Ridgway Ditch & Otonowanda Reservoir

Feasibility Study

Ridgway Ditch & Otonowanda Reservoir

Town of Ridgway
Ouray County, Colorado

January, 2011

AG File No. 09-139

Prepared for:

TOWN OF RIDGWAY

Town of Ridgway
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INTRODUCTION

The Town of Ridgway (Town) is seeking recommendations concerning its available water supplies in meeting future demands and potential alternatives to ensure that a reliable, firm supply is available particularly under extended drought conditions. Additionally, the Town is seeking advice regarding system improvements that may be necessary or required to enhance the Town's overall delivery and storage system.

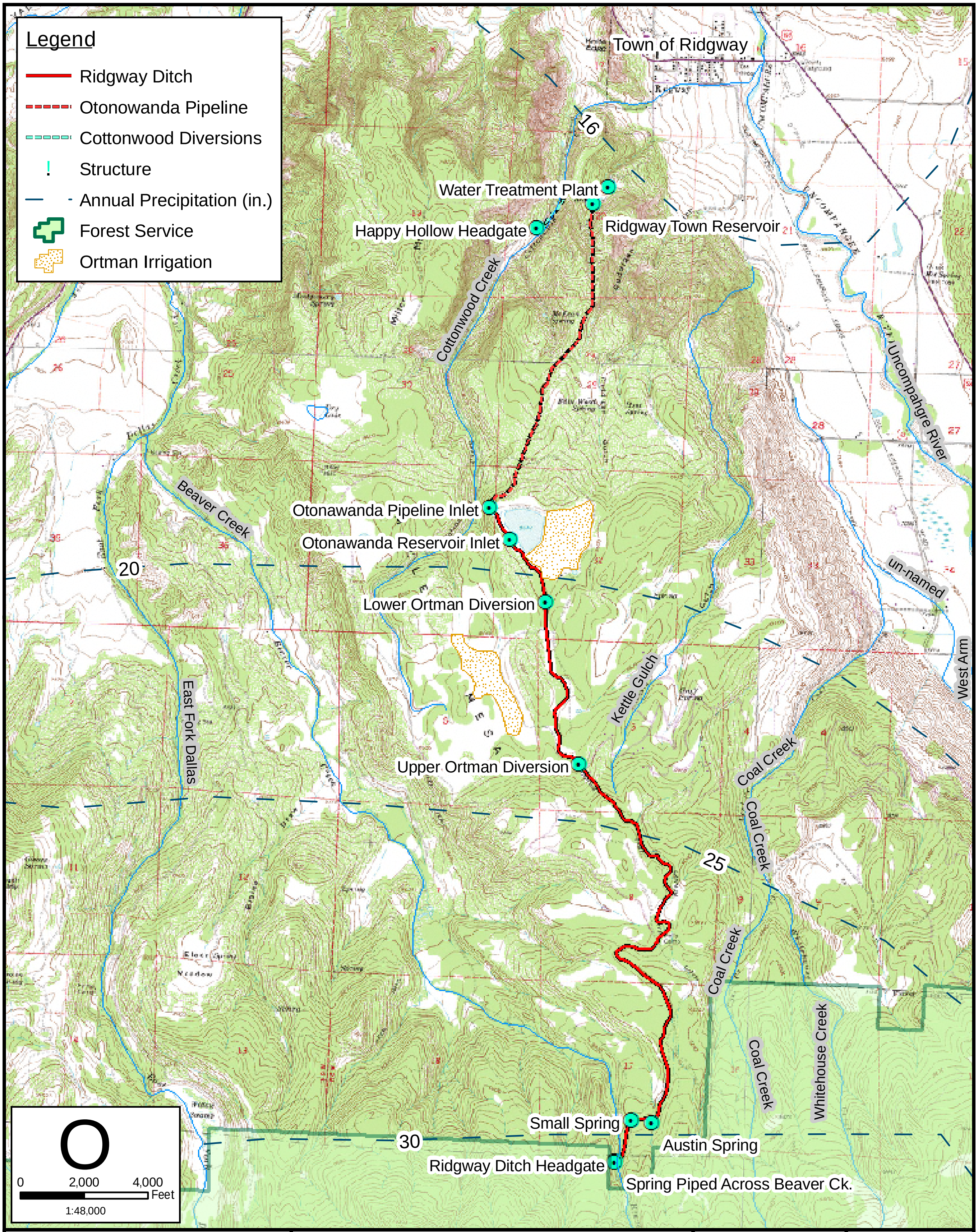
The Town's current and future operations will be reviewed to determine water demands as well as potential water or storage needs that may exist under those scenarios. To the extent information is available, we have considered this in our evaluation. We have also coordinated our review with the Town's staff to assist with our understanding of the Town's water system and operations. This report will provide a summary of our findings and recommendations. This study is funded in part by a grant obtained from the Gunnison Basin Roundtable and the CWCB Water Supply Reserve Account.

CURRENT OPERATIONS

The Town currently derives the majority of its water from two sources (See Figure 1). The first is the Ridgway Ditch system and the second is the Happy Hollow system. Diversions from these two systems are delivered to the Town of Ridgway's Ridgway Reservoir, herein referred to as the "Town Reservoir", which serves as a series of settling basins for the water treatment plant and settled non potable water line. The water treatment plant treats water from the Town Reservoir and sends it to residents for indoor and outdoor use. The settled non potable water line delivers water to various public parks and buildings for landscape and park water, dust control, culvert cleaning and other incidental municipal and domestic uses.

The Ridgway ditch diverts water from the upper reaches of Beaver Creek, a tributary to Dallas Creek, which is a tributary to the Uncompahgre River. Flow rates are measured using a 30" Parshall flume. The ditch travels across a series of mesas and through multiple steep drops for over five miles before arriving at Lake Otonowanda. Two splitter boxes divert water in this reach for irrigation purposes. No official flow measurements are taken at these points, but Town staff estimate that each structure diverts approximate 1/3 of the total flow in the ditch during irrigation season thereby leaving approximately 1/3 of the total diversions for the Town's use. Once the ditch arrives at the reservoir, the majority of the water bypasses the reservoir and enters the Otonowanda Pipeline which then travels to the Town Reservoir. This pipeline has a significant elevation drop and currently carries water in an open channel flow condition. A pressure reducing valve located at the Town Reservoir is not currently used due to problems with debris. Prior to entering the pipeline, a wooden diversion structure allows the Town to divert water into Lake Otonowanda.

If water is diverted into Lake Otonowanda, there is currently no method of making releases since the lake sits in the bottom of a natural depression. Discussions with Town staff and the Ortmans indicate that at one time the reservoir inundated the entire bottom of the natural depression and included an outlet tunnel on the northwest side. Then in 1936, the Town purchased approximately 39 acres of the lake area and constructed a small dam across the middle of the basin in order to contain the water on the Town property. At some point, the outlet tunnel collapsed leaving the lake with no outlet.



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Town of Ridgway
Water Supply Delivery Infrastructure
Vicinity Map

Date: 8-04-2010

Job #: 09-139

Drawn By: CU

Figure: 1

Of: 17

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The Happy Hollow system consists of a wooden/concrete diversion structure on Cottonwood Creek. Flows are measured using a Parshall flume with a staff gage. After passing through the flume, water enters a pipeline which transports the water to the Town Reservoir. This diversion frequently diverts all water in Cottonwood Creek.

A vicinity map of the system is shown in Figure 1.

PHOTO 1 LAKE OTONOWANDA

PREVIOUS STUDIES

In 2005, Carter Burgess completed an evaluation for the Town of Ridgway which investigated the potential cooperative effort between the Town and the Tri-County Water Conservancy District to develop treated water supplies in the Upper Uncompahgre Valley.[1] The report provided findings regarding the Town's water demands, supplies, and alternatives. In addition to evaluating the Town's water treatment needs and options, an analysis of the Town's water rights was completed to determine storage options to offset potential calls from senior water rights particularly during extended drought conditions.

In completing our evaluation and analysis, and where appropriate, we used information provided in the 2005 Carter Burgess report. In this report, we will refer to the previous report as the "2005 Report". Information such as climate data was not re-created, and was taken as provided in the 2005 Report. To the extent this information is referenced, please refer to the tables and descriptions provided in the 2005 Report.

UPDATED TOWN WATER NEEDS

The 2005 Report provided estimates of the Town's water needs. In 2000, demographic projections were provided for Montrose and Ouray Counties by Region 10 League for Economic Assistance & Planning, Inc.[2] It was reported that the growth rate of Ridgway has and will continue to outpace Ouray County's growth rate by a factor of two to one.

EXISTING DEMANDS

POTABLE WATER DEMANDS

Existing water demands are based on the Town's treated water production. Demands for irrigation within the Town using treated water were assessed and are also included in demand projections.

We have received records from the Town regarding its treated water production over the period of 2002 through 2009 as shown in the following table.

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[1] Final Report – Ridgway/Tri-County Water Feasibility Study, Carter Burgess, November 7, 2005

[2] Region 10 League for Economic Assistance & Planning, Inc., *Comprehensive Economic Development Strategy*

TABLE 1 CURRENT TREATED WATER PRODUCTION

	2002	2003	2004	2005	2006	2007	2008	2009	Ave.
MG	55	51	48	51	55	55	64	60	55
Ac-ft	168	157	148	156	170	169	198	183	169

Over the period of 2002 through 2009, the Town's treated water production has averaged approximately 55 million-gallons per year or approximately 169 acre-feet, ranging from a low of 148 acre-feet in 2004 to a high of 198 acre-feet in 2008. The following graph reflects the Town's treated water production over the last eight years and generally represents a range of treated water production between 150 and 200 acre-feet per year.

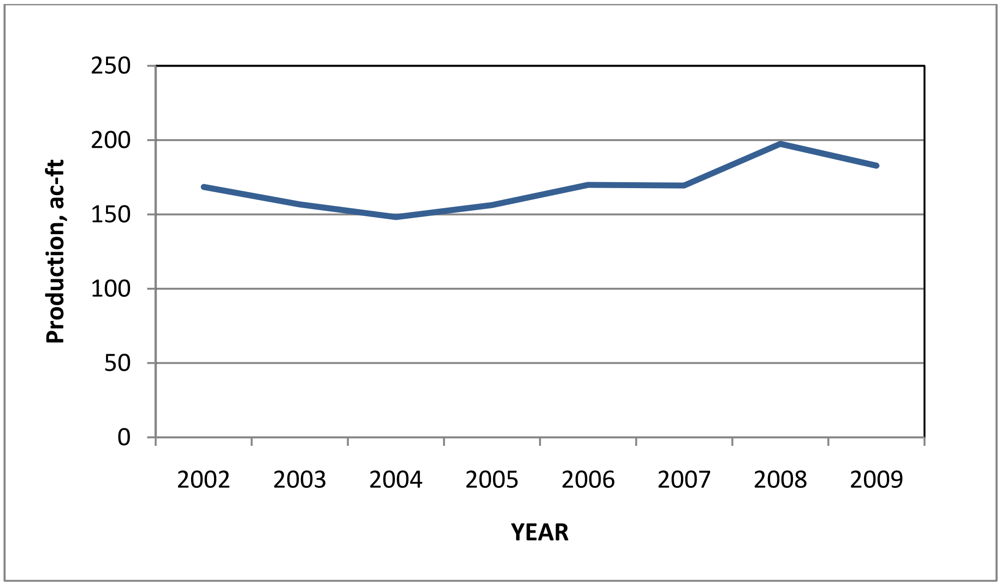

FIGURE 2 TREATED WATER PRODUCTION

Understanding that during 2002 and 2003 drought conditions were experienced throughout much of the State, these years may not be representative of normal or average conditions. During extended dry conditions, the demands for treated water supplies typically increase unless shortages are experienced and restrictions are mandated. Therefore, considering the production in 2004 versus 2009, a general increase of about 24% was experienced over this 5-year period.

SETTLED NON POTABLE WATER DEMANDS

Raw or non-treated water is applied to various areas within the Town for watering of parks and landscaped areas as well as for dust control on gravel roads. These areas include Cottonwood Park, Heritage Park, Hartwell Park, the Athletic Park, the school ball fields and the County Fairgrounds, as well as other locations throughout the Town, see Figure 9 at the end of the report. Currently, application of this water is not metered and therefore, the total settled non potable water demand was estimated using several methods.

From discussions with Joanne Fagan, the amount of water used for watering within the Town was determined from treated water production records and assuming watering demands during the summer months in addition to the treated water demands. Using this information, it was estimated that an additional 50-100% of the treated water volume is used for watering. For this analysis, we have used the period of May through October as the normal irrigation season within the Town. Assuming 50-100% of the treated water production volumes over the period of 2002 through 2009 as a basis for determining the settled non potable water demands, a total of approximately 56 to111 acre-feet may be used for non-treated purposes. The amount of water (acre-feet) varied throughout the season based on treated water production patterns as reflected in the following table.

TABLE 2 CURRENT SETTLED NON POTABLE WATER DEMAND

Ratio	May	Jun	Jul	Aug	Sep	Oct	Total
50%	8	12	12	10	8	6	56
100%	17	23	24	19	16	12	111

The total irrigated area within the Town is approximately 16.8 acres. Additional demands include drip irrigation for trees located near the public works buildings and water for dust control on streets. Assuming an irrigation water demand of 3.5 feet per acre of irrigation results in an annual irrigation demand of 59 acre-feet for irrigated areas. The watering of approximately 50 trees near the public works building is assumed to be five acre-feet per year. Town staff estimated that up to five tanker loads per day for six weeks are used in the spring to maintain roads. Assuming a five day work week and 5,000 gallons per tanker load results in an annual use of 2.3 acre-feet. This analysis results in an annual settled non potable water demand of approximately 66 acre-feet.

Town staff further estimated that there are 110 sprinkler zones with an average of 60 gpm per zone. Each zone runs approximately 45 minutes three times per week. These numbers result in an average usage of 128,000 gallons per day or 72 acre-feet during the months of May through October. Adding in the demand for dust control and drip irrigation brings the annual settled non potable water demand to 79.3 acre-feet.

Based on the various analyses described above, a conservative annual settled non potable water demand of 111 acre-feet was selected due to the lack of records on water used in this system and to account for system losses. Although the irrigation demand may be a bit conservative, applying this estimate would likely allow for additional in-house (treated) demands particularly during the summer as tourism increases. This amount was distributed throughout the year as shown in the bottom row of Table 2.

TOTAL EXISTING DEMANDS

Total existing demands include both treated and settled non potable water demands. Treated demands are on average, approximately 169 acre-feet per year. Raw water demands, as projected using the Town's treated water volumes, are estimated as an additional 111 acre-feet. In total, the Town's demands equate to 280 acre-feet per year.

FUTURE DEMANDS- (YEAR 2030)

FUTURE TREATED WATER DEMANDS

Information from the State Demographer's Office (SDO) was reviewed with respect to projected changes in population for Montrose and Ouray Counties. The following table presents a summary of the projected population growth for these counties through year 2035 in comparison to the State averages.

TABLE 3 PREDICTED POPULATION GROWTH RATE

COUNTIES	Average Annual Percent Change						
	00-05	05-10	10-15	15-20	20-25	25-30	30-35
COLORADO	2.2%	2.8%	2.8%	2.7%	2.3%	2.0%	1.4%
Montrose	2.2%	2.7%	2.7%	2.8%	2.5%	2.1%	1.5%
Ouray	2.4%	3.0%	3.1%	2.3%	0.7%	0.5%	0.4%

Similarly, we reviewed information from the State Demographer specifically for Ouray County and Ridgway with regard to change in population since year 2000. As indicated in the following table, the Town has experienced a higher rate of increase in population as compared with the State and County.

TABLE 4 HISTORICAL POPULATION GROWTH RATE

	Average Annual Rate Of Change			
	05-06	06-07	07-08	00-08
COLORADO	2.0%	1.9%	1.9%	1.8%
OURAY COUNTY	1.8%	3.6%	4.2%	2.8%
Ridgway	12.7%	5.9%	5.1%	5.1%
Unincorporated Area	-1.5%	2.9%	4.3%	2.4%

Excluding the period from 2005 to 2006, the Town's growth over the last couple of years has consistently been greater than 5% and for the period of 2000 through 2008, the change has averaged just over 5%, slightly under twice that experienced by the County as a whole. Therefore as an initial estimate, future water demands were projected at an average growth rate of 5%.

Using the Town's average annual treated water demand of 169 acre-feet (based on 2002 through 2009 period), we projected out the Town's future demands applying an annual growth rate of 5% from the year 2010 through 2030. Using this projection, the Town's treated water demand would increase to an average annual demand of approximately 448 acre-feet per year, or about 2.7 times current demand.

However, it should be noted that the Statewide Water Supply Initiative (SWSI) investigations projected that the average annual growth rate for the Gunnison Basin over the period of 2000 to 2030 would be 2%. Specifically for Ouray County, SWSI projected an average annual growth rate of 1.8% over the period of 2000 to 2030. Again, considering that Ridgway has had roughly twice the growth rate of the County, we projected treated water demands applying an average annual growth rate of 3.6% as reflected in the following chart. Under this approach, the Town's treated water demand would increase to an average annual demand of approximately 342 acre-feet per year or about twice the current demand.

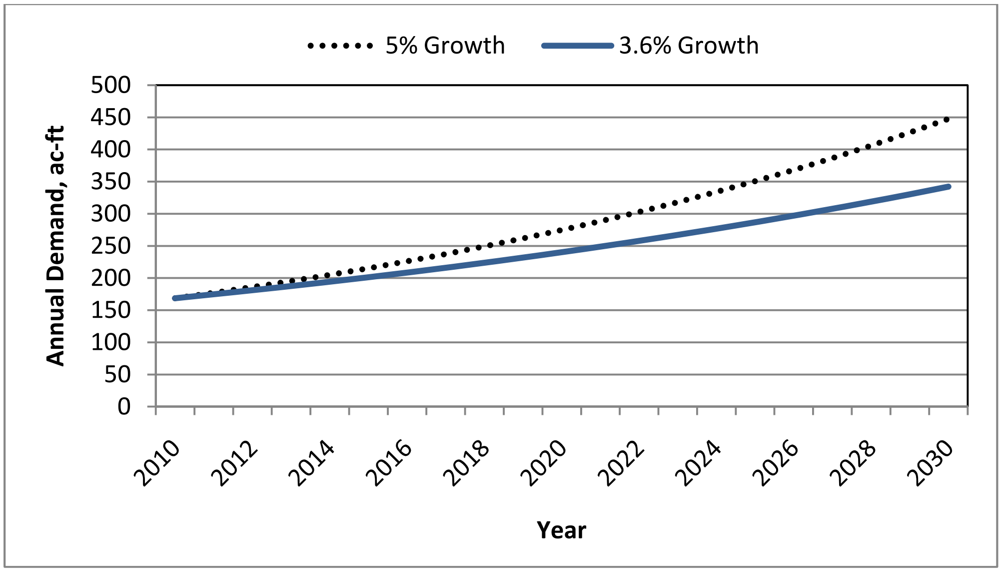

FIGURE 3 FUTURE TREATED WATER DEMAND

A study was completed in 2007 regarding Ouray County's build-out projections in which several build-out scenarios were examined.[3] From this study, potential annual growth rates for the County ranging from 3.0% to 4.7% were presented. The projections included various variables such as housing units and development patterns, master plan goals, and other factors. The range of growth provided from this study appears to support a future growth rate for the Town of less than 5%.

Considering the available information concerning the County and Town projected growth rates, we applied an average annual growth rate of 3.6% per year in our analysis to determine future treated water demands. We believe this is a reasonable estimate to project the Town's growth rate out to year 2030. Therefore, projected treated water demands are estimated at 342 acre-feet per year.

FUTURE RAW WATER DEMANDS

In determining the future settled non potable water demand, we applied a growth factor for irrigation of 35% over current conditions. This factor represents approximately six acres of new irrigation that may be constructed as a result of future development. In addition, according to Town staff there is potential for a private user to utilize water in the settled non potable water system. Using this factor, we determined that future settled non potable water demands may increase to a total of 149 acre-feet as reflected in the following table.

TABLE 5 FUTURE SETTLED NON POTABLE WATER DEMANDS

May	Jun	Jul	Aug	Sep	Oct	Total
22	31	32	26	22	16	149

TOTAL FUTURE DEMANDS

Total existing demands include both treated and settled non potable water demands. At an average annual growth rate of 3.6% per year, treated water demands are on average, projected at approximately 342 acre-feet per year. Future settled non potable water demands, as projected

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[3] Scenarios and Indicators for Ouray County build-out analysis, David M. Theobald, et al, January 25, 2007.

using the Town's treated water volumes and a growth factor of 35% for irrigated areas, are estimated as an additional 149 acre-feet. In total, the Town's demands equate to 491 acre-feet per year.

EXISTING WATER RIGHTS

The water rights owned by Ridgway are located in the Dallas Creek and the Cottonwood Creek drainage basins, both of which are tributary to the Uncompahgre River. The primary source of water is the Ridgway Ditch located on Beaver Creek which is tributary to Dallas Creek. However, the water rights diverted from Cottonwood Creek (Happy Hollow) have provided a consistent base supply for the Town. A summary of the Town's water rights are provided in Appendix A.

RIDGWAY WATER SYSTEM

The Town's Water System is comprised of several structures and facilities which include the following:

- Ridgway Ditch and associated springs
- Otonowanda Reservoir
- Town of Ridgway's Ridgway Reservoir (aka Town Reservoir)
- Otonowanda Pipeline
- Happy Hollow Ditch
- Happy Hollow Pipeline
- Happy Hollow Branch Pipeline

Generally speaking, the Ridgway Ditch diverts water from Beaver Creek and then can deliver water to Otonowanda Reservoir, to the Town Reservoir via the Otonowanda Pipeline, or both. The Happy Hollow facilities are used to deliver water from Cottonwood Creek to the Town Reservoir.

PHOTO 2 HAPPY HOLLOW DIVERSION

The Otonowanda Reservoir currently has a storage capacity of approximately 120 acre-feet. The estimated surface area of the reservoir according to a survey by Buckhorn Geotech is approximately 26 acres and the reservoir has a maximum depth of approximately six feet. As mentioned earlier the only way to withdraw water from the reservoir would be a diesel pump.

WATER RIGHT OWNERSHIP

It is our understanding that the Town's major water rights include a relatively senior 2cfs Sibert Ditch decree (appropriation date of June 1, 1882, adjudication date of May 15,1897, transferred to the headgate of the Ridgway ditch in CA-1496), a 25 cfs Ridgway Ditch decree, and a more junior 5 cfs Ridgway ditch decree all of which can be diverted at the headgate of the Ridgway ditch and are available for municipal and domestic use by the Town. Up to 2cfs of the 5cfs Ridgway ditch decree can be diverted from the Happy hollow diversions on Cottonwood Creek, instead of at the Ridgway ditch headgate. The Happy hollow decrees ,and the 5 cfs Ridgway ditch decree were decreed in CA-

1286 along with a 746 acre foot storage decree for Lake Otonowanda and a 14.9 a-f storage decree for the Ridgway Reservoir with an adjudication date of August 2, 1905. These decrees are shared with the successors in interest to the plaintiffs in civil action C-2649 (herein referred to as Ortmans) pursuant to the terms of a Stipulation and Decree in that case which allows those parties to use the water after the municipal and domestic needs of the Town, including storage in the reservoirs, is met.

There are five other water rights owned by the Town which include the Town Pump Station, Hyde-Sneva Ditch, Ridgway Spring No. 2, Ridgway Spring No. 3, and the Austin Spring. The three springs are tributary to the Ridgway Ditch conveyance system and flow into the Ridgway Ditch. The inflows of the three springs are downstream of the ditch's measuring flume; therefore, flows attributable to these rights are not included in the Ridgway Ditch diversion records. The Ridgway Springs are relatively small with a combined decreed amount of 0.048 cfs, or approximately 21.5 gallons per minute (gpm) and are not included in this analysis. The Austin Spring is decreed for 0.13 cfs and this amount was added to the Ridgway Ditch flows in the analysis. A summary of the Town's other various water rights is presented in the following table:

TABLE 6 MINOR WATER RIGHTS OWNED BY THE TOWN

Structure	Adjudication Date	Appropriation Date	Case No.	cfs
Ridway Spring No. 2	December 31, 1972	June 1, 1890	W-1305	0.022
Ridgway Spring No. 3	December 31, 1972	June 1, 1890	W-1305	0.026
Austin Spring	December 31, 1972	June 1, 1890	W-1305	0.13
Ridway Town Pump Station No. 1	December 31, 1999	October 6, 1999	99CW0265	1.0
Hyde Sneva Ditch	May 15, 1897	May 1, 1886	96CW076	0.1146

Robert Savath quit-claimed to the Town 0.1146 cfs (approximately 51 gpm) of the Hyde-Sneva Ditch which apparently corresponds to Priority No. 100. The Hyde-Sneva Ditch water right was originally decreed for a total of 17.5 cfs for Priorities No. 42 and 100, with appropriation dates of October 1, 1880 and May 1, 1886, respectively. In Case No. 96CW076, a change in place of use and an alternate point was confirmed for 1.1146 cfs of the Hyde-Sneva water right; Mr. Savath and the South Ridgway Partnership were co-applicants in the case. The alternate points of diversion included the Dallas Ditch headgate and at a well located on the South Ridgway Partnership Property in the SW¼ of the SW¼ of Section 19, Township 45 N, Range 8 West. This change only allowed the Hyde-Sneva water right to be diverted either through the Dallas Ditch or the identified well for irrigation uses. If the Town seeks to divert the Hyde-Sneva Ditch water right at a location other than the Dallas Ditch or the well located on the South Ridgway Partnership Property a change of use decree must first be obtained since the existing decree limits where the water can be diverted. This water right cannot be used by the Town as a municipal water source other than irrigation / watering of parks until the Town seeks a change of use which would allow such additional uses.

The Town Pump Station is a 1 cfs absolute water right located north of Ridgway which was decreed for municipal uses in Case No. 99CW265 with an appropriation date of October 6, 1999. The point of diversion for the Town Pump Station is located on a tributary to the Uncompahgre River north of Town. It is our understanding that the Town uses this water right primarily for irrigation and does not at this time convey water to the Town's water treatment plant. The irrigation demands associated with this source are considered to be included in the estimates for settled non potable water irrigation previously discussed. Additionally, the water right would be fairly junior in comparison to other basin water rights and would be susceptible to curtailment due to exercise of other senior water rights in the basin.

Based on the relatively small size of these water rights, location, and relatively junior priority, the water rights associated with the Town Pump Station, Hyde-Sneva Ditch, Ridgway Spring No. 2, and Ridgway Spring No. 3 are not included in the analysis concerning the water availability of the Town's existing water rights.

Water Right Calls

Calls placed by the Uncompahgre Valley Water Users Association (UVWUA) on the Uncompahgre River include senior water rights located in the Montrose area and the M&D Canal. The primary calling water rights under the UVWUA are associated with the M&D Canal. During dry years, calls by the M&D Canal will affect the entire Uncompahgre River upstream of Montrose, including the Bureau of Reclamation's Ridgway Reservoir and water rights in the Dallas Creek basin.

The river call chronology over the period of 2002 through 2009 was reviewed to determine the frequency of calls that may affect District 68 and in particular, the Town's water rights. Additionally, we had discussions with the Water Commissioner to confirm the frequency of the related senior calls.

The following table presents a summary of how senior water right calls affected the Ridgway Ditch during the recent drought cycle:

TABLE 7 HISTORICAL CALLS ON RIDGWAY DITCH

Year	Ridgway Ditch 2 cfs	Ridgway Ditch 25 cfs	Ridgway Ditch 5 cfs	Lake Otonowanda
2002	All rights out of priority July through September 2nd			
2003	In priority all year	Out of priority July 10 through September 10th		

As reflected in the above table, during a drought cycle such as that experienced in 2002, all of the water rights associated with the Ridgway Ditch would be out-of-priority basically during the entire summer (July through September). Similarly, during a year following a drought such as 2003, the Town's 2 cfs Ridgway Ditch right would be in-priority all year however, the remaining rights associated with the Ridgway Ditch would be called out during the July through September period.

Although not reflected in the above table, the Happy Hollow water rights would have been out-of-priority during the same time frame as the Ridgway Ditch junior water rights (25 cfs and 5 cfs) as the priority of these rights are similar to the Ridgway Ditch junior water rights if it were not for the fact that the Division Engineer assumed a futile call existed. Additionally, the Happy Hollow (2 cfs) and Ridgway (5 cfs) water rights cannot divert more than a combined rate of 5 cfs. In previous years, the Happy Hollow water rights have not been subjected to curtailment due to senior water right calls as it has been determined that curtailment of the Happy Hollow rights would not benefit the Uncompahgre River. However, it should be noted that future administration may change and such conditions may no longer apply.

Within the Dallas Creek basin, local water rights administration include the water rights associated with the Johnson, Dallas, Hosner Rowell, Reed Overman, James Stewart, Oakes Jerome, Scott McNeil, Henry Trenchard, Sherbino, and the Mayoral Sisson Ditches. However, the Reed Overman, Sherbino, and the Mayoral Sisson water rights are located on the West Fork of Dallas Creek and cannot place a call on the Ridgway Ditch whereas calls from the remaining water rights may affect diversions by the Ridgway Ditch. The Johnson, Dallas, Henry Trenchard, Hosner Rowell, and Oakes Jerome Ditches have priorities senior to the Town's Ridgway Ditch 2 cfs water right. The following table presents a summary of the relative priority of the Dallas Creek water rights with respect to the Ridgway Ditch water rights. It should be noted that transfers in and out of some of the ditches has occurred over time which is not reflected below.

TABLE 8 DALLAS CREEK WATER RIGHTS

Ditch	Adjudication Date	Appropriation Date	Amount (cfs)	Admin No.
Johnson Ditch	5/15/1897	8/01/1877	3.0	10075.00000
Dallas Ditch	5/15/1897	10/01/1880	11.625	11232.00000
Henry Trenchard Ditch	5/15/1897	6/01/1881	4.875	11475.00000
Hosner Rowell Ditch	5/15/1897	4/01/1882	2.0	11779.00000
Oakes Jerome Ditch	5/15/1897	5/01/1882	3.0	11809.00000
Ridgway Ditch 2 cfs	5/15/1897	6/01/1882	2.0	11840.00000
Hosner Rowell Ditch	5/15/1897	5/10/1883	7.5	12183.00000
Oakes Jerome Ditch	5/15/1897	5/01/1884	2.0	12540.00000
Johnson Ditch	5/15/1897	6/01/1884	2.0	12571.00000
Scott McNeil Ditch	5/15/1897	10/15/1884	2.0	12707.00000
Hosner Rowell Ditch	5/15/1897	4/01/1887	0.375	13605.00000
James Stewart Ditch	5/15/1897	5/01/1888	0.5	14001.00000
Scott McNeil Ditch	5/15/1897	5/01/1888	2.0	14001.00000
Ridgway Ditch 25 cfs	5/15/1897	6/01/1890	25.0	14762.00000
Hosner Rowell Ditch	5/15/1897	4/01/1893	0.25	15797.00000
Ridgway Ditch 5 cfs	8/2/1905	6/01/1890	5.0	19904.14762
Otonowanda Reservoir	8/2/1905	6/01/1890	746 ac-ft	20269.14762

According to the Water Commissioner, there has not typically been a call placed by the senior Dallas Creek water rights. The Hosner Rowell Ditch has placed a call in the past but has not placed a call in recent years. In the future, consideration may need to be given to the potential of local calls being placed within the Dallas Creek basin.

Diversion Records

The diversion records from the DWR for the Ridgway Ditch and Cottonwood Creek water rights are provided in Appendix A. The records show that numerous years of data are missing for the Ridgway Ditch. Records are available for the Happy Hollow Ditch for the period from 2003 through 2008. Diversion records are not available for Happy Hollow Pipeline, Happy Hollow Branch Pipeline, or Otonowanda Reservoir. In addition, there are two sets of diversion records for the Ridgway Ditch, one set represents the senior 2 cfs and the junior 25 cfs water rights, and it is believed that the other set may represent the junior 5 cfs water right which is only for the period from 1970 through 1989. It may also be that this information represents the combined diversions from the Ridgway and Cottonwood Creek system based on the 5 cfs limitation set forth in the Case No. 1286. The water commissioner was not aware of two sets of diversion records and could not explain their existence.

Beginning in 1999, the diversion records appear to combine all the priorities of the Ridgway Ditch. Therefore for this analysis, we used the more recent period of record to evaluate the Town's water rights considering the affects of senior river calls. Additionally, inflows from the Austin Spring located along the ditch provide a decreed flow of 0.13 cfs which is not recorded in the diversion records. According to Town staff this water supply is very consistent and reliable and therefore the decreed flow rate of 0.13 cfs was added to all diversion records for periods when the Austin Spring is in-priority. A measuring device installed at this location would help determine the validity of this assumption. The spring was assumed to be out-of-priority for periods similar to the junior Ridgway Ditch rights.

The existing records show diversions occurring during the winter months; however, the last actual flume reading is typically recorded in early November and the next flume reading is typically in mid to late April the following year. Assuming that diversions remain at the level of the last observation in early November would likely overestimate the amount of water available at the headgate since river flows typically decrease throughout the winter. During this time period we assume that the ditch diverts the entire flow in Beaver Creek as was observed during our site visit in October. To obtain a more realistic estimate of winter diversions several methods were used to recreate an annual hydrograph for Beaver Creek at the Ridgway Ditch headgate. The StreamStats program published in 2009 by the USGS was used to create two alternate hydrographs. The first is based off the Southwest regression equations and the second uses the Mountain region regression equations. The basin lies in the Southwest region, however, it is very close to the border of the mountain region and the basin characteristics are similar to many of the basins used to develop those equations. It is recognized that the small basin area above the Ridgway Ditch headgate, 1.57 sq mi, and the high mean basin elevation of 11,200 ft are outside the applicable ranges of some of the regression equations. Therefore, we also examined data from nearby stream gages on basins with similar characteristics.

Available gage data on Beaver Creek is difficult to apply to the upper portion of the basin due to the low mean basin elevation and low mean precipitation for the basin above this gage near the confluence with Dallas Creek. Other nearby gages with similar basins to the Ridgway Ditch headgate include Red Mountain Creek, and the East and West Fork of Dallas Creek. Table 9 shows a comparison of these basins. Figure 10 at the end of this report depicts the locations and drainage basins for these gages.

Annual hydrographs for the basin above the Ridgway Ditch headgate were reconstructed by dividing the Ridgway Ditch basin area by the basin area in question and multiplying all discharge records by the resulting ratio. The results of this reconstruction are shown in Figure 4.

TABLE 9 GAGING STATIONS - BASIN SUMMARY

Basin	Basin Area sq. miles	Mean Basin Elevation ft	Mean Annual Precipitation in
Beaver Creek above R. Ditch	1.57	11200	36.34
Beaver Creek Gage	12.2	9380	27.33
East Fork Dallas Ck. Gage	16.5	10900	35.16
West Fork Dallas Ck. Gage	14.1	10200	30.81
Red Mountain Creek Gage	18.1	11400	40.37

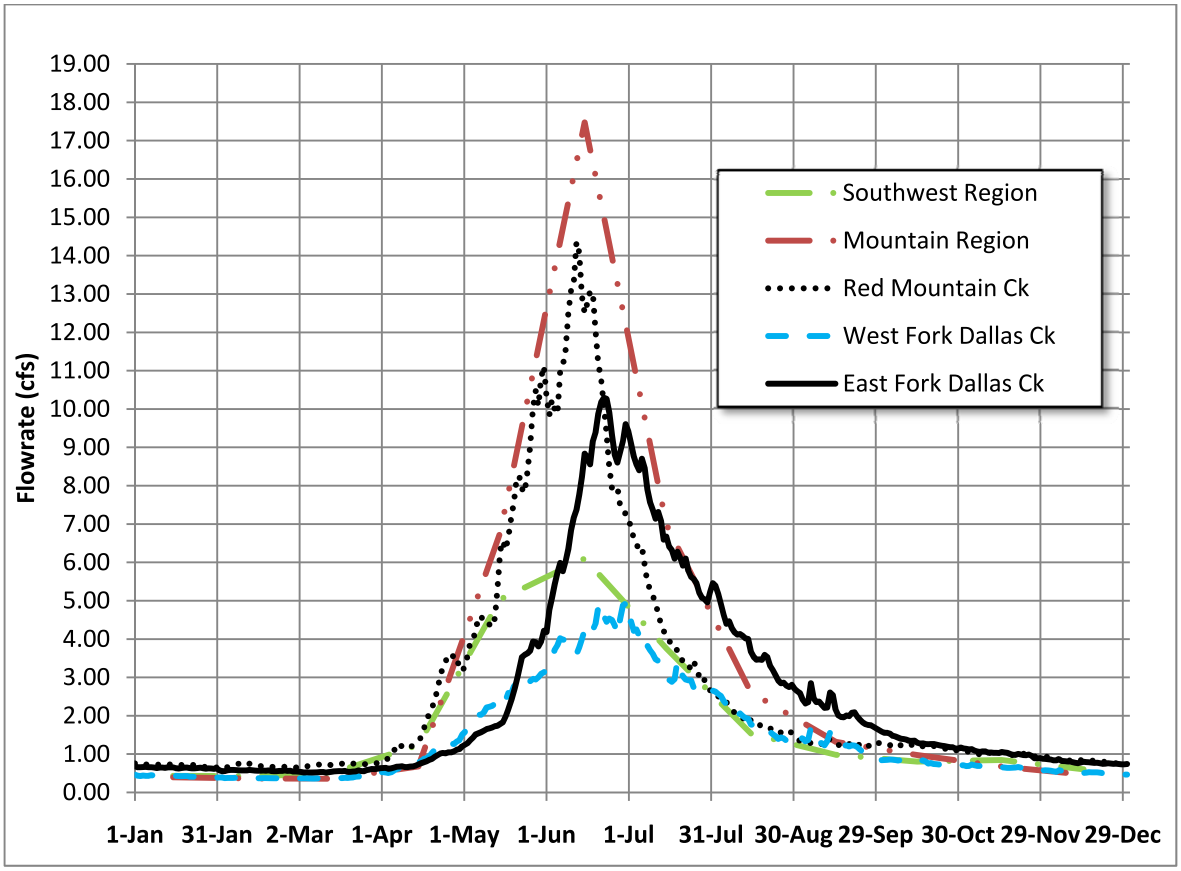

FIGURE 4 RECONSTRUCTED HYDROGRAPHS FOR BEAVER CREEK

This analysis results in highly variable flows from mid-April through the end of October, but relatively consistent results during the winter months. It was determined that the most appropriate data set to use for reconstructing winter flows would be East Dallas Creek since the basin characteristics for this gage were most similar to the basin in question. For our analysis we further examined data from the East Fork of Dallas creek to determine the winter flows available during a dry-year. During the period of record for the East Fork of Dallas Creek, the driest year was 1951. According to long-term gage data on the San Miguel River this year was not as dry as 2002, but it will suffice for this analysis since the winter flows are fairly consistent. The results of this analysis were used to estimate the volume of water available at the Ridgway Ditch headgate during the months of November 1st through April 30th.

