.

6.1.4.10 CONCLUSIONS

The operational analysis of the City of Laramie water system included the evaluation of the following elements:

- Pressure zones
- Additional storage
- Additional conveyance
- The proposed Spur Well field
- Existing pump stations
- The 20-inch and 24-inch transmission lines
- Increased treatment plant capacity

The system was analyzed for its ability to meet system demands for water, while maintaining adequate pressures. In addition, means to maximize operational efficiency were investigated. Demands were calculated using the information given in the Level I study.

6.1.5 PIONEER CANAL

6.1.5.1 CANAL LINING OPTIONS

Lining portions of the canal will reduce conveyance losses in a given reach. The canal is divided into six reaches with each reach between the autumn cross-sections. For this analysis three lining types are considered. The three lining types considered are:

1. High Density Polyethylene (HDPE).
2. Polyvinyl Chloride (PVC).
3. Clay earth.

Each lining type will require clearing and grubbing of the existing channel cross-section. For the HDPE and PVC alternatives, after clearing and grubbing, approximately 12" of the existing canal soil would first be removed, the liner installed, and then the soil placed back on top of the liner. For the clay earth liner, after clearing and grubbing, approximately 24" of the existing canal soil would first be removed, 12" of clay soil installed, 12" of existing canal soil installed, and the remaining existing canal soil disposed of. Refer to Figure 6-3 for typical cross sections of the various canal lining opportunities.
TYPICAL CROSS-SECTIONS -- PIONEER CANAL
LARAMIE WATER SUPPLY PROJECT

FIGURE 6-3
6.1.5.2 INFLUENCES OF CANAL LINING ON GROUNDWATER

As stated above in the introduction, the scope was modified to include research of water wells in the vicinity of the Pioneer Canal. The State Engineers Office Groundwater Division was contacted to provide a tabulation of well permit records. Research shows there are 43 permitted wells in the following Township/Range/Section/Quarter.

1) T14, R76, S19 SWNW 10) T14, R77, S25, SWNE
2) T14, R76, S29 SWSW 11) T14, R77, S26, SESE
3) T14, R76, S30 SWNE 12) T14, R77, S35, NESE
4) T14, R76, S31, NENE 13) T14, R77, S35, NESW
5) T14, R76, S31, NWNE 14) T14, R77, S35, NWSE
6) T14, R76, S31, SWNW 15) T14, R77, S35, NWSE
7) T14, R77, S25, NESE 16) T14, R77, S35, SESW
8) T14, R77, S25, NWSE 17) T14, R77, S35, SWNW
9) T14, R77, S25, SESE

Of the 43 permitted wells, 7 wells are 35 feet or less deep and an additional three wells are abandoned wells owned by the City of Laramie. There are an additional seven wells that have no recorded depth, yield or static water level. Of the remaining 26 wells, ten have a recorded depth of 100 feet or more.

Of the remaining 26 wells, the closest well is approximately 1500 feet from the Pioneer Canal and is about 80 feet higher in elevation. Therefore, lining the Pioneer Canal is expected to have little if any noticeable effect on well performance in the area. However, it is suggested that groundwater models for the existing condition and a lined canal condition be completed at the time of Level III final design to confirm that lining the canal will have little if any noticeable effect on well performance in the area.

6.1.6 APPURTENANCES

No improvements are warranted on the appurtenances observed because no significant deterioration was observed. The following three main pipeline appurtenances were analyzed as a part of this report: the inlet structure on the 36 inch raw water line at the outlet of Sodergreen Reservoir; the valve vault on the 24 inch line adjacent to HWY 230 (sta 977+00); and the vaults at each end of the tunnel under I-80 which contain the 24 inch treated water line and a 20 inch steel water line.
6.2  RECOMMENDED CAPITAL IMPROVEMENTS

6.2.1  36" RAW WATER LINE

6.2.1.1 INLET STRUCTURE

RBD, Inc. recommends that a structural engineer be retained to more fully evaluate the inlet structure during the next scheduled “draining” of Sodergreen Reservoir. Until full access to the interior of the inlet structure is obtained to verify assumptions used herein, the full scope of the project is unobtainable. However, City crews have been inside the inlet structure and have not reported structural problems. In addition, evidence was not found of any problems other than minor surface spalling. Therefore, RBD, Inc. believes the inlet structure is only in need of surface “cosmetic” repairs and possibly minor structural repairs.

When Sodergreen Reservoir is drained and the slide gate and minor structural repairs to the inlet structure are undertaken, RBD, Inc. recommends all of the debris in the inlet structure and the 36" pipeline near Sodergreen Reservoir be removed. The 36" pipeline should be videotaped to the common point with the taping completed under this study. Completion of this section will provide the City with a tape of the entire pipeline (less the small amount at the “fish trap” vault).

Finally, if Sodergreen Reservoir is “drained” to the point where the 36" pipeline in the reservoir is visible, RBD, Inc. recommends dredging out the silt load around the pipe entrance. Although the area will eventually refill with silt, this will significantly reduce the amount of sediment load into the WTP. In addition, installing a 90 degree vertical bend on the end of this pipe will also reduce the amount of silt entering the WTP through the pipeline.

6.2.1.2 NEW RAW WATER PIPELINE TO THE WTP

As discussed in Section 6.1.1- 36" Raw Water Line, RBD, Inc.’s evaluation demonstrated that the existing 36" pipeline does not have the ability to pass the future required hydraulic capacity of 15.75 MGD. In addition, the pipelines structural integrity has been jeopardized due to corrosion. Therefore, RBD, Inc. recommends that the existing pipeline be abandoned in place and be replaced with a new 42" pipeline as soon as possible.

There are many factors that must go into the final design of a new 42" pipeline. Based on RBD, Inc.’s analysis, the following recommendations are offered:

1)  The new pipeline should be constructed on the south side of Hwy. 230 as much as possible. This will greatly assist in the design of the cathodic protection system since interferences with the existing gas line will be minimized. This alignment will require acquisition of temporary construction and permanent access easements. A conceptual layout of this installation is shown in Figure 6-4 in the back pocket of this report.
2) The design must allow for future maintenance. Access manways should be installed with the pipeline at approximately every 2000 feet. Once installed, TV, maintenance and service equipment need only be able to extend 1000 feet.

3) Due to the WTP being constructed “on a mound”, it is not possible to construct a pipeline from Sodergreen Reservoir without a low spot (assuming the pipe has adequate cover to prevent it from freezing and that the cover isn’t provided by mounding dirt over the pipe). This low spot will collect debris and silt and therefore provisions must be made for cleaning and draining the pipe. An access manway should be constructed at the low spot (near the “fish trap” vault) and provisions made to pump silt and debris from this location without shutting down the entire pipeline. A conceptual layout of this installation is shown in Figure 6-4.

4) The City has expressed a desire to modify the in-plant piping to include an in-line mixer. Assuming the piping improvements extend from Sodergreen Reservoir completely to the existing mixing chamber and the mixing chamber is replaced with an in-line mixer, RBD, Inc. recommends modifying the piping to eliminate the existing mixing chamber. In addition, the weir at the mixing chamber which delivers flow to the treatment process should be eliminated. If these losses can be eliminated, it will improve the pipeline hydraulics at very little cost.

5) The existing pipeline has large radius bends. Future pipelines should be installed with the same large radius bends. Not only are they very efficient hydraulically they also allow cameras to track through the pipeline, greatly assisting in obtaining TV data.

