2.0 Plant Size and Treatment Process

2.1 Introduction

This section of the Second Plant Feasibility Study addresses sizing of the second plant by drawing on the demand projections and reviewing factors which affect the decision about plant size. Available water quality information from locations in the vicinity of potential plant sites is discussed in relationship to the requirements for treated water and potential treatment processes for the second plant. While water quality data are limited, they are used to define a water treatment process train for the new plant. Two options for plant treatment processes are evaluated and a recommendation is included for the treatment processes in the proposed new plant.

2.2 Existing Water Treatment Plant Capacity

Originally constructed in 1971, with additions in 1974 and 1998, the Gary Roberts Water Treatment Plant (GRWTP) is a conventional plant located below the Goose Pasture Tarn Reservoir. The plant expansions brought the total design rated capacity of the GRWTP to 5 MGD. The Town operated a second potable water treatment plant located at Peak 7 until 2004 when it was taken off line, removing 0.5 MGD of production capacity from the system.

While the GRWTP is rated to produce 5 MGD, operators have indicated that the plant can dependably produce 4 MGD year around. A difference in operational versus rated capacity is not unusual for water treatment plants of all sizes. Raw water variability and the age of the plant can impact the ability of a plant to meet all current regulations under high flow conditions. A portion of the Breckenridge water plant was designed and constructed 40 years ago when drinking water quality regulations were quite different than they are today. In spite of the age of part of the plant, it consistently produces water meeting current drinking water regulations to meet current demand requirements. Operating at a flow rate below the rated capacity may facilitate the ability to produce excellent drinking water, particularly under spring runoff water conditions. A detailed review of the GRWTP would be prudent to determine improvements necessary to ensure that it can produce water over the next 20 to 30 years at the design flow rate of 5 MGD.

2.3 Second Plant Capacity

Determination of the size of a future second water treatment plant is dependent on the following:

- The projected future water demand, which was developed at length in the previous section.
- The dependable year-around capacity of the existing water treatment plant
- The need for treatment redundancy, which impacts system flexibility and the ability to make repairs and improvements at both treatment plants
- Plans for incorporating new customers into the water service area
• The Town's philosophy on management of risk with respect to providing drinking water

The projections for future water demand (see Figure 1-12 in the previous section) show that if the system is expanded as proposed, the water demand at the 90 percent confidence limit (10% exceedance) will exceed 5 MGD by 2014 or 2015 and at the lower 75 percent confidence limit (25% exceedance), the demand is projected to exceed 5 MGD in 2022. Typically, to minimize the risk of not having enough treated water, utilities plan for meeting the 90 percent confidence limit of the demand projection.

Assuming the Roberts WTP can provide a firm 5 MGD in the future, the capacity requirements for a second plant are summarized in Table 2-1. The values in the table are based on the Monte Carlo simulation predicting peak day demand probabilities over time developed in the previous section. The 10 percent exceedance curve was utilized for this analysis, which means that the demand has a 10 percent chance of exceeding the value shown in the table as the peak day demand. Most water utilities elect to plan based on the projected 10 percent exceedance curve to minimize the risk of having insufficient water.

<table>
<thead>
<tr>
<th>Year</th>
<th>Peak Day Demand Projection ¹ (MGD)</th>
<th>Roberts WTP Firm Capacity (MGD)</th>
<th>Firm Capacity Required at Second WTP (MGD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2020</td>
<td>5.57</td>
<td>5</td>
<td>0.57</td>
</tr>
<tr>
<td>2025</td>
<td>6.23</td>
<td>5</td>
<td>1.23</td>
</tr>
<tr>
<td>2030</td>
<td>7.06</td>
<td>5</td>
<td>2.06</td>
</tr>
<tr>
<td>2040</td>
<td>9.27</td>
<td>5</td>
<td>4.27</td>
</tr>
</tbody>
</table>

¹. Projected peak demand with 10% chance that the demand will exceed this level.

In planning for water treatment expansion, the Town should be planning additional treatment capacity when the demand reaches 80% of the current production capacity. Nominally, the Roberts WTP can currently produce 5.0 MGD, so the Town should be planning additional capacity when the peak day demand reaches 80% of 5.0 MGD, or 4.0 MGD. Similarly, new plant capacity should be planned to serve 80 percent of the allowable SFE’s to account for fluctuations in daily transient population and water use. Table 2-2 shows both the water and planning SFE’s that could be expected to be served with new plant capacity. One planning SFE equates to 1.4 water SFE’s.

<table>
<thead>
<tr>
<th>Additional Treatment Plant Capacity</th>
<th>Water SFE’s Added to the System</th>
<th>Planning SFE’s Added</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5 MGD</td>
<td>2607</td>
<td>1862</td>
</tr>
<tr>
<td>3.0 MGD</td>
<td>5228</td>
<td>3734</td>
</tr>
</tbody>
</table>
The plant could be constructed in stages composed of 1.5 MGD each. This approach would minimize the initial cost for the plant construction, since only a single treatment train would be constructed at the outset. However, some parts of the plant would be designed to hold equipment adequate to treat the build-out capacity. The chemical feed equipment spaces, the rapid mix, and the raw water piping would be designed to meet the build-out condition because the cost of reconstructing these elements later would waste significant Town resources. In addition, the property acquired would need to be adequate for ultimate plant expansion.