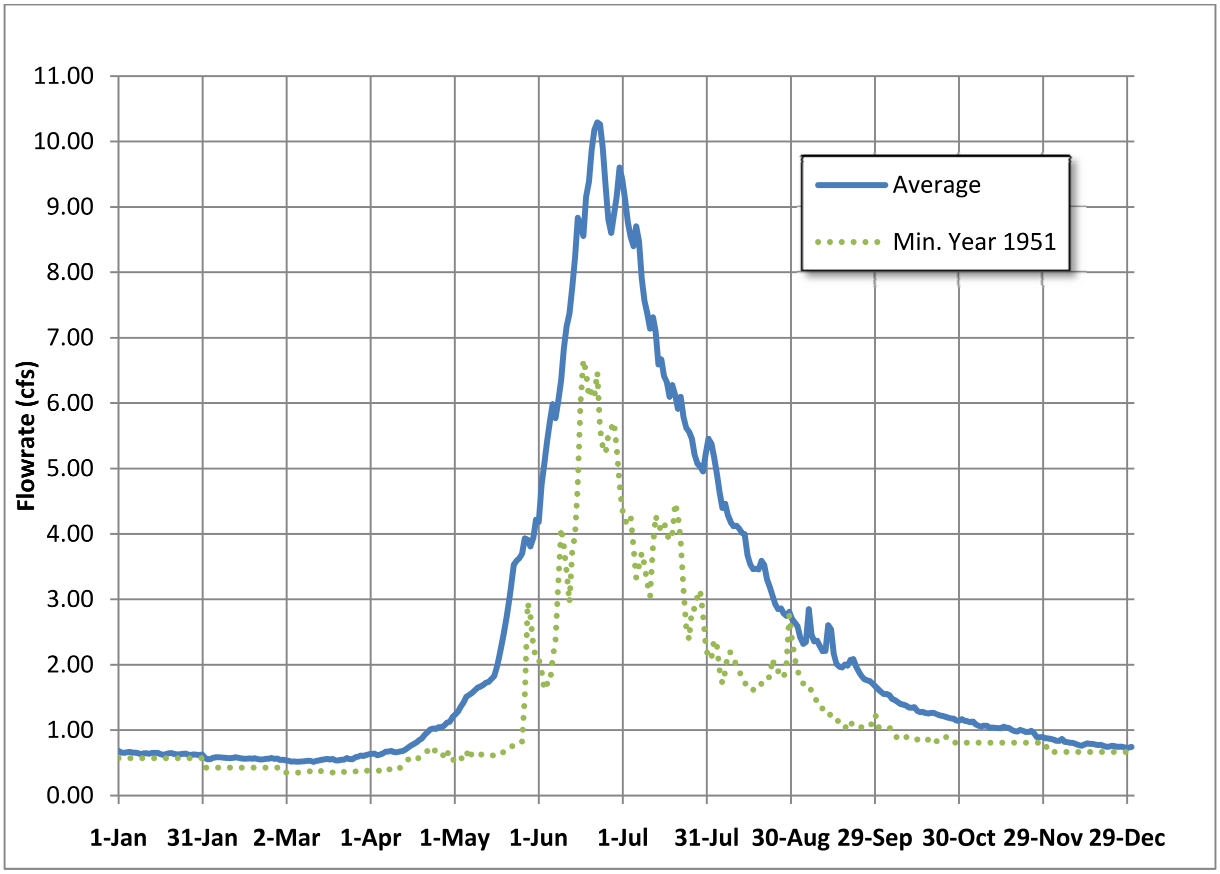

FIGURE 5 RECONSTRUCTED BEAVER CREEK HYDROGRAPH - AVERAGE AND DRY YEARS

This data was then combined with the diversion records of the Ridgway Ditch to obtain the amount of water available during an average year, as well as a 2002 and 2003 drought, see Table 8 below.

During 2002, it was assumed that all of the Town's water rights were called out June-Sept. The actual call period in 2002 did not include June. We felt that it is prudent to assume that the call could be placed earlier in the future due to uncertainties regarding the future climate and the potential effects of dust on snow events. Recent research has indicated that large amounts of dust deposited on the snowpack can cause the peak runoff in nearby streams to occur 50 days earlier. This phenomenon had a dramatic impact on the runoff hydrographs for streams in southern Colorado the last two years. In 2003, the only water right of the Town's remaining in-priority was the senior 2 cfs; therefore, the diversion records were adjusted to reflect no more than a 2 cfs diversion.

TABLE 10 ADJUSTED RIDGWAY DITCH DIVERSIONS (AF)

	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Total
Average	60[1]	48[1]	40[1]	32[1]	34[1]	48[1]	160[2]	243[2]	247[2]	162[2]	151[2]	139[2]	1349
2002	48[3]	41[3]	35[3]	24[3]	22[3]	31[3]	116[2]	0	0	0	0	91[2]	402
2003	48[3]	41[3]	35[3]	24[3]	22[3]	31[3]	79[2]	117[4]	119[4]	122[4]	96[4]	87[2]	836

1 – Value determined using average reconstructed Beaver Creek hydrograph
2 – DWR diversion records with 0.13 cfs addition for Austin Spring
3 – Value determined using dry year reconstructed Beaver Creek hydrograph
4 – Adjusted DWR diversion records to limit diversions to 2 cfs; No Austin Spring

RECOMMENDED OPERATIONAL ALTERNATIVES

We do not at this time believe that an exchange would be needed if Lake Otonowanda is enlarged and an outlet constructed. The existing Ridgway Ditch and Happy Hollow Ditch if operated efficiently when no call exists during the October through May period, plus the enlargement of Lake Otonowanda, would give the Town an adequate supply to meet demands during periods of river call and in addition meet the future water needs of the Town through year 2030.

OPERATIONAL SCENARIOS

In order to evaluate the Town's future operations for year 2030, we projected future demands as described above and applied these to anticipated diversions under the respective water rights. In doing so, we first used the Happy Hollow water rights in meeting demands directly when these rights would be in-priority. To the extent shortages or deficits occurred, water from the Ridgway Ditch and Otonowanda Reservoir were used to meet demands.

Two reservoir options were evaluated. Reservoir option 1 assumes that the reservoir will be located entirely on the Town's parcel of land and option 2 assumes that no dam will be constructed and the water will inundate the Ortman's property. The options will be discussed in more detail in this report.

Future planning should consider extended drought cycles such as that experienced over the 2002 and 2003 timeframe. In projecting such drought cycles, we assumed all of the Town's water rights would be called out during the June through September period during a 2002 drought and all but the senior 2 cfs Ridgway Ditch right would be called out during a 2003 drought. Demands during a 2002 drought would then need to be met from the Ridgway Ditch water stored in Otonowanda Reservoir. In a 2003 drought, demands could be met through direct diversion of the senior 2 cfs Ridgway Ditch right or by releasing water stored in Lake Otonowanda.

Our operational analysis considered five successive years in which we modeled, an average or normal year, followed by a 2002 drought, followed by two years similar to 2003, and finally a normal year. All water rights were assumed to be called out in a 2002 drought scenario from June through September. All water rights except the senior 2 cfs Ridgway Ditch water right were assumed to be called out in 2003 during the months of June through September. Further, it was assumed that during a drought scenario, all water diverted under the Ridgway Ditch will be taken by the Town and no water will be used by the Ortmans for irrigation purposes.

Over a five-year cycle, three years of which were considered drought conditions, the Town could meet projected 2030 demands during the summer months through releases made from Otonowanda Reservoir. This would require storage of the Ridgway Ditch water during the winter months for subsequent releases. During non-drought years, the Happy Hollow water rights are sufficient to meet demands with supplemental water released during the summer to offset potential deficits.

Under this five-year projection, approximately 336 acre-feet of storage would ensure that the Town would meet projected future demands under an extended drought cycle using reservoir option 1. Under option 1, it is assumed that the beginning storage volume is zero before entering into the beginning month of the water year. If option 2 is selected, approximately 377 ac-ft of storage is necessary in order to account for increased evaporation from the larger reservoir surface area. However, option 2 requires some initial water in storage at the beginning of the modeled scenario otherwise, a water supply deficit will occur during the drought cycles.

These scenarios assume that the Town will increase its portion of the headgate diversions from an initial value of 33% to 100% during the extended drought cycle. This will require shutting off water available to the Ortmans during this time. It is our understanding that this is within the bounds of the Town's agreement with the Ortmans. In addition, this analysis does not account for the implementation of any watering restrictions during the extended drought or ditch improvements that could decrease transit losses in the ditch. A detailed spreadsheet for the operational scenarios is provided in Appendix A.

LAKE OTONOWANDA

EXISTING CONDITIONS

Lake Otonowanda is a water storage reservoir owned by the Town of Ridgway. The reservoir is located in a natural depression, and the storage volume is defined by a dam crossing the depression. Water is delivered to the reservoir via the Ridgeway Ditch. A wooden diversion box located adjacent to the reservoir can send water to the reservoir or allow it to continue down the ditch and into the Otonowanda Pipeline which conveys it to the Town Reservoir. Historically the outlet of the reservoir was located on the northwest side of the reservoir through a natural embankment, herein referred to as "the saddle". The outlet consisted of a tunnel that collapsed at an unknown date. Due to a surface area exceeding 20 acres and a storage volume greater than 100 acre-feet, this structure would classify as a small jurisdictional dam according to rule 4.2.5 of the Dam Safety Branch of the State Engineer's Office (SEO). During our site visit to the reservoir water was ponded on both sides of the dam. The water on the downstream side likely originated from tailwater resulting from the flood irrigation of the Ortman property and overflow from the reservoir. To further assess the existing conditions at the reservoir site, various studies were performed including a topographic/bathymetric survey, a geotechnical investigation, and a wetlands study/survey of the site. A vicinity map of the area surrounding the reservoir is shown in Figure 11 at the end of this report.

SURVEY

A topographic and bathymetric survey of the basin area was completed by Buckhorn Geotech in October 2009. The survey boundary was expanded from the original proposal to include the entire natural depression and the location of any structures, ditches, wetlands flags, boreholes from the geotechnical study, and vegetation limits. Buckhorn was not able to locate any existing property pins during the survey; therefore the property boundary was not plotted at this time. A separate property boundary survey would be needed prior to proceeding with final design of the reservoir. Buckhorn Geotech has estimated that this would cost approximately $1,900. The current survey indicates that the existing reservoir has a storage capacity of approximately 119.7 ac-ft and a surface area of 26.2 acres. The water surface elevation is limited to 8541.90 ft by several breaches in the top of the 6-foot high dam that allow water to exit the reservoir. A copy of the completed survey is located in Appendix B.

GEOTECHNICAL STUDY

Buckhorn Geotech completed a geotechnical investigation on October 6th and 7th, 2009 in order to characterize the subsurface conditions of the site. This study included drilling six boreholes around the site to depths ranging from 11.0-41.5 feet. Three boreholes were completed along the existing dam alignment. Borehole #1, located near the north abutment, was drilled through the existing embankment. The embankment material was classified as a clayey silt to silty clay. Native materials immediately under the dam embankment ranged from lean clays near both abutment and a sequence of clayey sands, silty sands, and clays near the center of the dam. These materials predominantly have plasticity index's ranging from 17-36 and a percent fines of 65-85 percent.

Competent bedrock was only encountered on the south abutment at a depth of 23.5 ft. The bedrock is likely a volcanic breccia which was encountered in other boreholes around the reservoir. The other two boreholes along the dam alignment were advanced to 41.5 feet and failed to contact any competent bedrock formation although they did encounter what might be weathered bedrock.

Three other boreholes were drilled around the reservoir perimeter. Boreholes #2 and #4 were located adjacent to the reservoir on the north and south side respectively and borehole #3 was located near the top of the saddle on the northwest side. All three boreholes indicated 3-5 feet of clayey sand and gravel to silty clay and gravel. Bedrock in all three boreholes was classified as fresh to moderately weathered, medium strong to very strong volcanic breccia of intermediate composition. The information obtained from boring #3 shows igneous rock (basalt) at a depth of 3.3 feet. The upper 2 feet of the bedrock was highly fractured and weathered with a RQD of zero. The next 4 feet of core had a RQD of 52.5% and the value increased to 70.6% over the last 1.7 feet of drilled core. Although basalt can be very hard and difficult to excavate, it appears that the upper 3 to 5 feet of the material we encountered is typically fractured, weathered, and is probably rippable. It is our opinion that the upper 3 to 5 feet of bedrock would be difficult to bore but deeper, sound bedrock may be successfully bored. No groundwater was encountered in any of these boreholes regardless of the fact that borehole #2 extended nearly 10 ft below the water level in the reservoir.

Boreholes were not drilled within the reservoir, and therefore there is no measurement of the accumulated silt in the bottom of the reservoir. Boreholes would be required in the reservoir during final design. These boreholes would provide an estimate of the amount of silt presently in the reservoir. Based on the topography of undisturbed portion of the basin we estimate that the natural grade within the reservoir was likely between 8533 and 8535. The existing bottom is at 8536 ft which indicates that 1-3 feet of sediment has accumulated in the reservoir since it was constructed. There is not suitable area on the town's property to construct a sedimentation basin and therefore there is no way to capture the sediment prior to entering the reservoir. Any sediment currently in the reservoir could be mechanically removed during construction of the new dam but disposal of this sediment could be problematic depending on the content of the sediment. Also, removing this sediment may increase the seepage losses of the reservoir. The issue of sedimentation should be evaluated further during final design once information is available regarding the current sediment depth in the reservoir.

A geology map of the area is located in Figure 12 at the end of this report. The full geotechnical report is located in Appendix C.

ENVIRONMENTAL/WETLANDS STUDY

A wetlands delineation survey was performed by Environmental Solutions, Inc. (ESI) on October 6th and 7th, 2009. ESI flagged the boundary of the wetlands on site, collected study pit data and photo documentation of the site. The flagged wetlands boundary was surveyed by Buckhorn Geotech. The total surface area of the wetlands was determined to be 2.6 acres. ESI determined that the wetlands on site resulted strictly from the waterline of Otonowanda Reservoir. Furthermore, due to the lack of any defined natural channel entering or leaving the basin including the reservoir, it is ESI's opinion that these wetlands do not have any surface connectivity to any Traditionally Navigable Water (TNW) or Reasonable Permanent Water (RPW). This conclusion results in an Isolated Water classification which means they will not be subject to the jurisdiction of the Army Corps of Engineers. The Corps and EPA have both agreed with the conclusions of this report; therefore, no additional correspondence with these entities is required.

The full wetlands report is located in Appendix D.

REHABILITATION OPTIONS

As mentioned previously in this report, two reservoir options for providing the Town with the needed storage were analyzed. A preliminary design schematic for each option is located in Appendix E. These options are discussed in detail below. The survey of the existing reservoir indicated that the minimum elevation is approximately 8536.0 ft. The options discussed below assume that the bottom two feet of the reservoir would be dead storage to allow for siltation over time. The storage required according to the operational scenarios is provided as active storage above an elevation of 8538.0 ft.

EXPANSION ON PROPERTY – ALTERNATIVE A

To obtain the minimum of 336 acre-ft of active storage identified in the operational alternatives analysis while remaining on the parcel owned by the Town will require a significant dam enlargement. The dam embankment would need to be raised to a total height of 17 feet including three feet of freeboard. The proposed dam would have a crest elevation is 8555.0 ft, a water surface elevation of 8552.0 ft, a surface area of 29 acres, and a storage volume of 347 ac-ft. This water surface is approximately 10 ft higher than it was at the time of the survey. The three feet of freeboard is recommended as a minimum based on experience. The downstream toe of the dam is located approximately 10 feet from the assumed property line. No property pins were found during the survey. A property boundary survey during final design would be needed to verify that the toe of the embankment is a sufficient distance from the property line to allow access for maintenance activities.

The geotechnical investigation indicated a significant amount of clay on site but a general lack of more granular materials. It is assumed that the dam would consist of a homogenous embankment with a keyway to tie into the existing clay in the bottom of the natural depression. The existing embankment would be removed and compacted in order to ensure proper compaction of the entire embankment. The embankment side slopes were assumed to be 3.5H:1V on the upstream side and 4H:1V on the downhill slope. The upstream slope was set at a 3.5H:1V to provide stability during a rapid drawdown scenario and based on our experience with dam construction using similar materials. The downstream toe was assumed to coincide with the existing embankment toe.

Water ponding near the toe will reduce the overall stability of the embankment. The stipulation does require that successors on the Ortman's side keep water away from the dam. The flatter downstream slope was assumed in order to increase the stability of the embankment. Other methods of addressing the tail water that were evaluated included improved irrigation practices on the neighboring property or installing an electric submersible pump to carry water over the dam into Lake Otonowanda. Pumping this water would remove it from the toe of the dam and allow for a steeper slope to be used in the construction of the embankment. However, this method would essentially require the pump to operate when needed and a breakdown could compromise the stability of the steeper embankment. The installation of a sprinkler system would greatly increase the water application efficiency and nearly eliminate runoff from the fields if managed properly. This system could be pressurized using gravity by moving the diversion point for this area upstream on the Ridgway Ditch and piping the irrigation water 2,300 feet to the pasture. The NRCS EQIP program has often funded water conservation practices such as the conversion of flood irrigation to sprinkler irrigation. It is possible that this funding, supplied on a 50/50 cost share basis, could be obtained to cover half the cost of a sprinkler system. The cost of such a sprinkler system would be approximately $80,000. This is nearly 4 times the added cost of constructing the downstream slope at a 4H:1V, rather than a 2.5H:1V, and therefore the lower slope angle is the recommended option.

The natural drainage basin for this option would mainly consist of the reservoir surface area and the immediate slopes on the north and west side. The total drainage basin area would be 63 acres of which 29 are the reservoir water surface. We discussed this unique dam and reservoir with the State Engineer's Office (SEO) to determine how such a reservoir would be classified. According to Paul Perri with the SEO this dam would be viewed as a small, non-public hazard dam since a dam breach would only result in the flooding of the neighboring pasture. According to the SEO rules and regulations the inflow design flood (IDF) would be a 25-yr storm event. As a result of the small reservoir basin, however, we estimate that the reservoir, with three feet of freeboard, could easily store a much larger storm event in the freeboard without any issues. The PMP rainfall depth, according to the Hydrometeorological Report No. 49, is 10.3 inches. Assuming the dam was rated as a small, significant hazard dam, it would need to safely convey 45% of the PMP event. Even if the surrounding basin was completely impermeable this storm would result in a rise in the reservoir surface of 9.6 inches. The recommended spillway option consists of a 12" pipe installed through the saddle that would discharge into the Otonowanda Pipeline or Cottonwood Creek. The pipe would have a concrete inlet box with a crest elevation of 8552.0 ft. This pipe could be placed in the same trench as the siphon outlet but it will require an additional 4-6 ft of excavation through the breccia bedrock in the saddle for a distance of 140 ft. We would recommend digging a test pit in the saddle during the final design to determine the effort that would be required to excavate the bedrock to this depth. In addition, if the bedrock is sound enough to allow for a vertical excavation through the saddle this would reduce the construction costs. This spillway was used in the cost estimates, but an alternative spillway would be to place a pipe through the dam embankment that would discharge onto the Ortman Property. From that point it would pool on the backside of the dam. The elevation of this spillway could be set one ft above the normal operating surface of the reservoir. If the reservoir was never operated above the maximum elevation of 8552.0 then this spillway should never be needed as accumulated flood waters would not sufficiently fill the reservoir and flood water could be removed through the outlet. The SEO has concurred that this option would be acceptable from their standpoint.

The only instrumentation that would be required by the SEO for a NPH dam is a gage rod which could consist of a steel rail with weld marks installed on the side slope. A more robust and preferable alternative would be a level recording device installed in a stilling well near the outlet.

EXPANSION OFF PROPERTY – ALTERNATIVE B

During our field investigations in October 2009 we were approached by John Ortman, the owner of the adjacent property to the south and east. He suggested that the Town consider removing the dam completely and expanding Lake Otonowanda onto his property. The Town had not considered this option during the original scope of work but decided to investigate this option as part of the current study. The survey was expanded to include this area but no geotechnical work was performed at this time.

Removing the existing dam would allow Lake Otonowanda to occupy over twice the amount of surface area as Alternative A and would inundate 32.7 acres of the Ortman property. To obtain the required amount of storage associated with this option, the reservoir would have a water surface elevation of 8546.5 ft which results in a total water depth of 10.5 ft. The surface area of the reservoir when full would be 62 acres. When full the reservoir would inundate the existing access road on the north side and therefore a new road would need to be constructed. A spillway would likely not be required; however, we would recommend installing an overflow pipe through the saddle so that in the event the basin is drastically overfilled the water has a controlled release through the saddle. This pipe could be installed in the same excavation as the siphon outlet discussed later in this report. The only method of keeping the water level at or below the designed elevation would be through operation of the inlet and outlet. If the reservoir was full the inflow would need to approximately equal the outflow with a slight difference to account for seepage and evaporation. This is considered reasonable since the anticipated inflow rate would fill the reservoir very slowly, less than 1" per day, even with the outlet closed. The outlet could also be automated and programmed to operate the outlet such that it never lets the reservoir level rise above a certain point.

The low angle of the existing grade along the proposed reservoir perimeter would need to be adjusted to avoid creating a large mudflat when the reservoir is lowered. Grading the bank at a 6:1 slope around the water perimeter would decrease the amount of ground exposed as the reservoir drops and limit the amount of aquatic vegetation around the perimeter of the lake. It was assumed that this grading would only take place on the Ortman's property since the grading on the Town's property is already near the proposed slope of 6:1. This slope would extend 1-foot above the normal high water line and 3 feet below. Slopes for this grading would then be tied into existing grading at roughly a 2% slope to ensure proper drainage of the pasture around the reservoir. Excess material could be disposed of in the bottom of the reservoir or used as liner material if suitable. It is our understanding that the Town does not want to dispose of any excess material in the reservoir bottom and therefore a substantial amount of material would need to be hauled offsite. For purposes of cost estimating, we assumed that approximately half of the material would be used for the reservoir liner and the rest would need hauled off site.

Eliminating the dam has several advantages. Since the storage facility would only use the natural topography, the reservoir would be classified as a below ground storage facility and thereby qualify as an exempt structure according to the SEO Rules and Regulations. This would result in no regular inspections by the SEO. The overall cost per acre-ft of this option is lower than alternative A, however the cost, presented later in the report, does not currently include any amount for concessions that may be required by the Ortmans for utilizing 32.7 acres their property, which may or may not be significant. Another advantage to this option would be that return flows from irrigation on the Ortman's property would directly enter the reservoir rather than pond along the downstream toe of a dam embankment.

This option also has several significant disadvantages that may result in this option being unfeasible. As mentioned earlier, expanding the reservoir in this manner will drastically increase the surface area of the reservoir thereby increasing the amount of water lost each year to evaporation. During the five years modeled in the operational scenarios this amount varied from 57-93 acre-feet depending on the level of the reservoir. Increasing the reservoir footprint will likely increase the amount of seepage. The area east of the existing berm will take several years to build up a biological seal similar to the one that likely exists in the current reservoir. That said the Ortman's property is likely underlain by materials similar to those encountered on the Town's property, which included a significant amount of clayey soils overlaying the fractured breccia bedrock in the bottom of the depression. This option also has a lower water depth than alternative A. This could increase the likelihood of algae blooms creating water quality problems in the lake.

Preliminary discussions with the Ortmans indicate that they would be unwilling to convey fee title or an adequate perpetual easement to the town for the portion of their land onto which the reservoir would be expanded which is also subject to an Elk Foundation conservation easement. In addition they want a deeper pool than contemplated in this analysis which would increase costs. Further complicating the matter is the fact that their property is for sale. The Town needs to maintain an amiable relationship with Ortmans because of the water sharing provisions of the stipulation and decree referenced above and would be very reluctant to consider the use of eminent domain. As a result option B may not be a realistic possibility.

INLET STRUCTURE

Currently, the Ridgway Ditch delivers water to Lake Otonowanda through an aging wooden structure located on the south west side of the reservoir. This structure allows water to be diverted into the reservoir through an unprotected, earthen channel or continue down the ditch to the pipeline. The condition of this structure and the channel are considered poor. The general configuration of the structure is acceptable, but the wooden materials are deteriorating, and the channel is eroding. We recommend replacing this structure with a precast concrete diversion that will allow for more precise control and measurement of diversions into Lake Otonowanda. The structure should have stop logs to raise the water surface elevation and force water through a gated outlet into the reservoir. Water passing through the gated outlet will be measured using a v-notch weir before it falls into the channel. The channel between the diversion structure and the reservoir is very steep and prone to erosion; this channel should be lined with riprap or piped to the reservoir bottom to prevent further erosion. A schematic of a concrete diversion structure is included in Appendix E.

PHOTO 3 LAKE OTONOWANDA INLET STRUCTURE

OUTLET OPTIONS

Lake Otonowanda is built in a natural depression, and the outlet for the reservoir was a tunnel through the saddle. This tunnel outlet has since collapsed and the current condition is unknown. The geotechnical investigation showed that the saddle consists of volcanic breccia bedrock within a few feet of the ground surface. The recommended reservoir outlet will be in the same location and three different outlet types have been considered; a siphon outlet, a new tunnel, and reopening the historic tunnel. The first two configurations would require sealing the old tunnel to insure that no seepage or leakage is occurring there. This would likely involve drilling several holes in the vicinity of the tunnel entrance and pressure grouting the bedrock in that area. The maximum outlet capacity was determined by assuming that all water to meet the Town's future demand would be met using water from Lake Otonowanda. This would require a monthly demand of 83 acre-ft or an average daily flow rate of 1.35 cfs. For design purposes, and to extend the useful life of the siphon beyond 2030, a minimum flow rate of 2 cfs was assumed. The unique aspects of each outlet option are discussed below.

SIPHON OUTLET

Installing a gravity flow outlet by simply trenching through the saddle would require 27 ft of excavation through hard rock, which would likely prove cost prohibitive. A siphon outlet could be constructed to reduce the excavation depth required through the saddle to 7 ft. If a spillway pipe is installed through the saddle, the siphon pipe would be placed in the same trench which would be approximately 13 ft deep at the deepest point. The pipe would be buried below frost depth through the saddle. Consideration must be given to the inlet and outlet elevations as well as the maximum possible siphon lift at this altitude. The siphon would pull water from a wet well placed near the edge of the reservoir. This wet well would be filled by an 18" pipe connected to the inlet structure in the bottom of the reservoir. An isolation valve would be placed on the 18" pipe to allow the wet well to be pumped out for maintenance. The siphon would be primed using a sump pump in the wet well. This pump would require electric service in order to fill the siphon in a timely manner. Water pumped into the siphon would force air out of the siphon through a small blowoff line located at the high point in the siphon. This line would run back to the wet well where the air would be discharged. Once the air was purged from the system water releases through the siphon would be controlled using a valve located on the downstream end of the siphon. An actuator would need to be installed on this valve and tied to the SCADA system to allow the system to be remotely operated. A flow measuring device would need to be installed at some point along the pipeline. The recommended pipe for this scenario would be either welded steel or fused solid wall HDPE pipe. The welded nature of pipe joints would allow zero air infiltration when the siphon is operating or when the siphon is charged but not releasing water. The inlet to the siphon would require a screen to prevent debris from entering and plugging the siphon. This screen could also be sized to prevent small fish from entering the siphon; however this would increase the cost. Based on economics along, the siphon is the recommended outlet option for Lake Otonowanda; however, the complexity of this unique system will require more maintenance than other options. A schematic of the siphon outlet is included in Appendix E.

NEW TUNNEL OUTLET

The construction of a new gravity outlet was discussed with several experienced hard rock tunneling contractors. According to the Geotech Report the fractured breccia is very hard and abrasive. This bedrock was found to have RQD values around 70% at a depth of 5 ft below the top of bedrock which indicated that it is likely suitable for tunneling. The tunnel could be constructed using a 36" Tunnel Boring Machine (TBM) with disc cutters. This size far exceeds what is necessary for the Town's purposes and would result in spoil material that would need to be disposed of. Using a TBM would eliminate the need for construction workers to enter the tunnel thereby increasing the safety of the operation. Due to the steep slopes at the tunnel exit the tunnel would likely need to be bored from the reservoir side. Boring in the downhill direction will require the bore to be constructed at a very low grade in order to efficiently check the grade of the tunnel at regular intervals. Another option would involve using a laser to check the tunnel grade if the tunnel was installed on a steeper slope. This method, however, would require the TBM to be removed from the tunnel in order to check the grade which would significantly slow down the process and increase construction costs. As the tunnel was bored a 36" steel casing would be advanced with the bore. This casing would be grouted in place once the tunnel was completed.

Once the casing was installed, a 12" ID welded HDPE pipe would be inserted into the new casing pipe. This carrier pipe could be installed in a sloped configuration within the casing in order to increase the slope. A gated inlet structure would be constructed in the reservoir and the downstream end of the pipe would terminate at a discharge vault near the Otonowanda Pipeline intake. An acoustic flow meter would be inserted up the pipe to monitor the flow rate exiting the pipe. A overflow spillway could be incorporated into the gravity outlet tunnel behind the gate to provide a spillway for the reservoir. A schematic of this option is included in Appendix E.

The construction cost associated with this option are approximately 31% higher than a siphon outlet, thereby making this option not as favorable if evaluated strictly from an economical standpoint. However, the simplicity of this option will result in lower operation and maintenance costs which should be considered by the town prior to selecting the desired outlet configuration. Our current recommendation would be to further evaluate this option once additional information is obtained during the final design process. The overall cost of this option is highly dependent on the cost to construct the 36" bore. Additional borelogs along the proposed alignment would provide more refined information which may result in the costs for this option going down by as much as 15%.

REOPENING THE TUNNEL

One possible alternative to the option above would include cleaning out the old tunnel. Due to the lack of information available on the original tunnel construction methods, tunnel dimensions, and the extent of the collapse we cannot determine the feasibility of reopening the tunnel at this time. Based on the fractured nature of the bedrock encountered in the borelogs and the extended amount of elapsed time since the collapse, we feel that a significant portion of the tunnel could be blocked near either end and possibly in the center. We could investigate this option a little further if desired by the Town. A test excavation performed near the assumed tunnel entrance on the reservoir side could determine if the collapse was merely near the portal of the tunnel or if a significant portion of the tunnel length collapsed. If the tunnel entrance can be located and cleaned, an inspection camera could investigate the condition of the remaining tunnel. Removing any tunnel blockage could prove difficult since the tunnel was likely constructed under different working conditions that would not be allowed by today's worker safety standards. The original tunnel was likely a small hand excavated tunnel with minimal shoring protection. Worker access within the old tunnel would likely require a ventilation system, additional shoring, and other safety improvements. Reopening the tunnel would result in an opening much larger than is necessary to release water from the reservoir. A welded HDPE pipe would likely be inserted into the tunnel and the tunnel entrance around the pipe would need to be sealed and an inclined slide gate system placed on the upstream end. This option carries the greatest unknowns and risk when evaluated from a cost standpoint and is the least likely alternative at this time.

RESERVOIR SEEPAGE

Town staff has stated that, based on their long-term observations, they suspect a significant amount of reservoir storage is lost to seepage as indicated by the reservoir level dropping throughout the summer. Furthermore, the rate at which the lake level drops after being filled has decreased over time, which is likely accounted for by the formation of a biological seal on the undisturbed reservoir bottom. The annual evaporation at the reservoir site exceeds the precipitation rate by approximately 14 inches which would account for some of the drop in water level, but Town staff estimate that a significant amount of seepage is still occurring. Reviewing the grading of the reservoir below the water surface reveals relatively steep side slopes and a flat bottom. This may indicate that the borrow areas for the existing dam was the sides of the reservoir in order to steepen the bank around the perimeter. In doing so the native clays atop the fractured bedrock may have been removed and allowed seepage to occur there. Also, the collapsed outlet tunnel may account for a large amount of seepage since it was never properly sealed.

The analysis presented in this report assumes that the seepage equals the amount of rain falling on the surface of the reservoir. Given an average annual precipitation at the reservoir of 20 inches near the reservoir the resulting seepage would equate to 48 and 103 ac-ft per year for options 1 and 2, respectively. The validity of this assumption should be reviewed after a seepage test has been conducted at the reservoir site as discussed below. If the seepage test reveals that seepage out of the bottom of the reservoir is above what the Town is willing to accept, the reservoir area could be lined to reduce the overall seepage losses to an acceptable level.

Based information from the geotechnical report, it appears that a significant amount of clayey materials existing on the reservoir bottom that could be utilized as a clay liner. Boreholes 1, 5, and 6 indicate that there is at least 20 ft of predominantly CL and CH materials on the reservoir bottom. If later testing reveals insufficient quantities of clay material on site, the native soils could be mixed with a small percentage of imported clay bentonite to seal the reservoir where necessary. A synthetic liner is not recommended at this site due to the cost of installing such a large liner and the concern of unintended consequences of a complete reservoir seal on the water supply in Cottonwood Hollow. Borehole number 2 on the north shore of the existing lake indicates that there is no clay on top of the fractured bedrock. This area will be inundated by options 1 and 2 and therefore it is likely that at least the steeper side slopes of an enlarged reservoir will need to be lined to avoid excessive seepage losses through the fractured breccia bedrock. Additional geotechnical information from the reservoir bottom would need to be obtained during the final design to determine the extent of liner required; however, for cost estimating purposes, the steeper side slopes were assumed to be covered with a 2 ft thick native clay liner as shown on the reservoir schematics in Appendix E.

RESERVOIR SEEPAGE TEST

To quantify seepage out of the reservoir, a device to measure the reservoir water level should be installed. In this case, this will only be a temporary arrangement. For a temporary installation a staff gage can be installed on a vertical post placed near the shore of the reservoir. The total depth of the reservoir is currently approximately five feet, and over the seepage test we can expect the water surface would lower no more than two feet. The staff gage needs to be placed in an area that will allow measurement of at least two feet of water level change. There are several locations shown on the bathometric survey where this depth could be achieved close to the shore, such as near the old outlet, or at the edge of the berm. A Stevens Water brand Style C staff gage will measure water level in hundredths of a foot, and a three foot length can be purchased from www.stevenswater.com for $40. This can be mounted to a 4x4 pressure treated post driven into the reservoir bottom. A more permanent staff gage or stilling well should be installed during any enlargement activities.

During the test no water should be diverted into Lake Otonowanda, so that the seepage in the reservoir can also be measured with the staff gage. To quantify the reservoir seepage accurately both evaporation and precipitation also need to be accounted for. Evaporation from the lake surface will be estimated using the rates used in the operations scenario analysis. Precipitation will be measured using a battery powered, tipping bucket style rain gage. The selected rain gage should be capable of keeping a total rainfall record and should cost less than $50. Staff gage readings should be checked every other week and the cumulative rainfall recorded. The test duration should be two months or a period of time over which the reservoir drops two feet or more, whichever occurs first.

PERMITS AND REGULATIONS APPLYING TO THE PROPERTY

OURAY COUNTY

Conversations with the Ouray County Planning and Zoning Staff indicated that no permitting with them would be required for construction of a dam or reservoir outlet. The county has not adopted 1041 regulations which would eliminate the need to consult with multiple agencies. The reservoir is not visible from any surrounding public roads and is therefore outside of any specified view corridors that would require review by the county.

COLORADO DEPARTMENT OF PUBLIC HEALTH AND ENVIRONMENT (CDPHE)

Any construction activity that disturbs greater than one acre of land is required to prepare a Stormwater Management Plan for the site. This plan must be submitted to CDPHE for approval 10 days prior to the start of construction activities. Since this project will disturb more than one acre of land, this permit will be required.

CDPHE regulates fugitive dust emissions from land disturbing activities. No permit is needed for projects resulting in less than 25 acres in disturbance and a disturbance period of less than six months. Given the reservoir configurations evaluated in this report we do not feel that this permit will be required.

CDPHE also regulates discharge from dewatering activities associated with construction. A CDPHE Dewatering permit would be required for any groundwater exposed and pumped out of the construction site. This permit needs to be submitted 30 days prior to construction and will likely require sampling and monitoring of the discharge water.

STATE ENGINEER'S OFFICE (SEO)

This agency regulates the use of surface and groundwaters of the state. A permit is required to expose groundwater to evaporation/consumption and a plan for the replacement of the evaporative loss or use of the groundwater is needed. A substitute water supply plan (SWSP) would then be required during construction if groundwater was exposed. The SWSP will remain in effect until construction is complete and approved by the SEO. It will be necessary to show that any potential drawdown will not affect adjacent well owners. It is our current assumption that the reservoir would be fully drained during construction. It is likely that once the reservoir was drained no groundwater would be encountered during construction. It is possible, however, that some groundwater could remain near the boundary with the Ortman's property due to irrigation there, in which case a permit would be required for dewatering activities.

This agency also regulates water storage reservoirs throughout the State of Colorado. Reservoirs that store water above the natural ground surface and exceed one of the following criteria are under the SEO's jurisdiction: water surface area greater than 20 acres; water storage greater than 100 acre-ft; or having a normal high water surface greater than 10 ft from lowest point of *natural* ground. Reservoir option 1 in this study would be considered jurisdictional per the rules and regulations of the SEO. This dam would likely be considered a small, non-public hazard dam. The submittal to the SEO will require a Hydrology Report, Hazard Classification Report, Design Report, Construction Plans, and Specifications.

U.S. ARMY CORPS OF ENGINEERS (CORPS)

As mentioned earlier, the wetlands report prepared by Environmental Solutions recommends that the wetlands on site be considered non-jurisdictional. This conclusion was concurred by the Army Corp and the Environmental Protection Agency in a letter dated October 29, 2010. The letter is included in Appendix D. The wetlands on site have now been found non-jurisdictional and are not subject to Section 404 of the Clean Water Act. Obtaining the letter of concurrence from the Corps and the EPA is the final determination of the wetlands on the site. This letter confirms that no further action need to be taken with reference to the wetlands on site.