6) One of the most common large diameter pipelines is steel. Since the existing pipeline is steel, it has been somewhat assumed that any replacement pipeline would be steel. Although steel has many benefits, it requires some type of protective system to protect it from corrosion. Prior to replacement of any component of the 36" pipeline, an evaluation of inert pipeline materials should be completed.

During development of the recommended capital improvements, considerable discussions were held regarding the costs involved in replacing the entire pipeline. As a result of these discussions, it was decided by the Wyoming Water Development Commission that the project would be completed in two phases. The first phase involves approximately the first 1000' of pipeline from the WTP towards Sodergreen Reservoir. This portion of pipeline was selected since it significantly increases the flow capacity of the system, due to the high head losses from the “fish trap” vault into to WTP, and because the improvements will be compatible with a future replacement/enlargement of the raw water pipeline regardless of whether the source is Sodergreen Reservoir or the Laramie River. The second phase includes replacing the pipeline between the inlet structure and the westerly end of the pipeline completed in Phase 1 as well as rehabilitating the slide gates at the inlet structure.
# 6.2.1.3 COST ESTIMATES

## TABLE 6-2

**Conceptual Design Cost Estimate for 42-inch Pipeline**  
**City of Laramie Water Supply Master Plan, Level II**

<table>
<thead>
<tr>
<th>Description</th>
<th>Unit</th>
<th>Quantity</th>
<th>Unit Price</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Mobilization (10% of CCS)</td>
<td>LS</td>
<td>1</td>
<td>$152,250</td>
<td>$152,250</td>
</tr>
<tr>
<td>2 Pipeline Construction</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a. Rehabilitate Intake Structure</td>
<td>LS</td>
<td>1</td>
<td>$25,000</td>
<td>$25,000</td>
</tr>
<tr>
<td>b. Bore Hwy. 230</td>
<td>LF</td>
<td>120</td>
<td>$400</td>
<td>$48,000</td>
</tr>
<tr>
<td>c. Install 42&quot; Pipe (including fittings &amp; appurt.)</td>
<td>LF</td>
<td>8600</td>
<td>$125</td>
<td>$1,075,000</td>
</tr>
<tr>
<td>d. Construct Access Manholes</td>
<td>EA</td>
<td>5</td>
<td>$6,000</td>
<td>$30,000</td>
</tr>
<tr>
<td>e. In-Plant Appurtenances and Fittings</td>
<td>LS</td>
<td>1</td>
<td>$25,000</td>
<td>$25,000</td>
</tr>
<tr>
<td>f. Cathodic Protection</td>
<td>LS</td>
<td>1</td>
<td>$15,000</td>
<td>$15,000</td>
</tr>
<tr>
<td>3 Unlisted items (10% of above items)</td>
<td>LS</td>
<td>1</td>
<td></td>
<td>$152,250</td>
</tr>
</tbody>
</table>

1 Construction Cost Subtotal (CCS)                                           | $1,522,500 |
2 Engineering Costs (10% of #1)                                              | $152,250   |
3 Subtotal (#1+#2)                                                            | $1,674,750 |
4 Contingency (15% of #3)                                                     | $251,213   |
5 Construction Cost Total (#3+#4)                                             | $1,925,963 |

6 Prepare Final Designs and Specs. (10% of #5)                                 | $288,894   |
7 Permitting and Mitigation                                                   | $10,000    |
8 Legal Fees                                                                  | $10,000    |
9 Acquisition of Access and R.O.W.                                           | $8,000     |
10 Project Cost Total (#5+#6+#7+#8+#9)                                        | $2,242,857 |

*Note: Costs are 1996 based.  
Construction period assumed @ 6 months.*
6.2.2 24 INCH TRANSMISSION LINE

6.2.2.1 CORROSION

As discussed in Section 6.1.2 - "24 Inch Transmission Line", RBD, Inc.'s evaluation demonstrated that pipe replacement should be focused on those areas where leakage has already occurred. In addition, “Hot Spot” cathodic protection should be installed at specific locations where corrosion has been observed or is likely to occur based on soil conditions. Therefore, RBD, Inc. recommends that the following 24" pipeline replacement segments and ‘Hot Spot” cathodic protection segments be constructed as available funding permits.

Replace approximately 3,400 linear feet of 24" pipeline at the following locations:

1. From the east side of the mobile home park located east of the valve vault at HWY 230 to a point approximately 1,000 feet further east of the mobile home park (station 988+20 to station 998+20).

2. From the west side of the Laramie River east approximately 2,400 feet to the east side of Spring Creek located on the Big Horn Lumber Mill property (station 1062+00 to station 1086+00).

Install “Hot Spot” cathodic protection on a total of 4,850 linear feet at the following locations:

1. From a point approximately 250 feet west of Adams Street to a point approximately 1,750 feet west of Adams Street (1,500 feet) (station 1024+00 to station 1039+00).

2. From the east side of Spring Creek located on the Big Horn Lumber Mill property to 2nd & Sanders (approximately 1,350 feet) (station 1087+20 to station 1101+35) Note: Footage and stationing differ due to existing railroad casing pipe not needing cathodic protection.

3. From a point midway between 8th & Springcreek and 9th & Russell to a point midway between 10th & Russell and 11th & Russell (approximately 1,000 feet) (station 1027+00 to station 1037+00).

4. From a point approximately 50 feet east of 15th & Spring Creek to a point 500 feet further east (500 feet) (station 1053+00 to station 1058+00).

5. From the east side of the intersection of Park Ave. & Spring Creek to a point approximately 500 feet west (500 feet) (station 1170+50 to station 1175+50).

Refer to Figure 6-5 for a conceptual layout of each of the pipeline replacement segments as well as the “Hot Spot” cathodic protection segments. Refer to the Technical Appendix L for proposed 24" transmission line and hot spot cathodic protection details.
### 6.2.2.2 COST ESTIMATES

#### TABLE 6-3
Conceptual Design Cost Estimate* for Replacement of Approximately 3,400 Linear Feet of 24-inch DIP Transmission Line
City of Laramie Water Supply Master Plan, Level II

<table>
<thead>
<tr>
<th>Description</th>
<th>Unit</th>
<th>Quantity</th>
<th>Unit Price</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Mobilization (10% of CCS)</td>
<td>LS</td>
<td>1</td>
<td></td>
<td>$35,750</td>
</tr>
<tr>
<td>2 Section A Pipe Replacement**</td>
<td>LF</td>
<td>1000</td>
<td>$65</td>
<td>$65,000</td>
</tr>
<tr>
<td>Furnish and Install 24&quot; DIP</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Connection to Existing System</td>
<td>EA</td>
<td>2</td>
<td>$3,000</td>
<td>$6,000</td>
</tr>
<tr>
<td>3 Section B Pipe Replacement***</td>
<td>LF</td>
<td>2400</td>
<td>$85</td>
<td>$204,000</td>
</tr>
<tr>
<td>Furnish and Install 24&quot; DIP</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Connection to Existing System</td>
<td>LS</td>
<td>2</td>
<td>$3,000</td>
<td>$6,000</td>
</tr>
<tr>
<td>4 Restore Disturbances</td>
<td>AC</td>
<td>5</td>
<td>$1,000</td>
<td>$5,000</td>
</tr>
<tr>
<td>5 Unlisted items (10% of above items)</td>
<td>LS</td>
<td>1</td>
<td></td>
<td>$35,750</td>
</tr>
</tbody>
</table>