Constructing a 3 MGD plant has a number of advantages to the Town:

- By building the first two treatment trains simultaneously, the plant will have internal operational flexibility that would not exist with a single treatment train, particularly when the Roberts WTP is off line or when demands peak under water quality conditions where the water is more difficult to treat (during runoff).
- A 3 MGD plant provides flexibility in carrying the demand load while improvements to the existing plant are completed in the future. Because the “off season” periods in the Town are becoming shorter, the time periods when part of the Roberts WTP can be off line are also shortened. Thus, upgrades and improvements will become more difficult and likely more costly as the construction periods are shortened to only a month or two.
- Having 3 MGD capacity at the second plant allows for operational flexibility and the opportunity to have a second source of supply in the case of a wild fire in the Upper Blue River watershed.
- If water availability at the Tarn limits the Roberts WTP production to less than 5 MGD during extreme drought, then a plant that draws water from the lower end of the Blue River may be able to more reliably provide water to the Town. The Town might then be able to bypass water through Town to ease the stress of the drought on the river section through Town. In this instance, 3 MGD treatment capacity at the second plant provides additional flexibility for drought conditions.
- Construction of a 3 MGD plant can allow for maximizing the Town’s water rights, particularly if the new water plant intake is located below the Blue River Gauge near Dillon. Based on discussions with Tom Williamsen, the combination of a 3 MGD second plant and the location of the intake on this stretch of the River have significant advantages for flexibility of water storage in the Tarn. Water taken from below the Blue River Gauge near Dillon must be augmented at 5 percent. If water production from the second plant is maximized during periods when storage is desired, the overall water requirement is reduced.

The plant footprint discussed later in this section is based on 3 MGD production capacity with two 1.5 MGD treatment trains. Cost estimates are provided in Section 5.1 for a 1.5 MGD plant and for a 3 MGD plant.

The footprint also shows a layout that could incorporate an additional treatment train for future expansion to 4.5 MGD. The projection indicates that 4.5 MGD would be needed by 2034. Revisiting the demand projection in ten years using actual growth rates is
recommended to confirm the timing of additional capacity. Expansion to this capacity is carried through this study to provide the flexibility to expand on the proposed new site. Eventually, the Town may want the ability to utilize the new plant as a base-loading plant, particularly if there are water rights benefits associated with doing so.

2.4 Water Quality Data

Based on discussions with the Town staff and with Tom Williamson regarding water rights, source water for the second water plant is best taken from the Blue River from a location below the Blue River Gauge near Dillon on the River. This gauge is located at the Swan Mountain Road bridge over the Blue River, so is sometimes referred to as the Swan Mountain Road gauge. The Town sampled the water at both the Blue River Gauge at Highway 9 and at the Blue River Gauge near Dillon. In addition, samples were taken from the piezometers on the McCain property, even though that property is located well above the Blue River Gauge near Dillon along the river. Some consideration was given to trying to utilize water from beneath the McCain property because the Town owns the property and installing a collection system for the water is possible. Samples were also taken from two wells located near the lower end of the valley with the objective of understanding the quality of potential future ground water wells near potential plant sites in that area. The sample sites are shown on the map in Figure 2-1.

Water quality data resulting from the sampling is presented in Table 2-3. Parameters tested included a range of metals that have sometimes been present in high amounts in local water sources due to the historic mining in the area, alkalinity, fluoride, and total organic carbon. Sample results that indicate the water would require treatment, because they are near or over the MCL, are highlighted in grey.

The results indicate that the water in the pool under the McCain site would not be desirable for a drinking water source. Parameters at the McCain site that are elevated enough to require specific treatment for removal include arsenic, barium, beryllium, cadmium, chromium, iron and manganese and total organic carbon. The combination of these high parameters would make a treatment plant utilizing this water highly complex, very expensive and difficult to operate, so the McCain site water was eliminated as an option based on quality as well as on water rights concerns.

Sampling results from both the Blue River Gauge at Highway 9 and the Blue River Gauge near Dillon show high quality water that would be excellent for a drinking water source. Sufficient alkalinity is present to allow for coagulation for particulate removal and high levels of metals are not present so unusual treatment processes would not be required. These water samples were taken while the Iowa Hill Wastewater Treatment Plant was offline, so the quality could shift somewhat when that WTP is operational and discharging treated wastewater to the Blue River above the sample locations. Based on the available results, locating a water treatment plant intake on the Blue River at any location below the Blue River Gauge at Highway 9 appears to be a wise approach from a water quality standpoint. No treatment plant should be designed based on a single water quality sample, so additional sampling would be required prior to the Town moving ahead with
preliminary design. For design approval, the State requires sampling at the intake location and analyzing for the entire suite of National Primary Drinking Water Standards. Good practice argues for sampling several times in each season to capture the variability of the water supply characteristics and ensure that the design can address typical raw water conditions.