COST ESTIMATES

The summary of costs associated with developing reservoir options A and B are presented below. These options assume a siphon outlet and pipe spillway will be installed. More detailed information is provided in Appendix F.

TABLE 11 RESERVOIR STORAGE DEVELOPMENT COSTS

Description	Project Cost	Storage (ac-ft)	Cost/Ac-ft
Option 1 – On Property	$1,469,000	347	$4,233
Option 2 – Off Property	$1,115,000	377	$2,956

Based on this information, Option 2 appears to be most economical although concessions and fees required by the Ortmans could affect that dramatically. Further if the Ortmans remain unwilling or unable to convey good title to the property needed without unworkable strings, Option 2 is probably unpractical. The cost per acre-ft for both options is considered to be low based on other recent reservoir projects throughout the state. This is in large part due to the off-channel site, the presence of on-site materials, and the lack of any dam construction required (Option 2).

FUNDING SOURCES

There are a variety of sources available for obtaining funding for water resources type projects.

COLORADO WATER CONSERVATION BOARD (CWCB)

The CWCB has a long history of supplying low interest loans to assist in funding the construction of water projects around the state. They currently have approximately 15 million dollars to award to loan applicants in 2010 and applications can be submitted throughout the year. There is concern that based on last year's experience any money left in this fund could be taken by the state government to help balance the budget, so if the Town is interested in seeking funds for this project now would be the time to apply. Applications can be submitted at any board meeting, which are held every other month. The loan amount can include the construction cost as well as design fees. The CWCB can provide a loan up to 90% of the total project cost. The annual interest rate for municipalities ranges from 4-5.25% and depends on the applicant's income classification. The Town of Ridgway would fall in the middle income bracket which has an interest rate of 4.5%.

COLORADO WATER RESOURCES AND POWER DEVELOPMENT AUTHORITY (CWRPDA)

The CWRPDA provides funding for similar projects through their Small Water Resources Projects fund. The Town would likely qualify as a Category 3 borrower, which means that their system must serve either 650 taps if submitting under a revenue pledge or a population of 1,000 residents if submitting under a general obligation pledge. For more details on the different pledge types see their website and application forms at: http://www.cwrpda.com/SWRPsubmenu.htm. The interest rate for a loan is determined by the current market rate for AAA rated bonds. This program is operated as a pooled program which means that a bond issue will not be scheduled until a total bond size has been achieved.

STATE OF COLORADO - DEPARTMENT OF LOCAL AFFAIRS (DOLA)

Funding assistance may be available from the Energy and Mineral Impact Assistance Fund. Grants are made available for up to 75% of the project cost, however, the applicant is strongly encouraged to supply at least 50% of the project cost through cash, in kind contributions, or other sources. Applications can be submitted on either August 1st or December 1st for the following state fiscal year. According to DOLA, 26.26 million dollars is available each application period in 2010. Of that amount 20% is reserved for project awards of 200,000 dollars or less and the remainder is for projects ranging from 200,000 to 2,000,000 dollars. At the time of publication of this report, however, money in this fund has been redirected by the Colorado Legislature to help balance the State budget. It is unknown when money will be available in the future.

COLORADO RIVER WATER CONSERVATION DISTRICT (CRWCD)

The CRWCD recently combined their small and large grant programs into a single annual program with a total funding amount of $250,000. The objectives of this program that would apply to this project are developing new water supplies, improving existing water supply projects, and protecting pre-1922 Colorado River Compact water rights. Eligible applicants can receive up to $150,000 or 25% of the total project costs, whichever is less. The deadline for submitting an application is typically January 31st. Grants are voted on by the Board at the second quarterly board meeting in April. CRWCD expects that funding requests will be made for projects which have gone through normal planning and design, and have a well established time line for completion. As such, the majority of the projects are expected to be on a path for implementation in the year the application is submitted for River District consideration.

RIDGWAY DITCH

EXISTING CONDITIONS

The Ridgway ditch diverts water from the upper reaches Beaver Creek, and delivers it to Lake Otonowanda. The ditch travels across a series of mesas and through multiple steep drops for over 5 miles before arriving at Lake Otonowanda. The entire ditch is open with the exception of a few culverts at road crossings. Two structures divert water from the ditch in this reach for irrigation purposes. Once the ditch arrives at Lake Otonowanda, the majority of the water bypasses the reservoir and enters the Otonowanda Pipeline which then travels to the Town Reservoir. Prior to entering the pipeline, a wooden diversion structure allows the Town to divert water into Lake Otonowanda.

SEEPAGE

SOILS DATA

Soils data was obtained from the Natural Resources Conservation Service (NRCS), Montrose Field Service Center. The boundaries of soil types along the length of the ditch are shown on Figure 14 at the end of this report. Six soil types are encountered between the headgate and Lake Otonowanda. The location of each soil type can be seen on the attached map, and a description of the soil follows.

959 Shanley-Davoty Complex, with 25 to 65% slopes consists of 65% loam to very gravelly clay with a slow permeability and 15% clay loam with a slow permeability.

963/962 Beachcanyon-Cochetopa-Dippingvat Complex with 3 to 35% slopes consists of 30% stony to very stony loam to clay loam with a slow permeability, and 30% loam to sandy clay with a slow permeability and 15% extremely stony clay with a slow permeability

966 Ustic Haplocryalfs – Ustic Argicryolls – Rock outcrop Complex with 25 to 80% slopes, consists of 45% cobbly to very cobbly fine sandy loam to clay loam with a moderately slow permeability, and 30% cobbly clay loam with a moderately slow permeability, and 15% Rock Outcrops.

969 Tellura – Cochetopa Dewaggoner Complex with 3 to 30% slopes consists of 40% very cobbly loam to clay loam with a moderately slow permeability, 30% loam to clay with a slow permeability, and 15% loam to gravelly clay with a slow permeability.

980 Shanley-Davoty gravelly loams with 3 to 25% slopes consists of 65% gravelly to extremely gravelly loam to clay loam with a slow permeability, and 15% gravelly loam to clay with a slow permeability.

0-139 Papaspila-Taterheap Complex with 5 to 40% slopes consists of 60% loam to extremely cobbly sandy loam with a moderately high or high permeability, and 25% loam to very cobbly sandy loam with a moderately high or high permeability.

The Ridgway ditch encounters all six soils types in its path from the headgate to Lake Otonowanda. These six soils can be categorized by their permeability into three groups; Moderately High or High, Moderately Slow to Slow and Slow. The Figure 15 shows the areas of Moderately high or High and areas of Moderately Slow to Slow permeability along the ditch, and Table 12 summarizes the length of ditch that crosses the three categories of soil permeability.

The Moderately High or High, and Moderately Slow to Slow soil categories are encountered near the headgate, between stations 0+00 and 70+00. Between stations 70+00 and Lake Otonowanda, the soils are predominantly in the Slow category. Observations made during a site visit indicated that the ditch bottom was composed of heavy clay between stations 180+00 to 230+00. The upper reaches of the ditch contained a gravel bottom primarily composed of materials imported by water diversions from Beaver Creek. These gravels are excavated out as needed which would also remove any natural seal that would form from the finer sediments in the ditch. This seal, however, may not ever build up to a significant amount due to the granular nature of the sediments on Beaver Creek and the higher velocities in the Ridgway Ditch.

PHOTO 4 RIDGWAY DITCH STATION 11+00

Two permanent methods of addressing seepage in the ditch would include piping the ditch or installing a synthetic liner in the open ditch. In order to determine the necessary pipe and liner size a profile of the ditch was generated using data from the National Elevation Dataset (NED). The NED is published by the U.S. Geological Survey and has a resolution of 10 meters. A survey of the ditch centerline would be required during final design of any lining or piping project. The ditch profile and approximated ditch slopes are shown below in Figure 6. A design flow of 7 cfs was selected to evaluate the alternatives since this was the highest diversion amount on recent records and this also is the combined decreed diversion of the water rights decreed for domestic uses.

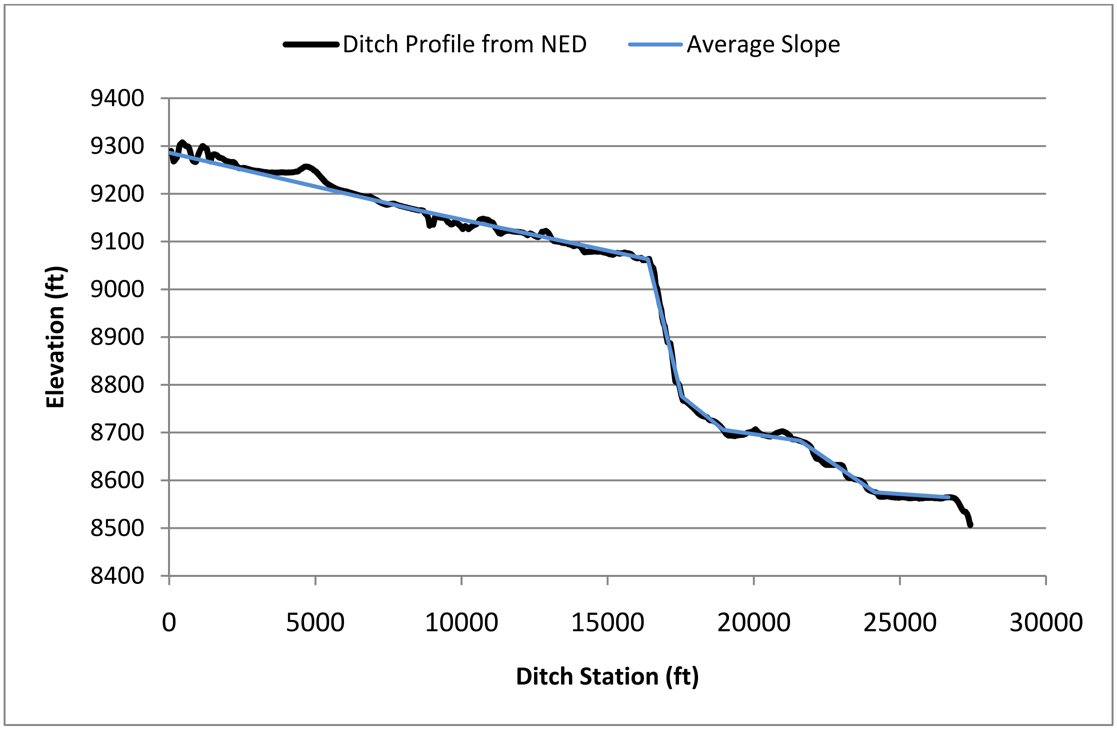

FIGURE 6 RIDGWAY DITCH PROFILE

PHOTO 5 RIDGWAY DITCH STATION 210+00

Pipe sizes were determined assuming that the pipes would run in an open channel flow condition and never be more than 75% full at a discharge of 7 cfs. Based on these assumptions a 15" diameter PVC pipe is needed between stations 0+00 and 241+50 and an 18" diameter PVC pipe is required between stations 241+50 and 262+00, where the ditch would empty into the reservoir due to the flatter grade. The material and installation cost of the PVC pipe was calculated to be $30/ft for the 15" and $35/ft for the 18" pipe. This assumes that the pipe would be buried in the existing access road and provided with a minimum of 15" of soil cover. A heavier walled PIP PVC pipe would be recommended due to the rocky soils on site. The high cost of importing proper backfill would likely require on site soils to be screened of large rocks and used as the majority of the backfill. A small amount of material would be imported in order to place the pipe on an acceptable subgrade and backfill under the haunches of the pipe. This cost does not include the cost of filling the old ditch. If the ditch is left in place it will continue to collect surface runoff during rain events and snowmelt. This water would need to be removed from the ditch periodically and either placed into the piped Ridgway Ditch as has occurred historically or discharged at natural drainage features. In addition, two inlet structures would be required to collect water from the Austin Spring and the other small spring located near station 12+00. Additional structures would be required to divert water for irrigation of the Ortman Property. The two steep drops along the ditch would likely be left as open channels to avoid creating extreme velocities in the pipe and the buildup of excess pressure in a closed conduit. It is acknowledged that this installation will place the pipe within the frost zone. The piped ditch, however, should have sufficient velocity to prevent freezing and the snow cover present in the winter time would tend to insulate the ground and not allow the frost to penetrate as deep as it would should the pipe be located in a plowed roadway.

Another method used to address ditch seepage is installing a synthetic liner across the bottom of the canal. These installations are typically buried under soil or concrete to prevent damage to the liner. In addition, an exposed liner installed on relatively steep ditches creates the potential for water to pass under the liner though any breach in the liner and build up a "balloon" at the downstream end where it cannot escape. If the ditch slope is sufficient this "balloon" could potentially cause overtopping of the ditch. The Ridgeway Ditch is located in a remote area with potential for damage to an exposed liner from wildlife, public land users, falling trees, and other hazards. Therefore, we do not recommend considering an exposed liner as an option for addressing the ditch seepage. If a liner was installed it would need to be buried under sufficient soil or concrete cover to prevent damage. Another problem that could arise with installing a liner would be the potential for high groundwater causing the liner to float. We anticipate that groundwater would be a problem along most of the ditch during the spring snowmelt. The groundwater could be addressed by either adding sufficient soil cover to prevent the liner from floating or installing a drain system under the liner to remove groundwater. A drain system for such a small ditch would be cost prohibitive since it would require the importation of granular materials and installation of a drain pipe. Concrete cover was also considered cost prohibitive due to the distance from a concrete plant and the amount of cover required to prevent floatation without a drain system. The soil cover along the bottom of the canal could be increased to approximately two feet deep to prevent floatation. This depth could be decreased to one foot at the top of the liner. The recommended side slopes for soil cover on a synthetic liner is 3:1 for stability. The ditch section required to carry 7 cfs with six inches of freeboard is shown in Appendix E.

A third option for lining the ditch would require mixing the native soils with bentonite to create a seal. This option would require stripping approximately six inches of soil from the ditch and stockpiling it adjacent to the ditch. Bentonite would then be incorporated at a rate of 3.00 to 3.75 lb/square foot into a 6" thick layer of native materials in the ditch cross section. The actual rate of amendment would be determined following geotechnical testing of the native soils. Once the bentonite is properly mixed in and compacted the stockpiled material would be placed atop the liner and compacted. This layer will serve to protect the liner from damage resulting from wildlife traffic, ditch maintenance, and drying.


PHOTO 6 RIDGWAY DITCH STATION 35+00

Table 12 depicts the estimated construction cost associated with piping and lining the ditch across various soil types. The cost-benefit of piping the ditch is likely higher in areas of higher permeability soils due to the increased reduction of seepage in these reaches. Depending on the amount of funding available for piping, the areas can be prioritized by permeability. The table above shows that the reaches of high permeability soils are short, only 3,300 feet and the cost of piping that section is relatively low. Additional measurement of flows at locations along the ditch will confirm that these areas are seeping and the seepage is of an amount that is concerning. Recommendations regarding flow measurement to quantify seepage are included in the next section. Sediment flow would need to be considered when piping the ditch. Much of the sediment could be excluded from the ditch by reconstructing the grate at the headgate which is discussed later in this report. The cost of lining the ditch across slow permeability soils was not included since there would be little benefit of installing a lining. Piping costs are still included for this reach since piping would still have some benefits in this reach such as decreased maintenance.

TABLE 12 DITCH LINING COSTS

Soil Permeability	Length	Piping Cost	Synthetic Liner	Bentonite Liner
Moderately High or High	3300 ft	$97,000	$185,000	$76,000
Moderately Slow to Slow	4200 ft	$124,000	$220,000	$84,000
Slow	18700 ft	$577,000	N/A	N/A

RECOMMENDED FLOW MEASUREMENT LOCATIONS

We recommend collecting additional diversion and flow measurement data prior to the final design of a reservoir at Lake Otonowanda. Installing additional devices and beginning to collect records as soon as possible will significantly aid in making educated decisions regarding seepage prevention and storage needs. Flow measurement devices at six locations along the Ridgway ditch are recommended if the Town budget allows. These locations will allow you to measure diversions and quantify seepage. The locations of these devices are depicted in Figure 14 at the end of this report.

Table 13 summarizes the locations, recommendations and costs, and is followed by a more detailed description of the recommended equipment and each location.

TABLE 13 FLOW MEASUREMENT RECOMMENDATIONS

Location	**Flume/Weir Recommendation**	**Recorder**	**Material Cost**
Beaver Creek Headgate	10 cfs EZ Flow Ramp Flume	Yes	$2,900
Austin Spring	3.5 cfs EZ Flow Ramp Flume	No	$1,000
Intermediate Seepage Measurement	10 cfs EZ Flow Ramp Flume	No	$700
Upper Ortman Splitter Box	10 cfs EZ Flow Ramp Flume	No	$700
Ortman's	Measure distance to stop logs	No	$0
Ridgway	Measure distance to stop logs	No	$0
Lower Ortman Splitter Box	7 cfs EZ Flow Ramp Flume	No	$550
Ortman's	Measure distance to stop logs	No	$0
Ridgway	Measure distance to stop logs	No	$0
Lake Otonowanda Diversion	7 cfs EZ Flow Ramp Flume	Yes	$2,750
		Total	**$8,600**

EQUIPMENT

Measuring flow in the Ridgway Ditch can be accomplished without significantly impeding flow by using flumes set in the centerline of the ditch. Recent research has shown that long throated flumes such as ramp flumes are preferred over Parshall flumes because of unacceptably high errors resulting from Parshall flumes used under submerged conditions[4]. Ramp flumes can be customized and calibrated to most ditch sections, or prefabricated units are available for a very reasonable price. These EZ Flow Ramp Flumes are available in four sizes with maximum flows of 3.5, 7, 10 and 20 cfs. A staff gage is installed on the flume and will measure directly in cfs, accurate to +/-3%. Proper installation and location of the ramp flumes is essential to their accuracy. The entrance conditions just upstream of the flume need to be tranquil. A calm pool must form to achieve accurate measurements. The flume must not be set at a corner, or just downstream of a drop or gate. There must be sufficient drop across the flume to insure that the flume will not become fully submerged, although too much drop at the exit of the flume may cause excessive scour and erosion. Typically 2"-3" of drop in water surface elevation is sufficient for these flumes to remain accurate. The flume must also be level and secured in place. The cost of 3.5, 7, and 10 cfs flumes are approximately $450, $550, and $700, respectively and they can be ordered from Nu-Way Flumes in Delta, Colorado.

From our experience with data recorders, we recommend using a Sutron brand Stage Discharge Recorder. This type of recorder uses a float type water level sensor. This technology is very reliable and relatively simple for troubleshooting and maintenance. Data can be stored in the internal memory for up to six months, at which time the battery will need to be recharged and the data taken out of memory. The recorder also digitally displays a direct reading of flow, which is convenient for staff or water commissioners monitoring the site. The cost of each recorder with a battery is approximately $1,600. This recorder would be housed in a fabricated hinged steel

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[4] US Department of the Interior Bureau of Reclamation. *Water Measurement Manual.* Denver: US Government Printing Office, 2001.

enclosure atop a stilling well for protection and security. The stilling well would be fabricated from 15" corrugated metal pipe and would include a hinged lid, a platform for the recorder, and a 1" diameter pipe to connect it to the flume. The estimated cost of materials and fabricating each stilling well is $600.

HEADGATE FLUME

Currently flows are measured with a 30" Parshall flume, and occasionally a 9" Parshall flume is set in the larger flume to measure low flows. Setting the 9" Parshall requires significant effort and time to insure that flows are accurately measured and water is not bypassing the flume. We recommend that a flume capable of measuring both high and low flows is utilized in this location, without the need to switch devices.

The 30" Parshall flume that is currently being used in this location, if properly installed, should be able to read flows between 1.11 cfs and 59.14 cfs with +/-5% accuracy. This range should be sufficient to capture the flows diverted through this headgate. Currently, it is our opinion that the flume is not accurately reporting flows, due to its alignment and entrance conditions. The flow entering the flume is supercritical; at the low flows we observed resulting in a lower than actual reading. This flume could be left in place, if the entrance condition is improved to provide a calm pool upstream of the structure, and the flume is currently level.

If it is found that the physical condition or levelness of the 30" Parshall flume is unsatisfactory, we would recommend replacing it with a rectangular ramp flume. A 10 cfs EZ Flow Ramp flume would be of adequate size to handle the flows diverted at this location and would be capable of reading flows as low as 0.1 cfs. Upon installation of this flume, adequate entrance and exit conditions would need to be verified. We also recommend installing a stilling well and data logger at this location. This would save a significant amount of time spent by Town staff traveling to and from the headgate and may allow for readings to be tracked throughout the winter when access is even more difficult. Freezing temperatures may pose a problem with the data logger throughout the winter. The effects of freezing weather could be reduced by installing a deep stilling well that would allow water to gain geothermal warmth and possibly keep the station operational throughout the winter. We would recommend checking on the recorder a couple times during the winter using a snowmobile.

AUSTIN SPRING

The Austin Spring is decreed for 0.13 cfs, and water records have not been filed with the state. The water from Austin Spring is split before it enters the ditch, with a portion of the flow bypassing the ditch through a pipe elevated above the ditch. We recommend using a 90 degree V-notch with a depth of at least one foot. This will allow measurement of up to 2.5 cfs, which should be more than sufficient. The V-notch could be constructed out of 1/4" to 3/8" steel sheet material and driven or buried in the ground 18" or more, depending on the local soil conditions. The installed weir should maintain at least 6" of space between the base of the notch and the downstream ground level. The area downstream of the weir should be lined with small rock to prevent erosion. The plate will need to

PHOTO 7 AUSTIN SPRING SPLITTER BOX

be wide enough to span the width of the channel without allowing water to bypass the V-notch. The weir should be located in a straight reach of channel and there should be sufficient channel depth upstream of the plate to allow water to pool. This will raise the water surface elevation at the division structure and the division structure may need to be adjusted to account for this. The aging division structure may require some maintenance prior to adjustment. A staff gage would need to be installed four feet upstream of the V-notch.

INTERMEDIATE SEEPAGE MEASUREMENT

We are interested in the amount of seepage occurring in the upper reaches of the ditch due to information obtained from the NRCS soils report. To determine the amount of seepage, it would be helpful to have an additional flow measurement location near the end of the higher permeability soils near station 58+00. This location was chosen because of proximity to the road. Installation of the flume in this location and subsequent measurements could be taken without disturbing neighboring properties. A measurement at this location could show if excessive seepage is occurring in the higher permeability soils.

In this location we would like to be able to measure up to 10 cfs. A 10 cfs EZ Flow Ramp flume would be of adequate size to handle the flows diverted at this location. The staff gage installed on the flume will measure directly in cfs and is accurate to +/-3%. Upon installation of this flume, adequate entrance and exit conditions would need to be verified. Readings on this flume would not need to be obtained on a regular basis. Periodic reading made up to four times a year would suffice. Readings on all other flumes would need to be taken at the same time during a period where headgate diversions are remaining fairly steady and diversions by the Ortmans are steady.

UPPER ORTMAN SPLITTER BOX

Historically flows have been divided at this diversion with approximately 1/3 of the flow going to irrigate the Ortman's land, and 2/3 continuing down the ditch to Lake Otonowanda. A splitter box was installed at this location to control the amount of flow going in each direction. We recommend installing a flume upstream of the diversion and modifying the operation of the splitter boxes to allow for an approximate measurement of the flows in both directions.

PHOTO 8 UPPER ORTMAN SPLITTER BOX

The 24" Parshall flume that is currently located just downstream of the Upper Ortman splitter box could potentially be relocated and reset to this location in order to save the purchase cost of another flume. It is not serving a purpose in its current location due to erosion underneath the flume. When resetting this flume, care should be taken to insure that all required conditions for the installation of a Parshall flume are met. This includes setting the flume level on a good foundation to prevent future settlement, creating an area for a calm pool to develop upstream of the structure, and insuring there is sufficient drop downstream of the structure to prevent submergence. Due to difficulties often encountered when resetting flumes, including the questionable condition of the flume, we recommend installing a ramp flume upstream of the splitter box, and downstream of the road crossing. A 10 cfs EZ Flow Ramp Flume will be adequate in this location.

In addition, this splitter box could be reconfigured to allow the percentage of flow passing in each direction to be estimated. First, the area upstream of the entrance to the box needs to be cleaned out, and the banks raised as necessary to bring them level with the top of the box. This will allow for a calm pool to form upstream of the diversion. Second, stop logs should be added to the left channel to eliminate that potential flow path. Third, the stop logs in the right channel should be modified to insure that water does not leak between or around them. A sheet of plastic attached to the front of the stop logs could accomplish this. The stop logs in this channel can be adjusted to divide the flow as desired. By measuring the distance from the top of the box to the stop logs, the percentage of water being diverted in each direction can be determined, and an approximate flow assigned. These measurements need only be taken when the stop logs are adjusted. The following schematic illustrates modification to the operation of the splitter box.

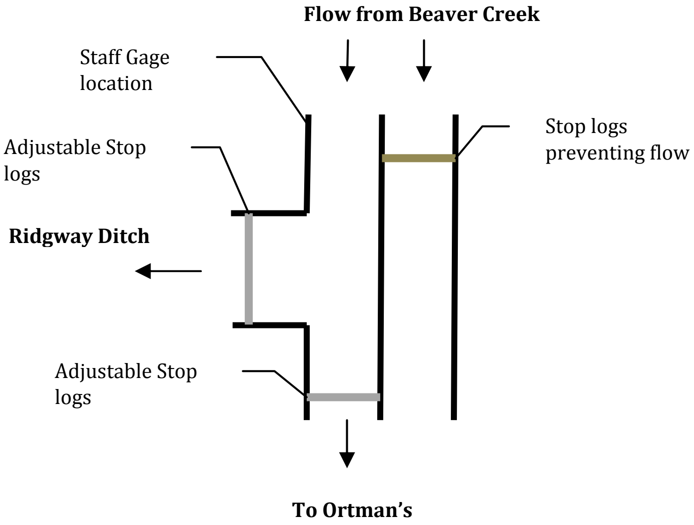


FIGURE 7 UPPER ORTMAN SPLITTER BOX SCHEMATIC

LOWER ORTMAN SPLITTER BOX

The same approach can be taken with this splitter box as was taken with the Upper Ortman splitter box. Historically flows have been divided at this diversion with approximately ½ of the flow going to irrigate the Ortman's land and ½ continuing down the ditch towards the reservoir and pipeline. The same modifications will be applied to this box. Although more work may need to be done to the entrance of this splitter box. We would recommend placing a 7 cfs EZ Flow Ramp Flume upstream of this structure. Each time the stop logs are adjusted measurements from the top of the box to the top of the stoplogs should be taken to determine how the flow is being split.

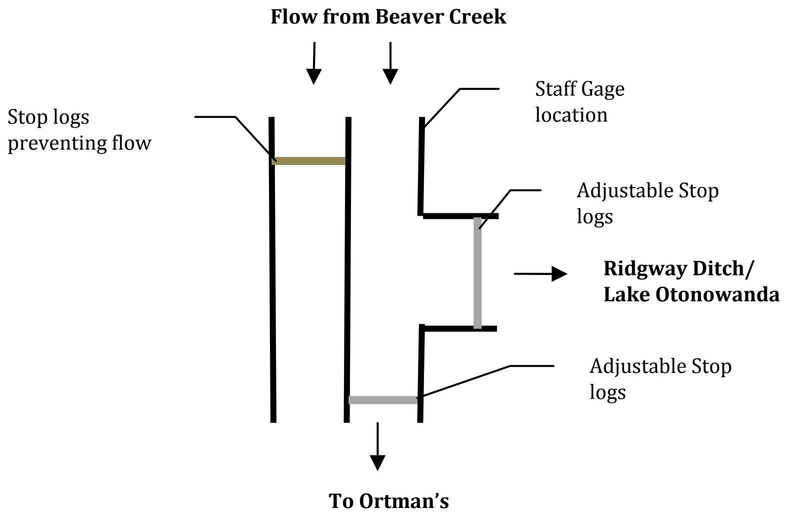


FIGURE 8 LOWER ORTMAN SPLITTER BOX SCHEMATIC

LAKE OTONOWANDA DIVERSION

We recommend placing one 7 cfs EZ Flow Ramp Flume with a recorder downstream of the diversion to Lake Otonowanda. Ultimately we are most interested in the amount of water diverted to the water treatment plant, than we are to the reservoir. The amount of water delivered to the reservoir could be estimated by calculating the difference between the lower Ortman flume and the Lake Otonowanda flume. When the diversion to Lake Otonowanda is rebuilt during the reservoir construction, we recommend incorporating flow measurement into the new diversion. This is addressed in the Lake Otonowanda section of this report.

PRIORITIZED INSTALLATION

If there is not sufficient funding to install all of the recommended flow measurement devices, we recommend prioritizing the installations as follows. Accurately measuring the diversions at the Ridgway Ditch Headgate is the first priority. As mentioned above the cost of materials for this structure would be $2,900.

Second in priority would be Lake Otonowanda 7 cfs EZ Flow Ramp flume. The estimated cost for this flume is $2,750. This includes a stilling well and recorder in the EZ Flow Ramp flume.

Third in priority would be the Austin Spring and two Ortman diversion flumes. Installing these flumes will allow for accounting of all additions and diversions from the Ridgway ditch system as well as determining seepage losses between each device. The materials for these three flumes are estimated to cost a total of $2,250.

The lowest priority flume is the intermediate seepage measurement. This flume will provide valuable information in making a decision regarding piping of the ditch, but the flume will not provide information about diversions or demands.

HEADGATE DIVERSION

The existing diversion structure is a Tyrolean style intake which is well suited for high mountain streams that carry a large amount of debris during spring runoff. The intake grate is situated on the bottom of the main channel and consists of upside down rails welded together. Town staff recently rebuilt this grate and the spacing on the rails is approximately ½" however this varies due to the amount of wear on the rail edges. The remaining structure is composed of wood timbers. Water falling through the intake grate enters a rectangular channel with an inoperable swing gate. The swing gate was historically used to flush sediment from the box. At the end of this box stop logs can direct water back to Beaver Creek or into the Ridgway Ditch. The 30" Parshall flume is located approximately 100 ft downstream of the wooden channel.

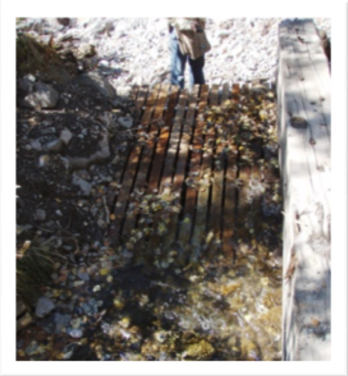


PHOTO 9 RIDGWAY DITCH HEADGATE SCREEN

PHOTO 10 RIDGWAY DITCH HEADGATE

It is our understanding that the Town is interested in improving or replacing this structure in order to decrease the amount of maintenance it requires. Granular material passing through the intake grate accumulates in the rectangular channel as well as the main canal. This requires frequent cleaning during the peak runoff season. In 2010, very high runoff resulted in increased flows in the ditch which caused significant erosion of the ditch banks. Lastly, the existing stop log setup in the structure offers only a limited ability to precisely control the amount of water entering the ditch.

MODIFICATIONS TO EXISTING STRUCTURE

Recommended improvements to the existing structure would include replacing the existing grate and installing an adjustable gate on the structure to control flows into the Ridgway Ditch. The existing grate is composed of light crane rails placed side by side and upside down. The grate is approximately four feet wide and eight feet long. We estimate that of the 32 square feet of grate only 18 square feet actually lies above the channel below. This configuration has worked well in the past, however, the rail edges have become worn which has widened the gap between rails and allowed more sediment to pass through the screen and into the ditch. Replacing this screen with a prefabricated profile bar screen with smaller openings could remove a greater amount of sediment from the flow entering the ditch. This grate would be sized to pass a flow rate in excess of the decree to allow for some clogging between maintenance visits. We estimate that a new grate could keep out materials greater than 1/8" in diameter. The smaller openings in the grate may be more prone to icing during the winter and the old rack, or something similar, may need to be installed during the winter season. Other modifications to the existing structure would be required to allow the grate to attach to it securely; however, we would need to pull out the old grate to

PHOTO 11 RIDGWAY DITCH HEADGATE BOX

investigate this further. The rectangular wooden channel that the screened water passes into is approximately three feet wide by two feet deep. A square slide gate could be mounted near the downstream end six inches above the channel invert which would allow for some sediment storage in the upstream channel. This sediment could be flushed by removing stop logs on the bypass channel for a brief amount of time. Rodney Hunt manufactures a variety of gates including timber gates. Once of these gates could be incorporated into the existing structure with limited modification.

REPLACEMENT STRUCTURE

Eventually the structure will need to be reconstructed since it is currently composed of wood. At this point we would recommend replacing the structure with something composed of concrete. Due to the remote location, however, cast in place concrete is likely to be very expensive. We would recommend exploring the possibility of constructing the structure out of precast concrete sections that would be bolted or welded together at the site. This option would also decrease the amount of down time experienced on the ditch since formwork, pouring, and curing time would all take place off site. Many precast suppliers are willing to custom build structures such as this one for a reasonable price. Another option could include constructing the majority of the structure out of wood and only casting the portion of the structure containing the gates out of concrete.

If the structure was completely rebuilt we would recommend a few changes to the existing configuration. A new grate would be incorporated to reduce the amount of sediment entering the ditch. A long weir wall installed in the channel downstream of the grate would serve to limit the fluctuations in water level regardless of the amount of water entering the channel through the screen. This would result in decreasing the frequency of adjustments to the stop logs and slide gate. The top of this wall would be set at an elevation such that the Ridgway Ditch can still divert up to 10 cfs prior to water spilling over the wall. A slide gate on the Ridgway Ditch would be used to control the amount of flow in the ditch. A stop gate would be installed near the downstream section of the weir wall near the bypass channel to allow sediment to be flushed from the upstream channel. A schematic of this layout is presented in Appendix E.

AUTOMATION

Other options for improving the headgate system would be to add automated or remote control to this structure. Automating the headgate would allow the Town to select a desired flow rate and the headgate system would make any adjustments necessary to maintain that flow rate. This option would involve installing a new headgate structure with a slide gate and actuator that could be controlled by a Programmable Logic Controller that would receive a signal from a water level sensor on the flume. The actuator would make necessary adjustments to the slide gate to maintain the desired flow rate at the flume. A more expensive, and complicated option would involve everything mentioned in the automated option but would also allow for the user to make changes and monitor the system from a remote location such as the water treatment plant. Due to the lack of electric power at the site any controls would require the installation of a solar panel and battery system.

PERMITTING ISSUES

Rehabilitation of existing agricultural headgates is exempt from Corps permitting as long as the ditch is owned and operated by an agricultural user and the maintenance is required for agricultural purposes. Since the ditch is owned by the Town, the Corps will likely require a permit to be obtained to perform significant maintenance on the headgate in the channel of Beaver Creek. This work could probably be covered by a Nationwide Permit No. 3 which is for pre-existing structures. This permit would not require a pre-construction notification or wetland delineation.

The owner of the land that the headgate lies on is listed as Wolf Land Company LP. Based on Colorado law you have the right to maintain your headgate as needed. Any specific easement or agreements regarding this parcel and the ditch should be reviewed by the Town's attorney prior to proceeding with any major rehabilitation work.

COST ESTIMATES

The costs of the various improvements are summarized below.

TABLE 14 RIDGWAY DITCH COST ESTIMATES

Description	Cost
Ditch Piping/Lining	
Moderately High to High Permeability	$76,000 - $185,000
Moderately Slow to Slow Permeability	$87,000 - $235,000
Slow Permeability	$680,700
Flow Measurement	
Priority No. 1	$2,900
Priority No. 2	$2,750
Priority No. 3	$2,250
Priority No. 4	$700
Headgate Modifications	$15,000
Headgate Replacement - Wood	$68,000
Headgate Replacement - Concrete	$88,000

FUNDING SOURCES

Funding sources listed for Lake Otonowanda would also apply to projects on the Ridgway Ditch. In addition, the program below may be a potential funding source for ditch projects.

WATERSMART PROGRAM – BUREAU OF RECLAMATION

This program is focused on funding projects that employ water and energy efficiency improvements and improve environmental conditions in the western United States. In 2010 this program awarded grants totaling 12.8 million dollars. Preference was given to projects that accomplished more than one of the program goals listed above. This program had funding requests totaling over $84 million which implies that the process was very competitive. The likelihood of using this program to fund ditch improvements such as lining or piping is probably rather low. In order to be competitive the proposed project would need incorporate renewable energy or some sort of water-energy connection.

Happy Hollow Diversion

PHOTO 12 HAPPY HOLLOW HEADGATE FLUME

At the Happy Hollow Diversion there is currently a 9” Parshall flume in an unconventional configuration. It appears that the exit conditions are satisfied, but it appears that water is entering the flume too quickly, creating waves at the staff gage. In addition, the flume is located immediately downstream of an existing sluice gate which would affect the readings, however, it is our understanding that this gate is not used. This may contribute to inaccurate readings. This could be helped by installing a stilling well on the existing Parshall flume, but space is limited around the flume. A tube could be run out from the side of the flume to an area downstream where there is adequate space to install the stilling well. The trench for the tube would need to be hand dug due to the close proximity of other parts of the diversion structure. Installing a stilling well is only necessary if a data recorder will be installed. Because of the close proximity of this diversion to Town, and the relatively low fluctuations in flow, a data recorder is probably not necessary.

The existing headgate structure is composed of wood and does not allow for much adjustability should that be required at some point. The wooden portion of the structure could be replaced by a structure similar to the one proposed for the Lake Otonowanda Inlet, see Appendix E. This structure could be tied into the existing concrete around the entrance to the Happy Hollow Pipeline.

CONCLUSIONS

As shown in the analysis of this report the Town currently has sufficient water rights to provide them with a reliable source of water into the year 2030. Water rights on the Ridgway Ditch and Happy Hollow make up the vast majority of water used by the Town. Water diverted under these rights is brought to the Ridgway Town Reservoir where the treatment plant is located. From there the water is treated and sent to users within the Town, or it enters a settled non potable water line and is used to irrigate parks and sporting fields around town.

The lack of any useable storage on the system will create water shortages during a call scenario such as the one experienced in the 2002 drought. The Town currently has the right to store 746 ac-ft of water in Lake Otonowanda but the existing reservoir is not constructed to the decreed size and lacks an outlet to access storage water. The amount of storage needed to provide the Town with a reliable water supply into the year 2030 was calculated using several assumptions.