1 Construction Cost Subtotal (CCS) $357,500
2 Engineering Costs (10% of #1) $35,750
3 Subtotal (#1+#2) $393,250
4 Contingency (10% of #3) $39,325
5 Construction Cost Total (#3+#4) $432,575
6 Prepare Final Designs and Specs. (10% of #5) $43,258
7 Permitting and Mitigation, Legal Fees, R.O.W. $5,000
8 Project Cost Total (#5+#6+#7) $480,833

*Note: Costs are based on 1996 dollars.
** Assumes replacement of 1,000 linear feet of pipeline located near western end of line.
*** Assumes replacement of 2,400 linear feet of pipeline located from west side of Laramie River to Canal on Big Horn Lumber Mill property.
Construction period assumed @ 6 months.
Table 6-4
Conceptual Design Cost Estimate for “Hot Spot” Cathodic Protection on the 24-inch DIP Transmission Line
City of Laramie Water Supply Master Plan, Level II

<table>
<thead>
<tr>
<th>Description</th>
<th>Unit</th>
<th>Quantity</th>
<th>Unit Price</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Mobilization (5% of CCS)</td>
<td>LS</td>
<td>1</td>
<td></td>
<td>$9,592</td>
</tr>
<tr>
<td>2 Hot Spot Cathodic Protection*</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a. Furnish and install magnesium anodes and joint bonds per joint length of pipe**</td>
<td>EA</td>
<td>280</td>
<td>$600</td>
<td>$168,000</td>
</tr>
<tr>
<td>b. Furnish and install test stations</td>
<td>EA</td>
<td>15</td>
<td>$150</td>
<td>$2,250</td>
</tr>
<tr>
<td>c. Restore Roadway</td>
<td>SY</td>
<td>120</td>
<td>$20</td>
<td>$2,400</td>
</tr>
<tr>
<td>3 Unlisted items (5% of above items)</td>
<td>LS</td>
<td>1</td>
<td></td>
<td>$9,592</td>
</tr>
</tbody>
</table>

| 1 Construction Cost Subtotal (CCS) | $191,833 |
| 2 Engineering Costs (5% of #1) | $9,592   |
| 3 Subtotal (#1+#2) | $201,425 |
| 4 Contingency (10% of #3) | $20,143  |
| 5 Construction Cost Total (#3+#4) | $221,568 |
| 6 Prepare Final Designs and Specs. (5% of #5) | $11,078  |
| 7 Permitting and Mitigation, Legal Fees, R.O.W. | $2,000   |
| 8 Project Cost Total (#5+#6+#7+#8+#9) | $234,646 |

*Note: Costs are 1996 based and assume a total of 4,850 linear feet of "Hot Spot" cathodic protection.
** Assumes existing pipe joint length is 18 feet and that 2 magnesium anodes are required per joint of pipe.
Construction period assumed @ 6 months.
6.2.3 20" TRANSMISSION LINE

As discussed in Section 6.1.3 - “20 Inch Transmission Line”, RBD, Inc.’s evaluation demonstrated that no capital improvements are warranted on this pipeline because the pipe excavations revealed no corrosion problems and the soil sample data does not indicate particularly aggressive soil.

6.2.4 CITY DISTRIBUTION SYSTEM

6.2.4.1 INTRODUCTION

Included below are RBD, Inc.’s recommendations for improvements to the water transmission, distribution and storage (TD&S) system. Construction of these recommended improvements will improve pressures to those areas currently experiencing problems, enhance the reliability of the system, and allow the system to accommodate growth at the pace and in the locations indicated in the Level I study.

6.2.4.2 PRESSURE ZONES

RBD, Inc. recommends that the upper three existing pressure zones (Zones 3 through 5) be combined into one new zone, Zone 3. (See Figure 6-6) The means to accomplish this would include construction of a pipeline to connect the Imperial Heights subdivision with the Indian Hills subdivision, and a new water storage tank to serve the combined zone. Zones 3, 4 and 5 all lie between 7270’ and 7410’ in elevation, a range of just 140 feet. If constructed with an overflow elevation of 7490 feet, the proposed tank would provide a minimum static pressure in Zone 3 of approximately 35 psi at the extreme upper portion of Imperial Heights, and a maximum static pressure of 95 psi at the low end of the Alta Vista subdivision. Although a static pressure of 35 psi is slightly lower than optimal, many portions of the existing distribution system experience pressures in this range during peak demand periods. A large capacity pipeline would ensure that system pressures in Imperial Heights during periods of peak demand would not drop significantly. The static pressure of 95 psi, which is likely at the west end of the Alta Vista subdivision, is somewhat higher than what is found elsewhere in the system, and has raised concerns by City staff that an increase in the frequency of waterline breaks could occur. However, the water distribution systems are typically designed to withstand pressures well in excess of 95 psi, and the surge pressures that occur when the existing pumps turn on and off may exceed this pressure now.

The proposed storage tank for Zone 3 would allow more consistent pressures to be maintained throughout the new zone because the pump stations would only need to pump the day’s average demand to the new tank. During hours where short-term demands exceed the day’s average demand, the tank would supply the difference between the short-term peak rate and the average rate. This is the purpose of equalization storage. Currently, some pumps run 24 hours per day. If a tank is constructed, the pumps would not need to react to sudden changes in system pressures or demands; they would instead pump at a constant rate, and turn on and off based on the water level in the tank. In the event of a fire in Zone 3, the
tank would provide the needed fire flows, and the existing pump stations could be utilized to augment these flows as well as refill the tank once the fire demand ended. In addition, a high incidence of waterline breaks can often be correlated with pressure surges caused by valves opening and closing, and by pumps turning on and off. A phenomenon known as water hammer occurs when the rate of flow in a pipeline changes too quickly, resulting in pressure waves traveling down the pipeline. When these pressure waves encounter any obstruction, such as a valve, a sudden pressure increase occurs, potentially damaging or destroying the valve. Likewise, these pressure waves can cause the pressure in a section of pipe to surge, which can lead to leaks or breaks. The construction of the proposed water tank, and a properly designed system of pumps and valves to fill the tank, will likely reduce such pressure surges, thereby offsetting any adverse effects of increased static pressures.

The boundaries of Zones 1 and 2 would remain unchanged. Although some areas in Zones 1 and 2 experience low pressures during periods of high demand, RBD, Inc. recommends that it would be more practical and cost effective to remedy these deficiencies by constructing additional conveyance, or by other means as discussed later in this report. The following table gives the elevations served by existing Zones 1 and 2 and by the proposed Zone 3:

<table>
<thead>
<tr>
<th>Pressure Zone</th>
<th>Tank Water Surface Elev.</th>
<th>Elevations Served</th>
<th>Range of Static Pressures</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zone 1</td>
<td>7260'</td>
<td>below 7180'</td>
<td>35 psi to 45 psi</td>
</tr>
<tr>
<td>Zone 2</td>
<td>7350'</td>
<td>7180' to 7270'</td>
<td>35 psi to 75 psi</td>
</tr>
<tr>
<td>Zone 3</td>
<td>7490'</td>
<td>7270' to 7400'</td>
<td>35 psi to 95 psi</td>
</tr>
</tbody>
</table>

### 6.2.4.3 ADDITIONAL STORAGE

To serve the projected demands over the next 25 years, a period chosen by the City for the planning of water supply projects, RBD, Inc. recommends that the City construct a water storage tank to serve Zone 3, with an overflow elevation of 7490', and with a capacity of approximately 1.2 MG. According to the City of Laramie Water Supply Master Plan, Level I, Pressure Zones 3 through 5 currently account for approximately 7 percent of the total system demand. Based on a Peak Day demand of 554 gpcd a population of 29,137 (see Section 5 of the Level I study), the Peak Day demand for the entire system in 1995 is estimated as 16.1 MGD, which yields a Peak Day demand for the new Zone 3 of 1.13 MGD. According to the planning information given in the Level I study, much of the projected growth in and around the City of Laramie will occur in the proposed Zone 3. Because of this, it is estimated that Zone 3 will account for approximately 10% of the total system demand in 25 years. Based on a Peak Day demand of 554 gpcd and a population of 37,963 (see Section 5 of the Level I study), the anticipated total Peak Day demand in the year 2020 will be approximately 21.0 MGD, of which 2.1 MGD will be by residents of Zone 3.