Because surface water quality could be impacted by the aftermath of a wildfire at the Roberts WTP or even at a second WTP which takes water from the Blue River, the availability of adequate ground water in the area of the potential second water plant sites is of interest. In order to get an idea of water quality of local ground water sources, the Town sampled two private wells, one located at DnR Kennels and one at the Veterinary Hospital across Swan Mountain Road from the wastewater treatment plant. The sample results are presented in the two right columns of the table. Water quality in both wells is excellent, although the well on the DnR Kennel site has elevated iron which would require removal in treatment and it also contains relatively high alkalinity. This implies high hardness as well, although hardness was not directly measured. In general, the well results indicate that ground water available in the area is suitable for drinking water use. Having a supply of well water near the second treatment plant would provide a supplemental source that could be utilized in conjunction with treated surface water or alone in the event that the surface water in the Blue River was adversely impacted by wildfire. Additional discussion of the impacts of wildfire on surface supplies can be found in section 7 of this study.
Figure 2-1: Map of Water Quality Sample Sites in Breckenridge
Table 2-3: Source Water Parameters at Several Locations in Breckenridge

<table>
<thead>
<tr>
<th>Parameter</th>
<th>MCL or SMCL (mg/L)</th>
<th>Blue River Gauge at Highway 9 Gold Hill on Blue River</th>
<th>Blue River Gauge near Dillon Gauge on Blue River</th>
<th>McCain N.E. Piezometer</th>
<th>McCain N.W. Piezometer</th>
<th>Well at 16152 HWY 9 Vet Hospital</th>
<th>Well at DnR Kennels HWY 9</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample Date</td>
<td>2/7/2013</td>
<td>2/7/2013</td>
<td>2/7/2013</td>
<td>3/12/2013</td>
<td>3/12/2013</td>
<td>3/12/2013</td>
<td>3/12/2013</td>
</tr>
<tr>
<td>Antimony</td>
<td>0.0060</td>
<td>&lt;0.00080</td>
<td>&lt;0.00080</td>
<td>&lt;0.00080</td>
<td>0.0029</td>
<td>&lt;0.00080</td>
<td>&lt;0.00080</td>
</tr>
<tr>
<td>Arsenic</td>
<td>0.010</td>
<td>&lt;0.0016</td>
<td>0.0016</td>
<td>0.0088</td>
<td>0.012</td>
<td>0.13</td>
<td>&lt;0.0021</td>
</tr>
<tr>
<td>Barium</td>
<td>2.0</td>
<td>0.065</td>
<td>0.057</td>
<td>1.2</td>
<td>0.46</td>
<td>2.7</td>
<td>0.32</td>
</tr>
<tr>
<td>Beryllium</td>
<td>0.0040</td>
<td>&lt;0.00040</td>
<td>&lt;0.00040</td>
<td>0.0022</td>
<td>0.012</td>
<td>0.0046</td>
<td>0.00059</td>
</tr>
<tr>
<td>Cadmium</td>
<td>0.0050</td>
<td>0.0022</td>
<td>0.0020</td>
<td>0.0012</td>
<td>0.0066</td>
<td>0.33</td>
<td>0.00033</td>
</tr>
<tr>
<td>Chromium(4)</td>
<td>0.10</td>
<td>&lt;0.0040</td>
<td>0.069</td>
<td>0.038</td>
<td>0.095</td>
<td>0.030</td>
<td>&lt;0.0040</td>
</tr>
<tr>
<td>Iron(2)</td>
<td>0.3</td>
<td>&lt;0.080</td>
<td>&lt;0.080</td>
<td>42.6</td>
<td>32.5</td>
<td>270</td>
<td>14.7</td>
</tr>
<tr>
<td>Manganese(2)</td>
<td>0.05</td>
<td>&lt;0.0020</td>
<td>&lt;0.0020</td>
<td>1.5</td>
<td>1.5</td>
<td>44.9</td>
<td>0.42</td>
</tr>
<tr>
<td>Mercury</td>
<td>0.0020</td>
<td>&lt;0.0010</td>
<td>&lt;0.0010</td>
<td>&lt;0.0010</td>
<td>&lt;0.0010</td>
<td>&lt;0.0010</td>
<td>&lt;0.0010</td>
</tr>
<tr>
<td>Nickel</td>
<td>&lt;0.0040</td>
<td>&lt;0.0040</td>
<td>0.052</td>
<td>0.037</td>
<td>0.66</td>
<td>0.022</td>
<td>&lt;0.0040</td>
</tr>
<tr>
<td>Selenium</td>
<td>0.050</td>
<td>&lt;0.00080</td>
<td>&lt;0.00080</td>
<td>0.0022</td>
<td>0.0073</td>
<td>&lt;0.00080</td>
<td>&lt;0.00080</td>
</tr>
<tr>
<td>Sodium</td>
<td>4.9</td>
<td>4.5</td>
<td>5.5</td>
<td>4.1</td>
<td>3.8</td>
<td>5.3</td>
<td>5.1</td>
</tr>
<tr>
<td>Thallium</td>
<td>0.0020</td>
<td>&lt;0.00040</td>
<td>&lt;0.00040</td>
<td>&lt;0.00053</td>
<td>&lt;0.00040</td>
<td>0.001</td>
<td>&lt;0.00040</td>
</tr>
<tr>
<td>Alkalinity</td>
<td>53.9</td>
<td>57.7</td>
<td>64.4</td>
<td>53.8</td>
<td>61</td>
<td>63.3</td>
<td>59.3</td>
</tr>
<tr>
<td>Cyanide</td>
<td>0.20</td>
<td>0.0050</td>
<td>&lt;0.005</td>
<td>0.04</td>
<td>&lt;0.005</td>
<td>&lt;0.0050</td>
<td>&lt;0.0050</td>
</tr>
<tr>
<td>Fluoride</td>
<td>4.0</td>
<td>&lt;0.10</td>
<td>&lt;0.10</td>
<td>0.04</td>
<td>0.11</td>
<td>0.04</td>
<td>&lt;0.10</td>
</tr>
<tr>
<td>Total Organic Carbon(3)</td>
<td>&lt;0.50</td>
<td>&lt;0.50</td>
<td>0.63</td>
<td>303</td>
<td>8.4</td>
<td>0.61</td>
<td>&lt;0.50</td>
</tr>
</tbody>
</table>