- The population growth rate was set at 3.6%
- Settled non potable water demands were increased by 35%
- Transit losses in the Ridgway Ditch and Happy Hollow systems were set at 35% and 0%, respectively.
- Annual Evaporation was set at 2.8 ft annually
- Reservoir Seepage and Rainfall were assumed to cancel one another; annual rainfall at Lake Otonowanda is 20 inches
- Five successive years were modeled, an average year, followed by a 2002 drought, followed by two years similar to 2003, and finally a normal year
- All water rights were assumed to be called out in a 2002 drought scenario from June through September
- All water rights except the senior 2 cfs Ridgway Ditch water right were assumed to be called out in 2003 during the months of June through September
- During a drought scenario all water diverted under the Ridgway Ditch will be taken by the Town and no water will be used by the Ortmans for irrigation purposes

Two options were investigated to enlarge the existing Lake Otonowanda and construct a new outlet to provide storage water to the Town. The first reservoir option was located entirely on the 39 acre parcel owned by the Town and the second option removes the dam completely and will inundate the Ortman's property for storage purposes. The two options are compared in the table below.

Parameter	Option 1- On Property	Option 2 – Off Property
Construction Cost	Higher Cost but still considered low compared to other recent storage projects in Colorado	Lower Cost but does not include easement or land purchase cost
Land Purchase or Easements Required	None	Required, Ortmans may not be willing to grant; Condemnation not likely an option - need to maintain amiable relationship with Ortmans.
Annual Evaporation & Seepage	Lower	Higher
Water Quality – Algae Blooms	Less likely	More likely

Improvements to the Ridgway Ditch could result in significant water and maintenance cost savings for the Town. Piping or lining the ditch across the higher permeability soils could potentially save 100-200 acre-feet per year based on transit loss estimates of 252 acre-feet in an average year. These savings could allow the Town to refill storage more quickly following a drought year, but will not decrease the amount of storage needed since the ditch would be called out in an extreme drought year. The Beaver Creek diversion structure is located at a remote location and could benefit from some improvements. At a minimum the inlet grate could be replaced to screen out finer debris and the stop log configuration could be replaced with a gated system that would allow for additional control over the division of flows. An entirely new structure could be constructed to further decrease maintenance and improve operations. This structure could be configured to provide for a more regular flow of water in the Ridgway Ditch regardless of daily fluctuations in the level of Beaver Creek.

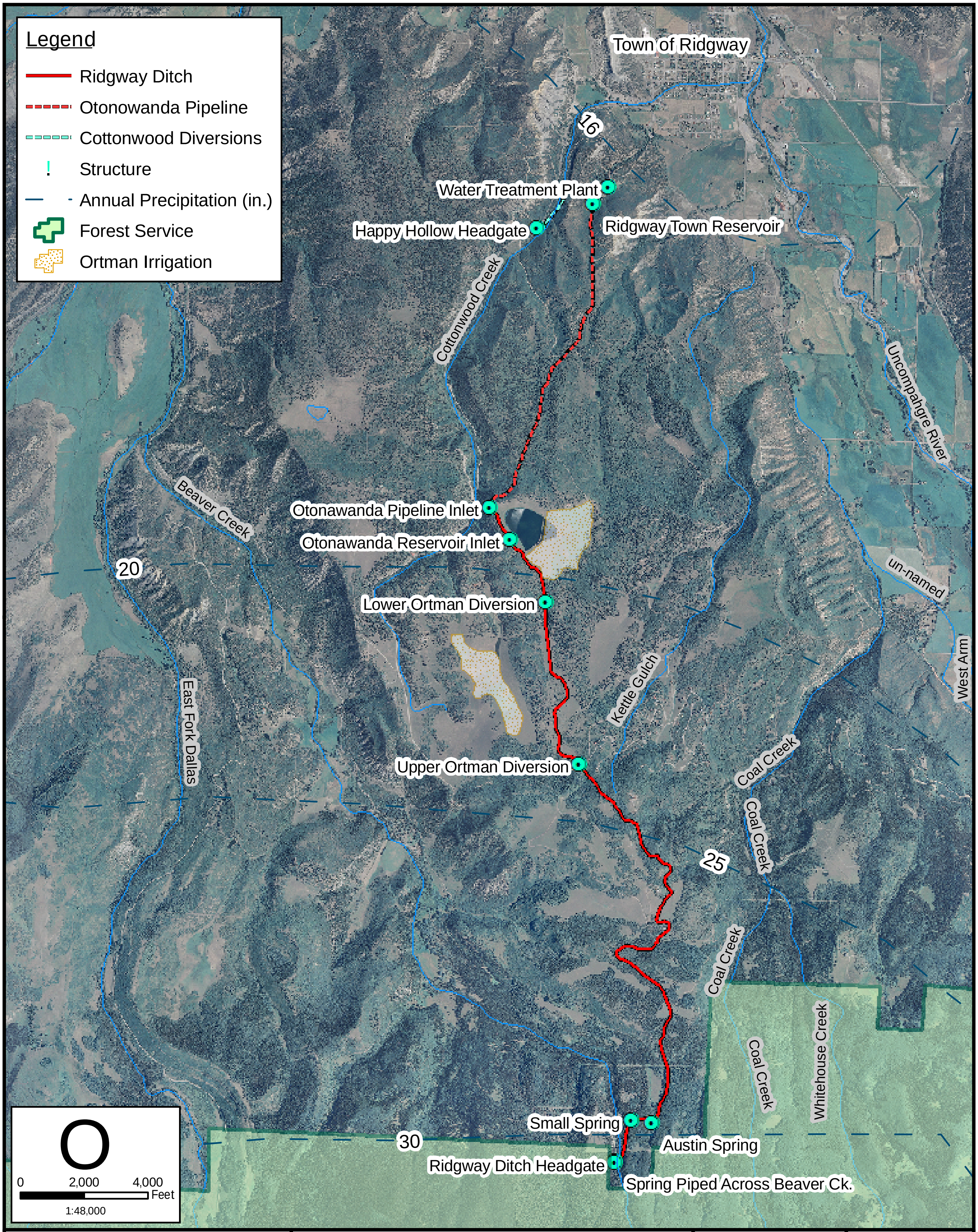
Other structures that benefit from replacement someday include the Lake Otonowanda Reservoir Inlet and the Happy Hollow Diversion. These structures do not allow the user a lot of control over the division of water and could be improved by installing new structures with gated diversions.

RECOMMENDATIONS FOR ADDITIONAL STUDY

This study relied upon several key assumptions that should be verified prior to proceeding with the construction of any storage facility. We recommend that the Town investigate the following items and determine if the findings will influence the conclusions of this report.

- Hold necessary conversations with Ortmans regarding the use of their property and what concessions they would require to store water on their property.
- Install a recording device at the Ridgway Ditch headgate and replace the flume if the existing one isn't level. *Estimate Cost of materials $2,900.*
- Install a measuring device and recorder on the Ridgway Ditch before it enters the Otonowanda Pipeline. *Estimated Cost of materials $2,750.*
- Conduct a seepage test at Lake Otonowanda.
- Perform a property line survey. *Estimated Cost $1,900.*
- Investigate the existing tunnel entrance and outlet to further determine the feasibility of reopening the tunnel.
- Revisit assumptions, operational scenarios, and conclusions presented in this report after addressing the items listed above.

After these items have been completed the project could proceed to final design. Final design would include additional boreholes and test pits in the reservoir area and along the outlet alignment in order to determine a more precise amount of onsite materials available for construction and to narrow down the costs of boring a new outlet tunnel.



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Town of Ridgway
Water Supply Delivery Infrastructure
Vicinity Map

Date: 8-04-2010
Job #: 09-139
Drawn By: CU

Figure: 9
Of: 17

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Town of Ridgway

Areas irrigated with Settled Non Potable Water

Date: 8-04-2010

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Figure: 10

Of: 17

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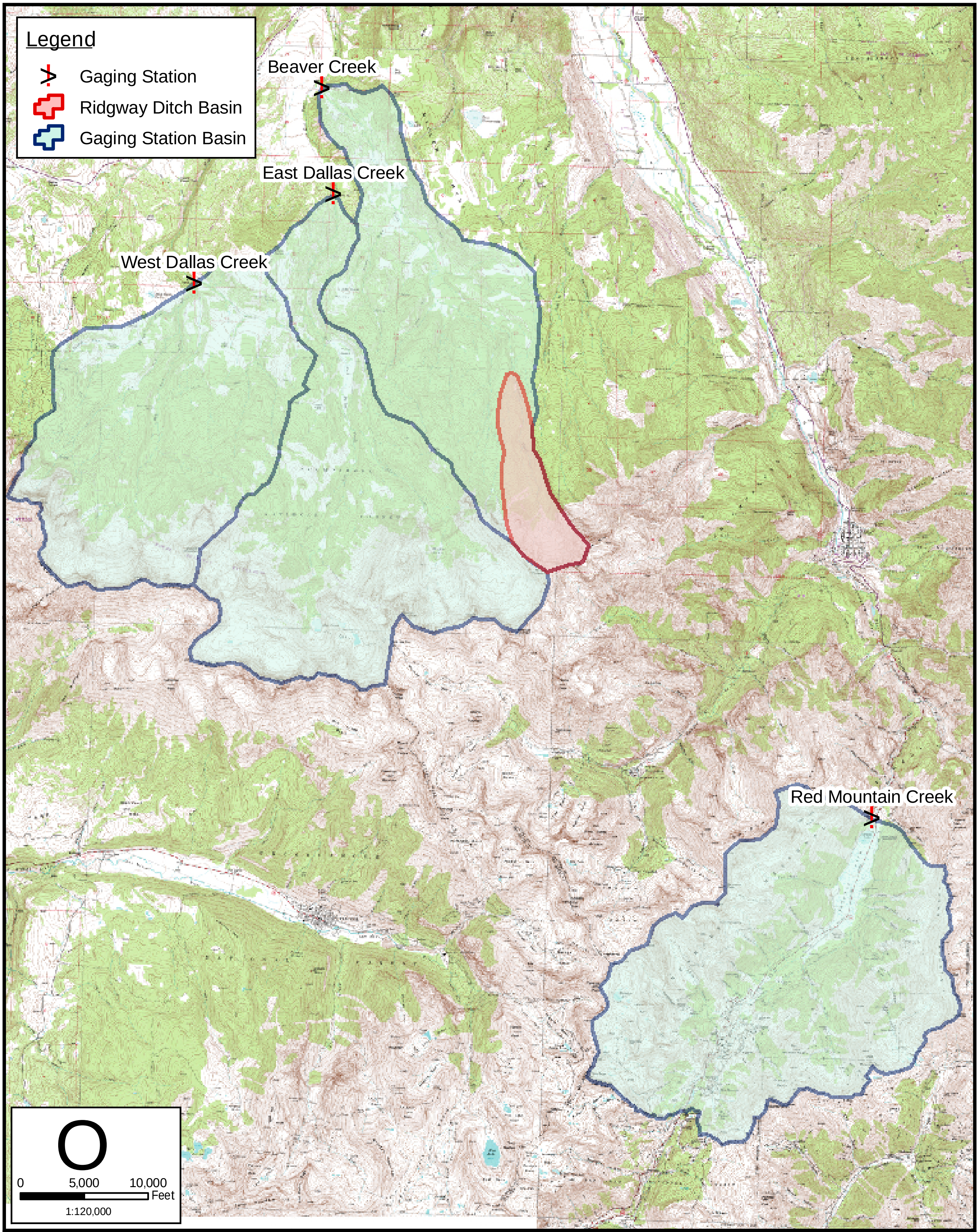

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Town of Ridgway
Historic Stream Gage Locations

Date: 8-04-2010

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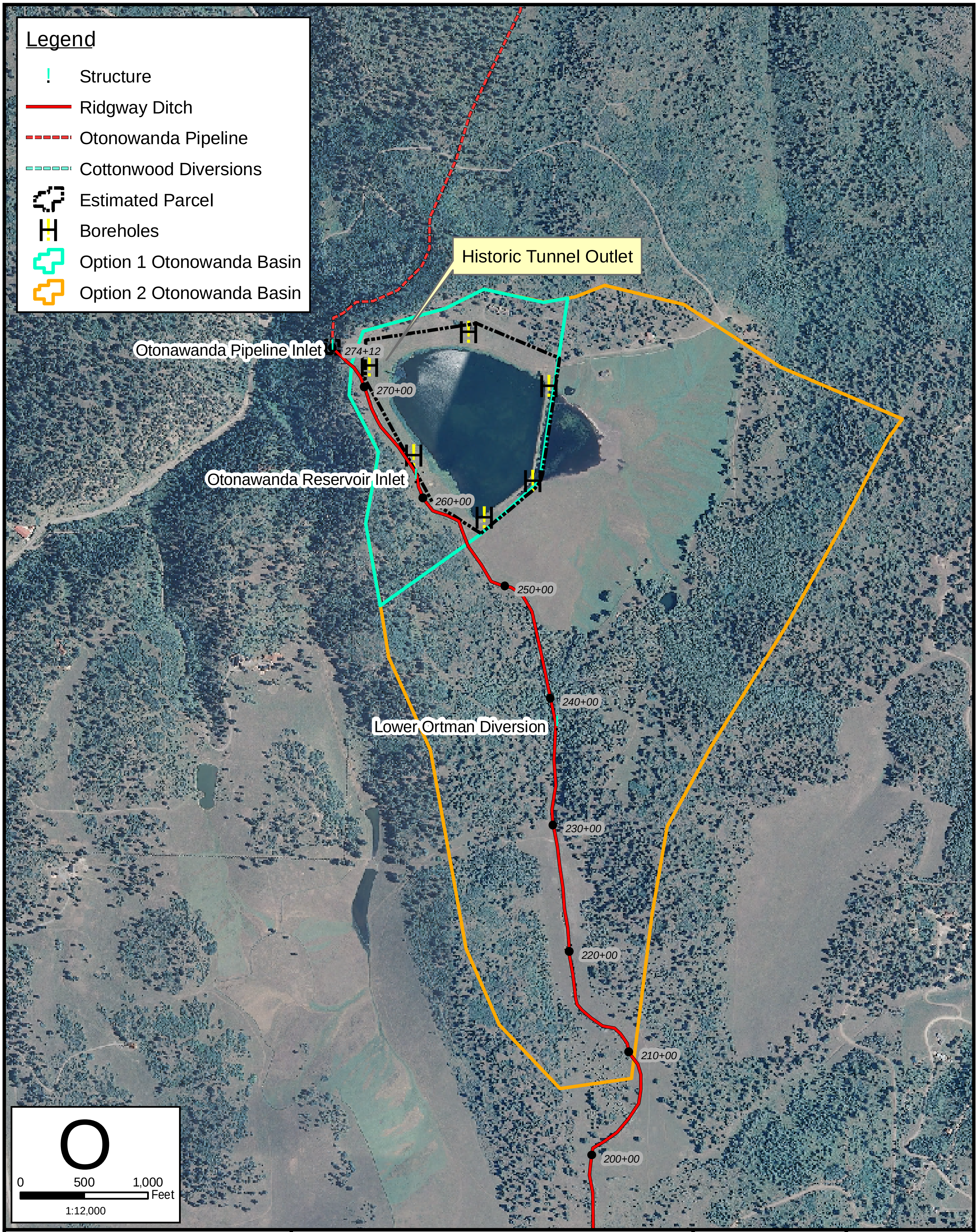

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Town of Ridgway

Lake Otonowanda

Date: Aug 02, 2010

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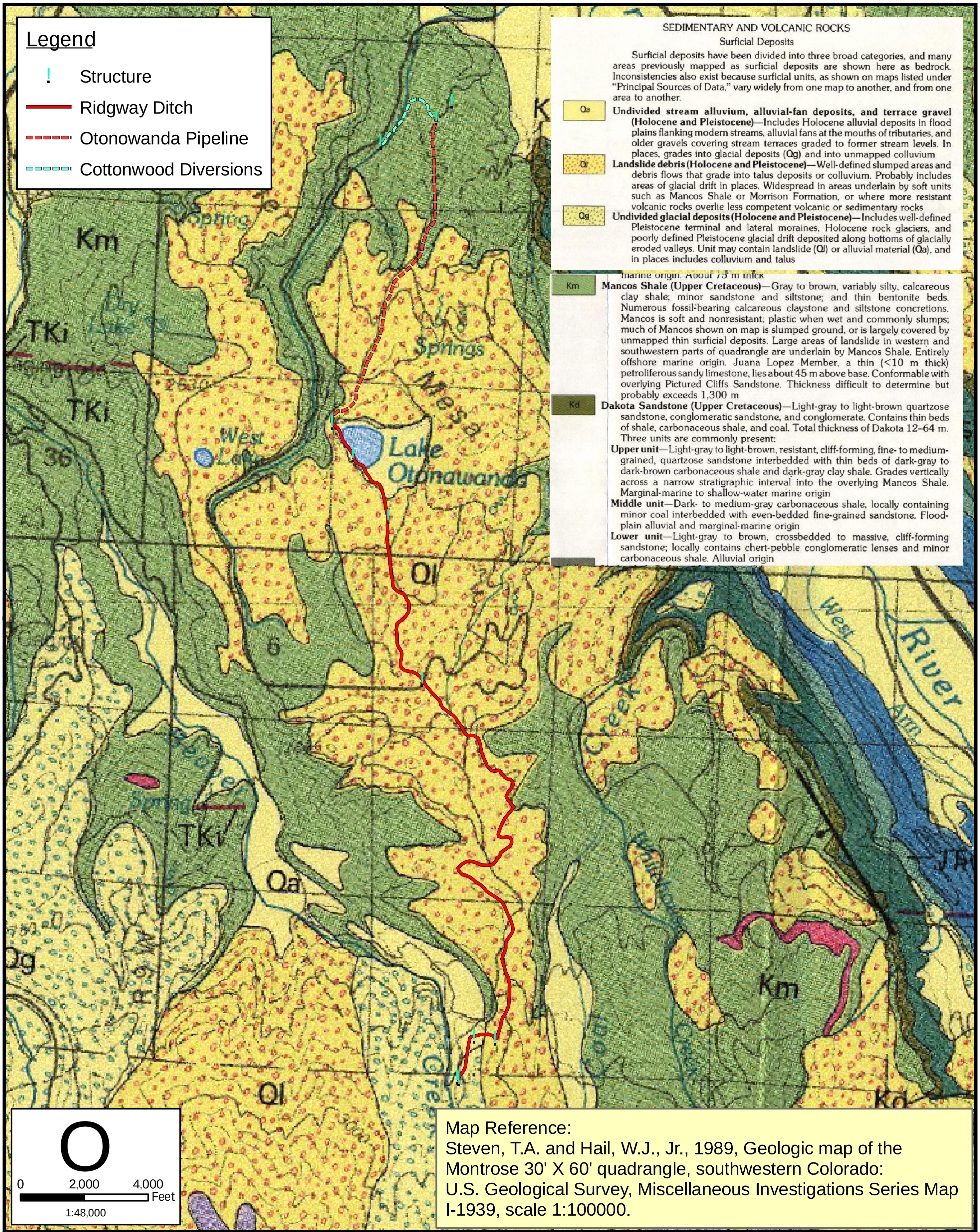

SEDIMENTARY AND VOLCANIC ROCKS

Surficial Deposits

Surficial deposits have been divided into three broad categories, and many areas previously mapped as surficial deposits are shown here as bedrock. Inconsistencies also exist because surficial units, as shown on maps listed under "Principal Sources of Data," vary widely from one map to another, and from one area to another.

Qa **Undivided stream alluvium, alluvial-fan deposits, and terrace gravel (Holocene and Pleistocene)**—Includes Holocene alluvial deposits in flood plains flanking modern streams, alluvial fans at the mouths of tributaries, and older gravels covering stream terraces graded to former stream levels. In places, grades into glacial deposits (Qg) and into unmapped colluvium

Ql **Landslide debris (Holocene and Pleistocene)**—Well-defined slumped areas and debris flows that grade into talus deposits or colluvium. Probably includes areas of glacial drift in places. Widespread in areas underlain by soft units such as Mancos Shale or Morrison Formation, or where more resistant volcanic rocks overlie less competent volcanic or sedimentary rocks

Qg **Undivided glacial deposits (Holocene and Pleistocene)**—Includes well-defined Pleistocene terminal and lateral moraines, Holocene rock glaciers, and poorly defined Pleistocene glacial drift deposited along bottoms of glacially eroded valleys. Unit may contain landslide (Ql) or alluvial material (Qa), and in places includes colluvium and talus

marine origin. About 75 m thick

Km **Mancos Shale (Upper Cretaceous)**—Gray to brown, variably silty, calcareous clay shale; minor sandstone and siltstone; and thin bentonite beds. Numerous fossil-bearing calcareous claystone and siltstone concretions. Mancos is soft and nonresistant; plastic when wet and commonly slumps; much of Mancos shown on map is slumped ground, or is largely covered by unmapped thin surficial deposits. Large areas of landslide in western and southwestern parts of quadrangle are underlain by Mancos Shale. Entirely offshore marine origin. Juana Lopez Member, a thin (<10 m thick) petroliferous sandy limestone, lies about 45 m above base. Conformable with overlying Pictured Cliffs Sandstone. Thickness difficult to determine but probably exceeds 1,300 m

Kd **Dakota Sandstone (Upper Cretaceous)**—Light-gray to light-brown quartzose sandstone, conglomeratic sandstone, and conglomerate. Contains thin beds of shale, carbonaceous shale, and coal. Total thickness of Dakota 12–64 m. Three units are commonly present:

Upper unit—Light-gray to light-brown, resistant, cliff-forming, fine- to medium-grained, quartzose sandstone interbedded with thin beds of dark-gray to dark-brown carbonaceous shale and dark-gray clay shale. Grades vertically across a narrow stratigraphic interval into the overlying Mancos Shale. Marginal-marine to shallow-water marine origin

Middle unit—Dark- to medium-gray carbonaceous shale, locally containing minor coal interbedded with even-bedded fine-grained sandstone. Flood-plain alluvial and marginal-marine origin

Lower unit—Light-gray to brown, crossbedded to massive, cliff-forming sandstone; locally contains chert-pebble conglomeratic lenses and minor carbonaceous shale. Alluvial origin

Map Reference:
Steven, T.A. and Hail, W.J., Jr., 1989, Geologic map of the Montrose 30' X 60' quadrangle, southwestern Colorado:
U.S. Geological Survey, Miscellaneous Investigations Series Map I-1939, scale 1:100000.

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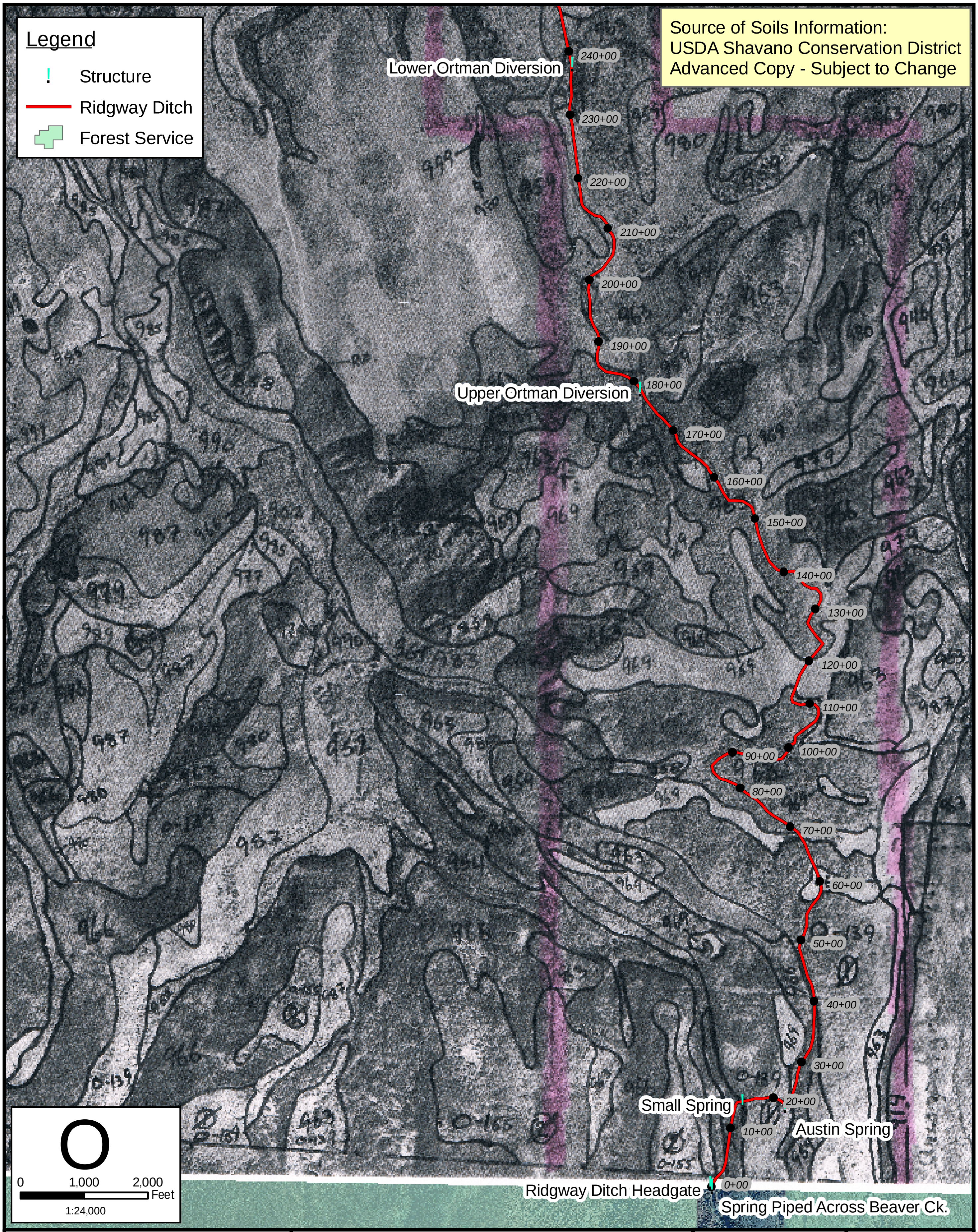
Town of Ridgway
Raw Water System
Geology Map

Date: 8-04-2010
Job #: 09-139
Drawn By: CU

Figure:
13
Of: 17



Source of Soils Information:
USDA Shavano Conservation District
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Town of Ridgway

Ridgway Ditch Soil Types

Date: 12-21-2010

Job #: 09-139

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Figure: 14

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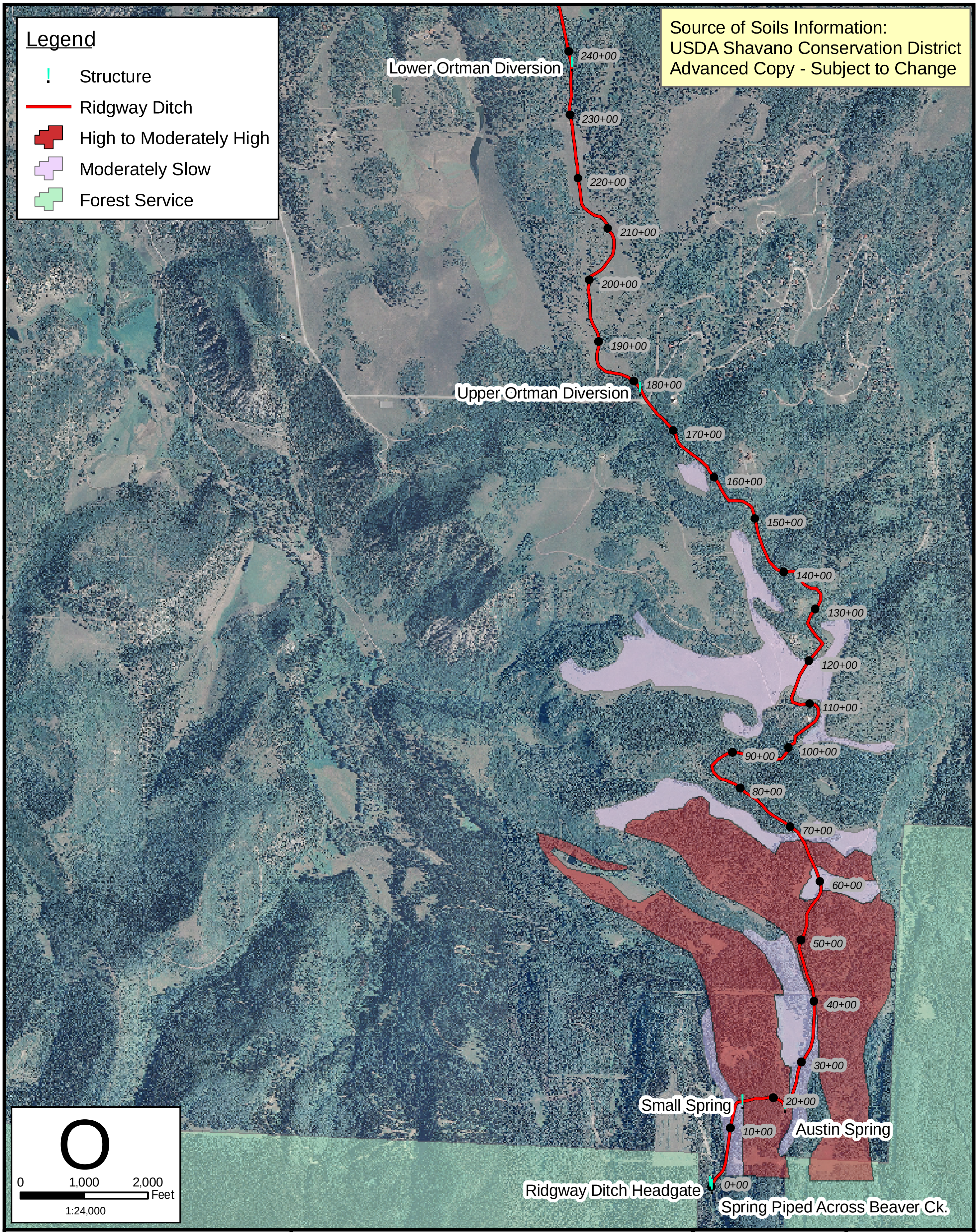

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Town of Ridgway

Ridgway Ditch Soil Permeability

Date: 12-21-2010
Job #: 09-139
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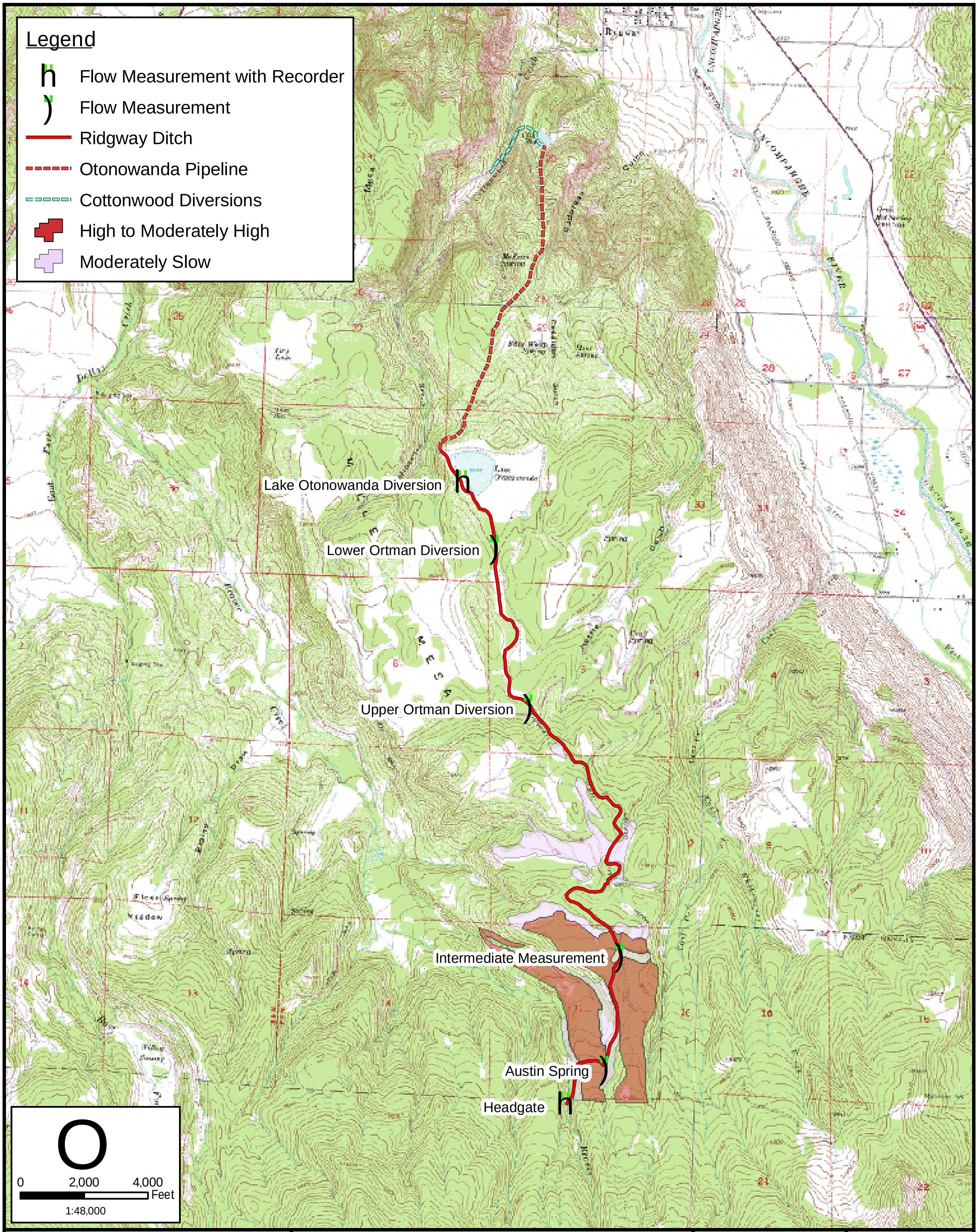

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Town of Ridgway

Recommended Flow Measurement Locations

Date: 12-21-2010

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Town of Ridgway

Ridgway Town Reservoir

Date: 12-20-2010

Job #: 09-139

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Figure: 17

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APPENDICES

APPENDIX A

TOWN OF RIDGWAY

Summary of Water Rights

					Amount			
	Structure	**Adjudication Date**	**Appropriation Date**	**Case No.**	**cfs**	**ac-ft**	**Source**	**Comments**
	Ridgway Ditch	May 15, 1897	June 1, 1882	CA1496	2		Beaver Creek	Transferred from Siebert Ditch; domestic purposes.
	Ridgway Ditch	May 15, 1897	June 1, 1890	CA0939	25		East, West, & Middle Forks of Beaver Creek; West Fork of Coal Creek.	Referenced uses include irrigation and storage in Otonowanda Reservoir by Ridgway Water & Power Co.
Ridgway Water System	Ridgway Ditch	August 2, 1905	June 1, 1890	CA1286	5		East & Middle Forks of Beaver Creek; discharge to Otonowanda Lake.	Limited to a total of 5 cfs including deliveries from Happy Holly Ditch, Happy Hollow Pipeline, and Happy Hollow Branch Pipeline.
	Otonowanda Reservoir	August 2, 1905	June 1, 1890	CA1286		746	Beaver Creek; delivery from Ridgway Ditch.	
	Ridgway Town Reservoir	August 2, 1905	June 1, 1890	CA1286		14.9	Beaver Creek & Cottonwood Creek; delivery from Ridgway Ditch, Otonowanda Pipeline, Happy Hollow Ditch, Happy Hollow Pipeline.	
	Ridgway Pipeline	August 2, 1905	June 1, 1890	CA1286			Beaver Creek & Cottonwood Creek; delivery supply line.	
	Otonowanda Pipeline	August 2, 1905	June 1, 1890	CA1286			Beaver Creek; delivery from Otonowanda Reservoir to Ridgway Reservoir.	
	Happy Hollow Ditch	August 2, 1905	March 1, 1892	CA1286	2		Cottonwood Creek; discharges to Ridgway Reservoir.	Used in conjunction with deliveries from Ridgway Ditch, Otonowanda Reservoir, Ridgway Reservoir, Ridgway Pipeline, and Otonowanda Pipeline.
	Happy Hollow Pipeline	August 2, 1905	March 1, 1892	CA1286			Cottonwood Creek; discharges to Ridgway Reservoir.	
	Happy Hollow Branch Pipeline	August 2, 1905	March 1, 1892	CA1286			Cottonwood Creek; connects with Happy Hollow Pipeline.	
	Ridway Spring No. 2	December 31, 1972	June 1, 1890	W-1305	0.022		Beaver Creek	Co-mingled with Ridgway Ditch diversions.
	Ridgway Spring No. 3	December 31, 1972	June 1, 1890	W-1305	0.026		Beaver Creek	
	Austin Spring	December 31, 1972	June 1, 1890	W-1305	0.13		Beaver Creek	
	Ridway Town Pump Station No. 1	December 31, 1999	October 6, 1999	99CW0265	1.0		Uncompahgre River	Diverts wastewater tributary to Uncompahgre River.
	Hyde Sneva Ditch	May 15, 1897	May 1, 1886	96CW076	0.1146		Dallas Creek	Quit claimed from Robert Savath to the Town of Ridgway; 0.1146 cfs (51.4 gpm) of the Hyde Sneva Ditch; no change of use, only change in place of use.