The storage requirements for the new Zone 3, discussed previously in this report, are 1.07 MG now, 1.15 MG in the year 2020, and 1.19 MG in the year 2045. Clearly the main
component of the storage for the new Zone 3 is fire flow storage. RBD Inc. recommends that the City construct the storage required to serve projected demands in the year 2045. The difference between the anticipated storage required in 25 years and in 50 years is very marginal in terms of cost, especially due to economies of scale. Large tanks cost less per gallon to construct than small tanks, which means that the cost difference between a 1.1 MG tank and a 1.2 MG tank may be insignificant.

No specific location for the Zone 3 storage tank is proposed herein. Numerous potential sites exist along the ridge east of Laramie, and the ultimate site will need to be determined based on such factors as availability of land, existing development proposals, acquisition of easements for the pipeline, utility conflicts, etc. As long as the overflow elevation of the tank is located at 7490 feet, and it is connected to the distribution systems in Imperial Heights and Indian Hills with pipeline of adequate capacity, the tank can be located on any site having the necessary elevation.

6.2.4.4 ADDITIONAL CONVEYANCES

Section 3.3.3 of this report identified several areas where low system pressures were frequently experienced, and where the distribution system was unable to deliver adequate fire flows under Peak Day conditions. The following recommended improvements to the distribution system are expected to alleviate or eliminate most of the problems identified.

6.2.4.5 PRESSURE ZONE 3

As discussed previously, RBD, Inc. recommends that the City of Laramie construct a pipeline to link the Alta Vista and Indian Hills subdivisions with the Imperial Heights subdivision. One possible alignment for the pipeline, and location for the tank, is shown in Figure 6-7. The pipeline must have adequate hydraulic capacity to carry Peak Day flows plus fire flows to the subdivisions without excessive head loss. The proposed storage tank will be tied into this pipeline somewhere between the two subdivisions. Therefore, in determining the required size of the pipeline we consider it as two separate pipelines; one running from the new tank to Imperial Heights, and one connecting the new tank to Indian Hills. RBD, Inc. has also taken into account the abilities of the existing pump stations to augment fire flows. Because the Imperial Heights Pump Station does not have a fire pump, the section of the proposed pipeline serving Imperial Heights is considered the critical reach. The total demand under fire flow conditions is calculated by adding the Peak Hour system demand, which is estimated to be 300 gpm for Imperial Heights, to the fire flow requirement, which we have assumed to be 4000 gpm. Under Peak Hour conditions the Imperial Heights Pump Station can deliver approximately 1000 gpm to the most distant point in the Imperial Heights subdivision. Therefore the proposed pipeline must be able to deliver 3300 gpm from the new tank with less than 35 feet of head loss (which is equivalent to 15 psi). If the proposed tank is located two miles or less from Imperial Heights, an 18-inch pipe would provide sufficient hydraulic capacity. If the tank is between two and four miles distant from Imperial Heights, a 20-inch pipe would be sufficient. However, since the exact locations future development are unknown, RBD, Inc. recommends that this segment of the pipeline
be 20 inches in diameter so that the system can deliver the needed flows regardless of where development occurs.

The section of the proposed pipeline serving the Alta Vista and Indian Hills subdivisions will not normally have to carry as much fire flow, given the respective capacities of the pump stations serving those areas. However, a minimum diameter of 18 inches is recommended for this section of the proposed pipeline for the following three reasons: 1) The growth projections given in the Level I study are only estimates, and represent a single view of the future. The proposed pipeline should have sufficient capacity to accommodate a reasonable range of growth scenarios. 2) One of the purposes of the proposed storage tank and pipeline is to increase the reliability of the distribution system. The pipeline should be adequately sized to provide Peak Day demands plus fire flows even in the event of a power failure to one of the pump stations. 3) The material costs for a pipeline project are a small percentage of the total project cost. It is therefore much more economical to oversize a pipeline slightly than it is to go back later and build another pipeline to augment system capacity. Final determination of the diameters for both sections of the proposed pipeline should be made after the location of the proposed tank is decided.

6.2.4.6 PUMP STATION MODIFICATIONS

The City of Laramie has requested that all proposed improvements to the City’s water TD&S system utilize existing infrastructure. Therefore, in formulating our recommendations, RBD, Inc. has endeavored to make efficient use of the City’s existing pump stations. RBD, Inc. proposes that, with minor modifications to some of the pumps and the control logic governing their operation, the existing pump stations can not only continue to be used, but can be operated more efficiently than before. The increased efficiency will occur because the pumps will be pumping to maintain the water level in the new tank, rather than to directly meet changing demands. It will be possible to eliminate most of the existing variable speed drives, which will allow the existing pumps to operate more efficiently. High service pumps will likely operate only during fire flow situations. The pumps will need to pump at the average rate of demand for the day, rather than to meet peaks in demand. This will reduce demand charges and operating costs for electrical power.

According to our modeling studies, the existing pumps are capable of pumping the required flows to the overflow elevation of the proposed Zone 3 storage tank under existing peak day demands. In addition, it appears that the pump stations have sufficient capacity to serve the needs of Zone 2 and the proposed Zone 3 under conditions of peak demand at the 25-year horizon, again, without significant modification of the pumps. However, under both these scenarios some of the pumps appear to operate at points on their head/discharge curves that are not at optimal efficiency. RBD, Inc. therefore recommends that simultaneously with constructing the pipeline and storage tank proposed herein, the pumps be further evaluated and modified by the City where necessary so that they operate more efficiently. These modifications are completed by trimming or changing impellers, or by the addition or removal of bowls or stages from the pumps.
Another recommended modification to the pump stations involves the current use of variable speed drives (VSD’s). The purpose of VSD’s is to allow a pump to deliver varying flow rates at a constant pressure, allowing a system with no storage tank to be supplied by pumps without sudden changes in pressures. However, when a system is served directly by a tank, the most efficient mode of operation is to let the tank regulate system pressures, and use the pumps to keep the tank full. Once a storage tank is constructed to serve Zone 3, RBD, Inc. recommends that the City remove the VSD’s on its pumps serving the proposed Zone 3. Because of the inefficiencies inherent in VSD’s, the City should realize some energy savings by removing the VSD’s.

6.2.4.7 CORTHELL HILL

RBD, Inc. recommends that the City construct an 18-inch diameter pipeline from a point of connection near the Hi-Lo Pump Station to the system on Corthell Hill. RBD, Inc. further recommends that the proposed pipeline connect to the dead-end lines at the southeast end of the development in order to loop these lines, thus providing additional system connectivity. (See Figure 6-8.) A pipeline 18 inches in diameter will not only provide sufficient conveyance to provide adequate fire flows and system pressures under existing peak demand conditions, but it will also have additional capacity to serve future development in the area. According to the Level I study, significant development is anticipated in the undeveloped areas around Corthell Hill. An 18-inch pipeline constructed along the alignment shown will serve as a backbone to the distribution system in that area.