(1) MCL = Maximum Contaminant Level; SMCL = Secondary Maximum Contaminant Level. If left blank, no MCL or SMCL exists.
(2) Parameters with a SMCL.
(3) A regulatory treatment technique and disinfection byproduct limits create an operational target for TOC of less than 2 mg/L in finished water.
(4) MCL is for total chromium. New MCLs are expected for chromium (III) and chromium(VI) in the near future.
As follow-up to the initial sampling results shown in Table 2-2, the Town arranged to take raw water samples during the spring 2013 runoff period near the Upper Blue Sanitation District diversion on the Blue River. The diversion is located just downstream from the Blue River Gauge near Dillon, so the water quality is representative of conditions between the gauge and Dillon Reservoir on the river. The results of this sampling are shown in Table 2-3.

Water quality during runoff at this location does not differ significantly from the water quality at the Tarn. Turbidities at the Tarn during this period ranged from 2.99 to 8.16 NTU, as compared to the range of 2.17 to 11.8 NTU on the river. The data do show somewhat higher levels of dissolved iron and manganese than the single sample taken in February, but the levels are not outside the range that is readily removed in treatment. The organic carbon in the water should be removed through coagulation as water with a SUVA number above 2 typically responds well to coagulation for removal of TOC.

Algae and diatom counts for the raw water at the diversion are also within the range experienced at the Tarn, as illustrated in Figure 2-2. The Roberts WTP consistently removes algae and diatoms to meet regulatory requirements, so similar treatment technology in the proposed new plant can be expected to remove them as well.

### Table 2-4: Source Water Quality at Upper Blue Sanitation District Diversion

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Turbidity</td>
<td>NTU</td>
<td>2.17</td>
<td>6.19</td>
<td>11.8</td>
<td></td>
</tr>
<tr>
<td>pH</td>
<td></td>
<td>8</td>
<td>7.9</td>
<td>7.78</td>
<td></td>
</tr>
<tr>
<td>Flow at Blue River near Dillon gauge</td>
<td>cfs</td>
<td>287</td>
<td>336</td>
<td>560</td>
<td></td>
</tr>
<tr>
<td>Total Alkalinity</td>
<td>mg/L as CaCO₃</td>
<td>43.3</td>
<td>42.3</td>
<td>38.6</td>
<td></td>
</tr>
<tr>
<td>Total Organic Carbon</td>
<td>mg/L</td>
<td>2.6</td>
<td>2.6</td>
<td>2.6</td>
<td></td>
</tr>
<tr>
<td>Dissolved Organic Carbon</td>
<td>mg/L</td>
<td>2.4</td>
<td>2.5</td>
<td>2.5</td>
<td></td>
</tr>
<tr>
<td>SUVA, Specific UV Absorbance at 254 nm⁻¹</td>
<td>L/mg-m</td>
<td>2.88</td>
<td>2.72</td>
<td>3.00</td>
<td></td>
</tr>
<tr>
<td>UV Absorbance at 254 nm</td>
<td>m⁻¹</td>
<td>6.9</td>
<td>6.8</td>
<td>7.5</td>
<td></td>
</tr>
<tr>
<td>Total Iron</td>
<td>mg/L</td>
<td>0.209</td>
<td>0.44</td>
<td>0.98</td>
<td></td>
</tr>
<tr>
<td>Dissolved Iron</td>
<td>mg/L</td>
<td>0.0167</td>
<td>0.033</td>
<td>0.027</td>
<td></td>
</tr>
<tr>
<td>Total Manganese</td>
<td>mg/L</td>
<td>0.0166</td>
<td>0.047</td>
<td>0.091</td>
<td></td>
</tr>
<tr>
<td>Dissolved Manganese</td>
<td>mg/L</td>
<td>0.0086</td>
<td>0.0076</td>
<td>0.0089</td>
<td></td>
</tr>
<tr>
<td>Nondiatomaceous Algae</td>
<td>#/100L</td>
<td>1362581</td>
<td>34589189</td>
<td>26902703</td>
<td></td>
</tr>
<tr>
<td>Diatoms</td>
<td>#/100L</td>
<td>31745192</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Note 1:** Calculated as UV abs. at 254 nm/Dissolved Organic Carbon
From the perspective of water quality and water rights, placing an intake on the river between Swan Mountain Road and the confluence with Dillon Reservoir has significant advantages and the least impacts to other users of the river. The site has the advantage of maximum available flow and maximum dilution of any potential contaminants because it is downstream of all tributaries. An intake located between the Highway 9 Bridge and Swan Mountain Rd could also be a possibility from a water quality perspective, but additional water quality data would be required to confirm that the treatment process proposed in the next section is adequate. The drinking water community has a long-standing guideline to adhere to the multi-barrier approach to treatment. Part of this approach is finding and using the highest quality raw water available – a condition that is most likely met at an intake site close to Dillon Reservoir.