TABLE
Summary of Ridgway Ditch Diversions

Ridgway Ditch "2 cfs" right (af)

Year	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Total
1999	65.0	71.9	71.9	65.0	89.6	89.2	77.1	106.1	122.9	116.9	81.4	95.3	1052.3
2000	28.0	28.9	28.9	27.0	28.9	28.0	77.4	119.0	122.9	63.1	101.8	120.6	774.5
2001	119.0	122.9	122.9	111.0	122.9	119.0	122.9	119.0	122.9	105.4	119.0	115.5	1422.6
2002	104.1	107.6	107.6	97.2	107.6	78.8	106.8	119.0	115.4	74.9	50.6	83.2	1152.9
2003	64.8	66.1	61.5	55.5	61.5	61.9	70.7	117.3	118.5	121.8	95.5	78.5	973.7
2004	59.5	61.5	61.5	57.5	65.2	115.4	111.0	100.6	122.9	122.9	119.0	122.9	1120.0
2005	119.0	122.9	122.9	103.1	61.5	79.3	122.9	119.0	122.9	122.9	119.0	122.9	1338.5
2006	116.1	113.1	113.1	102.2	113.1	111.4	122.9	119.0	122.9	104.6	112.8	106.5	1357.8
2007	92.0	99.6	99.6	89.9	76.2	59.5	122.9	119.0	122.9	122.9	119.0	122.9	1246.6
2008	119.0	122.9	122.9	115.0	122.9	25.3	60.5	119.0	122.9	61.2	93.0	120.1	1204.7
2009	112.6	122.9	122.9	111.0	104.1	89.2	122.9	119.0	122.9	89.2	83.7	109.1	1309.7
Average	90.8	94.6	94.2	85.0	86.7	77.9	101.7	116.0	121.9	100.5	99.5	108.9	1177.6

Ridgway Ditch "25 cfs" right (af)

Year	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Total
1999	0.0	0.0	0.0	0.0	0.0	0.0	0.0	203.0	116.6	56.5	3.8	5.7	385.6
2000	0.0	0.0	0.0	0.0	0.0	0.0	85.0	94.4	156.2	3.2	0.0	10.9	349.7
2001	0.0	0.0	0.0	0.0	0.0	0.0	15.8	245.6	189.6	62.3	75.3	8.6	597.2
2002	0.0	0.0	0.0	0.0	0.0	0.0	1.4	3.7	3.4	0.0	0.0	0.0	8.5
2003	0.0	0.0	0.0	0.0	0.0	0.0	0.0	107.1	50.3	39.4	29.7	0.0	226.5
2004	0.0	0.0	0.0	0.0	0.0	0.0	0.0	51.9	162.2	83.6	110.1	59.3	467.1
2005	0.0	0.0	0.0	0.0	0.0	0.0	60.2	89.9	120.2	111.9	87.6	85.2	554.9
2006	0.0	0.0	0.0	0.0	0.0	0.0	90.5	136.5	81.3	69.2	17.9	12.6	407.9
2007	0.0	0.0	0.0	0.0	0.0	0.0	172.8	111.1	79.2	88.1	92.4	20.5	564.1
2008	0.0	0.0	0.0	0.0	0.0	0.0	99.8	125.7	180.2	35.0	66.4	33.2	540.2
2009	0.0	0.0	0.0	0.0	0.0	0.0	28.7	140.9	152.1	36.6	0.0	5.8	364.1
Average	0.0	0.0	0.0	0.0	0.0	0.0	50.4	119.1	117.4	53.3	43.9	22.0	406.0

Ridgway Ditch "5 cfs" right (af)

Year	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Total
1999	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2000	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2001	14.9	15.4	15.4	13.9	15.4	14.9	0.0	0.0	0.0	0.0	0.0	0.0	89.7
2002	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2003	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2004	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2005	29.7	30.7	30.7	23.8	0.0	28.6	0.0	0.0	0.0	0.0	0.0	0.0	143.6
2006	65.0	0.0	0.0	0.0	0.0	5.2	0.0	0.0	0.0	0.0	0.0	0.0	70.2
2007	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2008	16.1	18.4	18.4	17.3	18.4	1.8	0.0	0.0	0.0	0.0	0.0	0.0	90.4
2009	20.8	30.7	30.7	27.8	11.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	122.0
Average	13.3	8.7	8.7	7.5	4.2	4.6	0.0	0.0	0.0	0.0	0.0	0.0	46.9

Ridgway Ditch total diversions (af)

Year	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Total
1999	65.0	71.9	71.9	65.0	89.6	89.2	77.1	309.1	239.5	173.4	85.2	101.0	1437.9
2000	28.0	28.9	28.9	27.0	28.9	28.0	162.5	213.3	279.1	66.3	101.8	131.5	1124.2
2001	133.9	138.3	138.3	124.9	138.3	133.9	138.8	364.6	312.5	167.7	194.3	124.1	2109.5
2002	104.1	107.6	107.6	97.2	107.6	78.8	108.3	122.7	118.8	74.9	50.6	83.2	1161.4
2003	64.8	66.1	61.5	55.5	61.5	61.9	70.7	224.4	168.8	161.2	125.2	78.5	1200.2
2004	59.5	61.5	61.5	57.5	65.2	115.4	111.0	152.5	285.2	206.5	229.0	182.2	1587.0
2005	148.7	153.7	153.7	126.9	61.5	107.9	183.1	208.9	243.1	234.8	206.5	208.1	2037.0
2006	181.1	113.1	113.1	102.2	113.1	116.6	213.4	255.5	204.2	173.8	130.7	119.2	1835.9
2007	92.0	99.6	99.6	89.9	76.2	59.5	295.7	230.0	202.1	211.1	211.4	143.4	1810.7
2008	135.0	141.4	141.4	132.3	141.4	27.1	160.2	244.7	303.2	96.1	159.3	153.3	1835.4
2009	133.4	153.7	153.7	138.8	116.0	89.2	151.6	259.9	275.0	125.9	83.7	114.9	1795.7
Average	104.1	103.2	102.8	92.5	90.8	82.5	152.0	235.0	239.2	153.8	143.4	130.9	1630.4

State of Colorado

Annual Water Diversion Report

HydroBase

Structure Name: **HAPPY HOLLOW DITCH**

Division: **4** **Water District:** **68** **Structure Id:** **591**

Location:

Q10	Q40	Q160	Section	Twnshp	Range	PM
	NW	SW	31	45N	8W	N

Distance From Section Lines: From N/S Line: From E/W Line:

UTM Coordinates (NAD 83): Northing (UTM y): 4224592 Easting (UTM x): 256730 Spotted from PLSS distances from section lines

Latitude/Longitude (decimal degrees):

IYR	**Identifier**	**FDU**	**LDU**	**DWC**	**Max Q/Date**	**NOBS**	**NUS**	**Nov**	**Dec**	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Annual Total**	**Unit**
2004	Total	2004-05	10-2004	161	1.31 05-24	9	0	0.00	0.00	0.00	0.00	0.00	0.00	20.79	72.40	60.44	70.71	57.96	48.28	330.57	AF
2005	Total	2004-11	10-2005	365	1.59 10-31	14	0	38.08	39.35	39.35	37.93	57.80	53.36	57.28	58.73	58.26	67.84	68.43	86.90	663.30	AF
2006	Total	2005-11	10-2006	365	1.59 11-01	14	0	93.19	92.85	92.85	83.86	92.85	87.35	62.40	40.33	44.99	55.46	47.43	53.06	846.60	AF
2007	Total	2006-11	10-2007	365	1.13 10-31	15	0	56.29	57.80	57.80	52.21	46.49	38.08	40.19	45.94	35.07	35.80	34.67	38.00	538.34	AF
2008	Total	2007-11	10-2008	366	1.13 11-01	12	0	50.28	34.43	34.43	32.21	34.43	36.54	38.64	34.75	42.86	31.32	31.14	29.99	431.03	AF

Legend for identifier string coding:

Source (S): 1 - Natural Streamflow, 2 - Reservoir Storage, 3 - Ground water (wells), 4 - Transbasin, 5 - Non-stream (springs, run-off), 6 - Combined, 7 - Transdistrict, 8 - Re-used, 9 - Multiple, R - Remeasured and rediverted
From (F): From structure WDID

Use (U): 0 - Storage, 1 - Irrigation, 2 - Municipal, 3 - Commercial, 4 - Industrial, 5 - Recreation, 6 - Fishery, 7 - Fire, 8 - Domestic, 9 - Stock, A - Augmentation, B - Export from basin, C - Cumulative accretion to river, D - Cumulative depletion from river, E - Evaporation, F - Federal reserve, G - Geothermal, H - Household use only, K - Snow making, M - Minimum streamflow, N - Net effect of river, P - Power generation, Q - Other, R - Recharge, S - Export from state, T - Transmountain export, W - Wildlife, X - All beneficial use

Diversion Type (T): 0 - Administrative record only, 1 - Exchange, 2 - Trade, 3 - Carrier, 4 - Alternate point of diversion, 5 - Re-used, 6 - Replacement to river, 7 - Released by river, 8 - Released to stream, A - Augmented, G - Geothermal, S - Reservoir substitution

Group (G): Group structure WDID

Report Date: 2010-01-26

HydroBase Refresh Date: 2009-12-01

Town of Ridgway Treated Water Production

	2009*	2008*	2007	2006	2005	2004	2003	2002
January	4,234,930	3,474,221	3,021,900	2,775,770	2,888,600	2,898,100	2,725,200	3038400
February	4,517,354	3,456,337	2,725,400	2,580,300	2,618,200	2,810,800	1,798,900	2887400
March	4,245,767	3,468,098	3,259,600	3,945,380	2,998,600	2,784,000	2,380,870	3237100
April	3,699,728	3,382,411	3,046,500	2,989,930	2,989,930	2,722,900	2,812,160	3896800
May	5,844,847	5,081,963	4,433,500	7,140,370	4,993,120	4,977,500	4,975,740	6368300
June	6,899,483	9,327,441	8,024,300	8,699,340	6,201,130	6,732,900	7,080,460	7899500
July	8,585,637	7,877,548	8,172,000	6,864,350	8,632,790	6,345,100	8,228,860	7723800
August	8,726,116	8,167,293	6,353,000	5,432,290	5,373,030	5,470,500	5,526,320	5692900
September	7,262,106	6,847,320	6,082,700	4,965,540	4,846,050	4,526,800	4,003,440	3761800
October	5,564,003	5,131,927	3,718,200	3,383,800	3,691,890	3,329,400	3,790,930	3520100
November	4,058,563	3,029,000	3,250,600	2,687,820	2,775,600	4,783,326	3248900	
December	4,082,857	3,346,200	3,346,130	3,027,220	2,932,300	2,973,520	3613100	
Total	**59,579,971**	**64,355,979**	**55,212,300**	**55,373,800**	**50,948,380**	**48,305,900**	**51,081,729**	**54,890,102**

* includes 200 gpm recirculation for 12+ hrs to heat water for each CIP 1.8+ MG

Town of Ridgway

Applegate Group, Inc.

Evaporation

Month	Percent of Annual Evaporation (A)	Average Monthly Temperature (B)	Gross Lake Evaporation (ft) (C)	Total Lake Evaporation (ac-ft) (D)
January	1.0%	22.4	0.00	0.00
February	3.0%	27.0	0.00	0.00
March	6.0%	34.6	0.19	4.85
April	9.0%	41.7	0.28	7.27
May	12.5%	50.4	0.39	10.10
June	15.5%	58.0	0.48	12.52
July	16.0%	63.5	0.49	12.93
August	13.0%	62.1	0.40	10.50
September	11.0%	54.5	0.34	8.89
October	7.5%	43.5	0.23	6.06
November	4.0%	32.0	0.00	0.00
December	1.5%	22.8	0.00	0.00
Total	**100%**		**2.79**	**73.11**

Evaporation
June-September =
44.83 acre-feet

Total Exposed Water Surface = 26.2 acres
Gross Annual Evaporation = 37 inches - Estimated using NOAA Technical Report NWS 33

(A) Percent of annual evaporation from SEO guidelines for elevations above 6,500 feet
(B) Average monthly temperature (^{o}F) from Ridgway weather station data.
(C) Gross lake evaporation = (A) * [Gross Annual Evaporation/12]
(D) Total lake evaporation = (B) * Total Exposed Water Surface

TOWN OF RIDGWAY

Summary of Projected Future Operational Scenarios - Reservoir Option 1, On Property

Transit Loss	35%	
Otonowanda Reservoir max storage	347	ac-ft
Otonowanda Reservoir Beginning Storage	0	ac-ft

33 % total diversions by Town from Ridgway Ditch during irrigation season (May - Sep); Happy Hollow rights not called out.

YEAR 1 - normal year		1-Nov	1-Dec	1-Jan	1-Feb	1-Mar	1-Apr	1-May	1-Jun	1-Jul	1-Aug	1-Sep	Oct	Total
Supply	Total Ridgway Ditch Average Diversion (1)	60	48	40	32	34	48	158	240	245	159	149	136	1349
	Town Ridgway Ditch Average Diversion	60	48	40	32	34	48	52	79	81	52	49	136	712
	Transit Loss	21	17	14	11	12	17	18	28	28	18	17	48	249
	Net Town Ridgway Ditch Diversion	39	31	26	21	22	31	34	51	53	34	32	88	463
	Happy Hollow Ditch Average Diversion	55	52	52	48	55	52	43	50	47	49	43	51	598
	Total Diversions	**94**	**84**	**78**	**69**	**77**	**83**	**77**	**101**	**100**	**83**	**75**	**140**	**1061**
Demands	Projected Treated Water Demands, 2030	21	20	19	18	20	20	34	47	48	39	32	25	342
	Projected Raw Water Demand, 2030							22	31	32	26	22	16	149
	Total Demand	**21**	**20**	**19**	**18**	**20**	**20**	**56**	**78**	**80**	**65**	**54**	**41**	**491**
	Demand Met by Happy Hollow	21	20	19	18	20	20	43	50	47	49	43	41	392
	Happy Hollow surplus	34	32	33	30	35	32						10	206
	Happy Hollow deficit	0	0	0	0	0	0	(13)	(28)	(33)	(16)	(11)	0	(100)
Lake O	Demands Met From Lake Otonowanda	0	0	0	0	0	0	13	28	33	16	11	0	100
	Evaporation Depth (ft)	0	0	0	0	0.19	0.28	0.39	0.48	0.49	0.40	0.34	0.23	2.80
	Otonowanda Surface Area (ac)	0	22	24	24	25	25	26	26	26	27	27	27	
	Evaporative Loss	0	0	0	0	5	7	10	13	13	11	9	6	74
	End of Month Storage	39	70	96	117	134	158	169	181	188	195	207	290	
	Ridgway Ditch Surplus	0	0	0	0	0	0	0	0	0	0	0	0	0

100 % total diversions by Town from Ridgway Ditch during irrigation season (May - Sep); All Ridgway Ditch & Happy Hollow rights called out June through September.

YEAR 2 - 1st drought cycle		Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Total
Supply	Total Ridgway Ditch 2002 Diversion (2)	48	41	35	24	22	31	112	0	0	0	0	89	402
	Town Ridgway Ditch 2002 Diversion	48	41	35	24	22	31	112	0	0	0	0	89	402
	Transit Loss	17	14	12	8	8	11	39	0	0	0	0	31	141
	Net Town Ridgway Ditch Diversion	31	27	23	16	14	20	73	0	0	0	0	58	261
	Happy Hollow Ditch 2002 Diversion	44	45	46	36	35	33	31	0	0	0	0	34	304
	Total Diversions	**75**	**72**	**69**	**52**	**49**	**53**	**104**	**0**	**0**	**0**	**0**	**92**	**565**
Demands	Projected Treated Water Demands, 2030	21	20	19	18	20	20	34	47	48	39	32	25	342
	Projected Raw Water Demand, 2030							22	31	32	26	22	16	149
	Total Demand	**21**	**20**	**19**	**18**	**20**	**20**	**56**	**78**	**80**	**65**	**54**	**41**	**491**
	Demand Met by Happy Hollow	21	20	19	18	20	20	31	0	0	0	0	34	183
	Happy Hollow surplus	23	25	27	18	15	13							121
	Happy Hollow deficit	0	0	0	0	0	0	(25)	(78)	(80)	(65)	(54)	(7)	(308)
Lake O	Demands Met From Lake Otonowanda	0	0	0	0	0	0	25	78	80	65	54	7	308
	Evaporation Depth (ft)	0	0	0	0	0.19	0.28	0.39	0.48	0.49	0.40	0.34	0.23	2.80
	Otonowanda Surface Area (ac)	29	29	30	30	30	30	30	30	28	26	24	20	
	Evaporative Loss	0	0	0	0	6	8	12	14	14	10	8	5	77
	End of Month Storage	321	347	347	347	347	347	347	255	162	86	24	70	
	Ridgway Ditch Surplus	0	0	23	16	9	12	36	0	0	0	0	0	96

100 % total diversions by Town from Ridgway Ditch during irrigation season (May - Sep); Junior Ridgway Ditch & Happy Hollow rights called out June through September.

YEAR 3 - 2nd drought cycle		Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Total
Supply	Total Ridgway Ditch 2003 Diversion(3)	48	41	35	24	22	31	76	123	124	127	101	84	836
	Town Ridgway Ditch 2003 Diversion	48	41	35	24	22	31	76	123	124	127	101	84	836
	Transit Loss	17	14	12	8	8	11	27	43	43	45	35	29	293
	Net Town Ridgway Ditch Diversion	31	27	23	16	14	20	50	80	81	83	66	55	544
	Happy Hollow Ditch 2003 Diversion	44	45	46	36	35	33	21	0	0	0	0	32	292
	Total Diversions	**75**	**72**	**69**	**52**	**49**	**53**	**71**	**80**	**81**	**83**	**66**	**87**	**836**
Demands	Projected Treated Water Demands, 2030	21	20	19	18	20	20	34	47	48	39	32	25	342
	Projected Raw Water Demand, 2030							22	31	32	26	22	16	149
	Total Demand	**21**	**20**	**19**	**18**	**20**	**20**	**56**	**78**	**80**	**65**	**54**	**41**	**491**
	Demand Met by Happy Hollow	21	20	19	18	20	20	21	0	0	0	0	32	171
	Happy Hollow surplus	23	25	27	18	15	13							121
	Happy Hollow deficit	0	0	0	0	0	0	(35)	(78)	(80)	(65)	(54)	(9)	(320)
Lake O	Demands Met From Lake Otonowanda	0	0	0	0	0	0	35	78	80	65	54	9	320
	Evaporation Depth (ft)	0	0	0	0	0.19	0.28	0.39	0.48	0.49	0.40	0.34	0.23	2.80
	Otonowanda Surface Area (ac)	24	25	25	26	26	26	27	27	26	26	26	26	
	Evaporative Loss	0	0	0	0	5	7	10	13	13	10	9	6	74
	End of Month Storage	101	128	151	166	176	188	193	182	170	177	180	220	
	Ridgway Ditch Surplus	0	0	0	0	0	0	0	0	0	0	0	0	0

100 % total diversions by Town from Ridgway Ditch during irrigation season (May - Sep); Junior Ridgway Ditch & Happy Hollow rights called out June through September.

YEAR 4 - 3rd drought cycle		Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Total
Supply	Total Ridgway Ditch 2003 Diversion(3)	48	41	35	24	22	31	76	123	124	127	101	84	836
	Town Ridgway Ditch 2003 Diversion	48	41	35	24	22	31	76	123	124	127	101	84	836
	Transit Loss	17	14	12	8	8	11	27	43	43	45	35	29	293
	Net Town Ridgway Ditch Diversion	31	27	23	16	14	20	50	80	81	83	66	55	543
	Happy Hollow Ditch 2003 Diversion	44	45	46	36	35	33	21	0	0	0	0	32	292
	Total Diversions	**75**	**72**	**69**	**52**	**49**	**53**	**71**	**80**	**81**	**83**	**66**	**87**	**835**
Demands	Projected Treated Water Demands, 2030	21	20	19	18	20	20	34	47	48	39	32	25	342
	Projected Raw Water Demand, 2030							22	31	32	26	22	16	149
	Total Demand	**21**	**20**	**19**	**18**	**20**	**20**	**56**	**78**	**80**	**65**	**54**	**41**	**491**
	Demand Met by Happy Hollow	21	20	19	18	20	20	21	0	0	0	0	32	171
	Happy Hollow surplus	23	25	27	18	15	13							121
	Happy Hollow deficit	0	0	0	0	0	0	(35)	(78)	(80)	(65)	(54)	(9)	(320)
Lake O	Demands Met From Lake Otonowanda	0	0	0	0	0	0	35	78	80	65	54	9	320
	Evaporation Depth (ft)	0	0	0	0	0.19	0.28	0.39	0.48	0.49	0.40	0.34	0.23	2.80
	Otonowanda Surface Area (ac)	19	28	28	29	29	29	30	30	29	29	29	29	
	Evaporative Loss	0	0	0	0	6	8	12	14	14	12	10	7	82
	End of Month Storage	251	277	300	316	324	336	339	327	314	320	322	347	
	Ridgway Ditch Surplus	0	0	0	0	0	0	0	0	0	0	0	14	14

33 % total diversions by Town from Ridgway Ditch during irrigation season (May - Sep); Happy Hollow rights not called out.

YEAR 5 - normal year		Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Total
Supply	Total Ridgway Ditch Average Diversion (1)	60	48	40	32	34	48	158	240	245	159	149	136	1349
	Town Ridgway Ditch Average Diversion	60	48	40	32	34	48	52	79	81	52	49	136	712
	Transit Loss	21	17	14	11	12	17	18	28	28	18	17	48	249
	Net Town Ridgway Ditch Diversion	39	31	26	21	22	31	34	51	53	34	32	88	463
	Happy Hollow Ditch Average Diversion	55	52	52	48	55	52	43	50	47	49	43	51	598
	Total Diversions	**94**	**84**	**78**	**69**	**77**	**83**	**77**	**101**	**100**	**83**	**75**	**140**	**1061**
Demands	Projected Treated Water Demands, 2030	21	20	19	18	20	20	34	47	48	39	32	25	342
	Projected Raw Water Demand, 2030							22	31	32	26	22	16	149
	Total Demand	**21**	**20**	**19**	**18**	**20**	**20**	**56**	**78**	**80**	**65**	**54**	**41**	**491**
	Demand Met by Happy Hollow	21	20	19	18	20	20	43	50	47	49	43	41	392
	Happy Hollow surplus	34	32	33	30	35	32						10	206
	Happy Hollow deficit	0	0	0	0	0	0	(13)	(28)	(33)	(16)	(11)	0	(100)
Lake O	Demands Met From Lake Otonowanda	0	0	0	0	0	0	13	28	33	16	11	0	100
	Evaporation Depth (ft)	0	0	0	0	0.19	0.28	0.39	0.48	0.49	0.40	0.34	0.23	2.80
	Otonowanda Surface Area (ac)	19	30	30	30	30	30	30	30	30	30	30	30	
	Evaporative Loss	0	0	0	0	6	8	12	14	15	12	10	7	83
	End of Month Storage	347	347	347	347	347	347	347	347	347	347	347	347	
	Ridgway Ditch Surplus	39	31	26	21	16	23	10	9	5	6	11	82	280

Notes

(1) November-April diversions based on average water availibility determined by synthetic hydrology for Beaver Creek @ R. Ditch headgate. May-Oct based on average R. Ditch diversion records (2000-2009)

(2) November-April diversions based on dry year availibility determined by synthetic hydrology for Beaver Creek @ R. Ditch headgate. May and Oct based on 2002 R. Ditch diversion records

(3) November-April diversions based on dry year availibility determined by synthetic hydrology for Beaver Creek @ R. Ditch headgate. May-Oct based on 2003 R. Ditch diversion records for senior 2 cfs right

Total Ridgway Ditch diversions (average & dry-year) includes additional 0.09 cfs for spring inflow not reflected in headgate diversions.

Evaporative loss based on 37 inches of evaporation per year; no evaporation in months where average temp less than 32 degrees.

Seepage and Rainfall are assumed to balance one another due to uncertainties regarding seepage rate

Happy Hollow Ditch diversions based on average of 2004 - 2009 records.

Year 2030 treated water demands based on growth rate of 3.6% per year.

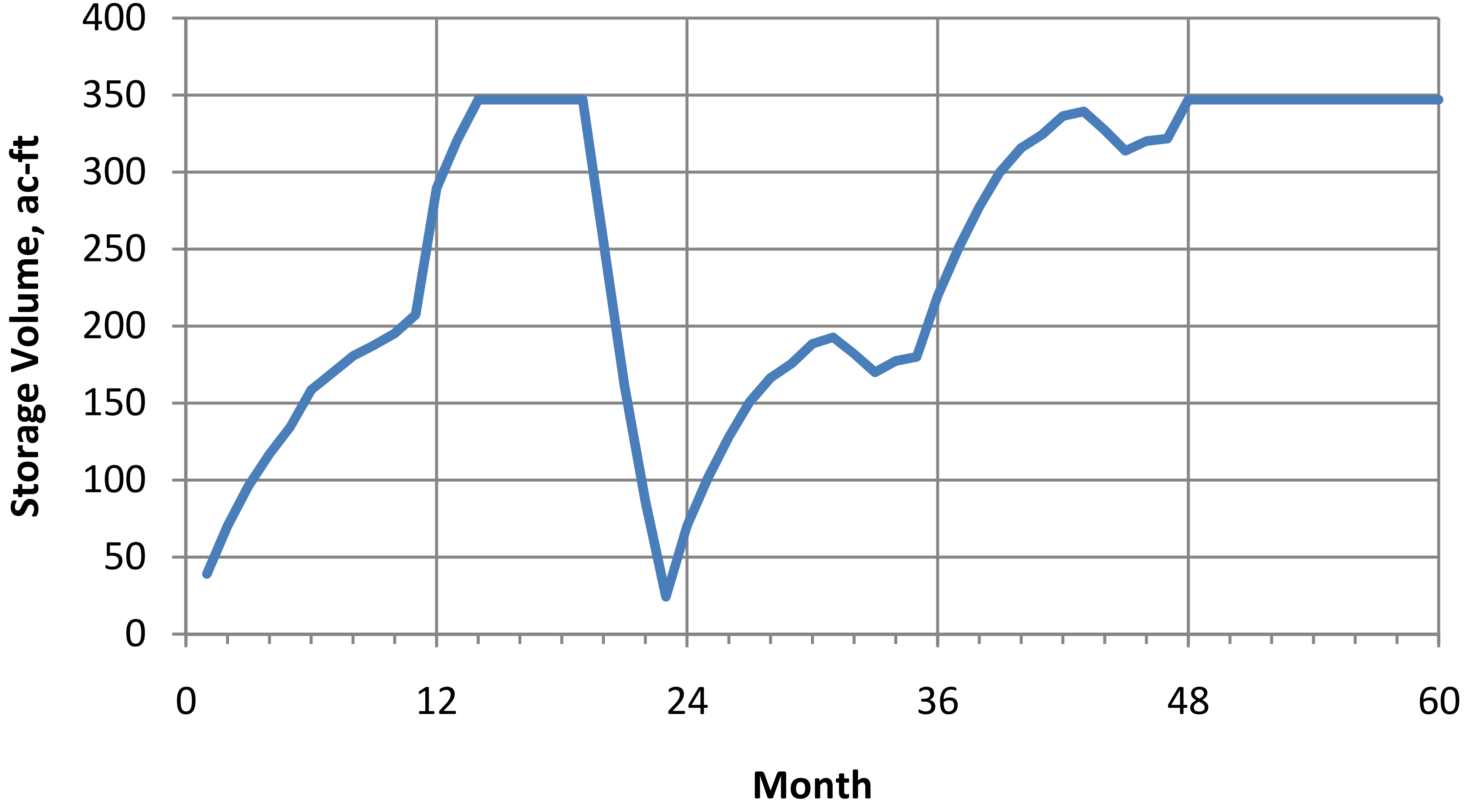
Year 2030 raw water demands based on growth rate of 35% of current existing irrigation demands.

Treated water demands first met by Happy Hollow supply; remaining demands met by releases from Otonowanda Reservoir.

The portion of the Ridgway Ditch diverted for the towns use is all placed into Otonowanda Reservoir except when it is full at which point is is shown as a surplus

Adjusted storage limited to maximum projected capacity accounting for deliveries and evaporation loss.

Otonowanda Operations - Option 1



TOWN OF RIDGWAY

Summary of Projected Future Operational Scenarios - Reservoir Option 2, Off Property

Transit Loss	35%	
Otonowanda Reservoir max storage	377	ac-ft
Otonowanda Reservoir Beginning Storage	50	ac-ft

33 % total diversions by Town from Ridgway Ditch during irrigation season (May - Sep); Happy Hollow rights not called out.

YEAR 1 - normal year		1-Nov	1-Dec	1-Jan	1-Feb	1-Mar	1-Apr	1-May	1-Jun	1-Jul	1-Aug	1-Sep	Oct	Total
Supply	Total Ridgway Ditch Average Diversion (1)	60	48	40	32	34	48	158	240	245	159	149	136	1349
	Town Ridgway Ditch Average Diversion	60	48	40	32	34	48	52	79	81	52	49	136	712
	Transit Loss	21	17	14	11	12	17	18	28	28	18	17	48	249
	Net Town Ridgway Ditch Diversion	39	31	26	21	22	31	34	51	53	34	32	88	463
	Happy Hollow Ditch Average Diversion	55	52	52	48	55	52	43	50	47	49	43	51	598
	Total Diversions	**94**	**84**	**78**	**69**	**77**	**83**	**77**	**101**	**100**	**83**	**75**	**140**	**1061**
Demands	Projected Treated Water Demands, 2030	21	20	19	18	20	20	34	47	48	39	32	25	342
	Projected Raw Water Demand, 2030							22	31	32	26	22	16	149
	Total Demand	**21**	**20**	**19**	**18**	**20**	**20**	**56**	**78**	**80**	**65**	**54**	**41**	**491**
	Demand Met by Happy Hollow	21	20	19	18	20	20	43	50	47	49	43	41	392
	Happy Hollow surplus	34	32	33	30	35	32						10	206
	Happy Hollow deficit	0	0	0	0	0	0	(13)	(28)	(33)	(16)	(11)	0	(100)
Lake O	Demands Met From Lake Otonowanda	0	0	0	0	0	0	13	28	33	16	11	0	100
	Evaporation Depth (ft)	0	0	0	0	0.19	0.28	0.39	0.48	0.49	0.40	0.34	0.23	2.80
	Otonowanda Surface Area (ac)	32	37	41	44	46	47	49	49	49	49	49	49	
	Evaporative Loss	0	0	0	0	9	13	19	24	24	20	17	11	137
	End of Month Storage	89	120	146	167	180	198	200	200	196	195	199	276	
	Ridgway Ditch Surplus	0	0	0	0	0	0	0	0	0	0	0	0	0

100 % total diversions by Town from Ridgway Ditch during irrigation season (May - Sep); All Ridgway Ditch & Happy Hollow rights called out June through September.

YEAR 2 - 1st drought cycle		Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Total
Supply	Total Ridgway Ditch 2002 Diversion (2)	48	41	35	24	22	31	112	0	0	0	0	89	402
	Town Ridgway Ditch 2002 Diversion	48	41	35	24	22	31	112	0	0	0	0	89	402
	Transit Loss	17	14	12	8	8	11	39	0	0	0	0	31	141
	Net Town Ridgway Ditch Diversion	31	27	23	16	14	20	73	0	0	0	0	58	261
	Happy Hollow Ditch 2002 Diversion	44	45	46	36	35	33	31	0	0	0	0	34	304
	Total Diversions	**75**	**72**	**69**	**52**	**49**	**53**	**104**	**0**	**0**	**0**	**0**	**92**	**565**
Demands	Projected Treated Water Demands, 2030	21	20	19	18	20	20	34	47	48	39	32	25	342
	Projected Raw Water Demand, 2030							22	31	32	26	22	16	149
	Total Demand	**21**	**20**	**19**	**18**	**20**	**20**	**56**	**78**	**80**	**65**	**54**	**41**	**491**
	Demand Met by Happy Hollow	21	20	19	18	20	20	31	0	0	0	0	34	183
	Happy Hollow surplus	23	25	27	18	15	13							121
	Happy Hollow deficit	0	0	0	0	0	0	(25)	(78)	(80)	(65)	(54)	(7)	(308)
Lake O	Demands Met From Lake Otonowanda	0	0	0	0	0	0	25	78	80	65	54	7	308
	Evaporation Depth (ft)	0	0	0	0	0.19	0.28	0.39	0.48	0.49	0.40	0.34	0.23	2.80
	Otonowanda Surface Area (ac)	26	58	60	62	63	63	63	63	56	46	36	27	
	Evaporative Loss	0	0	0	0	12	18	25	30	27	18	12	6	148
	End of Month Storage	308	334	357	373	375	377	377	269	162	79	13	57	
	Ridgway Ditch Surplus	0	0	0	0	0	0	23	0	0	0	0	0	24

100 % total diversions by Town from Ridgway Ditch during irrigation season (May - Sep); Junior Ridgway Ditch & Happy Hollow rights called out June through September.

YEAR 3 - 2nd drought cycle		Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Total
Supply	Total Ridgway Ditch 2003 Diversion(3)	48	41	35	24	22	31	76	123	124	127	101	84	836
	Town Ridgway Ditch 2003 Diversion	48	41	35	24	22	31	76	123	124	127	101	84	836
	Transit Loss	17	14	12	8	8	11	27	43	43	45	35	29	293
	Net Town Ridgway Ditch Diversion	31	27	23	16	14	20	50	80	81	83	66	55	544
	Happy Hollow Ditch 2003 Diversion	44	45	46	36	35	33	21	0	0	0	0	32	292
	Total Diversions	**75**	**72**	**69**	**52**	**49**	**53**	**71**	**80**	**81**	**83**	**66**	**87**	**836**
Demands	Projected Treated Water Demands, 2030	21	20	19	18	20	20	34	47	48	39	32	25	342
	Projected Raw Water Demand, 2030							22	31	32	26	22	16	149
	Total Demand	**21**	**20**	**19**	**18**	**20**	**20**	**56**	**78**	**80**	**65**	**54**	**41**	**491**
	Demand Met by Happy Hollow	21	20	19	18	20	20	21	0	0	0	0	32	171
	Happy Hollow surplus	23	25	27	18	15	13							121
	Happy Hollow deficit	0	0	0	0	0	0	(35)	(78)	(80)	(65)	(54)	(9)	(320)
Lake O	Demands Met From Lake Otonowanda	0	0	0	0	0	0	35	78	80	65	54	9	320
	Evaporation Depth (ft)	0	0	0	0	0.19	0.28	0.39	0.48	0.49	0.40	0.34	0.23	2.80
	Otonowanda Surface Area (ac)	26	37	40	43	45	45	46	46	44	41	41	41	
	Evaporative Loss	0	0	0	0	8	13	18	22	21	17	14	9	123
	End of Month Storage	88	115	138	153	159	167	163	143	123	124	122	158	
	Ridgway Ditch Surplus	0	0	0	0	0	0	0	0	0	0	0	0	0

100 % total diversions by Town from Ridgway Ditch during irrigation season (May - Sep); Junior Ridgway Ditch & Happy Hollow rights called out June through September.

YEAR 4 - 3rd drought cycle		Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Total
Supply	Total Ridgway Ditch 2003 Diversion(3)	48	41	35	24	22	31	76	123	124	127	101	84	836
	Town Ridgway Ditch 2003 Diversion	48	41	35	24	22	31	76	123	124	127	101	84	836
	Transit Loss	17	14	12	8	8	11	27	43	43	45	35	29	293
	Net Town Ridgway Ditch Diversion	31	27	23	16	14	20	50	80	81	83	66	55	543
	Happy Hollow Ditch 2003 Diversion	44	45	46	36	35	33	21	0	0	0	0	32	292
	Total Diversions	**75**	**72**	**69**	**52**	**49**	**53**	**71**	**80**	**81**	**83**	**66**	**87**	**835**
Demands	Projected Treated Water Demands, 2030	21	20	19	18	20	20	34	47	48	39	32	25	342
	Projected Raw Water Demand, 2030							22	31	32	26	22	16	149
	Total Demand	**21**	**20**	**19**	**18**	**20**	**20**	**56**	**78**	**80**	**65**	**54**	**41**	**491**
	Demand Met by Happy Hollow	21	20	19	18	20	20	21	0	0	0	0	32	171
	Happy Hollow surplus	23	25	27	18	15	13							121
	Happy Hollow deficit	0	0	0	0	0	0	(35)	(78)	(80)	(65)	(54)	(9)	(320)
Lake O	Demands Met From Lake Otonowanda	0	0	0	0	0	0	35	78	80	65	54	9	320
	Evaporation Depth (ft)	0	0	0	0	0.19	0.28	0.39	0.48	0.49	0.40	0.34	0.23	2.80
	Otonowanda Surface Area (ac)	26	48	51	53	54	55	55	54	52	50	50	49	
	Evaporative Loss	0	0	0	0	10	15	21	26	26	20	17	11	147
	End of Month Storage	189	216	239	254	258	263	256	232	207	205	200	234	
	Ridgway Ditch Surplus	0	0	0	0	0	0	0	0	0	0	0	0	0

60 % total diversions by Town from Ridgway Ditch during irrigation season (May - Sep); Happy Hollow rights not called out.

YEAR 5 - normal year		Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Total
Supply	Total Ridgway Ditch Average Diversion (1)	60	48	40	32	34	48	158	240	245	159	149	136	1349
	Town Ridgway Ditch Average Diversion	60	48	40	32	34	48	95	144	147	95	89	136	969
	Transit Loss	21	17	14	11	12	17	33	50	51	33	31	48	339
	Net Town Ridgway Ditch Diversion	39	31	26	21	22	31	62	94	96	62	58	88	630
	Happy Hollow Ditch Average Diversion	55	52	52	48	55	52	43	50	47	49	43	51	598
	Total Diversions	**94**	**84**	**78**	**69**	**77**	**83**	**105**	**143**	**143**	**111**	**101**	**140**	**1228**
Demands	Projected Treated Water Demands, 2030	21	20	19	18	20	20	34	47	48	39	32	25	342
	Projected Raw Water Demand, 2030							22	31	32	26	22	16	149
	Total Demand	**21**	**20**	**19**	**18**	**20**	**20**	**56**	**78**	**80**	**65**	**54**	**41**	**491**
	Demand Met by Happy Hollow	21	20	19	18	20	20	43	50	47	49	43	41	392
	Happy Hollow surplus	34	32	33	30	35	32						10	206
	Happy Hollow deficit	0	0	0	0	0	0	(13)	(28)	(33)	(16)	(11)	0	(100)
Lake O	Demands Met From Lake Otonowanda	0	0	0	0	0	0	13	28	33	16	11	0	100
	Evaporation Depth (ft)	0	0	0	0	0.19	0.28	0.39	0.48	0.49	0.40	0.34	0.23	2.80
	Otonowanda Surface Area (ac)	26	56	58	60	61	62	63	63	63	63	63	63	
	Evaporative Loss	0	0	0	0	12	17	25	30	31	25	21	14	176
	End of Month Storage	273	304	330	351	362	375	377	377	377	377	377	377	
	Ridgway Ditch Surplus	0	0	0	0	0	0	23	36	32	21	26	74	212

Notes

(1) November-April diversions based on average water availibility determined by synthetic hydrology for Beaver Creek @ R. Ditch headgate. May-Oct based on average R. Ditch diversion records (2000-2009)

(2) November-April diversions based on dry year availibility determined by synthetic hydrology for Beaver Creek @ R. Ditch headgate. May and Oct based on 2002 R. Ditch diversion records

(3) November-April diversions based on dry year availibility determined by synthetic hydrology for Beaver Creek @ R. Ditch headgate. May-Oct based on 2003 R. Ditch diversion records for senior 2 cfs right

Total Ridgway Ditch diversions (average & dry-year) includes additional 0.09 cfs for spring inflow not reflected in headgate diversions.