6.2.4.8 THE SPUR WELL FIELD

The City of Laramie Water Supply Master Plan, Level I, discussed in detail the development of the Spur Wells as an additional groundwater source for the City. Subsequent to the completion of the Level I study, the City decided to proceed with the development of this new source to meet increasing demands for water. The Level I study included a proposed alignment for the pipeline connecting the well field to the Zone 2 distribution system. RBD, Inc. agrees with the proposal to bring the Spur Well water into Zone 2, but proposes an alternate alignment. Instead of building the pipeline to connect to the distribution system at the intersection of 30th and Reynolds Streets, as shown in the Level I study, RBD, Inc. recommends that the City connect the pipeline to the distribution system at the intersection of Beaufort and 15th Streets. (See Figure 6-2.) This alternate alignment does not cross the proposed Zone 3 at any point near the City, but instead stays in Zone 2. The advantage to this alignment is that the pipeline would be able to directly serve any new developments in Zone 2 north of the City. In addition, as discussed in Section 6.1.4, this alternate alignment will allow the pipeline in Reynolds Street to carry less flow, and therefore prevent it from exceeding its capacity.

Beginning at Beaufort and 15th Streets there is an existing system of 12-inch waterlines in Beaufort, 17th, W. Hill, and 18th Streets, ultimately connecting to the 12-inch line in Reynolds. The existing distribution system elements have the capacity to carry the peak 7-day flows from the Spur Wells, along with the Peak Hour demands projected for the year.
CITY OF LARAMIE
PROPOSED PIPELINE TO CORTHELL HILL

LEGEND
EXISTING PIPELINE
PROPOSED PIPELINE

FIGURE 6-8
RBD, Inc.
209 South Meldrum
Fort Collins, Colorado 80521
(970) 482-5922
2020, without excessive pressure loss. By feeding the Spur Well water to a point in the distribution system which is distant from the high side of the Hi-Lo Pump Station, those portions of the distribution system which are currently remote will be less so. This will act to make pressures throughout Zone 2 more uniform during periods of high demand. In addition, available fire flows in the northern part of Zone 2 will be increased by virtue of the second feed.

6.2.4.9 PRIORITIES

The recommended improvements are given below in prioritized order, with the top priority given first:

Pressure Zone 3:

RBD, Inc. recommends that the City of Laramie combine the existing pressure zones 3, 4 and 5 into one zone, which is referred to herein as Zone 3. To achieve this, RBD, Inc. recommends that the City construct a 1.2 million gallon storage reservoir east of the City, with an overflow elevation of 7490'. In addition, RBD, Inc. recommends that a pipeline be constructed connecting the new tank to the Imperial Heights and Indian Hills subdivisions. The recommended diameter of the pipeline is 18" between the tank and Indian Hills, and 20" between the tank and Imperial Heights.

RBD, Inc. recommends that once construction of the proposed storage reservoir for Zone 3 is completed, along with the connecting pipelines, the City evaluate their pump stations and modify them to operate more efficiently. This will primarily involve modification or replacement of pump impellers, and removal of variable speed drives.

Corthell Hill:

RBD, Inc. recommends that the City construct a pipeline 18" in diameter from the Hi-Lo Pump Station to the terminus of the distribution system on Corthell Hill. This pipeline will alleviate low pressure problems reported there, and ensure that adequate fire flows can be delivered.

Spur Wells:

RBD, Inc. recommends that the City construct the pipeline from the Spur Well Field to a point in the distribution system located at the intersection of Beaufort and 15th Streets. This will improve system pressures in the northern portions of Zone 2, and allow further developments north of Zone 2 to connect directly into the new pipeline. RBD, Inc. further recommends that the 500,000 gallon tank proposed for the Spur Well project in the Level I study be eliminated as it would be unnecessary.
Water Treatment Plant Pump Station:

RBD, Inc. recommends that when the City has increased the capacity of its water treatment plant to an amount greater than what the 20-inch and 24-inch pipelines can deliver by gravity flow, it construct a pump station at the plant to deliver the increased flows through the existing pipelines.

6.2.4.10 COST ESTIMATES

TABLE 6-5
Conceptual Design Cost Estimate for Zone 3 Modifications
City of Laramie Water Supply Master Plan, Level II

<table>
<thead>
<tr>
<th>Description</th>
<th>Unit</th>
<th>Quantity</th>
<th>Unit Price</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Mobilization (10% of CCS)</td>
<td>LS</td>
<td>1</td>
<td>$400,000</td>
<td>$400,000</td>
</tr>
<tr>
<td>2 Storage Tank</td>
<td>LS</td>
<td>1</td>
<td>$400,000</td>
<td>$400,000</td>
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<tr>
<td>3 Transmission Mains</td>
<td>LF</td>
<td>10000</td>
<td>$65</td>
<td>$650,000</td>
</tr>
<tr>
<td>4 SCADA System</td>
<td>LS</td>
<td>1</td>
<td>$25,000</td>
<td>$25,000</td>
</tr>
<tr>
<td>5 Electrical/Mechanical</td>
<td>LS</td>
<td>1</td>
<td>$25,000</td>
<td>$25,000</td>
</tr>
<tr>
<td>6 Pump Station Modifications</td>
<td>LS</td>
<td>1</td>
<td>$25,000</td>
<td>$25,000</td>
</tr>
<tr>
<td>7 Unlisted items (10% of above items)</td>
<td>LS</td>
<td>1</td>
<td>$140,625</td>
<td></td>
</tr>
</tbody>
</table>

| 1 Construction Cost Subtotal (CCS)         |      |          |            | $1,406,250 |
| 2 Engineering Costs (10% of #1)           |      |          |            | $140,625   |
| 3 Subtotal (#1+#2)                         |      |          |            | $1,546,875 |
| 4 Contingency (15% of #3)                  |      |          |            | $232,031   |
| 5 Construction Cost Total (#3+#4)          |      |          |            | $1,778,906 |

| 6 Prepare Final Designs and Specs. (10% of #5) |      |          |            | $266,836   |
| 7 Permitting and Mitigation (2.5% of #5)    |      |          |            | $44,473    |
| 8 Legal Fees (2.5% of #5)                   |      |          |            | $44,473    |
| 9 Acquisition of Access and R.O.W. (2.5% of #5) |      |          |            | $44,473    |
| 10 Project Cost Total (#5+#6+#7+#8+#9)       |      |          |            | $2,179,160 |