2.5 Treatment Process

Treatment objectives for the proposed second water treatment plant are derived from the Colorado Primary Drinking Water Regulations. The treatment goals that are typically measured at a treatment plant are shown in Table 2-5. For those constituents not listed in the table, the treatment objective is to meet the maximum contaminant levels defined in
the State drinking water regulations or the secondary maximum contaminant levels as identified by EPA.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Town Water Department Objective</th>
</tr>
</thead>
<tbody>
<tr>
<td>Turbidity</td>
<td>0.3 NTU in 95% of readings each month</td>
</tr>
<tr>
<td>pH</td>
<td>7.2 – 7.8</td>
</tr>
<tr>
<td>TOC Removal</td>
<td>25% to 35% removal</td>
</tr>
<tr>
<td>Chlorine residual</td>
<td>0.2 mg/L at all points in the distribution system</td>
</tr>
<tr>
<td>Log Removal Credit for <em>Giardia</em></td>
<td>3-log removal</td>
</tr>
<tr>
<td>Log Removal Credit for <em>Viruses</em></td>
<td>4-log removal</td>
</tr>
<tr>
<td>Log Removal Credit for <em>Cryptosporidium</em></td>
<td>3-log removal</td>
</tr>
</tbody>
</table>

1. *Cryptosporidium* credit in conventional treatment plants is 3-log removal if the plant meets turbidity removal criteria (Interim Enhanced Surface Water Treatment Rule and Long-term 2 Enhanced Surface Water Treatment Rule).

In addition to treatment goals which target water quality objectives, efforts to minimize treatment costs associated with plant power, chemicals, and disposal of treatment wastes are incorporated into the evaluation of potential treatment technologies.

### 2.5.1 Treatment Requirements

Because the proposed raw water quality is excellent, treatment process requirements are relatively straightforward. The contaminants that must be removed from the water include naturally occurring particulates, pathogens, and organic matter. Treatment process train options should include treatment for iron and manganese since they are prevalent in the Goose Pasture Tarn water and could very likely be higher at some times of year than is shown by the water quality sampling results on the Blue River. When the Iowa Hill Wastewater Treatment Plant becomes operational, some shifts in water quality could occur, so the proposed treatment trains discussed here must be revisited with additional data taken from the new intake location at the time of preliminary design.

### 2.5.2 Rapid Mixing

The treatment train for the second plant should include robust rapid mixing for complete mixing of coagulant chemical into the raw water. HDR’s experience in treating cold, relatively clean Rocky Mountain water has shown that adequate mixing energy and mixing time in cold water conditions allows for optimization of flocculation and settling for removal of particulates and smaller colloidal and dissolved contaminants. In particular, removal of dissolved organic matter (typically measured as total organic carbon (TOC)) requires that each molecule of TOC come into contact with coagulant chemical for destabilization of the surface chemistry, which allows the TOC to be incorporated into floc as they are formed and settled. When the rapid mixing energy is robust, complete mixing of the chemical can occur and the flocculation process can be properly initiated.
2.5.3 Iron and Manganese Removal

Removal of iron and manganese will require the addition of potassium permanganate at a point in the process stream that provides a few minutes of contact time prior to the rapid mix. Potassium permanganate oxidizes iron and manganese relatively quickly, converting the dissolved fraction of these metals into colloidal and particulate forms that can then be coagulated and removed in flocculation and settling. Permanganate is available as either potassium permanganate, which is a dry chemical that must be mixed in water to be delivered to the treatment process, or sodium permanganate, which is a liquid chemical. Both chemicals provide the same permanganate ion to the water, which is the active ingredient that oxidizes iron and manganese. Liquid chemicals typically require less operator interaction with equipment, but sodium permanganate must be diluted prior to dosing because it comes delivered as a 20 percent solution and the doses that would be required are very low. Thus, a dilution tank will be necessary. The decision as to which type of permanganate fed can be made at the time of design. Similar space would be required for each type of feed system.

2.5.4 Flocculation

Options for mixing in flocculation include vertical mixers and paddle flocculators. Both of these options can provide good floc formation, providing the mixing energy is tapered between the three stages of flocculation and the flow path through the flocculator basins minimizes short-circuiting and forces the water to pass through the flocculation mixing zone. Typically, paddle flocculators are preferred.

![Figure 2-3: Typical Paddle Flocculator](image)

2.5.5 Settling

Settling can be accomplished in conventional basins with good flow distribution, an adequate length to width ratio and appropriate layout of effluent launders, without the addition of tube or plate settlers. Particles settle to the bottom of the basin by gravity and clarified water is removed off the surface of the basin in the launders. Use of inclined plate settlers can reduce the footprint of the settling basins by shortening the distance particles
must fall before hitting a surface. As the particles build up on the inclined plates, the weight of the solids causes the sludge to slough off and fall to the bottom of the basin. Typical loading rates for plate settlers are 0.3 - 0.7 gpm/sf of projected horizontal plate area, which translates to 2 – 6 gpm/sf of basin surface area. In contrast, settling basins without plates are generally designed at a maximum of 0.7 gpm/sf of basin surface area. A typical plate settler unit is shown in Figure 2-4. Whether a conventional basin design is used or inclined plates or tubes, a sludge collector should be included on the bottom of each basin. Current efficient models of sludge collectors vacuum the sludge solids off the bottom of the basin once a day for discharge to the waste management system.