Evaporative loss based on 37 inches of evaporation per year; no evaporation in months where average temp less than 32 degrees.

Seepage and Rainfall are assumed to balance one another due to uncertainties regarding seepage rate

Happy Hollow Ditch diversions based on average of 2004 - 2009 records.

Year 2030 treated water demands based on growth rate of 3.6% per year.

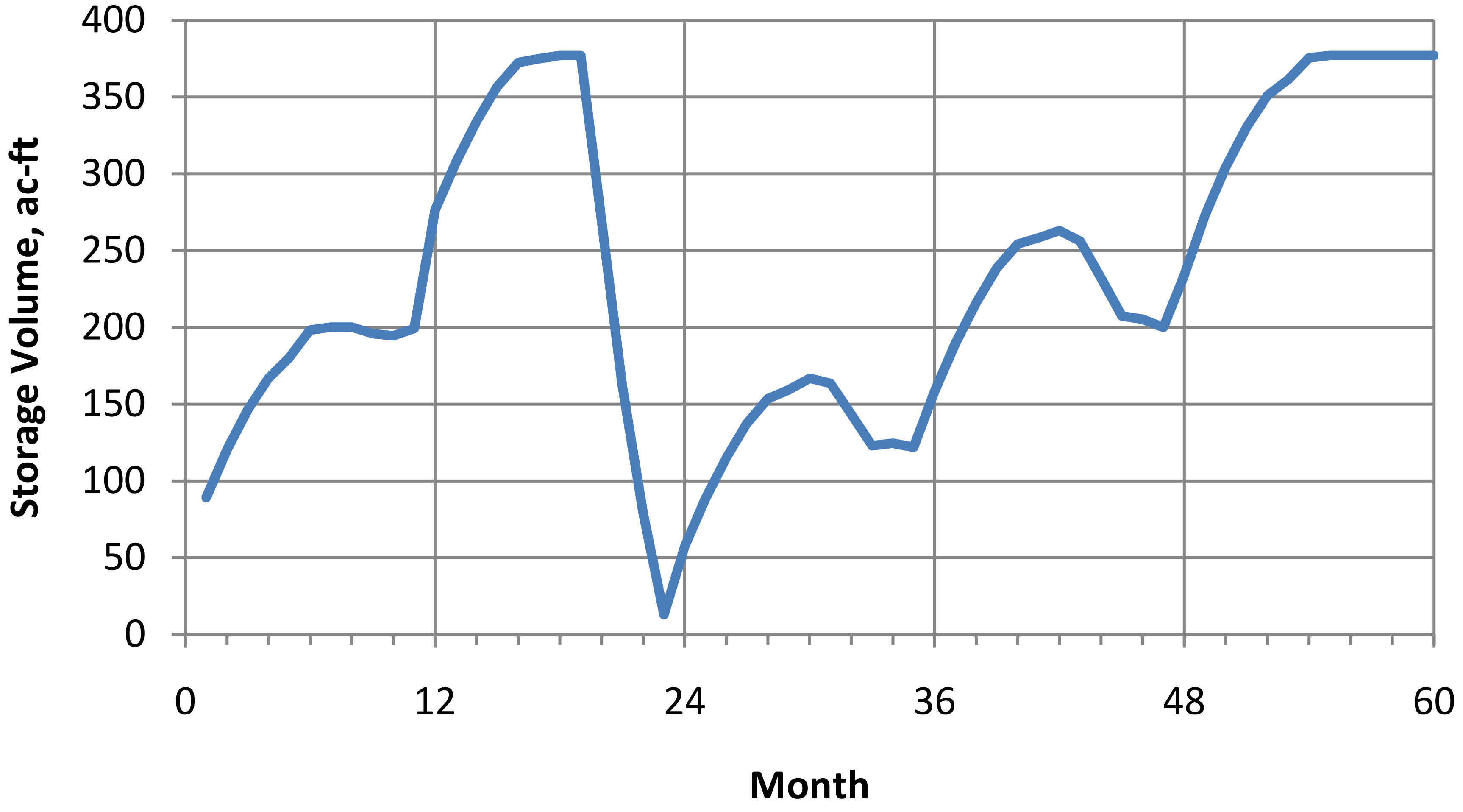
Year 2030 raw water demands based on growth rate of 35% of current existing irrigation demands.

Treated water demands first met by Happy Hollow supply; remaining demands met by releases from Otonowanda Reservoir.

The portion of the Ridgway Ditch diverted for the towns use is all placed into Otonowanda Reservoir except when it is full at which point is is shown as a surplus

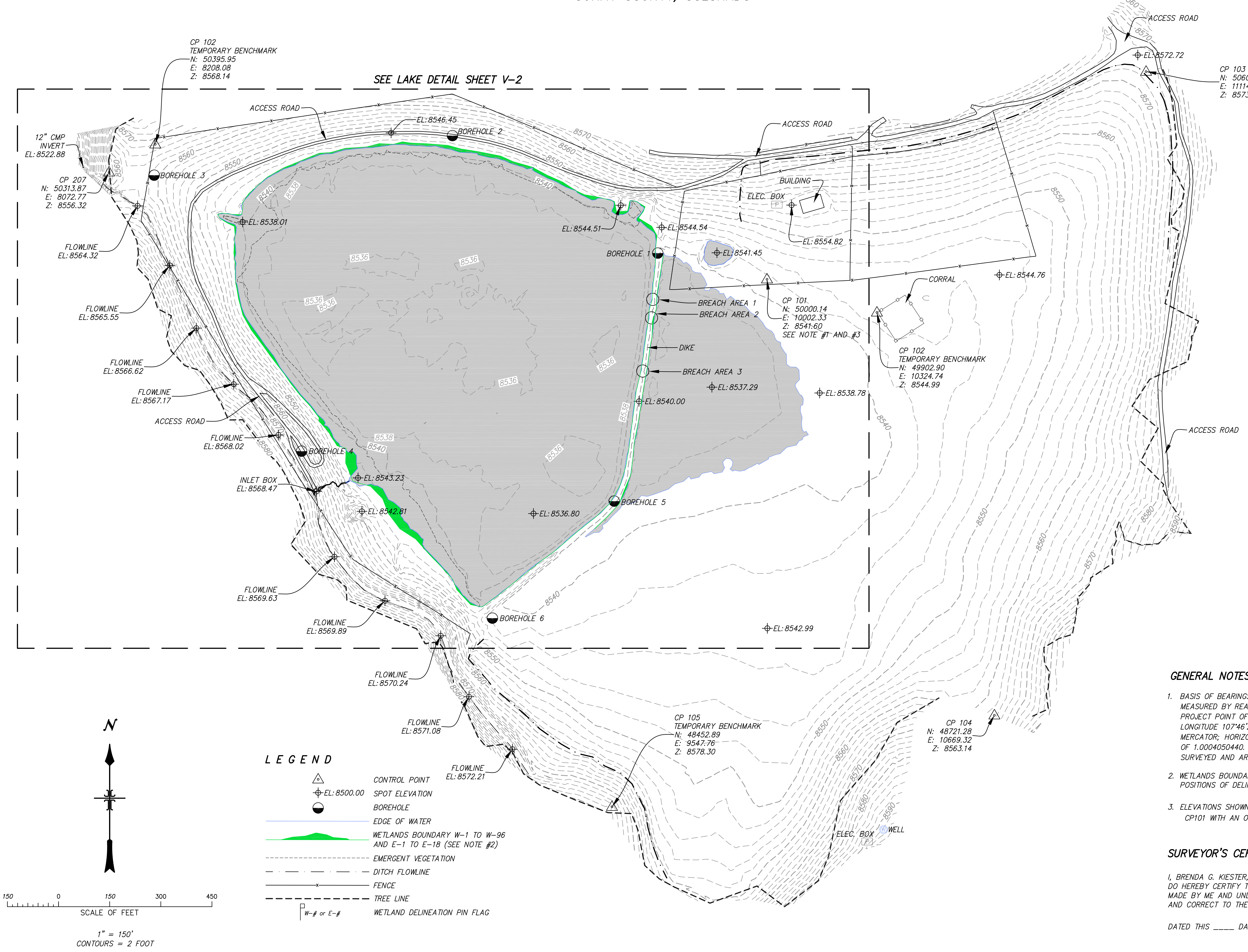
Adjusted storage limited to maximum projected capacity accounting for deliveries and evaporation loss.

Otonowanda Operations - Option 2



APPENDIX B

LAKE OTONAWANDA
TOPOGRAPHIC & BATHYMETRIC SURVEY
WITHIN
OURAY COUNTY, COLORADO



GENERAL NOTES

1. BASIS OF BEARINGS FOR FIELD MEASUREMENTS IS WGS84 GEODETIC NORTH AS MEASURED BY REAL TIME KINEMATIC GPS OR GPS L1/L2 OBSERVATIONS FROM A PROJECT POINT OF ORIGIN (CP101) OF LATITUDE 38°06'40.78591" NORTH AND LONGITUDE 107°46'23.77938" WEST (CP101). PROJECTION = TRANSVERSE MERCATOR; HORIZONTAL DATUM: NAD 83 (1992) WITH A GROUND SCALE FACTOR OF 1.0004050440. BOUNDARIES OF SUBJECT PROPERTY/IES HAVE NOT BEEN SURVEYED AND ARE NOT SHOWN HEREON AT CLIENT'S REQUEST.

2. WETLANDS BOUNDARY SHOWN HEREON WAS OBTAINED FROM FIELD MEASURED POSITIONS OF DELINEATION PERFORMED BY ENVIRONMENTAL SOLUTIONS, INC.

3. ELEVATIONS SHOWN HEREON ARE BASED ON A FIELD MEASURED POSITION FOR CP101 WITH AN OPUS DERIVED ELEVATION OF 8541.60 FEET, NGVD88.

SURVEYOR'S CERTIFICATE

I, BRENDA G. KIESTER, A REGISTERED LAND SURVEYOR IN THE STATE OF COLORADO, DO HEREBY CERTIFY THAT THIS PLAT AND THE SURVEY REFERRED TO HEREIN WERE MADE BY ME AND UNDER MY DIRECTION AND CONTROL AND THAT BOTH ARE TRUE AND CORRECT TO THE BEST OF MY KNOWLEDGE.

DATED THIS ____ DAY OF ___________, A.D. 20___.

BUCKHORN GEOTECH
Civil, Structural & Geotechnical Engineers
222 So. Park Ave. Montrose, Colorado 81401
970-249-6828 Fax. No. 970-249-0945
www.buckhorngeo.com

REVISIONS	DATE

APPLEGATE GROUP
LAKE OTONAWANDA
TOPOGRAPHIC AND BATHYMETRIC SURVEY

DESIGNED	N/A
DRAWN	EBP
DATE	11/04/09
PROJ. NO.	09-296
DRAWING NUMBER	V-1
OF <->	DWGS.

\\2003server\SURVEY\2009\09-296 Lake Otonawanda\dwg\Survey\09-296 Lake Otonawanda.dwg, 11/10/2009 10:43:20 AM, Adobe PDF 21X33.pc3

LAKE OTONAWANDA
TOPOGRAPHIC & BATHYMETRIC SURVEY
WITHIN
OURAY COUNTY, COLORADO

LAKE OTONAWANDA
SURFACE ELEV=8541.90
SURFACE AREA=26.20 ACRES
LAKE VOLUME=119.7 ACRE FEET
OPEN WATER
1047478.932 SQ. FT
24.04 ACRES
VEGETATION IN WATER
94058.594 SQ. FT
2.16 ACRES

BUCKHORN GEOTECH
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REVISIONS	DATE

APPLEGATE GROUP
LAKE OTONAWANDA
TOPOGRAPHIC AND BATHYMETRIC SURVEY – LAKE DETAIL

DESIGNED N/A
DRAWN EBP
DATE 11/04/09
PROJ. NO. 09-296
DRAWING NUMBER
V-2
OF <-> DWGS.

\\2003server\SURVEY\2009\09-296 Lake Otonawanda\dwg\Survey\09-296 Lake Otonawanda.dwg, 11/10/2009 10:42:29 AM, Adobe PDF 21X33.pc3

APPENDIX C

BUCKHORN GEOTECH
Civil, Structural & Geotechnical Engineers

222 South Park Ave. • Montrose, CO 81401
Ph.: (970) 249-6828 • FAX: (970) 249-0945

LAKE OTONAWANDA IMPROVEMENTS PROJECT
GEOTECHNICAL FEASIBILITY STUDY FOR RESERVOIR ENLARGEMENT

DAM I.D. #680112
WATER DIVISION 4
OURAY COUNTY, COLORADO

November 13, 2009

Prepared For:

Applegate Group, Inc.
118 West 6th Street, Ste. 100
Glenwood Springs, Colorado 81601

TABLE OF CONTENTS

APPENDICES

1.0 INTRODUCTION

The Town of Ridgway, Colorado is seeking to evaluate and implement improvements to its water collection and storage system, consisting of the Ridgway Ditch and Lake Otonawanda. The Ridgway Ditch is an open ditch that diverts water from the Beaver Creek drainage for the Town's municipal water supply. The raw water in the Ridgway Ditch may be either diverted into Lake Otonawanda for storage or routed into the Otonawanda Pipeline that conveys the water to the Town's water treatment plant.

The Town would like to augment its water supply to manage increasing water demands and accommodate senior calls on the river. Lake Otonawanda has a decreed storage of 746 acre-feet, but currently only has an estimated 75 to 150 acre-feet of storage capacity. The reservoir also has been reported to have significant seepage losses (Carter Burgess, 2005).

Buckhorn Geotech, Inc. (BGI) was retained by Applegate Group, Inc. (Applegate) to provide geotechnical engineering services related to assessing the feasibility of enlarging the storage capacity of Lake Otonawanda and providing improvements to reduce seepage losses. A geotechnical investigation was conducted to characterize the embankment fill and foundation conditions along the existing dam and around the reservoir rim. This report presents the findings of the geotechnical investigation and geotechnical recommendations for reservoir enlargement alternatives.

1.1 Project Description

Lake Otonawanda is a non-jurisdictional reservoir located in Ouray County, Colorado. The reservoir is located approximately 3 miles south of the Town of Ridgway on Miller Mesa, in the NW quarter of Section 32, T. 45 N., R. 8 W. The lake is in the western side of a broad, open topographic depression, with ridge-like hills to the north, east, and south. The Cottonwood Creek drainage is located immediately west of the lake. Lake Otonawanda is on Town property, however land surrounding the lake is privately owned. The project location is presented on Drawing 1 in Appendix A of this report.

The existing Otonawanda dam is an earthen embankment approximately 8 feet high at the maximum section and approximately 1,275 feet long. The dam crest is at an approximate elevation of 8,540 feet. The reservoir surface area was approximately 26 acres at the time of the geotech investigation and survey work. We have not been provided any documentation regarding the dam construction.

Land downstream of the dam has been used as irrigated pasture. At the time of the geotechnical investigation, land downstream of the northern leg of the dam was covered by ponded water. The ponded water is also present on aerial photos of the site.

The State Engineer's Office (SEO) records list the Otonawanda dam as non-jurisdictional, although significant improvements to the dam would likely result in a reclassification to a jurisdictional dam.

1.2 Scope of Geotechnical Services

The objective of the geotechnical investigation was to characterize foundation conditions along the existing dam alignment and subsurface conditions around the perimeter of the existing reservoir as a preliminary feasibility level study for enlargement of the reservoir capacity. Six boreholes were drilled at locations requested by Applegate. Due to private land surrounding the reservoir, no alternative dam locations were investigated.

Index laboratory tests were conducted to classify the soils encountered. Testing of engineering properties, such as in-situ permeability (i.e., packer testing), shear strength, or consolidation properties was outside our scope of services. Feasibility level evaluation of alternatives for reservoir enlargement is provided.

2.0 GEOLOGICAL SETTING

Lake Otonawanda is located on Miller Mesa, approximately 3 miles south of the Town of Ridgway. The lake is on the northern flanks of the Sneffel's Range of the San Juan Mountains. The San Juan Mountains of southwestern Colorado are an uplifted dome of Paleozoic and Mesozoic sedimentary formations that have been capped by volcanics. Uplift that accompanied the volcanic eruptions resulted in local warping and folding of the older sedimentary beds. A network of dikes, sills and stocks were emplaced during the late Cretaceous and Tertiary periods. The present landscape has been formed by glacial, alluvial, and mass wasting processes acting on the uplifted sedimentary and igneous formations.

Geology of the Lake Otonawanda area has been mapped by Steven and Hail (1989). According to this publication, Lake Otonawanda is sited on landslide debris overlying Cretaceous Mancos Shale bedrock. Mancos Shale is a gray to brown, variably silty, calcareous clay shale of marine origin and is locally fossiliferous. The Mancos is not resistant to erosion, moderately plastic when wet, and may contain swelling clays.

Subsurface conditions encountered during our geotechnical investigation conflict with the geology mapped by Steven and Hail (1989). Relatively shallow volcanic breccia was encountered in boreholes drilled on the north, west, and southwest sides of the reservoir. Additionally, outcrops of the volcanic rock were observed on the slope of the Cottonwood Creek drainage near the Otonawanda pipeline intake. Steven and Hail (1989) map intrusive sills, stocks, and dikes from the lower Tertiary and upper Cretaceous in the region surrounding Lake Otonawanda, as well as extensive volcanic rock further south towards the Sneffels range. It is likely that the volcanic breccias encountered during the geotechnical investigation overlie the Mancos Shale. However, no Mancos Shale was encountered during the drilling investigation. Further discussion of the site specific subsurface conditions are presented in Section 3.0.

2.1 Seismicity

Lake Otonawanda is located in the Colorado Plateau Seismotectonic Province in Colorado, where maximum credible earthquakes are estimated to be on the order of magnitude M5.5 to M6.5. The largest recorded earthquake in the region was the 1960 M5.5 (MM VI) Montrose/Cimarron event (Kirkham and Rogers, 2000).

The closest mapped potentially active fault to Lake Otonawanda is the Ridgway Fault, located approximately 4 miles to the north (Kirkham and Rogers, 1981) and (Widmann et al., 1998). The Ridgway Fault is an east-west trending normal fault (down to the south) that is well defined by the escarpment along the southern margin of Log Hill Mesa. A magnitude M6.5 to 6.75 maximum credible earthquake has been inferred for the Ridgway Fault.

According to the USGS Earthquake Mapping Hazard Project, an earthquake with a 5,000-year return frequency for the Otonawanda Dam site would have a peak ground acceleration (PGA) of 0.3739g (USGS, 2008).

3.0 GEOTECHNICAL INVESTIGATION

The geotechnical investigation for the feasibility study consisted of six boreholes drilled at locations adjacent to the existing dam and around the reservoir perimeter. The borehole locations were selected by Applegate prior to the investigation. The investigation was conducted on October 6 and 7, 2009. Investigation of other dam sites was outside our scope of services.

Three boreholes were drilled on or adjacent to the existing dam, including boreholes near each abutment and one borehole near the existing elbow in the dam. The narrow and irregular dam crest generally precluded access on the dam with the available drill rig. Therefore, only one borehole (BH#1) was drilled through the existing dam fill (at the left abutment). Borehole BH#5 was drilled just downstream of the dam elbow point and BH#6 was drilled just downstream of the dam at the right abutment.

The remaining three boreholes were drilled around the perimeter of the reservoir rim. One borehole was located on the north side of the reservoir (BH#2); one borehole was located in the saddle area northwest of the reservoir (BH#3); and one borehole was located on the southwest side of the reservoir (BH#4). All borehole locations were subsequently surveyed. These borehole locations are presented on Drawing 2 in Appendix A.

The boreholes were drilled with a Simco 2800 H.S. truck-mounted drill rig to depths ranging from 11 to 41.5 feet. The boreholes were drilled with a 4¼-inch solid stem continuous flight auger within soil. Soil samples were obtained from auger cuttings and with a modified California split spoon. Standard penetration tests (SPT) were conducted in general accordance with ASTM D1586. Bedrock, where encountered, was cored using an HQ wireline assembly. The recovered soil was logged and representative samples of subsurface materials encountered were brought back to our soils laboratory for detailed examination and testing. Upon completion of drilling, the boreholes were backfilled with native soil cuttings. Subsurface conditions encountered during drilling are discussed below and borehole logs are presented in Appendix B. A cross-section along the existing dam alignment is presented on Drawing 3 of Appendix A.

Existing Dam & Dam Foundation Boreholes

Boreholes BH#1, BH#5 and BH#6 were drilled on or immediately downstream of the existing dam. Due to the narrow and uneven dam crest, drill access was limited. Only BH#1, located at the left abutment, was drilled through the existing dam fill. The dam fill was characterized as damp to wet, soft, olive black clayey silt to silty clay. The phreatic surface was encountered at an approximate depth of 3 feet, and measured at 3.3 feet in the open borehole one day after drilling. One SPT yielded an N-value of 4 blows per foot (bpf) at a depth of 5 feet. Native foundation soils were encountered at approximately 7 feet below the dam crest.

Dam foundation soils encountered in the three boreholes along the dam alignment were somewhat variable, consisting mostly of lean clays at the left and right abutment boreholes (BH#1 and BH#6) and a sequence of clayey sands, silty sands, and clays at BH#5 near the maximum section of the dam. The surficial soils are interpreted to be landslide deposits derived from weathered Mancos Shale, upvalley glacial deposits and igneous bedrock.

At the left abutment, foundation soils were characterized as moist, very stiff, moderate brown lean and fat clay with some sand and trace gravel between depths of 7 feet (top of native soil) to approximately 25 feet. N-values in these clay soils ranged from 18 to 26 bpf. Below 25 feet, the soil type changed to moist, very stiff to hard, vari-colored (purple-green-brown) clayey sand and gravel. The gravel was of igneous lithology and crumbled easily when extruded from the split spoon liners. This soil is likely derived from intrusive and/or extrusive igneous rock in the area, and may have either weathered in-place or been transported to the area. Uncorrected N-values ranged from 41 to 52 bpf in this material, indicative of very stiff soil. Borehole BH#1 was terminated at a depth of 41.5 feet. No competent formational bedrock was encountered to this depth.

At the right abutment, foundation soils consisted of moist, firm to very stiff, moderate brown and moderate yellowish brown, lean clay with variable sand content. Uncorrected SPT N-values ranged from 8 to 24 bpf. These soils extended to a depth of 23.5 feet, at which point hard rock was encountered. The rock was augered for 2 feet to a depth of 25.5 feet, at which point auger refusal was virtually met. Based on subsurface conditions encountered in boreholes around the reservoir rim, the rock may be volcanic breccia. No groundwater was encountered in borehole BH#6 during or immediately following drilling.

Borehole BH#5 was drilled just downstream of the dam elbow, near the maximum section of the existing dam. Water from irrigation was ponded on the downstream side of the dam between the dam elbow and the left abutment, thereby precluding truck-mounted drill access to this area. Foundation soils at BH#5 were somewhat variable, and significantly sandier than soils encountered at the abutment boreholes. The foundation soils were generally characterized as moist to wet, firm to very stiff, moderate brown and moderate yellowish brown, clayey sand, clayey silt, and silty clay with trace gravel. The borehole was extended to a total depth of 41.5 feet without encountering bedrock. Uncorrected N-values ranged from 7 to 36 bpf, with an average of 20.6 bpf. Groundwater was measured in the open borehole at a depth of 8 feet after drilling.

Reservoir Rim Boreholes

Three boreholes were drilled around the reservoir rim. Borehole BH#2 was drilled on the north side of the reservoir; BH#3 was drilled in the saddle area northwest of the reservoir near the outlet pipe alignment; and BH#4 was drilled on the southwest side of the reservoir. Due to slopes and the necessity to set the drill rig up at relatively level locations, BH#2 and BH#4 were drilled on the existing access road. Borehole BH#3 was drilled on the gentle slopes above the access road.

Subsurface conditions at all three boreholes consisted of shallow igneous bedrock mantled by a thin soil cover of clayey sand and gravel to silty clay and gravel. Bedrock was encountered at approximate depths of 3 feet in BH#2 and BH#3 and at approximately 5 feet in BH#4, and cored approximately 10 feet in each borehole. The bedrock was characterized as fresh to moderately weathered, medium strong to very strong volcanic breccia of intermediate composition. Core recovery ranged from 40 to 100% with average recovery of 84% in the three boreholes. The rock quality designation (RQD) ranged from 0 to 71%, with an average RQD of 39%. RQD is defined as the sum of core pieces greater than 100 mm long divided by the total core run length. Typically, RQD less than 25% is an indication of “very poor” rock mass quality; RQD of 25 to 50% is indicative of “poor” rock mass quality; RQD of 50 to 75% is indicative of “fair” rock mass quality; RQD of 75 to 90% is indicative of “good” rock mass quality; and RQD greater than 90% is indicative of “excellent” rock mass quality. The recovered core and RQD values are indicative of highly fractured rock. The photograph below shows the nature of recovered core from BH#2.

Recovered core from BH#2 (3.3-14.5 feet). The gravel-sized pieces of rock are indicative of highly fractured, and possibly more weathered bedrock zones.

No groundwater was encountered in the boreholes during or immediately following drilling to depths of 11 to 15 feet.

4.0 LABORATORY TESTING

Index tests were conducted on soil samples collected during the geotechnical investigation. These included grain-size analysis (ASTM C117, C136, D422), Atterberg limits (ASTM D4318), and USCS classification (ASTM D2487). Testing of engineering properties, such as shear strength, hydraulic conductivity, and one-dimensional consolidation were outside our scope of services. Lab test results are summarized in Table 1 and presented in Appendix C.

One sample of the existing dam fill (DS1) was found to have a liquid limit (LL) of 38 and a plasticity index (PI) of 17. The sieve analysis indicated the sample to be composed of approximately 65% fines, 21% sand, and 14% gravel. Based on these test results, the dam fill sample classifies as sandy lean clay (CL) according the Unified Soil Classification System (USCS).

Index testing was also conducted on seven soil samples of native foundation soils obtained from the boreholes drilled along the existing dam alignment (samples DS2, DS3, DS8, DS9, DS10, DS15, and DS16). The samples were found to have liquid limits (LL) ranging from 33 to 54, with an average LL of 45.5 and plasticity indices (PI) ranging from 17 to 36, with an average PI of 28.5. Sieve analyses conducted on these samples indicated the soils to be composed of approximately 33 to 86% fines (silt and clay), 14 to 61% sand, and 0 to 6% gravel. Based on these test results, the samples classify as CL, CH, and SC according to the USCS. The similar composition of the dam fill and the foundation soils suggest that the embankment was constructed from local materials.

One sample of the soil interpreted to be highly weathered bedrock from BH#1 was also tested (sample DS5). This sample had a liquid limit of 46 and a plasticity index of 19. The sample was composed of approximately 17% fines, 33% sand, and 50% gravel. Sample DS5 classifies as clayey gravel with sand (GC).

Table 1. Index Test Results

Boring ID	Sample ID	Sample Depth (ft)	Natural Moisture (%)	Dry Density (pcf)	Atterberg Limits			Grain Size Distribution			USCS
					LL	PL	PI	% Gravel	% Sand	% Fines	
BH#01	DS1	5-6.5	29.5		38	21	17	13.8	20.9	65.3	CL
	DS2	10-11.5	25.0	99.5	48	17	31	2.2	29.2	68.6	CL
	DS3	15-16.5	26.6		54	18	36	0.3	14.2	85.5	CH
	DS5	25-26.5	24.9		46	27	19	49.6	33.2	17.2	GC
BH#05	DS8	5-6.5	15.2		33	16	17	5.8	61.2	33.0	SC
	DS9	10-11.5	33.1	89.3							
	DS10	15-16.5	25.8		50	17	33	0.0	14.1	85.9	CH
BH#06	DS15	5-6.5	22.7		45	18	27	0.4	30.0	69.6	CL
	DS16	10-11.5	28.5	94.3	43	16	27	0.0	14.5	85.5	CL

NOTES:
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
NP = Non-Plastic
USCS = Unified Soil Classification System

5.0 GEOTECHNICAL ASSESSMENT OF RESERVOIR ENLARGEMENT

This section provides a discussion of geotechnical issues relating to the dam foundation materials, reservoir rim, outlet works and potential borrow materials. Further discussion of reservoir enlargement alternatives is presented in Section 6.

5.1 Dam Foundation

As discussed in the *Geotechnical Investigation* (Section 3), the foundation along the existing dam alignment consists of variable soil types, generally consisting of lean and fat clays to clayey sands with minor beds and lenses of cleaner sands. Clayey soils were generally encountered at the existing dam abutments, with sandier soils towards at the middle section of the dam foundation. Competent bedrock was encountered at 23.5 feet at the right abutment, however no competent bedrock was encountered to depths of 41.5 feet at the left abutment or middle section of the dam. Clayey sand and gravel, interpreted to be highly weathered volcanic rock, was encountered at the left abutment at a depth of 25 feet. Consistency of the clayey soils was generally stiff to very stiff.

The surficial soils will be suitable to support an earthfill embankment of the anticipated size for this project (likely less than 10 to 15 feet high at the maximum section). Permeability of the foundation soils is estimated to range from low permeability clays to medium permeability clayey sands and gravels. Thin layers and lenses of higher permeability sands are likely present at various depths below the embankment. A cutoff trench may be adequate to address seepage through the foundation soils. Although not encountered in the geotechnical investigation, some low strength, compressible soils may be present near the surface. These soils should be removed and either reconditioned and compacted or replaced with suitable embankment fill.

Ponded water at the downstream toe of the dam will reduce the effective strengths of the foundation soils at the toe. We recommend either a change in the irrigation practices or grading the ground surface at the downstream toe of the dam to promote drainage away from the dam and prevent ponded water at the toe of the dam.

Shallow volcanic bedrock may be present further up the abutment slopes and may affect seepage for an enlarged embankment that extends further up the slopes beyond the existing abutments. The volcanic rock encountered around the reservoir rim was highly fractured and may be highly permeable relative to the dam foundation soils. Treatment of bedrock at the abutments, if present, may include dental concrete, a cutoff trench, and/or grouting. Packer testing is recommended to quantify permeability of the bedrock.

5.2 Reservoir Footprint and Rim

Shallow volcanic bedrock was encountered in three boreholes around the reservoir perimeter. The rock was generally highly fractured. Although packer testing was outside our scope of services, the fractured rock is likely highly permeable relative to the clayey foundation soils. Seepage through the fractured rock may corroborate the Town’s public works beliefs that springs in Cottonwood Creek drainage are from Lake Otonawanda (Carter Burgess, 2005). Packer testing is suggested to characterize permeability of the bedrock to better evaluate seepage losses through the rock formation.

Obviously the subsurface conditions within the reservoir footprint were not characterized as part of this study. The thickness and composition of sediment in the reservoir bottom is uncertain. However, it should be expected that fractured bedrock is present at shallow depths throughout a significant portion of the reservoir footprint, particularly around the margins of the reservoir. Based on subsurface conditions at borehole BH#5, thicker soil cover over bedrock may be present in the central and eastern portions of the reservoir bottom.

Slopes around the reservoir rim are mostly gently sloping. Potential for slope failures around the reservoir rim during fluctuations of the reservoir level is considered to be relatively low, considering the gentle slopes and shallow bedrock. Slope failures that may occur should be shallow and small and are not anticipated to affect the safe operation of the reservoir.

5.3 Outlet Works

It is our understanding that the existing outlet pipe/tunnel at the northwestern side of the reservoir has collapsed. The extent and nature of the collapse, as well as the original construction details, are somewhat uncertain.

Bedrock was encountered at a depth of 3.3 feet in the vicinity of the outlet works (BH#3). The near surface bedrock may be fractured enough to allow some excavation if needed for a siphon outlet. A new bored outlet pipe with a grouted annulus is an alternative to discharge into the Otonawanda pipeline.

5.4 On-Site Borrow Sources

On site borrow areas were not specifically investigated as part of this work. Depending on permission from private landowners and the plans for the reservoir enlargement, there is a local source for quality clay for use as low permeability embankment fill and/or low permeability soil liner for the reservoir bottom. Approximately 18 feet of lean and fat clay was encountered in BH#1 and approximately 23 feet of clay was encountered at BH#6. Soils at BH#5 tended to be more variable and sandier than soils at BH#1 and BH#6.

Strong, durable igneous rock is available on-site at relatively shallow depths. The rock may be suitable for rip rap or concrete aggregate if needed. Crushing and screening operations may be necessary.

It is possible that some local soils may be processed for use as filters, if needed for embankment construction. Otherwise, filter materials may need to be imported.

Further investigation is needed to better define extents and characteristics of local materials and to estimate quantities.

6.0 LAKE OTONAWANDA IMPROVEMENT ALTERNATIVES

Potential alternatives to increase the storage capacity and reduce seepage losses from Lake Otonawanda are discussed below. Alternatives considered for increasing the storage capacity include dredging the existing reservoir, enlargement of the existing dam, relocation of the dam to a downstream location, or complete removal of the dam (thus expanding the reservoir basin). Geotechnical and geological issues relating to these alternatives are presented.

6.1 Dredging of Reservoir

Reservoir dredging is one alternative presented in the Carter Burgess feasibility study (2005) to increase the storage capacity of Lake Otonawanda. Based on the uncertainty of the subsurface conditions within the reservoir footprint, including the depth of sediment, and thickness and composition of the underlying soils, the viability of reservoir dredging is likewise uncertain. Dredging will result in thinner soil cover over bedrock, and may even expose shallow bedrock in places. This could lead to increased seepage losses through the reservoir footprint, unless the reservoir bottom is treated or lined. Discussion of potential treatment of the reservoir area is presented in Section 6.4.

6.2 Enlargement of Existing Dam

Little information is available on the composition or density of the existing embankment fill. The one borehole in the existing dam at the left abutment indicated the fill to be soft (N-value of 4). Unless further characterization of the existing fill can demonstrate the in-place material is compatible with the design of a larger embankment, the design team should consider removal and reconstruction of the existing dam.

An increased reservoir level resulting from an enlargement of the dam may result in increased seepage losses through shallow fractured bedrock unless measures are taken to reduce seepage losses through the reservoir footprint.

6.3 Relocate or Remove Dam

Another alternative to increase the storage capacity of the reservoir would be to relocate the dam downstream of its present location, or remove the dam altogether to expand the reservoir to the natural basin. Either alternative would result in a extending the dam and/or reservoir on to the adjacent private landowner’s property. It is our understanding that this possibility is being discussed with the landowner.

This option would increase the surface area of the reservoir thereby increasing evaporation losses. Additional geotechnical investigation is advised to characterize subsurface conditions as appropriate for the reservoir enlargement.

6.4 Reduction of Seepage Losses

As referenced previously, the Town’s public works staff has been quoted as strongly believing springs and seeps in the Cottonwood Creek drainage (“Happy Hollow”) originates from seepage out of Lake Otonawanda. Although we are not aware of any data that confirm this, the highly fractured shallow bedrock encountered in three of the boreholes drilled for this study support the hypothesis. To confirm this hypothesis, packer testing of the fractured bedrock is recommended.

Installation of a low permeability liner throughout all or portions of the reservoir footprint would help to reduce seepage losses. Several liner alternatives are available including:

- Amendment of the native soils with bentonite;
- Placement of a compacted clay liner;
- Installation of a geomembrane liner; or
- Installation of a geosynthetic clay liner (GCL);

Bentonite amendment of the native soils or installation of a compacted clay liner are likely more economical alternatives than geosynthetic liners, especially if quality clay is locally available.

7.0 CLOSING CONSIDERATIONS

This report has been prepared in a manner consistent with local standards of professional geotechnical engineering practice. The interpretation of subsurface conditions is based on our training and years of experience, but is necessarily based on limited subsurface investigation and testing. As such, inferred ground conditions cannot be guaranteed to be exact. No other warranty, express or implied, is made. Additional geotechnical investigation, characterization, and testing will be needed to address the uncertainties discussed in this report and to develop design criteria for the chosen reservoir improvements. This may include additional drilling at proposed dam locations, reservoir rim or borrow areas, as well as in-situ permeability testing.

8.0 REFERENCES

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Widmann, B.L., Kirkham, R.M., and Rogers, W.P., 1998. “Preliminary Quaternary Fault and Fold Map and Database of Colorado”, Colorado Geological Survey Open File Report 98-8.

Respectfully Submitted, **BUCKHORN GEOTECH, INC.**	Reviewed by:
ELECTRONICALLY TRANSMITTED	
Brett R. Byler, P.E. Geotechnical Engineer	Thomas E. Griepentrog, P.E., P.G. Principal

APPENIDIX A

DRAWINGS

VICINITY MAP

MAP OBTAINED FROM TOPO!®VERSION 2.7.6©NATIONAL GEOGRPHIC HOLDINGS

MAP NUMBER

1

OF 3

INVESTIGATION BRB

DRAFTING CC

FIELD DATE 10/6–7/09

JOB NO. 09–296–GRP

APPLEGATE GROUP, INC.