*Note: Costs are based on 1996 dollars. Construction period assumed @ 6 months.
<table>
<thead>
<tr>
<th>Description</th>
<th>Unit</th>
<th>Quantity</th>
<th>Unit Price</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Mobilization (10% of CCS)</td>
<td>LS</td>
<td>1</td>
<td></td>
<td>$220,875</td>
</tr>
<tr>
<td>2 Well Field</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a. Drill Pilot Holes 5&quot; (2)</td>
<td>VLF</td>
<td>300</td>
<td>$15</td>
<td>$4,500</td>
</tr>
<tr>
<td>b. Drill 22&quot; Hole (2)</td>
<td>VLF</td>
<td>200</td>
<td>$170</td>
<td>$34,000</td>
</tr>
<tr>
<td>c. Install 16&quot; Steel Casing (2)</td>
<td>VLF</td>
<td>200</td>
<td>$25</td>
<td>$5,000</td>
</tr>
<tr>
<td>d. Cement Grout Cased Interval</td>
<td>EA</td>
<td>2</td>
<td>$7,000</td>
<td>$14,000</td>
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<tr>
<td>e. Drill 12&quot; to Total Well Depth</td>
<td>VLF</td>
<td>400</td>
<td>$70</td>
<td>$28,000</td>
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<tr>
<td>f. Install Test Pumps</td>
<td>EA</td>
<td>2</td>
<td>$4,000</td>
<td>$8,000</td>
</tr>
<tr>
<td>g. Well Production Tests (2)</td>
<td>HR</td>
<td>240</td>
<td>$50</td>
<td>$12,000</td>
</tr>
<tr>
<td>h. Furnish and Install 60 Hp Pump, Motor, Controller and Appurtenances</td>
<td>EA</td>
<td>2</td>
<td>$27,000</td>
<td>$54,000</td>
</tr>
<tr>
<td>i. Furnish and Install 125 Hp Pump, Motor, Controller and Appurtenances</td>
<td>EA</td>
<td>2</td>
<td>$46,000</td>
<td>$92,000</td>
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<tr>
<td>j. Pump House and Appurtenances</td>
<td>EA</td>
<td>2</td>
<td>$40,000</td>
<td>$80,000</td>
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<tr>
<td>k. Power Installation</td>
<td>LS</td>
<td>1</td>
<td>$10,000</td>
<td>$10,000</td>
</tr>
<tr>
<td>3 Pipeline Construction</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a. Furnish and Install 18&quot; PVC Pipe</td>
<td>LF</td>
<td>27000</td>
<td>$40</td>
<td>$1,080,000</td>
</tr>
<tr>
<td>b. Pipe Bedding</td>
<td>CY</td>
<td>4500</td>
<td>$25</td>
<td>$112,500</td>
</tr>
<tr>
<td>c. Road Restoration</td>
<td>SY</td>
<td>250</td>
<td>$20</td>
<td>$5,000</td>
</tr>
<tr>
<td>d. Bore Under 9th St.</td>
<td>LF</td>
<td>100</td>
<td>$130</td>
<td>$13,000</td>
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<tr>
<td>e. Valves and Appurtenances (½ mile spacing)</td>
<td>EA</td>
<td>11</td>
<td>$5,000</td>
<td>$55,000</td>
</tr>
<tr>
<td>f. Connection to Existing System</td>
<td>LS</td>
<td>1</td>
<td>$5,000</td>
<td>$5,000</td>
</tr>
<tr>
<td>4 Chlorination/Fluoridation System</td>
<td>EA</td>
<td>1</td>
<td>$60,000</td>
<td>$60,000</td>
</tr>
<tr>
<td>5 Restore Disturbances</td>
<td>AC</td>
<td>35</td>
<td>$1,000</td>
<td>$35,000</td>
</tr>
<tr>
<td>6 Unlisted items (10% of above items)</td>
<td>LS</td>
<td>1</td>
<td></td>
<td>$213,375</td>
</tr>
<tr>
<td>1 Construction Cost Subtotal (CCS)</td>
<td></td>
<td></td>
<td></td>
<td>$2,133,750</td>
</tr>
<tr>
<td>2 Engineering Costs (10% of #1)</td>
<td></td>
<td></td>
<td></td>
<td>$213,375</td>
</tr>
</tbody>
</table>
3 Subtotal (#1+#2) $2,347,125
4 Contingency (15% of #3) $352,069
5 Construction Cost Total (#3+#4) $2,699,194

6 Prepare Final Designs and Specs. (10% of #5) $404,879
7 Permitting and Mitigation $10,000
8 Legal Fees $10,000
9 Acquisition of Access and R.O.W. $8,000
10 Project Cost Total (#5+#6+#7+#8+#9) $3,132,073

*Note: Costs are based on unit costs in Level I report with 0.5 MG storage tank not included and length of pipeline changed. Construction period assumed @ 6 months.

TABLE 6-7
Conceptual Design Cost Estimate for Correll Hill
City of Laramie Water Supply Master Plan, Level II

<table>
<thead>
<tr>
<th>Description</th>
<th>Unit</th>
<th>Quantity</th>
<th>Unit Price</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Mobilization (10% of CCS)</td>
<td>LS</td>
<td>1</td>
<td></td>
<td>$71,957</td>
</tr>
<tr>
<td>2 18-inch DIP</td>
<td>LF</td>
<td>7,200</td>
<td>$75.00</td>
<td>$540,000</td>
</tr>
<tr>
<td>3 Bore Under Hwy 30</td>
<td>LF</td>
<td>120</td>
<td>$200.00</td>
<td>$24,000</td>
</tr>
<tr>
<td>4 18&quot; x 6&quot; Crosses</td>
<td>EA</td>
<td>5</td>
<td>$1,200.00</td>
<td>$6,000</td>
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<tr>
<td>5 18&quot; x 16&quot; Cross</td>
<td>EA</td>
<td>1</td>
<td>$2,500.00</td>
<td>$2,500</td>
</tr>
<tr>
<td>6 18-inch Manual Butterfly Valve</td>
<td>EA</td>
<td>2</td>
<td>$3,200.00</td>
<td>$6,400</td>
</tr>
<tr>
<td>7 Traffic Control</td>
<td>LS</td>
<td>1</td>
<td>$500.00</td>
<td>$500</td>
</tr>
<tr>
<td>8 Unlisted items (10% of above items)</td>
<td>LS</td>
<td>1</td>
<td>$70,085</td>
<td>$70,085</td>
</tr>
</tbody>
</table>

1 Construction Cost Subtotal (CCS) $719,571
2 Engineering Costs (10% of #1) $71,957
3 Subtotal (#1+#2) $791,528
4 Contingency (15% of #3) $118,729
5 Construction Cost Total (#3+#4) $910,257

6 Prepare Final Designs and Specs. (10% of #5) $136,539
7 Permitting and Mitigation (2.5% of #5) $22,756
8 Legal Fees (2.5% of #5) $22,756
9 Acquisition of Access and R.O.W. (2.5% of #5) $22,756
10 Project Cost Total (#5+#6+#7+#8+#9) $1,115,065

*Note: Costs are based on 1996 dollars. Construction period assumed @ 4 months.
6.2.5 PIONEER CANAL

6.2.5.1 WINTER FLOW STUDY

RBD, Inc. recommends the following five items be incorporated into future winter flow studies:

1) Continue the winter flow study throughout several winter seasons. During the winter of 1995-1996 is seems that performing velocity measurements twice a month was satisfactory. RBD, Inc. recommends the winter flow monitoring program be continued for five additional years. Reference is made to additional winter flow measuring in the costs on Table 6-10.

2) Move the two-foot parshall flume to the center or southeastern side of the 20-foot parshall flume. It is believed that moving the flume to the center or southeastern side of the larger flume will help prevent the flume from filling with snow. The prevailing wind is from the northwest and when the snow is blown into the flume it deposits along the Northwestern side of the flume. It was observed that the southeastern side of the flume was relatively snow free throughout most of the winter.

The sandbag dike and plastic tarp constructed for this study at the flume, appeared to have some minor leakage. Since the purpose of the study was to estimate the flow at the upstream and downstream ends of the canal, a good hydraulic seal as possible at the Parshall Flume is important to prevent bypass flow.