![Figure 2-4: Typical installation of plate settlers](image)

A few plants in the Rocky Mountain region are using alternative technologies for settling. One of those is dissolve air flotation (DAF). DAF units infuse small air bubbles at the bottom of a tank and as they float to the surface, particles tend to attach to the bubbles. The “float” is then skimmed off the top of the tank for disposal. The clarified water is removed through a perforated false floor or an underflow baffle wall. DAF technology is most effective in water with light particles (algae), high TOC, low turbidity, and cold temperatures. While this process could be applied in Breckenridge because of its small footprint and thick waste sludge characteristics, operationally it is much more complex than conventional settling (with or without plates) and it is power intensive due to the need for a large amount of compressed air, so it is not recommended.
Downstream of settling, the process must include a filtration step to remove the smallest particles that are too small to settle. Two options for filtration are generally considered for filtration: high rate mixed media or dual media filtration and microfiltration membrane filtration.

**Media filtration** is the process used in the Roberts WTP for final particle removal. It works well in that plant on water that is likely to be very similar to the water in the second water treatment plant. Particles are removed in media filters not by straining but by adhering to the filter grains or previously deposited particles. In a well-designed and operated media filter, particles are removed throughout the entire depth of the filter bed, giving the filter a high capacity for solids retention without clogging. The design of the filter media is developed to maximize the entire bed depth for particle removal and minimize the development of headloss as the filter operates. Whether the media configuration is mixed media or dual media is generally determined at the time of design. Colorado design criteria for water treatment plants require that design filtration rates not exceed 5 gpm/sf. Most filters are designed to operate near this rate.

Media filters are cleaned periodically (a typical filter run can last 48 to 72 hours) by backwashing with potable water. Water is pumped into the filter bed from the bottom at a rate that hydraulically mixes the filter media, knocking off the collected particles. As the backwash rate is slowly reduced the filter media settle in accordance with the relative density of the media materials to re-establish the filter bed as it was designed. The spent backwash water is generally piped to a clarifier or lagoon and left in a quiescent mode to settle the solids to the bottom so that the clarified water can be recycled to the head of the plant. Solids are then removed from the backwash clarifier and processed along with the sludge solids from settling.

**Microfiltration membranes** are an alternative to media filtration. Membranes are classified by the size of the pore openings through which the water passes. Any particle that is larger than the size of the membrane pores is retained on the membrane and removed from the treated water. Typical low pressure membranes used in water treatment are classified as...
either microfiltration or ultrafiltration membranes, depending on the size of the pore openings. These low-pressure membranes remove particles and pathogens such as *Cryptosporidium*, but they do not remove significant amounts of smaller particles. The relative sizes of membrane types and the types of particles removed are shown in Figure 2-6.

Membrane systems are designed to meet a flow requirement at a given water temperature because the flux through the membranes is dependent on the density of the water. Membranes designed for a given flow at cold water temperatures (near 0 °C) will likely be able to produce 25 percent more water in warm water conditions (near 18 °C). Membranes can be designed to operate in two stages so that the reject water from the first stage membranes is treated in a second stage membrane.

Figure 2-6: Relative size ranges of membrane types (Ref: Osmonics)

Waste flows from a membrane treatment plant with conventional pretreatment are composed of multiple flow streams: sludge from the sedimentation basin, the reject water from the second stage membranes, spent membrane backwash water, waste clean-in-place water, and flows from block and bleed valves. The reject stream will contain similar solids to the sludge withdrawn from a sedimentation basin and the backwash water will be similar to backwash water in a conventional plant. However, the clean-in-place water is harder to manage as a waste stream without being able to discharge it to the sewer because it contains high levels of cleaning chemicals (chlorine or acid), although it can be neutralized and blended with other flow streams. Under extremely optimal operating conditions when the plant is at maximum production and the backwash sequence is
optimized, the total waste stream from a two-stage membrane plant can be reduced to one percent of the flow.

**2.5.6 Disinfection**

Disinfection processes in water treatment plants are generally driven by regulatory requirements to provide disinfection contact time or show log-removal credit for specific pathogens identified in the regulations. In addition, systems are required to provide a disinfectant residual throughout the distribution system, so maintenance of either a chlorine or chloramine residual in the water leaving the water treatment plant is essential.

Currently, Breckenridge disinfects using free chlorine, so without any pressing reason to make a change to chloramines, free chlorine is recommended for the second water treatment plant. Blending types of chlorine in the distribution system is technically not realistic because it becomes difficult to maintain the required chlorine residual. The typical reason utilities change to chloramines for disinfection is to reduce the production of disinfection byproducts (DBPs). Breckenridge does not currently have an issue meeting the DBP rule and based on treatment options and raw water quality information, DBPs are not expected to be an issue going forward.

New chlorine facilities are generally using liquid hypochlorite chemical, either delivered as a liquid and stored at the plant, or generated on-site. The selection of chlorine delivery method should be made at the time of pre-design for the plant. Both approaches provide the same result from the standpoint of disinfection credit.

Ultraviolet (UV) light disinfection is currently used by some water treatment plants to inactivate *Cryptosporidium*. In the event that raw water sampling results at the time of design shown that the water contains a significant level of *Cryptosporidium*, then UV disinfection may be necessary downstream of media filtration to obtain regulatory compliance. Microfiltration membranes typically provide adequate removal of *Cryptosporidium* so that UV disinfection downstream of membranes is generally not required.