LAKE OTONAWANDA IMPROVEMENTS

OURAY COUNTY, COLORADO

BUCKHORN GEOTECH

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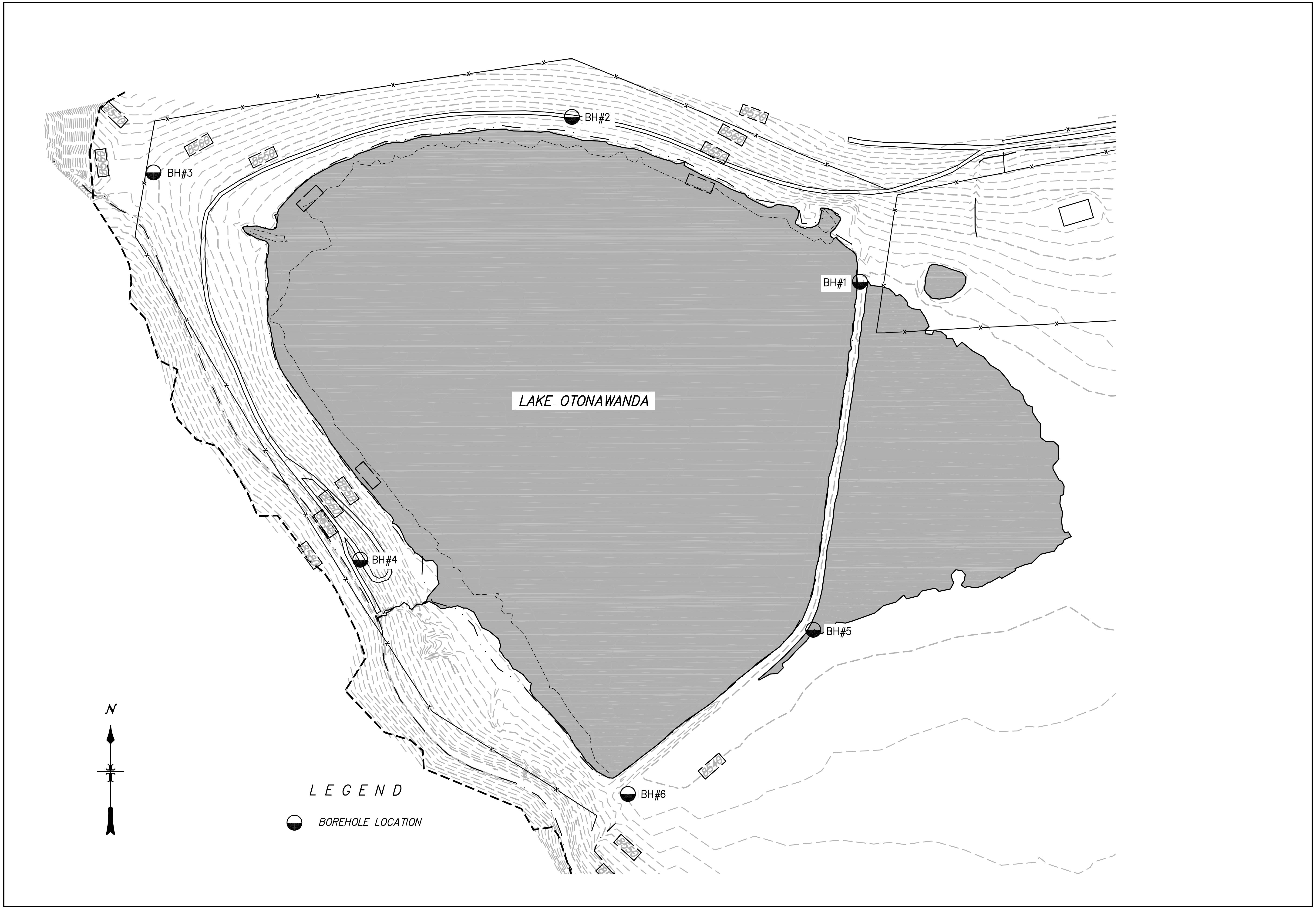

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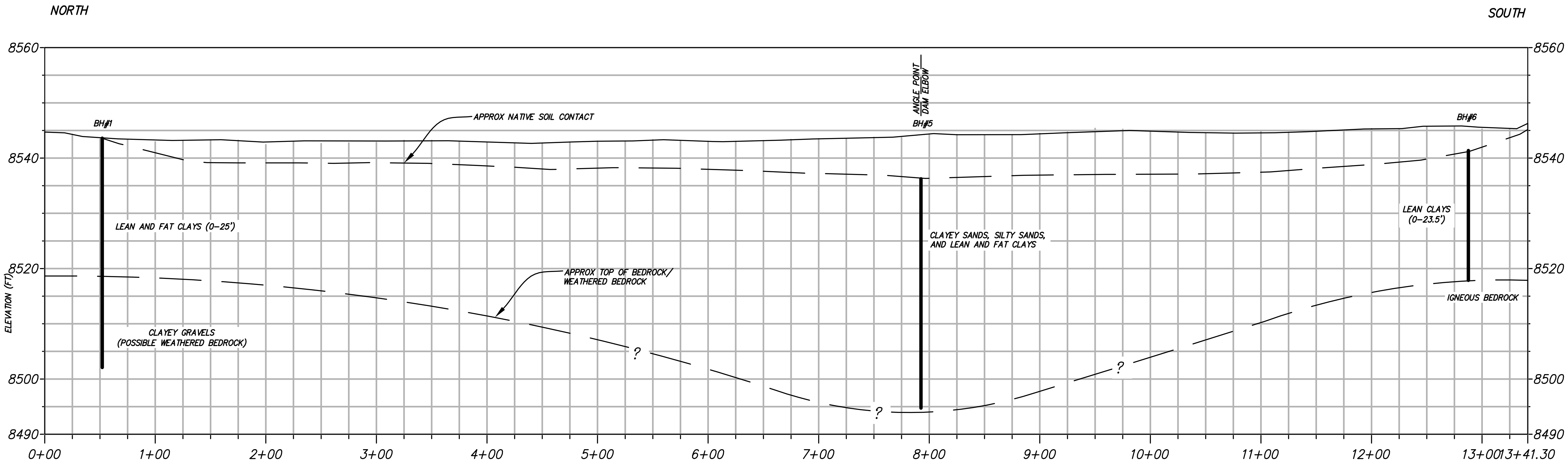
REVISIONS	DATE

APPLEGATE GROUP, INC.
LAKE OTONAWANDA IMPROVEMENTS
SITE PLAN

DESIGNED N/A
DRAWN
DATE 11/04/09
PROJ. NO. 09-296-GRP
DRAWING NUMBER
2
OF 3 DWGS.



GEOLOGIC PROFILE ALONG EXISTING DAM ALIGNMENT

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REVISIONS	DATE

APPLEGATE GROUP, INC.
LAKE OTONAWANDA
GEOLOGIC PROFILE

DESIGNED	BRB
DRAWN	BMB
DATE	11/11/09
PROJ. NO.	09-296

DRAWING NUMBER
3
OF 3 DWGS.

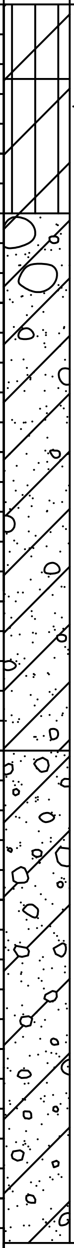
APPENDIX B

BOREHOLE LOGS

BOREHOLE LOG – BOREHOLE BH#1

BOREHOLE LOCATION: Left abutment existing dam
GPS Lat/Long: Approx. 38.11155N 107.77440W
NOTES:

DRILLER: Scott Drilling
DRILL RIG: Simco 2800 HS
DRILL STEM: 4.25" solid flight auger
SAMPLER: Modified California split spoon



DEPTH (ft.)	WATER LEVEL	SAMPLE TYPE	SAMPLE NUMBER	SPT BLOW COUNTS	*SPT 'N' VALUE (bpf)	RECOVERY (in.)	SUBSURFACE DESCRIPTION	FIELD & LABORATORY TEST RESULTS
0							olive black, damp, clayey SILT [FILL] (0–2.5')	
	▽	☒	GS1					
5		CA	DS1	2 2 2	4	8/18	olive black, moist to wet, soft, clayey SILT [FILL] (2.5–7') few cobbles below 7', becoming stiffer [NATIVE]	DS1 (CL) LL=38 PL=21 PI=17 GF=13.8% SF=20.9% F200=65.3% MC=29.5%
10			DS2	5 11 15	26	14/18	moderate brown, moist, very stiff, lean CLAY, with some sand and trace gravel, moderate plasticity	DS2 (CL) LL=48 PL=17 PI=31 GF=2.2% SF=29.2% F200=68.6% DD=99.5 pcf MC=25.0%
15			DS3	6 10 11	21	16/18	moderate brown, moist, very stiff, fat CLAY, with some sand, moderate plasticity becoming stiffer at 18'	DS3 (CH) LL=54 PL=18 PI=36 GF=0.3% SF=14.2% F200=85.5% MC=26.6%
20			DS4	6 9 9	18	14/18	moderate brown, moist, very stiff, lean CLAY, some sand and trace to little gravel	
25			DS5	14 22 22	44	8/18	vari-colored, purple-green-brown, moist, very stiff, clayey SAND and GRAVEL, appears to be highly weathered volcanic rock, crumbles easily; drills like stiff clay [POSSIBLE WEATHERED VOLCANIC BRECCIA]	DS5 (GC) LL=46 PL=27 PI=19 GF=49.6% SF=33.2% F200=17.2% MC=24.9%
30			DS6	17 21 31	52	10/18	vari-colored, purple-green-brown, moist, hard, clayey SAND and GRAVEL, appears to be highly weathered igneous rock, crumbles easily [POSSIBLE WEATHERED VOLCANIC BRECCIA]	
35								
40			DS7	10 17 24	41	12/18	vari-colored, moist, very stiff, clayey SAND and GRAVEL [POSSIBLE WEATHERED VOLCANIC BRECCIA] end of borehole @41.5' groundwater at 3.3' on 10/7/09	
45								

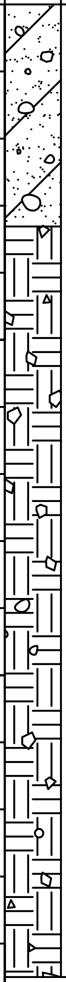
* SPT N-values not corrected for energy or depth; stratigraphic transitions are approximate and are inferred from cuttings & drillers comments

BOREHOLE LOG 1 OF 6	INVESTIGATION BRB DRAFTING SJ FIELD DATE 10/06/09 JOB NO. 09-296-GRP	APPLEGATE GROUP, INC. LAKE OTONAWANDA IMPROVEMENTS OURAY COUNTY, COLORADO	BUCKHORN GEOTECH Civil, Structural, and Geotechnical Engineers, Inc. 222 South Park Avenue Montrose, Colorado 81401 Phone (970) 249-6828 Fax (970) 249-0945

BOREHOLE LOG – BOREHOLE BH#2

BOREHOLE LOCATION: North side of reservoir on existing road
GPS Lat/Long: Approx. 38.11155N 107.77440W
NOTES: Hit rock at 3', moved to 10' and hit rock again at 3'

DRILLER: Scott Drilling
DRILL RIG: Simco 2800 HS
DRILL STEM: 4.25" solid flight auger (0-3.3'); HQ core (3.3-14.5')
SAMPLER:



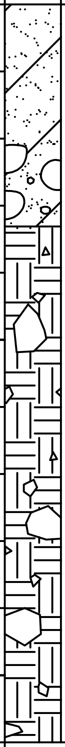
DEPTH (ft.)	GRAPHIC	WATER LEVEL	SAMPLE TYPE	SAMPLE NUMBER	*SPT 'N' VALUE (bpf)	RECOVERY (ft.)	R.Q.D. (ft./ft.)	STRENGTH INDEX	WEATHERING INDEX	SUBSURFACE DESCRIPTION	FIELD & LABORATORY TEST RESULTS
0										dark yellowish brown, dry, clayey SAND and GRAVEL (0–3.3')	
										coring began @3.3' with HQ wireline	
5			RUN 1			1.4/1.7	1.1/1.7			dusky blue green, strong to very strong, fresh to slightly weathered, VOLCANIC BRECCIA, intermediate composition	
			RUN 2			4.7/5.0	2.9/5.0				
10			RUN 3			1.8/4.5	0/4.5				
15										end of borehole @14.5' no groundwater encountered	
20											

BOREHOLE LOG 2 OF 6			
INVESTIGATION	BRB	APPLEGATE GROUP, INC.	**BUCKHORN GEOTECH**
DRAFTING	SJ	LAKE OTONAWANDA IMPROVEMENTS	Civil, Structural, and Geotechnical Engineers, Inc. 222 South Park Avenue Montrose, Colorado 81401
FIELD DATE	10/06/09	OURAY COUNTY, COLORADO	Phone (970) 249-6828 Fax (970) 249-0945
JOB NO.	09-296-GRP		

BOREHOLE LOG – BOREHOLE BH#3

BOREHOLE LOCATION: Saddle area west of reservoir
GPS Lat/Long: Approx. 38.11217N 107.77952W
NOTES:

DRILLER: Scott Drilling
DRILL RIG: Simco 2800 HS
DRILL STEM: 4.25" solid flight auger (0-3'); HQ core (3-11')
SAMPLER:

DEPTH (ft.)	GRAPHIC	WATER LEVEL	SAMPLE TYPE	SAMPLE NUMBER	*SPT 'N' VALUE (bpf)	RECOVERY (ft.)	R.Q.D. (ft./ft.)	STRENGTH INDEX	WEATHERING INDEX	SUBSURFACE DESCRIPTION	FIELD & LABORATORY TEST RESULTS
0										dark yellowish brown, dry, clayey SAND, some gravel; rock content increases @2–3'	
										coring began @3.3' with HQ wireline	
5			RUN 1			$\frac{1.7}{1.7}$	$\frac{0}{1.7}$			greenish gray, strong, slightly weathered, highly fractured VOLCANIC BRECCIA, intermediate composition	
			RUN 2			$\frac{4.0}{4.0}$	$\frac{2.1}{4.0}$				
10			RUN 3			$\frac{1.7}{1.7}$	$\frac{1.2}{1.7}$				
										end of borehole @11', driller loses bit and reamer in hole no groundwater encountered during drilling or on 10/7/09	
15											
20											

BOREHOLE LOG
3
OF 6

INVESTIGATION BRB
DRAFTING SJ
FIELD DATE 10/06/09
JOB NO. 09-296-GRP

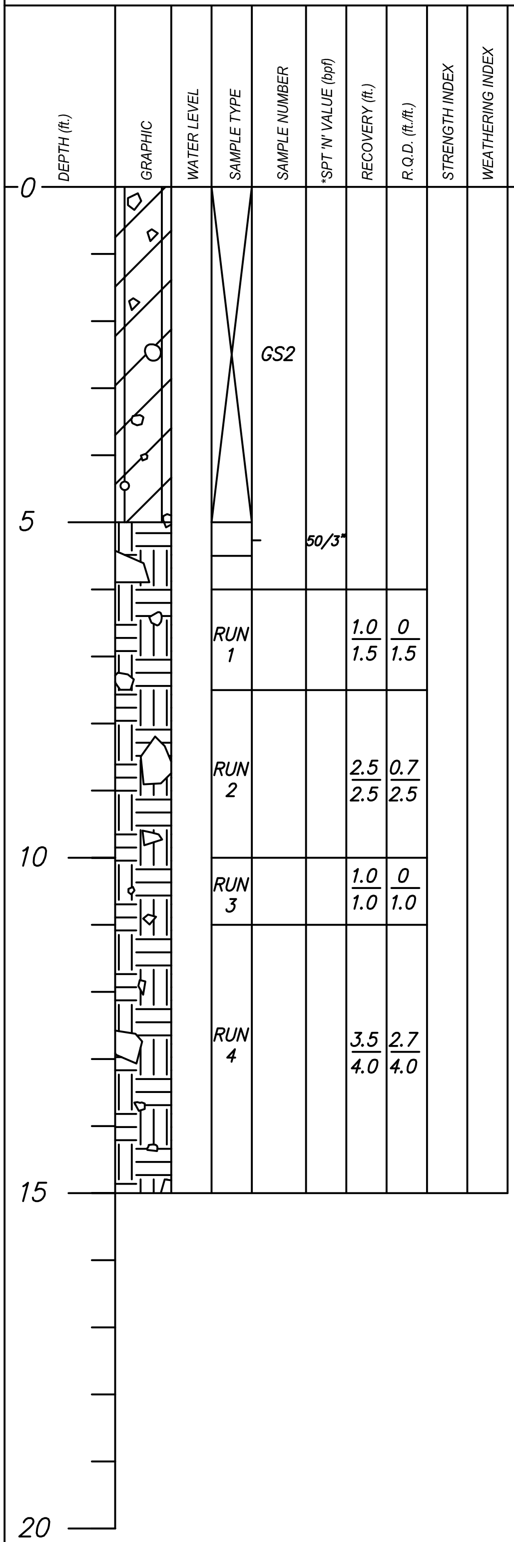
APPLEGATE GROUP, INC.
LAKE OTONAWANDA IMPROVEMENTS
OURAY COUNTY, COLORADO

BUCKHORN GEOTECH
Civil, Structural, and Geotechnical Engineers, Inc.
222 South Park Avenue
Montrose, Colorado 81401
Phone (970) 249-6828 Fax (970) 249-0945

BOREHOLE LOG – BOREHOLE BH#4

BOREHOLE LOCATION: Southwest side of reservoir
GPS Lat/Long: Approx. 38.10994N 107.77811W
NOTES:

DRILLER: Scott Drilling
DRILL RIG: Simco 2800 HS
DRILL STEM: 4.25" solid flight auger (0-5'); HQ core (5-15')
SAMPLER:



SUBSURFACE DESCRIPTION

dark yellowish brown, damp, silty CLAY with some gravel (0–5')

bedrock encountered @ 5'

began coring @6' with HQ wireline

dusky blue green, medium strong to very strong, slightly to moderately weathered, highly fractured, VOLCANIC BRECCIA; intermediate composition, fracture surfaces tend to be irregular and rough to very rough, some very thin clay infill, some calcite on fractures

end of borehole @15'
no groundwater encountered

FIELD & LABORATORY TEST RESULTS

BOREHOLE LOG
4
OF 6

INVESTIGATION	BRB
DRAFTING	SJ
FIELD DATE	10/07/09
JOB NO.	09–296–GRP

APPLEGATE GROUP, INC.
LAKE OTONAWANDA IMPROVEMENTS
OURAY COUNTY, COLORADO

BUCKHORN GEOTECH
Civil, Structural, and Geotechnical Engineers, Inc.
222 South Park Avenue
Montrose, Colorado 81401
Phone (970) 249-6828 Fax (970) 249-0945

BOREHOLE LOG – BOREHOLE BH#5

BOREHOLE LOCATION: Downstream of dam elbow

GPS Lat/Long: Approx. 38.10945N 107.77474W

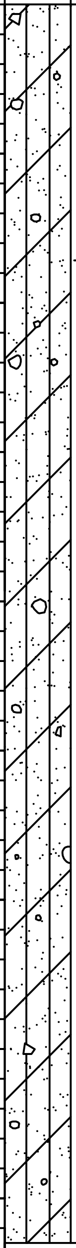
NOTES:

DRILLER: Scott Drilling

DRILL RIG: Simco 2800 HS

DRILL STEM: 4.25" solid flight auger

SAMPLER: Modified California split spoon



DEPTH (ft.)	WATER LEVEL	SAMPLE NUMBER	SPT BLOW COUNTS	*SPT 'N' VALUE (bpf)	RECOVERY (in.)	SUBSURFACE DESCRIPTION	FIELD & LABORATORY TEST RESULTS
0							
5		DS8	10 8 10	18	18/18	moderate brown, moist, very stiff, clayey SAND with trace gravel	DS8 (SC) LL=33 PL=16 PI=17 GF=5.8% SF=61.2% F200=33.0% MC=15.2%
	▽					split spoon wet	
10		DS9	2 4 3	7	18/18	moderate brown, moist to wet, firm, clayey SILT and clayey SAND with trace gravel	DS9 DD=89.3 pcf MC=33.1%
15		DS10	4 8 12	20	13/18	moderate brown, moist to wet, very stiff, silty CLAY with some sand	DS10 (CH) LL=50 PL=17 PI=33 GF=0.0% SF=14.1% F200=85.9% MC=25.8%
20		DS11	6 9 6	15	14/18	moderate brown, moist to wet, moderately dense, silty SAND with little clay	
25		DS12	9 13 17	30	8/18	moderate yellowish brown, wet, very stiff, clayey SAND with little to trace gravel; split spoon mostly full of slough	
30		DS13	10 9 9	18	4/18	moderate yellowish brown, wet, clayey SAND to sandy CLAY; split spoon mostly full of slough	
35							
40		DS14	16 16 24	36	18/18	dark yellowish brown, moist, hard, CLAY with little to some sand, little silt end of borehole @41.5' groundwater measured @8.0' immediately after drilling	
45							

* SPT N–values not corrected for energy or depth; stratigraphic transitions are approximate and are inferred from cuttings & drillers comments

BOREHOLE LOG
5
OF 6

INVESTIGATION	BRB
DRAFTING	SJ
FIELD DATE	10/07/09
JOB NO.	09–296–GRP

APPLEGATE GROUP, INC.

LAKE OTONAWANDA IMPROVEMENTS

OURAY COUNTY, COLORADO

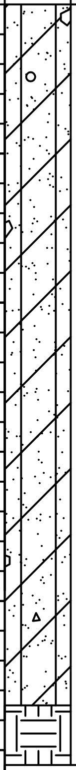
BUCKHORN GEOTECH

Civil, Structural, and Geotechnical Engineers, Inc.
222 South Park Avenue
Montrose, Colorado 81401
Phone (970) 249-6828 Fax (970) 249-0945

BOREHOLE LOG – BOREHOLE BH#6

BOREHOLE LOCATION: Downstream of right abutment
GPS Lat/Long: Approx. 38.10854N 107.77613W
NOTES:

DRILLER: Scott Drilling
DRILL RIG: Simco 2800 HS
DRILL STEM: 4.25" solid flight auger
SAMPLER: Modified California split spoon



DEPTH (ft.)	GRAPHIC	WATER LEVEL	SAMPLE TYPE	SAMPLE NUMBER	SPT BLOW COUNTS	*SPT 'N' VALUE (bpf)	RECOVERY (in.)	SUBSURFACE DESCRIPTION	FIELD & LABORATORY TEST RESULTS
0									
5				DS15	7 11 13	24	8/18	moderate yellowish brown, moist, very stiff, silty lean CLAY, with some sand and trace gravel	DS15 (CL) LL=45 PL=18 PI=27 GF=0.4% SF=30.0% F200=69.6% MC=22.7%
10				DS16	3 5 5	10	18/18	moderate yellowish brown, moist, stiff, silty lean CLAY, with little sand	DS16 (CL) LL=43 PL=16 PI=27 GF=0.0% SF=14.5% F200=85.5% DD=94.3 pcf MC=28.5%
15				DS17	2 3 5	8	18/18	moderate yellowish brown, moist, firm, sandy lean CLAY, with little gravel	
20				DS18	3 7 8	15	16/18	moderate brown, moist, stiff, silty lean CLAY, with little sand and trace gravel	
								bedrock encountered @23.5' [POSSIBLE VOLCANIC BRECCIA]	
25								end of borehole @25.5', no groundwater encountered to TD immediately after drilling	
30									
35									
40									
45									

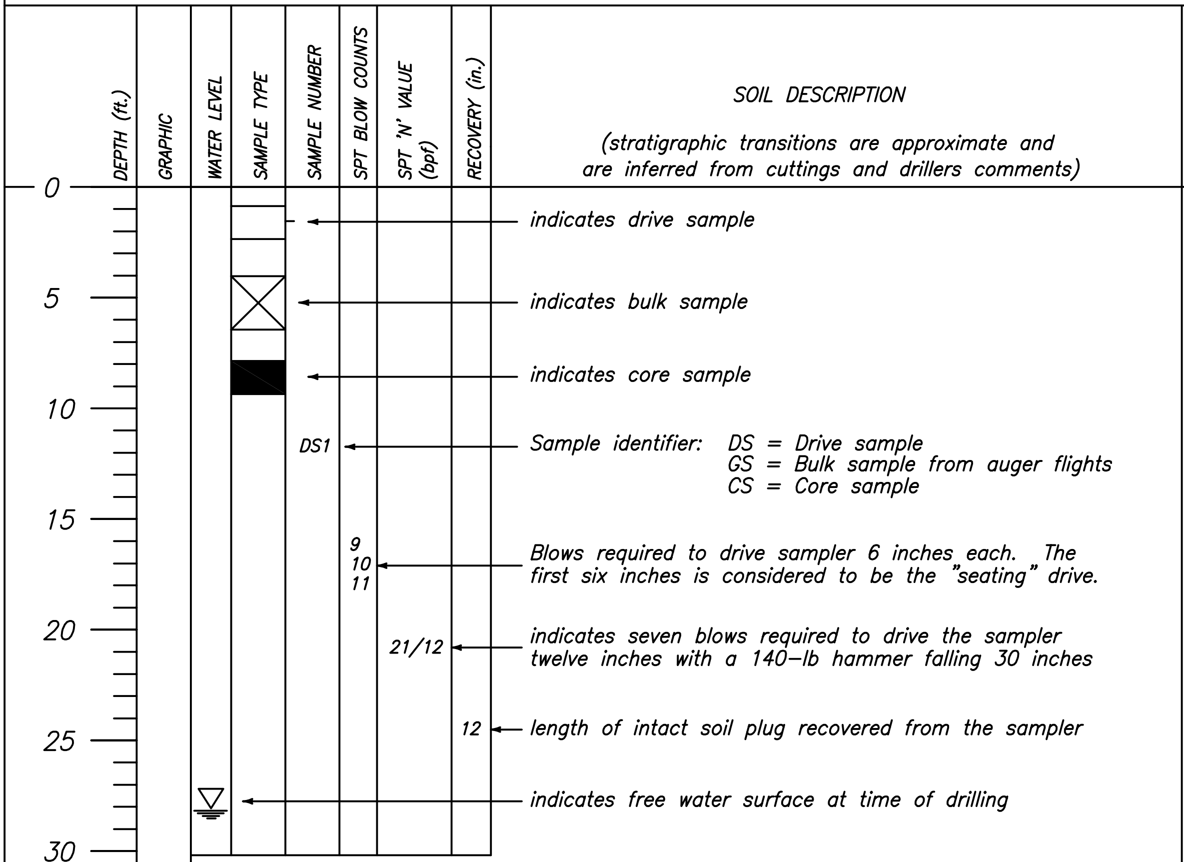
* SPT N–values not corrected for energy or depth; stratigraphic transitions are approximate and are inferred from cuttings & drillers comments

BOREHOLE LOG 6 OF 6	INVESTIGATION	BRB	APPLEGATE GROUP, INC. LAKE OTONAWANDA IMPROVEMENTS OURAY COUNTY, COLORADO	BUCKHORN GEOTECH Civil, Structural, and Geotechnical Engineers, Inc. 222 South Park Avenue Montrose, Colorado 81401 Phone (970) 249-6828 Fax (970) 249-0945
	DRAFTING	SJ		
	FIELD DATE	10/07/09		
	JOB NO.	09–296–GRP		

BOREHOLE LOG KEY

BOREHOLE LOCATION:
SURFACE ELEVATION:
NOTES:

DRILLER:
DRILL RIG:
DRILL STEM:
SAMPLER:



indicates drive sample

indicates bulk sample

indicates core sample

Sample identifier: DS = Drive sample
GS = Bulk sample from auger flights
CS = Core sample

Blows required to drive sampler 6 inches each. The first six inches is considered to be the "seating" drive.

indicates seven blows required to drive the sampler twelve inches with a 140–lb hammer falling 30 inches

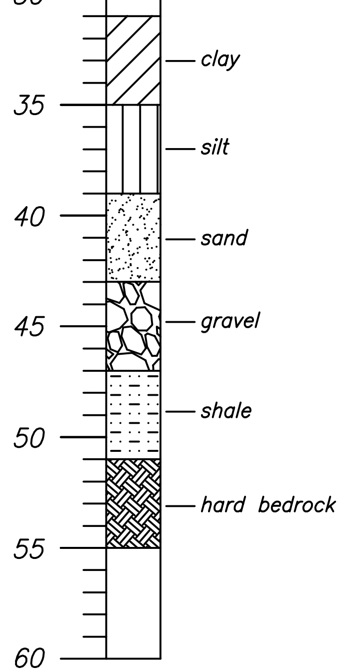
length of intact soil plug recovered from the sampler

indicates free water surface at time of drilling

FIELD & LABORATORY TEST RESULTS

Notes in this column indicate tests performed and test results

DD: dry density in cubic pounds per foot (pcf)
MC: % moisture content
LL: Liquid Limit
PL: Plastic Limit
PI: Plasticity Index
GF: Gravel fraction (%)
SF: Sand fraction (%)
F200: Silt/Clay (%)
Sh: Shear resistance
P: Penetration resistance
CBR: California Bearing Ratio
SP: swelling pressure
TM: total movement under consolidation pressure
psf: pounds per sq. ft.
pcf: pounds per cu. ft.
UCS: Unconfined Compressive Strength



Unified Soil Classification System (ASTM D–2487)

CL = lean clay to sandy/gravelly lean clay
ML = silt to sandy/gravelly silt
CH = high plasticity clay to sandy/gravelly high plasticity clay
MH = high elasticity silt to sandy/gravelly high elasticity silt
SW = well–graded sand or well–graded sand with gravel
SP = poorly graded sand or poorly graded sand with gravel
SM = silty sand to silty sand with gravel
SC = clayey sand to clayey sand with gravel
GW = well–graded gravel or well–graded gravel with sand
GP = poorly graded gravel or poorly graded gravel with sand
GM = silty gravel or silty gravel with sand
GC = clayey gravel or clayey gravel with sand

Rock Weathering Classification

W1 = Fresh
W2 = Slightly weathered
W3 = Moderately weathered
W4 = Highly weathered
W5 = Completely weathered
W6 = Residual soil, no structure

Intact Rock Strength Classification

R0 = Extremely weak rock, 35 – 150 psi
R1 = Very weak rock, 150 – 725 psi
R2 = Weak rock, 725 – 3625 psi
R3 = Medium strong rock, 3625 – 7250 psi
R4 = Strong rock, 7250 – 14500 psi
R5 = Very strong rock, 14500 – 36000 psi
R6 = Extremely strong rock, >36000 psi

RQD = Rock Quality Designation

BOREHOLE LOG KEY	INVESTIGATION	BOREHOLE LOG KEY	BUCKHORNGEOTECH
1	DRAFTING		Civil, Structural, and Geotechnical Engineers, Inc. 222 South Park Avenue Montrose, Colorado 81401 Phone (970) 249-6828 Fax (970) 249-0945
	FIELD DATE		
OF 1	JOB NO.		

FIELD SOIL IDENTIFICATION TERMS

RELATIVE DENSITY OF COHESIONLESS SOILS

DESCRIPTION	FIELD IDENTIFICATION	N VALUE
Very Loose	Easily penetrated with hand shovel	0 – 4
Loose	Easily penetrated with 1/2" rebar pushed by hand; easily excavated with hand shovel	4 – 10
Moderately Dense	Easily penetrated with 1/2" rebar driven with 5 lb. hammer; difficult to excavate with hand shovel	10 – 30
Dense	Penetrated 1 ft. with driven rebar; must be loosened with pick to excavate	30 – 50
Very Dense	Penetrated only a few inches with driven rebar; very difficult to excavate even with pick	>50

CONSISTENCY OF COHESIVE SOILS

DESCRIPTION	FIELD IDENTIFICATION	UNDRAINED SHEAR STRENGTH (psf)	N VALUE (Approx.)
Very Soft	Extrudes between fingers when squeezed	<250	0 – 2
Soft	Moulded by light finger pressure	250 – 500	2 – 4
Firm	Moulded by strong finger pressure	500 – 1000	4 – 8
Stiff	Indented by thumb	1000 – 2000	8 – 15
Very Stiff	Indented by thumbnail	2000 – 4000	15 – 30
Hard	Difficult to indent with thumbnail	>4000	>30

SOIL CONSTITUENTS

MODIFIER	trace	little	some	–ey or –y	and
% (by weight)	0 – 5	5 – 12	12 – 20	20 – 30	> 30

SHEET	INVESTIGATION	SOIL IDENTIFICATION TERMS	**BUCKHORN GEOTECH**
1	DRAFTING		Civil, Structural, and Geotechnical Engineers, Inc. 222 South Park Avenue Montrose, Colorado 81401 Phone (970) 249-6828 Fax (970) 249-0945
OF 1	FIELD DATE		
	JOB NO.		

APPENDIX C

INDEX LABORATORY TEST RESULTS

BUCKHORN GEOTECH
Civil, Structural & Geotechnical Engineers
222 South Park Ave. • Montrose, CO 81401
Ph.: (970) 249-6828 • FAX: (970) 249-0945

Sieve Analysis and Atterberg Limits

Project Name: Lake Otonawanda Improvements
Project Location: Lake Otonawanda
Client: Applegate Group, LLC.
Test Location: BH#1 @5.0-6.5'
Sample #: DS1

Date: 10/12/2009
Project #: 09-296-GRP
Sample by: BRB
Tested by: SJ/CC

Sieve Analysis

ASTM C136 / C117

Sieve	Opening (mm)	% Passing
3"	76.2	100.0
3/4"	19.0	92.7
3/8"	9.5	88.6
#4	4.75	86.2
#10	2.0	83.6
#40	0.425	78.2
#200	0.075	65.3

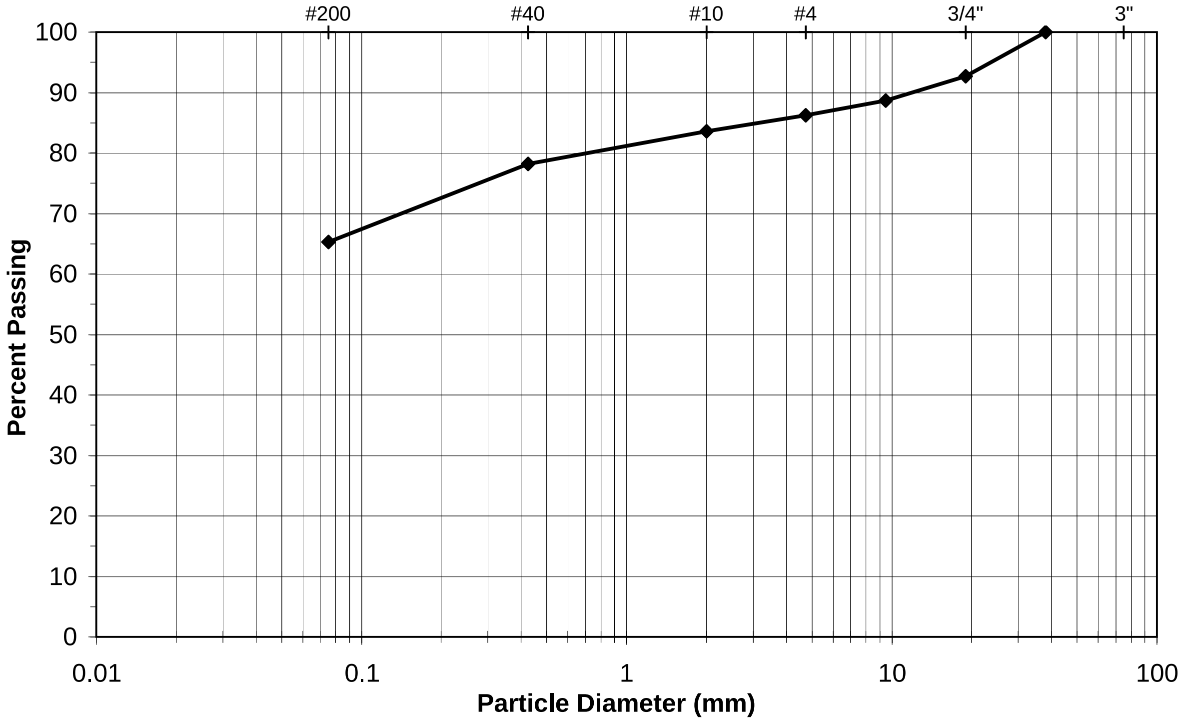
Atterberg Limits

ASTM D4318

Liquid Limit (LL)	38
Plastic Limit (PL)	21
Plasticity Index (PI)	17

Natural Moisture Content (%) = 29.5%

Soil Description: very dark brown to black sandy lean CLAY
USCS Classification: CL



Clay/Silt	Fine	Medium	Coarse	Fine	Coarse
FINES	**SAND**			**GRAVEL**	

% Fines = 65.3 % Sand = 20.9 % Gravel = 13.8

BUCKHORN GEOTECH

Civil, Structural & Geotechnical Engineers

222 South Park Ave. • Montrose, CO 81401
Ph.: (970) 249-6828 • FAX: (970) 249-0945

Sieve Analysis and Atterberg Limits

Project Name Lake Otonawanda Improvements
Project Location Lake Otonawanda
Client Applegate Group, LLC.
Test Location BH#1 @10.0-11.5'
Sample # DS2

Date 10/12/2009
Project # 09-296-GRP
Sample by BRB
Tested by SJ/CC

Sieve Analysis

ASTM C136 / C117

Sieve	Opening (mm)	% Passing
3"	76.2	100.0
3/4"	19.0	100.0
3/8"	9.5	98.2
#4	4.75	97.8
#10	2.0	94.2
#40	0.425	85.1
#200	0.075	68.6

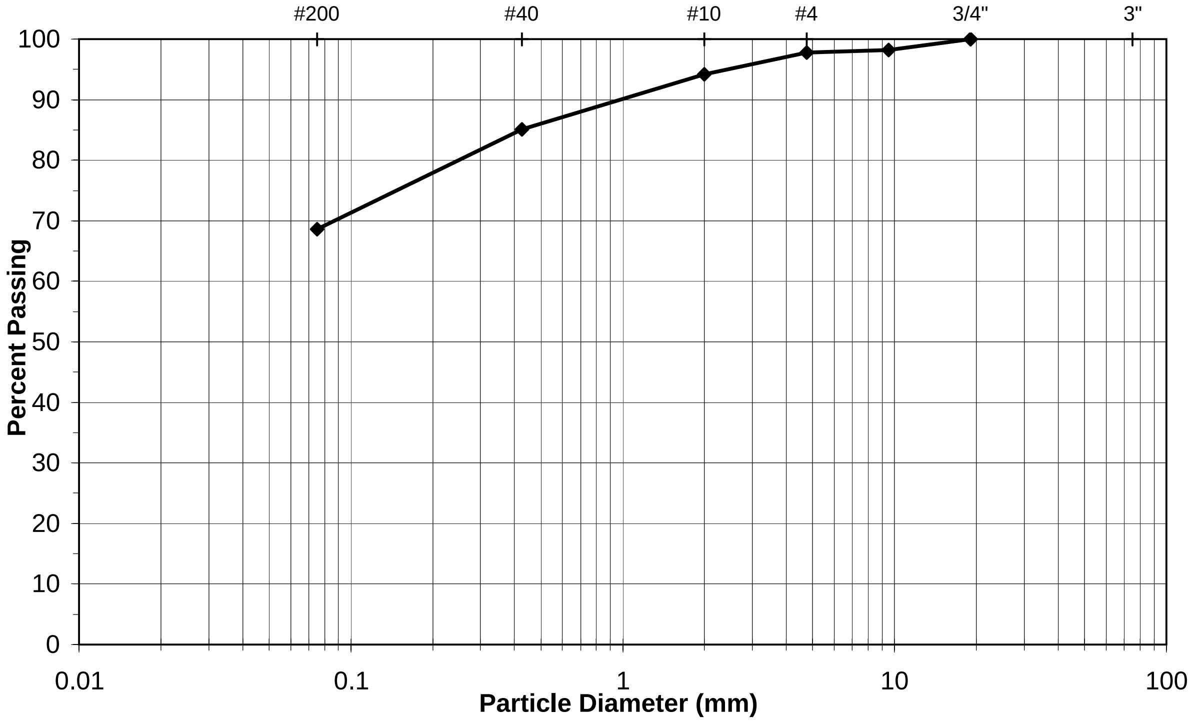
Atterberg Limits

ASTM D4318

Liquid Limit (LL)	48
Plastic Limit (PL)	17
Plasticity Index (PI)	31

Natural Moisture Content (%) = 23.3%

Soil Description reddish brown to dark brown sandy lean CLAY
USCS Classification CL