RBD, Inc. recommends that a steel panel dike with a steel stiffener plates be constructed in the throat of the 20-foot flume during the winter flow studies. The arrangement would consist of a steel plate forming the dike with a steel diaphragm "backbone" of the dike. The steel plate is also fitted with a bracket for holding the 2-foot parshall flume during future winter flow studies. The steel panel dike would be fitted with neoprene gaskets to form a water tight seal. Refer to Figure 6-9 for a conceptual layout. A conceptual design cost estimate is provided on Table 6-8.

3) Construct a solar shed over the two-foot flume. Constructing the solar shed will help the flume warm earlier in the day and stay warmer longer into the afternoon.

4) Construct an air or carbon dioxide bubbler for the head pool of the 2-foot parshall flume. The bubbler would be a steel pipe with holes drilled approximately every four inches. A hose attached to the pipe connects to a pressurized bottle of air or carbon dioxide. The gas bubbling to the surface every few seconds will help keep the canal cross-section and head pool free of ice and reduce or eliminate pressure flow. For this arrangement to be effective the bubbler must be started prior to ice forming in the autumn. This technique is useful for preventing ice formation instead of breaking-up ice after formation has begun. Refer to Table 6-8 for costs.
EXISTING 20 FOOT PARSHELL FLUME
PLAN VIEW

1" SCHED. 40 GALVANIZED PIPE
3" (TYP.)
1/16"-1/8" DRILLED HOLE

BUBBLER PIPE DETAIL

TEMPORARILY INSTALLED 2 FOOT FLUME FOR WINTER FLOW MEASUREMENT

VERTICAL STEEL RIBS (TYP.)
STEEL PANEL BOLTED TO FLOOR
FLUME WALL
AIR HOSE TO TANK

FLOOR OF FLUME

STEEL PANEL DIKE

SECTION A

TOP OF 20 FT FLUME WALL (BEYOND)

FLOW

ADJUSTABLE STAND

NOTE: INSULATE 2 FOOT FLUME WITH STRAW BALES

3/8" BOLT
3/8" STEEL PANEL DIKE

FLUME WALL
NEOPRENE GASKET
STRUCTURAL DIAPHRAM

RED HEAD FASTENER (TYP.)
A chainsaw was the most effective, efficient tool for getting through the ice at the winter cross-section. Several techniques for getting through the ice and keeping the ice open were tried during the study. However, because of the open location, cutting through the ice with a chainsaw when the ice is about three inches or greater in thickness seem to be the best solution.

5) During the Autumn Flow Study, there appears to be a difference in the flow reported by Western Water Consultants and the flow estimated by RBD, Inc. at the 20-foot flume. It was observed during the autumn that vegetation had grown on the walls and floor of the flume thus increasing the surface roughness. As the surface roughness increases the discharge is reduced (under the same head). It would be beneficial to accurately measure the flow in the flume throughout the year. Therefore, RBD, Inc. recommends that the 20-foot flume be painted with a product that resists the growth of vegetation in the flume. Perhaps, developing new flow vs. head rating curve should be considered.

6.2.5.2 CANAL LINING

Lining portions of the canal will reduce conveyance losses in a given reach. The canal has been divided into six reaches. As seen on the graph on page 4-4, the autumn flow data shows the variability in conveyance losses between cross-section stations. In order to eliminate conveyance losses and thus eliminate costs to the City of Laramie for conveyance losses as well as maximizing the available water right from the Laramie River, RBD, Inc. recommends that the canal be lined its entire length. However, if the decision is made to line selected sections, the reaches with the highest conveyance losses are recommended for lining first. The Cost Estimate in Table 6-9 is divided by reach to readily identify coss if selected reaches are chosen for lining.

6.2.5.3 INFLUENCES OF CANAL LINING ON GROUNDWATER

RBD, Inc. recommends that further investigation of potential influences between the surface water in the Pioneer Canal and its influence on groundwater be done at the time of the Level III project. A review of the state well permit records show there may be several wells that are various combinations of depth and recorded discharge that further investigation of the interaction of surface water and groundwater are warranted. The investigation should include a hydro geologic investigation to determine if it is possible that the wells and canal are hydraulically connected. If it is determined that there may a potential removal of water source by lining the canal, then a groundwater modeling exercise should be undertaken to further investigate and quantify the effects of lining the canal. RBD, Inc. recommends that two computer models be "built" as follows:

1. With water flowing in the canal in its existing condition.
2. With the canal lined thus removing surface water as a source from the subject wells.

Reference is made to a groundwater study in the costs on Table 6-10.
### 6.2.5.4 COST ESTIMATES

#### TABLE 6-8
Conceptual Design Cost Estimate for Winter Flow Measurement at the 20-foot Parshall Flume
City of Laramie Water Supply Master Plan, Level II

<table>
<thead>
<tr>
<th>Description</th>
<th>Unit</th>
<th>Quantity</th>
<th>Unit Price</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Mobilization (10% of CCS)</td>
<td>LS</td>
<td>1</td>
<td>$2,100</td>
<td>$2,100</td>
</tr>
<tr>
<td>2 Steel Panel Dike, Gaskets and Fittings</td>
<td>LS</td>
<td>1</td>
<td>$10,000</td>
<td>$10,000</td>
</tr>
<tr>
<td>3 Adjustable Stand for 2-foot Flume</td>
<td>Each</td>
<td>1</td>
<td>$200</td>
<td>$200</td>
</tr>
<tr>
<td>4 1-inch Pipe Bubbler Manifold</td>
<td>Each</td>
<td>1</td>
<td>$500</td>
<td>$500</td>
</tr>
<tr>
<td>5 500 Lb Bubbler Tank and regulator</td>
<td>Each</td>
<td>1</td>
<td>$5,000</td>
<td>$5,000</td>
</tr>
<tr>
<td>6 Annual Installation and removal including straw insulation for 2-foot flume</td>
<td>LS</td>
<td>1</td>
<td>$2,000</td>
<td>$2,000</td>
</tr>
<tr>
<td>7 Annual filling of Air Tank</td>
<td>LS</td>
<td>1</td>
<td>$200</td>
<td>$200</td>
</tr>
<tr>
<td>8 Plastic Tarp House over 2-foot flume</td>
<td>LS</td>
<td>1</td>
<td>$1,000</td>
<td>$1,000</td>
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<td>1 Construction Cost Subtotal (CCS)</td>
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<tr>
<td>2 Engineering Costs (10% of #1)</td>
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<td></td>
<td>$2,100</td>
</tr>
<tr>
<td>3 Subtotal (#1+#2)</td>
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<td>4 Contingency (15% of #3)</td>
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<tr>
<td>8 Legal Fees (2.5% of #5)</td>
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<td>$664</td>
</tr>
<tr>
<td>9 Acquisition of Access and R.O.W. (2.5% of #5)</td>
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<td></td>
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<td>$664</td>
</tr>
<tr>
<td>10 Project Cost Total (#5+#6+#7+#8+#9)</td>
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<td></td>
<td></td>
<td>$32,542</td>
</tr>
</tbody>
</table>

Note: Costs are based on 1996 dollars.
# TABLE 6-9
Conceptual Design Cost Estimate for Lining the Pioneer Canal
City of Laramie Water Supply Master Plan, Level II