**2.6 Treatment Options**

Three treatment train options are suggested for consideration by the Town. Option 1 is illustrated in Figure 2-7 and is composed of a conventional treatment train constructed on-site utilizing plate settlers and media filtration. Backwash water is managed in a backwash clarifier, with sludge solids combining with sludge from the settling basins to be treated on site. The clarified backwash water is recycled to the rapid mix at the head of the plant. Disinfection credit is obtained in the clearwell downstream of the filters. A space-holder is identified for potential future UV disinfection, should it become a requirement. Sludge is processed in a waste management system that can be detailed at the time of design, but which is assumed to be drying beds for the purposes of this study.

Option 2 is shown in Figure 2-8 and consists of conventional pretreatment (rapid mix, floculation, plate settlers) followed by two-stage membrane filtration. The waste streams are all shown going to a waste management system similar to the one in Option 1.
Membrane cleaning waste would require neutralization prior to discharge to the waste management system. Disinfection credit is obtained in the clearwell downstream of the membranes, but a placeholder is shown for potential future UV disinfection, similar to Option 1.

In the case of Breckenridge, implementing membranes in place of media filtration would very likely create a larger footprint for the plant overall due to the need for space for specialized membrane backwash and clean-in-place equipment. In addition, membranes have a higher cost than media filtration for plants in the size range of 3 MGD, both for construction and operation. Plant staff would have a significant learning curve to adapt to membrane treatment operations, whereas conventional treatment is already familiar.

Raw water will have to be pumped from the intake to the treatment plant regardless of the treatment process. Membranes would require re-pumping within the plant to drive the water through the membranes, first through the first stage membranes and then again for the portion of water treated through the second stage membranes. The only driver towards membranes would be if the source water had high levels of Cryptosporidium, but current water quality information shows very low levels of Cryptosporidium at the Tarn and levels further down on the Blue River would not be expected to increase significantly unless the Iowa Hill Wastewater Plant effluent flows contain Cryptosporidium.

Option 3 is to implement conventional treatment utilizing package treatment units that are installed in a building constructed on site and connected to a clearwell that is also constructed on site. The building configuration for this option would depend on the selected package unit, but would not be significantly smaller than the building square footage for Option 1. Ideally, the proposed package treatment units would have similar processes to the plant design for Option 1, that is, rapid mixing, flocculation, settling and media filtration. Tonka Equipment Company can provide three 1-mgd treatment trains consisting of pretreatment tanks equipped with rapid mixing, a 3-stage tapered flocculation system, and plate settlers. Each tank feeds two Simul-Wash air/water backwash filters. Each treatment train operates as a unit, with the finished water piped together downstream of the filters to the clearwell. Treatment concerns arising from the use of package treatment units include:

- Each individual train has its own rapid mix chamber, which requires splitting the coagulant chemical feed into three parts based on flow to each train, or provision of multiple chemical feed pumps, one for each treatment train. This adds unnecessary complexity to optimizing a treatment plant. The rapid mix chambers provided will not provide robust mixing, which HDR has found to be essential in cold water, low alkalinity water applications.
- While the design parameters of these units meet State design criteria, the configuration of the settling basin does not allow for optimal flow pathways for water approaching the plate settlers.
- Each train has a dedicated filter with two cells that are backwashed separately. This means that when each filter cell is backwashed, the flow in the adjacent cell is doubled. Altering flow to a media filter significantly in a short period of time creates
a hydraulic surge that causes a turbidity spike in the filtered water. Since the filters in the other two treatment trains do not participate in picking up the flow when one filter is off-line for backwashing, this sort of filter impact will occur daily.

- The manifold piping to connect three treatment trains will be significant and require thoughtful design to allow the trains to be operated automatically, as much as possible.

Of these three options, the conventional treatment train with media filtration constructed on site is the recommended approach. Figure 2-9 and Figure 2-10 show the plant footprint and the waste management facilities footprint for Option 1. Design criteria utilized to develop the footprint for Option 1 are shown in Table 2-6: These criteria are conservative so that the plant footprint is not underestimated for either space or cost.
### Table 2-6: Design Criteria for Option 1

<table>
<thead>
<tr>
<th>General Design Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plant Flow (MGD)</td>
</tr>
</tbody>
</table>

**Rapid Mix Basin**

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Dimensions (ft)</th>
<th>Volume (gal)</th>
<th>Detention Time (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6 x 6 x 6</td>
<td>1,300</td>
<td>&lt; 1 min</td>
</tr>
</tbody>
</table>

**Flocculation Basins (3-stage)**

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Dimensions (ft)</th>
<th>Total Volume (gal)</th>
<th>Volume Per Basin (gal)</th>
<th>Total Detention Time (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>12 x 14 x 56</td>
<td>103,700</td>
<td>51,850</td>
<td>20</td>
</tr>
</tbody>
</table>

**Sedimentation Basins with Plate Settlers**

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Dimensions (ft)</th>
<th>Total Volume (gal)</th>
<th>Volume Per Basin (gal)</th>
<th>Design Overflow Rate (gpm/sf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>12 x 16 x 80</td>
<td>210,637</td>
<td>105,318</td>
<td>2</td>
</tr>
</tbody>
</table>

**Media Filters**

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Dimensions (ft)</th>
<th>Design filtration rate (gpm/sf)</th>
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</thead>
<tbody>
<tr>
<td>3</td>
<td>12 x 13 x 26</td>
<td>5</td>
</tr>
</tbody>
</table>

**Clearwell (Disinfection)**

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Dimensions (ft)</th>
<th>Total Volume (gal)</th>
<th>Detention Time (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>12 x 72 x 16</td>
<td>106,250</td>
<td>51</td>
</tr>
</tbody>
</table>

### 2.7 Waste Stream Management

HDR and Gary Roberts met with Andy Carlberg of the Upper Blue Sanitation District to discuss waste discharges to the sanitary sewer. Mr. Carlberg indicated that the wastewater district would be unwilling to accept process waste streams from a second water treatment plant. According to Mr. Carlberg, any material containing a significant amount of alum or polymer causes the biological floc in the wastewater plant to coagulate, affecting the treatment and the downstream sludge dewatering processes. Therefore, waste streams must be either recycled to the head of the plant (clarified backwash water) or treated on site.