<table>
<thead>
<tr>
<th>Description</th>
<th>Unit</th>
<th>Quantity</th>
<th>Unit Price</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Mobilization (10% of CCS)</td>
<td>LS</td>
<td>1</td>
<td>$5,000</td>
<td>$5,000</td>
</tr>
<tr>
<td>2 Reach A PVC Lining</td>
<td>SY</td>
<td>9956</td>
<td>$7.50</td>
<td>$75,959</td>
</tr>
<tr>
<td>3 Reach B PVC Lining</td>
<td>SY</td>
<td>11733</td>
<td>$7.50</td>
<td>$88,000</td>
</tr>
<tr>
<td>4 Reach C PVC Lining</td>
<td>SY</td>
<td>11733</td>
<td>$7.50</td>
<td>$88,000</td>
</tr>
<tr>
<td>5 Reach D PVC Lining</td>
<td>SY</td>
<td>11733</td>
<td>$7.50</td>
<td>$88,000</td>
</tr>
<tr>
<td>6 Reach E PVC Lining</td>
<td>SY</td>
<td>11884</td>
<td>$7.50</td>
<td>$89,133</td>
</tr>
<tr>
<td>7 Reach F PVC Lining</td>
<td>SY</td>
<td>11582</td>
<td>$7.50</td>
<td>$88,867</td>
</tr>
<tr>
<td>8 Clearing and Grubbing Reach A</td>
<td>LF</td>
<td>2240</td>
<td>$5</td>
<td>$11,200</td>
</tr>
<tr>
<td>9 Clearing and Grubbing Reach B</td>
<td>LF</td>
<td>2640</td>
<td>$5</td>
<td>$13,200</td>
</tr>
<tr>
<td>10 Clearing and Grubbing Reach C</td>
<td>LF</td>
<td>2640</td>
<td>$5</td>
<td>$13,200</td>
</tr>
<tr>
<td>11 Clearing and Grubbing Reach D</td>
<td>LF</td>
<td>2640</td>
<td>$5</td>
<td>$13,200</td>
</tr>
<tr>
<td>12 Clearing and Grubbing Reach E</td>
<td>LF</td>
<td>2674</td>
<td>$5</td>
<td>$13,370</td>
</tr>
<tr>
<td>13 Clearing and Grubbing Reach F</td>
<td>LF</td>
<td>2606</td>
<td>$5</td>
<td>$13,030</td>
</tr>
<tr>
<td>14 Grading Reach A</td>
<td>LF</td>
<td>2240</td>
<td>$4</td>
<td>$8,960</td>
</tr>
<tr>
<td>15 Grading Reach B</td>
<td>LF</td>
<td>2640</td>
<td>$4</td>
<td>$10,560</td>
</tr>
<tr>
<td>16 Grading Reach C</td>
<td>LF</td>
<td>2640</td>
<td>$4</td>
<td>$10,560</td>
</tr>
<tr>
<td>17 Grading Reach D</td>
<td>LF</td>
<td>2640</td>
<td>$4</td>
<td>$10,560</td>
</tr>
<tr>
<td>18 Grading Reach E</td>
<td>LF</td>
<td>2674</td>
<td>$4</td>
<td>$10,696</td>
</tr>
<tr>
<td>19 Grading Reach F</td>
<td>LF</td>
<td>2606</td>
<td>$4</td>
<td>$10,424</td>
</tr>
<tr>
<td>20 Seeding and Mulching Reach A</td>
<td>LS</td>
<td>1</td>
<td>$5,000</td>
<td>$5,000</td>
</tr>
<tr>
<td>21 Seeding and Mulching Reach B</td>
<td>LS</td>
<td>1</td>
<td>$5,000</td>
<td>$5,000</td>
</tr>
<tr>
<td>22 Seeding and Mulching Reach C</td>
<td>LS</td>
<td>1</td>
<td>$5,000</td>
<td>$5,000</td>
</tr>
<tr>
<td>23 Seeding and Mulching Reach D</td>
<td>LS</td>
<td>1</td>
<td>$5,000</td>
<td>$5,000</td>
</tr>
<tr>
<td>24 Seeding and Mulching Reach E</td>
<td>LS</td>
<td>1</td>
<td>$5,000</td>
<td>$5,000</td>
</tr>
<tr>
<td>25 Seeding and Mulching Reach F</td>
<td>LS</td>
<td>1</td>
<td>$5,000</td>
<td>$5,000</td>
</tr>
</tbody>
</table>

1 Construction Cost Subtotals (CCS)         | $759,585|
2 Engineering Costs (10% of #1)             | $75,959 |
3 Subtotal (#1+#2)                          | $835,544|
4 Contingency (15% of #3)                   | $125,332|
5 Construction Cost Total (#3+#4)           | $960,875|

6 Prepare Final Designs and Specs. (10% of #5) | $144,131|
7 Permitting and Mitigation (2.5% of #5)   | $24,022 |
8 Legal Fees (2.5% of #5)                   | $24,022 |
9 Acquisition of Access and R.O.W. (2.5% of #5)| $24,022 |
10 Project Cost Total (#5+#6+#7+#8+#9)      | $1,177,072|
Costs are based on 1996 dollars.
Cost of PVC Material provided by Bowman Supply, Denver

### TABLE 6-10

Conceptual Design Cost Estimate for Winter Flow Study and Groundwater Study
City of Laramie Water Supply Master Plan, Level II

<table>
<thead>
<tr>
<th>Description</th>
<th>Unit</th>
<th>Quantity</th>
<th>Unit Price</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Mobilization (10% of CCS)</td>
<td>LS</td>
<td>1</td>
<td></td>
<td>$7,522</td>
</tr>
<tr>
<td>2 Winter Flow Study (per year)</td>
<td>EACH</td>
<td>5</td>
<td>$10,000</td>
<td>$50,000</td>
</tr>
<tr>
<td>3 Groundwater Study</td>
<td>LS</td>
<td>1</td>
<td>$15,000</td>
<td>$15,000</td>
</tr>
<tr>
<td>4 20-foot Flume Rating Study</td>
<td>EACH</td>
<td>1</td>
<td>$2,500</td>
<td>$2,500</td>
</tr>
<tr>
<td>5 Small Chainsaw</td>
<td>EACH</td>
<td>1</td>
<td>$200</td>
<td>$200</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Description</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Construction Cost Subtotal (CCS)</td>
<td>$75,222</td>
</tr>
<tr>
<td>2 Engineering Costs (10% of #1)</td>
<td>$7,522</td>
</tr>
<tr>
<td>3 Subtotal (#1+#2)</td>
<td>$82,744</td>
</tr>
<tr>
<td>4 Contingency (15% of #3)</td>
<td>$12,412</td>
</tr>
<tr>
<td>5 <strong>Construction Cost Total (#3+#4)</strong></td>
<td>$95,156</td>
</tr>
<tr>
<td>6 Prepare Final Designs and Specs. (10% of #5)</td>
<td>$14,273</td>
</tr>
<tr>
<td>7 Permitting and Mitigation (2.5% of #5)</td>
<td>$2,379</td>
</tr>
<tr>
<td>8 Legal Fees (2.5% of #5)</td>
<td>$2,379</td>
</tr>
<tr>
<td>9 Acquisition of Access and R.O.W. (2.5% of #5)</td>
<td>$2,379</td>
</tr>
<tr>
<td>10 <strong>Project Cost Total (#5+#6+#7+#8+#9)</strong></td>
<td>$116,566</td>
</tr>
</tbody>
</table>

Note: Costs are based on 1996 dollars.

### 6.2.6 APPURTENANCES

As discussed in Section 6.1.6 - "Appurtenances", RBD, Inc.'s evaluation demonstrated that no capital improvements are warranted on the three appurtenances analyzed because the corrosion evaluation revealed no corrosion problems.