Sludge sources in the plant consist of the sludge withdrawals from the backwash clarifier and the sedimentation basins. The solids produced in the proposed second plant will consist of alum-based floc containing a small amount natural organic matter and natural silt. Alum sludge from plants that have little silt content are gelatinous and thixotropic, which means that they become more fluid when stirred or shaken and are more solid when left standing. The concentration of solids in the waste stream is expected to be approximately 1.0 percent after blending the sedimentation sludge and settled sludge from the backwash clarifier.
The options for managing sludge on site include gravity dewatering in drying beds or the use of a mechanical dewatering process such as a screw press, belt press or centrifuge.

Mechanical dewatering processes produce variable results in dewatering alum sludge. When the influent to the dewatering process has 1.0 percent solids, the resulting dewatered cake can be expected to be 13 to 16 percent solids. Depending on the characteristics of the waste stream, polymer addition for any of the mechanical processes can range from 3 to 10 lbs/ton of solids. Plants have reported getting better dewatering results with lower polymer doses by adding solids conditioners such as diatomaceous earth or lime, reaching 20 to 25 percent cake solids. A layout of the mechanical dewatering system is shown in Figure 2-12.

Pilot testing of centrifuge, screw press, or belt press technology is recommended prior to design of a mechanical dewatering system. Suppliers will typically bring a small unit out to the plant site (could be done at the Gary Roberts WTP) and operate it for a week for a minimal charge (in the range of $5,000). For this study, a screw press has been assumed to be the best approach. A photograph of a screw press is shown in Figure 2-11. The equipment and a polymer feed system to provide polymer to the process would require a building footprint of 3,000 sf.

 HDR's normal recommendation for dewatering alum-based sludge is to use drying beds. Gravity dewatering in a drying bed involves placing the sludge on a sand or wedge wire filter surface so that the water can drain from the sludge through the filter material. A decanting system to collect water off the top of the sludge is useful and in some areas underdrains are useful for collecting water drained from the sludge. When the climate is dry, as in Breckenridge, the underdrain system is not necessary. Drying beds are typically designed to handle from 6 to 12 inches of sludge, with three to four beds available so that they can be cycled in spreading sludge. Freezing is very effective at breaking down the structure of alum sludge, so that when it thaws the material is fairly coarse and granular.
like sand. Dried sludge that meets the Paint Filter Test (which tests for moisture content) is generally hauled to a sanitary landfill.

Sanitary waste and the discharge from process analyzers will be discharged to the wastewater system. A commercial tap for the WTP will be required by Upper Blue Sanitation District.

2.8 Manpower Requirements

The proposed second water treatment plant would be classified by the State as a Class A plant, which means that the Operator in Responsible Charge of the plant must hold a Class A water operator certification. On a day-to-day basis, one operator and one maintenance/mechanical staff member should be assigned to the plant during the day when the plant is operating. While many of the functions within the plant can be automated, typical plant operational tasks of calibrating analytical equipment, recording process and regulatory information, completing analytical work for process checks and checking or changing chemical feeds require an operator. Filter backwash initiation and sequences can be automated, but often operators prefer to initiate backwashing manually. The required operator and maintenance/mechanical staff time consists of one full-time equivalent (FTE) for operating and one FTE for maintenance/mechanical.

If the plant is operated around the clock which is preferable for maintaining treatment continuity, the plant could be unmanned at night with an operator on call for alarm response. Plants can be set to automatically shut down on certain alarms or to call out for alarms, depending on the preferences of the utility. Control room, process laboratory and office spaces are provided in the proposed plant footprint to allow operators to work on site.

The manpower required for both waste management options is estimated to be an eighth time appointment by one maintenance/mechanical staff member. If the recommended drying beds are installed the operator will need to manipulate the sludge in order to insure it dries properly, probably on a weekly basis. The alternative option, utilizing mechanical equipment, will also require an operator to be available to adjust the feed rates of the sludge to the equipment and the thickening polymer to ensure the percent solids output is adequate for disposal on a daily basis. Operator time for the mechanical option is estimated to be slightly higher than for the drying beds due to the need for maintenance of the mechanical equipment and daily attention.

2.9 Treatment Recommendation

The recommended treatment process train for the second water treatment plant is a conventional treatment train utilizing plate settlers and high rate media filtration. The treatment process selection is based on existing water quality data and the objective of producing high quality treated drinking water.
Construction of a 3 MGD treatment plant divided into two 1.5 MGD trains will provide Breckenridge with adequate water for the next 20 years. Expansion by adding a third 1.5 MGD train, if it is needed in the future, is incorporated into the footprint of the plant for evaluation of plant sites for adequate space.