Clay/Silt	Fine	Medium	Coarse	Fine	Coarse
FINES	**SAND**			**GRAVEL**	

% Fines = 68.6
% Sand = 29.2
% Gravel = 2.2

BUCKHORN GEOTECH
Civil, Structural & Geotechnical Engineers
222 South Park Ave. • Montrose, CO 81401
Ph.: (970) 249-6828 • FAX: (970) 249-0945

Sieve Analysis and Atterberg Limits

Project Name: Lake Otonawanda Improvements
Project Location: Lake Otonawanda
Client: Applegate Group, LLC.
Test Location: BH#1 @15.0-16.5'
Sample #: DS3
Date: 10/12/2009
Project #: 09-296-GRP
Sample by: BRB
Tested by: SJ

Sieve Analysis

ASTM C136 / C117

Sieve	Opening (mm)	% Passing
3"	76.2	100.0
3/4"	19.0	100.0
3/8"	9.5	100.0
#4	4.75	99.7
#10	2.0	98.6
#40	0.425	94.4
#200	0.075	85.5

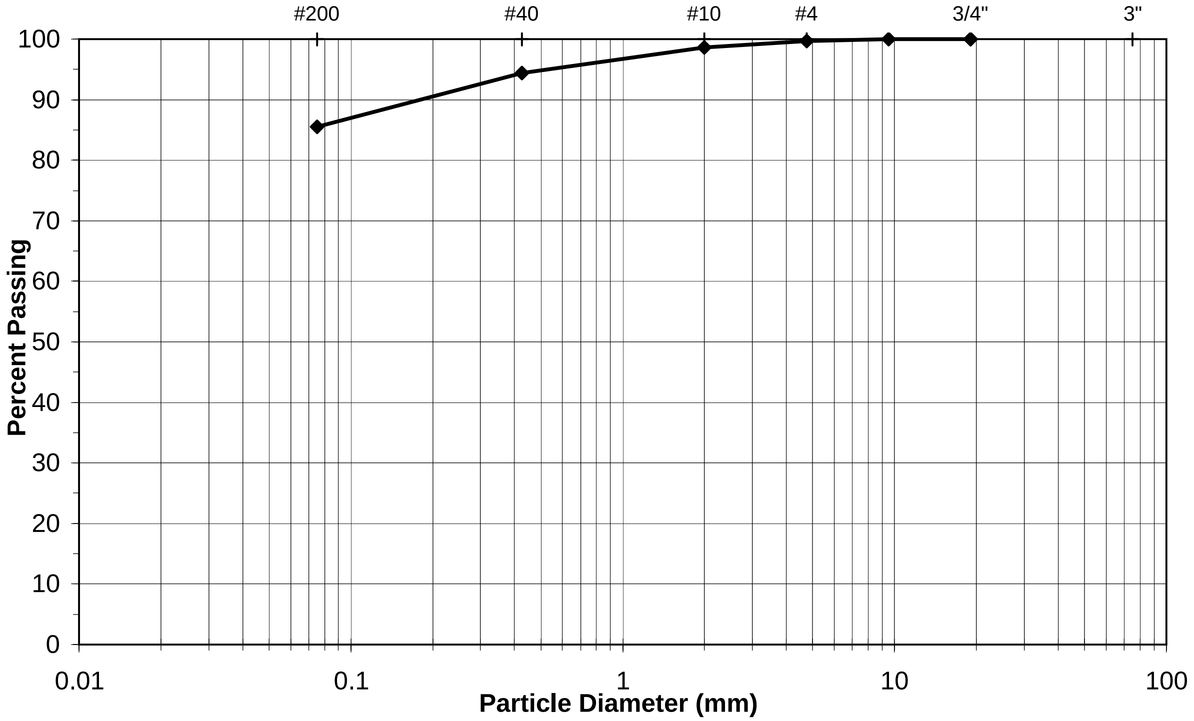
Atterberg Limits

ASTM D4318

Liquid Limit (LL): 54
Plastic Limit (PL): 18
Plasticity Index (PI): 36

Natural Moisture Content (%) = 26.6%

Soil Description: orange brown to reddish brown fat CLAY
USCS Classification: CH



Clay/Silt	Fine	Medium	Coarse	Fine	Coarse
FINES	**SAND**			**GRAVEL**	

% Fines = 85.5
% Sand = 14.2
% Gravel = 0.3

BUCKHORNGEOTECH
Civil, Structural & Geotechnical Engineers
222 South Park Ave. • Montrose, CO 81401
Ph.: (970) 249-6828 • FAX: (970) 249-0945

Sieve Analysis and Atterberg Limits

Project Name	Lake Otonawanda Improvements	Date	10/12/2009
Project Location	Lake Otonawanda	Project #	09-296-GRP
Client	Applegate Group, LLC.	Sample by	BRB
Test Location	BH#1 @25.0-26.5'	Tested by	SJ/CC
Sample #	DS5		

Sieve Analysis

ASTM C136 / C117

Sieve	Opening (mm)	% Passing
3"	76.2	100.0
3/4"	19.0	83.4
3/8"	9.5	65.3
#4	4.750	50.4
#10	2.000	38.6
#40	0.425	27.8
#200	0.075	17.2

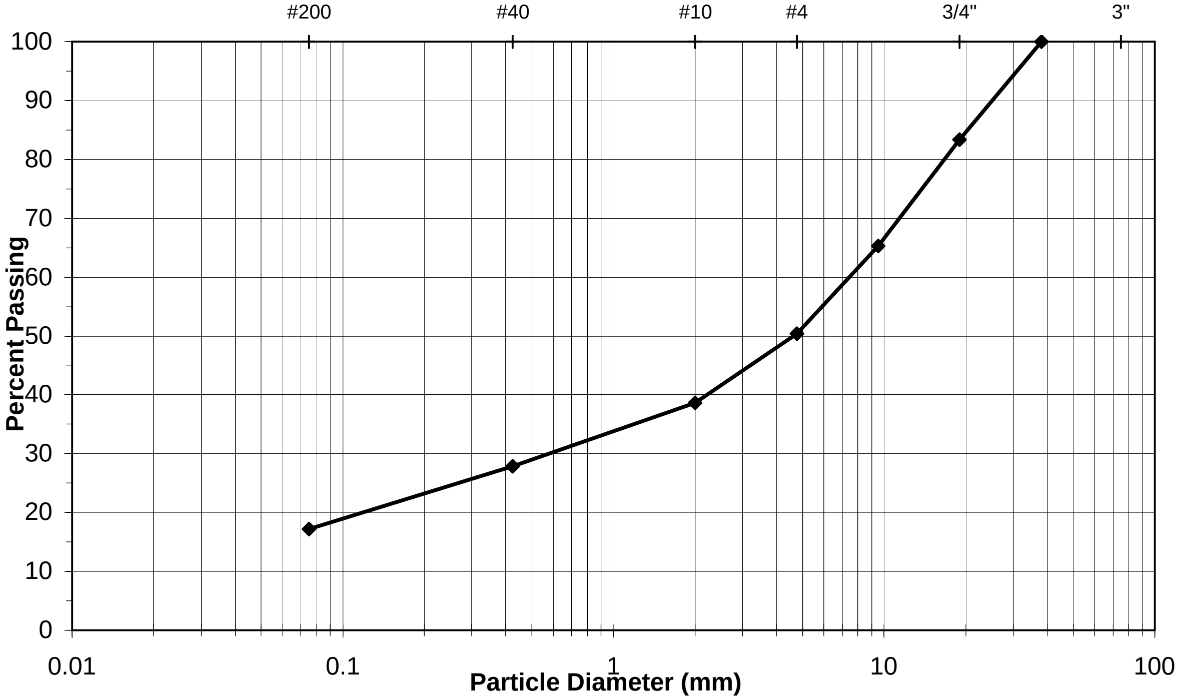
Atterberg Limits

ASTM D4318

Liquid Limit (LL)	46
Plastic Limit (PL)	27
Plasticity Index (PI)	19

Natural Moisture Content (%) = 24.9%

Soil Description dark gray clayey GRAVEL with sand
USCS Classification GC



Clay/Silt	Fine	Medium	Coarse	Fine	Coarse
FINES	SAND			GRAVEL	

% Fines = 17.2 % Sand = 33.2 % Gravel = 49.6

Sieve Analysis and Atterberg Limits

Project Name	Lake Otonawanda Improvements	Date	10/12/2009
Project Location	Lake Otonawanda	Project #	09-296-GRP
Client	Applegate Group, LLC.	Sample by	BRB
Test Location	BH#5 @5.0-6.5'	Tested by	SJ
Sample #	DS8		

Sieve Analysis

ASTM C136 / C117

Sieve	Opening (mm)	% Passing
3"	76.2	100.0
3/4"	19.0	100.0
3/8"	9.5	98.5
#4	4.75	94.2
#10	2.0	80.4
#40	0.425	51.8
#200	0.075	33.0

Atterberg Limits

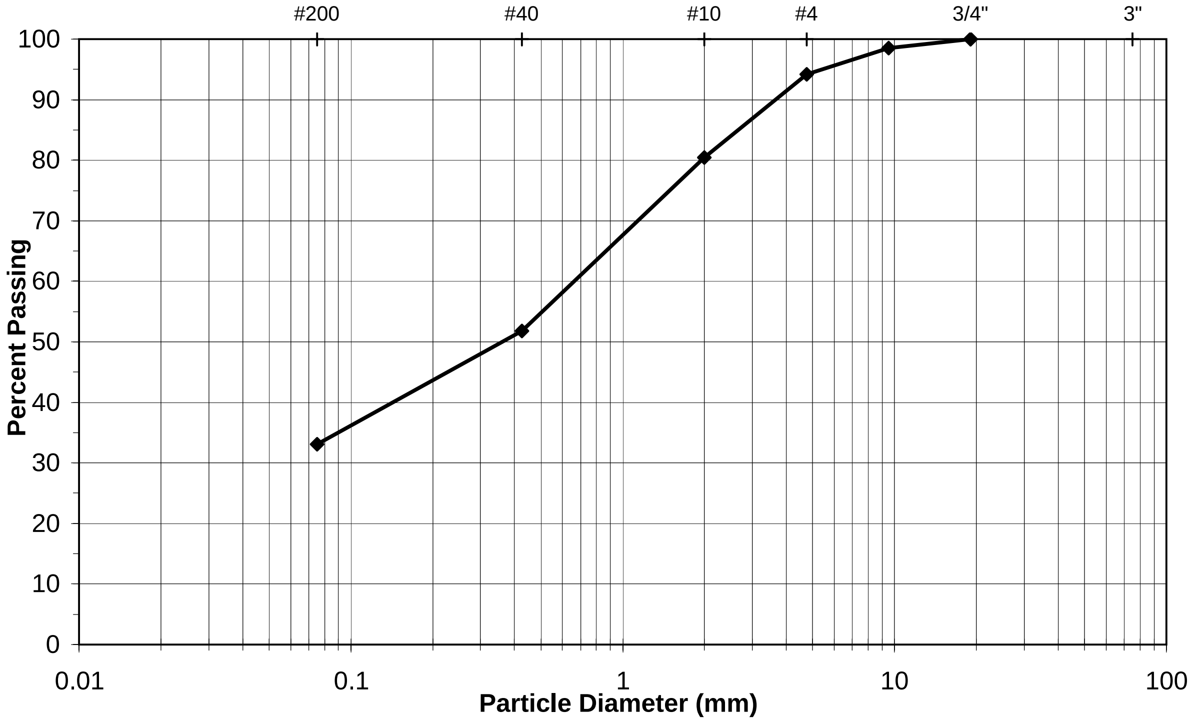
ASTM D4318

Liquid Limit (LL)	33
Plastic Limit (PL)	16
Plasticity Index (PI)	17

Natural Moisture Content (%) = 15.2%

Soil Description dark brown clayey SAND

USCS Classification SC



Clay/Silt	Fine	Medium	Coarse	Fine	Coarse
FINES	SAND			GRAVEL	

% Fines = 33.0

% Sand = 61.2

% Gravel = 5.8

Sieve Analysis and Atterberg Limits

Project Name: Lake Otonawanda Improvements
Project Location: Lake Otonawanda
Client: Applegate Group, LLC.
Test Location: BH#5 @15.0-16.5'
Sample #: DS10

Date: 10/12/2009
Project #: 09-296-GRP
Sample by: BRB
Tested by: SJ

Sieve Analysis

ASTM C136 / C117

Sieve	Opening (mm)	% Passing
3"	76.2	100.0
3/4"	19.0	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.0	98.7
#40	0.425	93.9
#200	0.075	85.9

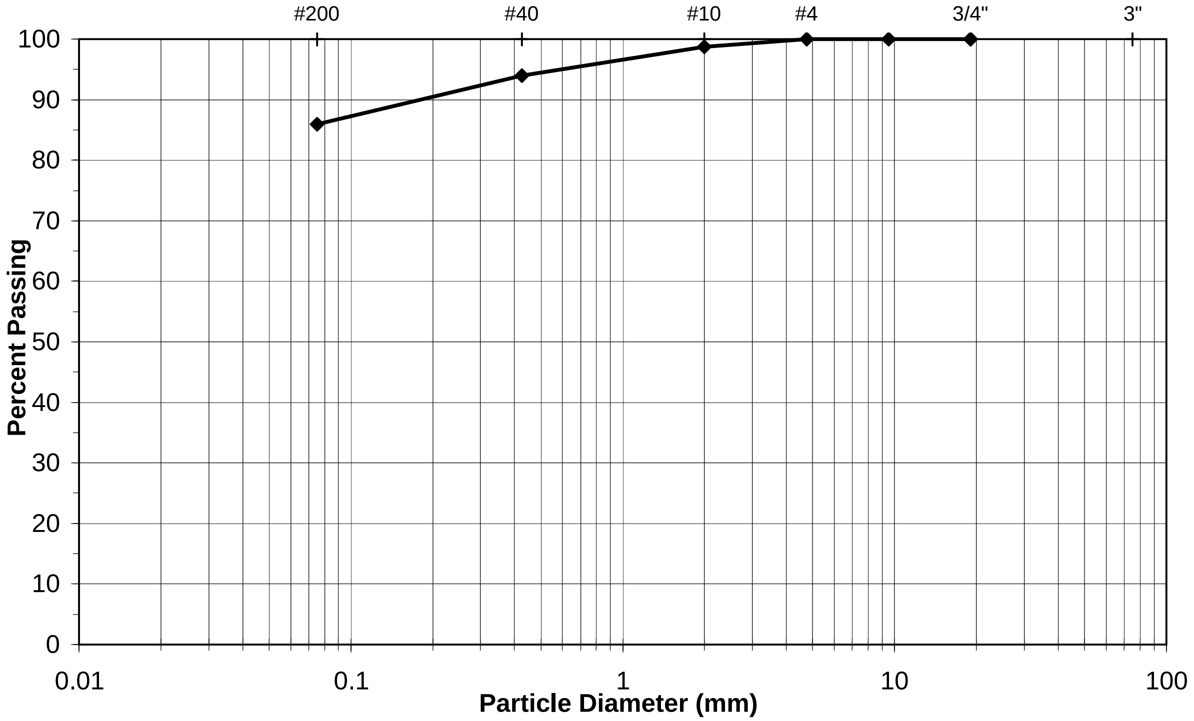
Atterberg Limits

ASTM D4318

Liquid Limit (LL)	50
Plastic Limit (PL)	17
Plasticity Index (PI)	33

Natural Moisture Content (%) = 25.8%

Soil Description: brown to reddish brown fat CLAY
USCS Classification: CH



Clay/Silt	Fine	Medium	Coarse	Fine	Coarse
FINES	SAND			GRAVEL	

% Fines = 85.9 % Sand = 14.1 % Gravel = 0.0

BUCKHORN GEOTECH
Civil, Structural & Geotechnical Engineers
222 South Park Ave. • Montrose, CO 81401
Ph.: (970) 249-6828 • FAX: (970) 249-0945

Sieve Analysis and Atterberg Limits

Project Name Lake Otonawanda Improvements
Project Location Lake Otonawanda
Client Applegate Group, LLC.
Test Location BH#6 @5.0-6.5'
Sample # DS15

Date 10/12/2009
Project # 09-296-GRP
Sample by BRB
Tested by SJ

Sieve Analysis

ASTM C136 / C117

Sieve	Opening (mm)	% Passing
3"	76.2	100.0
3/4"	19.0	100.0
3/8"	9.5	100.0
#4	4.75	99.6
#10	2.0	95.5
#40	0.425	84.9
#200	0.075	69.6

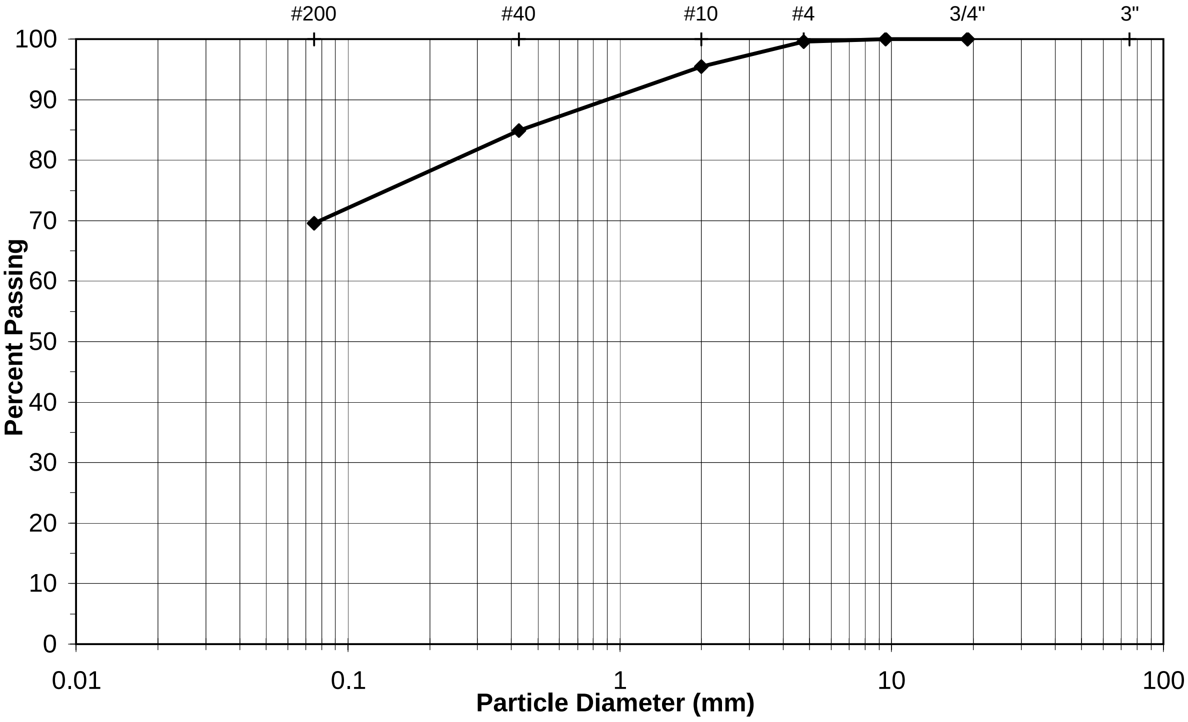
Atterberg Limits

ASTM D4318

Liquid Limit (LL)	45
Plastic Limit (PL)	18
Plasticity Index (PI)	27

Natural Moisture Content (%) = 22.7%

Soil Description reddish brown sandy lean CLAY
USCS Classification CL



Clay/Silt	Fine	Medium	Coarse	Fine	Coarse
FINES	SAND			GRAVEL	

% Fines = 69.6 % Sand = 30.0 % Gravel = 0.4

Sieve Analysis and Atterberg Limits

Project Name	Lake Otonawanda Improvements	Date	10/12/2009
Project Location	Lake Otonawanda	Project #	09-296-GRP
Client	Applegate Group, LLC.	Sample by	BRB
Test Location	BH#6 @10.0-11.5'	Tested by	SJ/CC
Sample #	DS16		

Sieve Analysis

ASTM C136 / C117

Sieve	Opening (mm)	% Passing
3"	76.2	100.0
3/4"	19.0	100.0
3/8"	9.5	100.0
#4	4.75	100.0
#10	2.0	98.7
#40	0.425	95.4
#200	0.075	85.5

Atterberg Limits

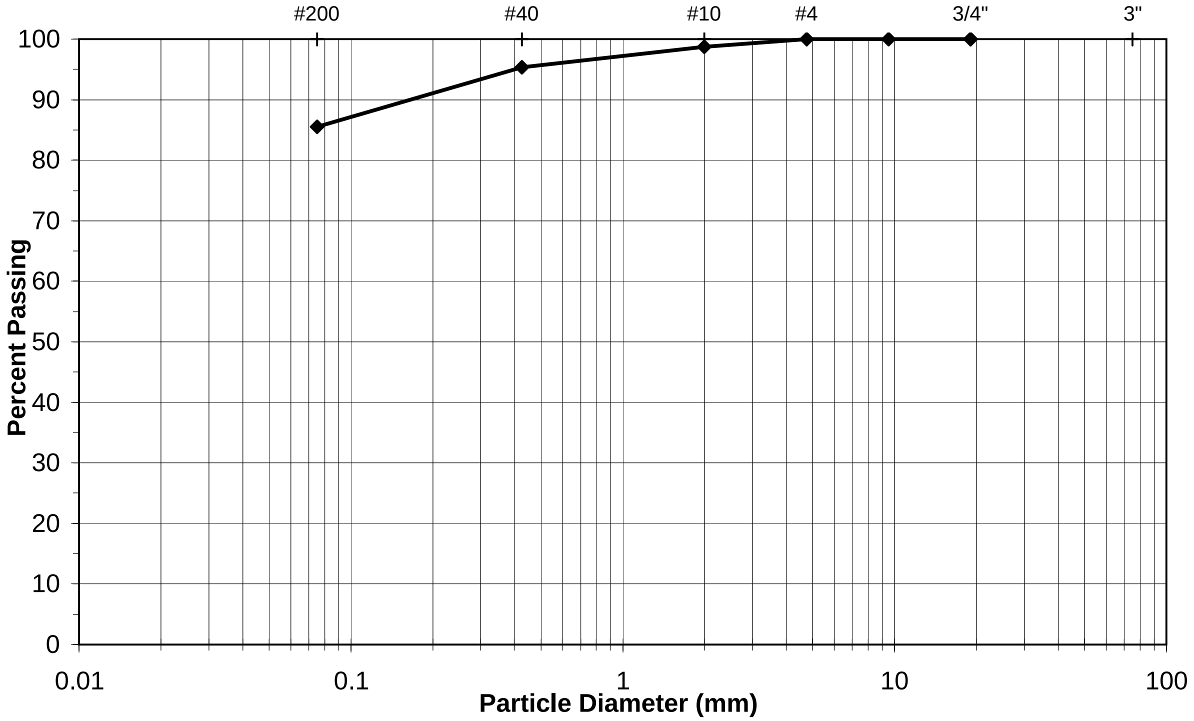
ASTM D4318

Liquid Limit (LL)	43
Plastic Limit (PL)	16
Plasticity Index (PI)	27

Natural Moisture Content (%) = 24.6%

Soil Description reddish brown lean CLAY

USCS Classification CL



Clay/Silt	Fine	Medium	Coarse	Fine	Coarse
FINES	SAND			GRAVEL	

% Fines = 85.5

% Sand = 14.5

% Gravel = 0.0

Dry Density

Project Name Lake Otonawanda Improvements
Project Location Lake Otonawanda
Client Applegate Group, LLC.

Date 10/8/2009
Project # 09-296-GRP
Sample by BRB
Test by SJ

Sample #	Test Location	Sample description	Wet Density (pcf)	Moisture Content (%)	Dry Density (pcf)
DS2	BH#1 @10.0-11.5'	reddish brown to dark brown sandy lean CLAY (CL)	124.4	25.0	99.5
DS9	BH#5 @10.0-11.5'	reddish brown, very moist, CLAY with sand	118.9	33.1	89.3
DS16	BH#6 @10.0-11.5'	reddish brown lean CLAY (CL)	121.1	28.5	94.3

APPENDIX D

INVENTORY PLANNING
ES
PERMITTING COMPLIANCE
Environmental Solutions, Inc.
600 CR 216 Rifle, Colorado 81650
(970) 618-6841 phone ~ (970) 625-1673 fax
email sdahmer@mailcw.com

April 28, 2010

Attn: Steve Moore
U.S. Army Corps of Engineers
Colorado/Gunnison Basin Regulatory Office
400 Rood Ave., Room 142
Grand Junction, CO 81501-2563

Steve:

Enclosed is a wetland delineation report and detailed supporting documentation for a JD which was conducted around the perimeter of Otonawanda Reservoir approximately 2.5 miles south of Ridgway. The reservoir is an off-channel structure located on an isolated, elevated bench well above surrounding natural drainages which are all tributary to the Uncompahgre River. The reservoir is situated entirely on private property owned by the Town of Ridgway, which entity also owns the water rights used to fill the reservoir. The attached delineation report, including maps, photos, data sheets and a completed Rapanos JD form, contains a detailed review of site conditions. This letter provides a brief summary of the major findings on the site.

The only water source into the reservoir basin is a ditch and pipeline headgate system that is operated at-will by the Town. Water can be drawn from three different sources, including Beaver Creek, East Fork Beaver Creek or West Fork Coal Creek. Once the water is in the system, it can either be diverted from the ditch to fill the reservoir or bypass the reservoir entirely and be sent directly into the pipeline feeding the Town's water treatment facility. Water entry to both the reservoir and the water treatment facility is entirely controlled via a decreed multiple-headgate ditch and pipeline delivery system operated by the Town.

The reservoir is situated on top of Miller Mesa in a small, isolated topographical depression basin (< 360 acres) between the Cottonwood Creek and Kettle Gulch watersheds with no defined drainage channels of any kind entering the basin. There are no mapped drainage channels on USGS quads and there is no evidence on the site itself to indicate any kind of defined-channel surface drainage into or out of the basin, which isn't surprising given the very small size of this "watershed." There is no spillway on the existing dam as there is no possibility of a natural flood entering the basin and there is no way for water to escape the reservoir other than the 12" outlet pipe feeding directly into the Town's water treatment facility.

Since there is no spillway and no natural inlet or outlet channel connecting the reservoir basin to any watershed and the only outlet is a pipeline directly feeding the Town's water treatment facility, there is no surface connection to any TNW or RPW. Further, the basin in which it is located has no drainage feature or water source connecting to any Waters of the U.S. and there is no way for any potential pollutants to enter such waters from this system since it is directly piped to the Town's water treatment facility. Therefore, this reservoir and its associated perimeter wetlands meet the definition of an Isolated Water per criteria set forth by the SWANCC decision.

The Town plans to enlarge this facility in order to meet growing water demands in the local community. In preparation for that work, the Town contracted Environmental Solutions, Inc. to conduct a formal delineation and serve as its representative to the USACE. We are providing notice to the USACE of the isolated nature of this system and would like to request a concurrence letter from the USACE for the project record.

Please contact me if you would like to arrange a site visit or have any further questions. I can be reached at 618-6841.

Sincerely,

Steve D. Dahmer
Enclosures

Cc: Craig Ullmann, Applegate Group, Inc.

WETLAND DELINEATION REPORT
TOWN OF RIDGWAY
OTONAWANDA RESERVOIR
OURAY COUNTY, COLORADO

PREPARED FOR:

Applegate Group, Inc.
Attn: Craig Ullmann
118 West 6th Street Suite 100
Glenwood Springs, CO 81601

October 2009

WETLAND DELINEATION REPORT

Town of Ridgway
Otonawanda Reservoir
Ouray County, Colorado

Site Description

The subject property is owned by the Town of Ridgway, 201 North Railroad Street, PO Box 10, Ridgway, Colorado 81432. Ridgway is a home rule municipality located in Ouray County.

The property is located approximately 2.5 miles south of Ridgway, Colorado atop an upland ridge known as Miller Mesa and is situated in an isolated topographical depression between Cottonwood Creek and Kettle Gulch, both tributaries to the Uncompahgre River (Figure 1). The property is accessed from Highway 62 by turning west from Highway 62 into the town of Ridgway and following Sherman Street west through town to Amelia Street. Turn left on Amelia Street and travel south approximately 0.3 miles and turn right onto County Road 5. Proceed south on CR5 approximately 1.8 miles to the driveway entrance, which is on the left side of the road marked by a double green gate and a mailbox marked 5300 CR5. The entry is locked since the driveway serves several private residences and the gravel driveway access continues another 1.5 miles up the mesa to the reservoir. The project area sits roughly between 8530-8560 feet in elevation. The map coordinates of the reservoir are NW ¼ of Section 32, of Township 45 North, Range 8 West of the 6th P.M.

The project area is situated on a 40-acre parcel. The reservoir itself, when full, covers 26.2 surface acres and holds approximately 120 AF of water. The reservoir is an off-channel structure built entirely on uplands and filled via a multiple headgate ditch delivery system (Figures 2-4). The system can bypass the reservoir and deliver water directly into the pipeline feeding the Town's water treatment facility. The only exit for water in the reservoir itself is also via the pipeline feeding directly to the water treatment facility. There is no spillway and no natural inlet or outlet channel connecting the reservoir basin to any watershed. The entire reservoir area was surveyed for wetlands and jurisdictional waters of the U.S. A total of 2.6 acres of wetlands meeting all three standard jurisdictional criteria were discovered on the site, all of which were entirely lucustrine-associated around the reservoir perimeter (Figure 5). Hydrology for the entire study area is clearly dependent on the reservoir, which is filled exclusively by the ditch and headgate system which can draw water from three different sources, including Beaver Creek, the East Fork Beaver Creek and the West Fork of Coal Creek. All diversion points are approximately 3.5 linear miles above the reservoir.

A survey to delineate wetlands and waters of the U. S. was conducted on October 6 & 7, 2009 by Steve Dahmer of Environmental Solutions, Inc. (ESI) using the routine onsite determination method as described in the 1987 Federal Manual for Identifying and Delineating Jurisdictional Wetlands in conjunction with the Western Mountains, Valleys and Coast Region Supplement. Site conditions were normal at the time of the delineation, with the reservoir full, no snow cover of any kind and all plant species intact and clearly identifiable. ESI collected study pit data, flagged the jurisdictional boundary and provided photo documentation of the wetlands. Surveying and mapping was completed by Buckhorn Geotech engineering based in Montrose, Colorado.

Vegetation

The delineation area consists chiefly of grass-grasslike and forb vegetation with a few shrub species present. The surrounding uplands are a grass-shrub mix giving way to oakbrush and ponderosa pine forest, with some areas of aspen. The wetlands are supported exclusively around the perimeter of the reservoir itself. The dominant vegetation of the wetland area consists of Beaked sedge (*Carex utriculata*), Water Sedge *(Carex aquatilis)*, Broad-leaved Cat-tail *(Typha latifolia),* and Creeping spikerush (*Eleocharis macrostachya*), along with Colorado rush *(Juncus confusus),* Longstyle rush *(Juncus longistylus),* Hard-stem bulrush *(Scirpus acutus)* and minor occurrences of Coyote willow *(Salix exigua)* and a single specimen of Whiplash willow *(Salix lasiandra)*.

Extensive stands of Gambel's oak (*Quercus gambelli*) intermixed with Ponderosa pine (*Pinus ponderosa*), some Aspen (*Populus tremuloides*), Rubber rabbitbrush *(Chrysothamnus nauseosus),* Douglas rabbitbrush *(Chrysothamnus viscidiflorus),* Cudweed sagewort *(Artemisia ludoviciana),* Western yarrow *(Achillea millefolium)* and Idaho fescue (*Festuca idahoensis*) dominate the surrounding uplands.

Understory plants in the forested upland areas are diverse, including Canadian Bluegrass *(Poa compressa),* Western wheatgrass *(Agropyron smithii)),* Lupine *(Lupinus argenteus),* Orchardgrass (*Dactylis glomerata*), Timothy *(Phleum pratense),* Alsike Clover *(Trifolium hybridum),* Virginia Strawberry *(Fragaria ovalis),* Snowberry (*Symphoricarpos oreophilus*), Wood's rose (*Rosa woodsii*), Mullein *(Verbascum thapsis)* and Bearberry (*Arctostaphylos uva-ursi*), along with some weed species such as Canadian Thistle *(Cirsium arvense),* Houndstongue (*Cynoglossum officinale*), Horseweed *(Conyza canadensis),* Tumblemustard *(Sysimbrium altissimum)* and Dandelion *(Taraxicum officinale).* Table 1 shows a complete list of plants encountered on the property.

Hydrology

Eight (8) study pits were excavated across the property to inspect for hydrology indicators, four of which were recorded on routine data forms.

It is clear that the hydrology supporting wetlands stems directly from the waterline perimeter of Otonawanda Reservoir itself, with a narrow band of wetlands surrounding the reservoir. All wetland areas and hydrology indicators are immediately adjacent to the reservoir shoreline. The entire site is located on top of Miller Mesa in a small, isolated topographical depression basin (< 360 acres) between the Cottonwood Creek and Kettle Gulch watersheds with no defined drainage channels of any kind entering the basin. The drainage area is so small and the basin topography is isolated from the two drainages named above such that a dam break analysis may not even be required by the State

Engineer's Office (SEO). Engineers from Applegate Group, Inc. ran a preliminary HEC-RAS model of a 100-year flood event for this isolated basin and determined the water level in the reservoir would rise only about 1 foot under such extreme circumstances (Craig Ullmann, pers. comm.). This flood-event rise would be attributable to sheet flow and direct precipitation in the small basin since there are no mapped drainage channels on USGS quads and there is no evidence on the site to indicate any kind of defined-channel, natural surface drainage into or out of the basin. All water for the reservoir is transported in a ditch and pipeline system up to a divider box west of the reservoir where it can either be sent into the basin or bypass the reservoir and feed directly into a pipeline supplying the Town's water treatment facility. There is no surface connectivity to any RPW or TNW, nor is there any way for potential pollutants to exit the basin and reach any such water as the only drainage possible is the pipeline feeding directly into the Town's water treatment facility. Therefore, this reservoir is clearly isolated according to SWANCC.

Soils

A total of eight (8) study pits were dug by hand to evaluate the soils on the property and results for four representative pits were recorded on routine data forms.

Native soils encountered on the property were comprised mainly of Shanley-Davoty gravelly loams, 3-25% slopes throughout most of the basin. There were also some Aquic Argicryolls 1-12% slopes in a few locations of emergent vegetation in the reservoir itself. Field inspections confirmed the types mapped by NRCS in the area.

The soils ranged from sandy to gravelly clay loam materials, with coarse sand and small gravels generally appearing below 4 inches and becoming more abundant deeper in the profile. Soils were generally deep and well-drained. Munsell color at the surface was generally consistent at 10YR 3/2, though it ranged to 10YR 2.5/1 in some locations. Soil pits in the clearly wet areas tended to show very weak hydric evidence in the form of low chroma colors, and a few with some very minor oxidized root channels, which further demonstrates the lack of any native wetland in this area pre-dating the reservoir. The soils were "hydric" only by virtue of being saturated at the time of inspection by waters anthropogenically diverted into the basin. If it were not for hydrology being present at the time of inspection, these soils would show no hydric evidence at all. Soil pits in non-jurisdictional areas were completely lacking in all indicators.

Summary

The property supports a single, distinct wetland area that is lucustrine-supported by the man-made Otonawanda Reservoir. The reservoir itself contains 26.2 surface acres of open water when full and supports a 2.6-acre fringe wetland that meets jurisdictional criteria as set forth in 1987 Manual. The wetland type consists almost entirely of a grass/grass-like and forb community.

It is important to note that the sole source of water supporting the wetland areas on this property consist of anthropogenically-controlled waters, which can be diverted from three sources (Beaver Creek, East Fork Beaver Creek or West Fork Coal Creek). The Town can release these waters at-will into the reservoir basin or bypass the reservoir and send them directly into the Town’s water treatment facility. If the water is stored in the reservoir, the only outlet for that water is also via the 12-inch pipeline that terminates in the Town’s water treatment facility, from whence treated waters are sent out to households and businesses in Ridgway. There are no natural springs, streams, gullies or other possible tributary water sources to the reservoir site, nor is there any historical evidence to suggest the area ever supported a wetland community prior to the reservoir construction. Further, the basin in which it is located has no drainage feature or water source connecting to any Waters of the U.S. Therefore, this reservoir is clearly isolated according to SWANCC.

In order to meet future water demands for the Town of Ridgway, the Town wishes to enlarge this off-channel reservoir and improve the existing structures to match the decreed storage in the water right and improve water conservation and efficiency in the system. Since this reservoir meets the requirements of an isolated Water/wetland, it is not regulated under the Clean Water Act and therefore no permit is required from the USACE.

Table 1: Common plants found on the Otonawanda Reservoir study area.

ID	Scientific Name	Common Name	Indicator Status	Stratum
1	Populus tremuloides	Quaking Aspen	FAC	T
2	Pinus ponderosa	Ponderosa Pine	FACU	T
3	Carex lanuginosa	Wooly Sedge	OBL	H
4	Fragaria ovalis	Virginia Strawberry	FACU	H
5	Phleum pratense	Timothy	FACU	H
6	Delphinium barbeyi	Arapahoe Peak Larkspur	FAC	H
7	Lupinus argenteus	Common Lupine	NI	H
8	Taraxicum officinale	Dandelion	FACU	H
9	Dactylis glomerata	Orchardgrass	FACU	H
10	Typha latifolia	Broad-leaved Cat-tail	OBL	H
11	Potentilla fruticosa	Shrubby Cinquefoil	FACW	H
12	Potentilla gracilis	Soft Cinquefoil	FAC	H
13	Artemisia ludoviciana	Cudweed Sagewort	FACU	H
14	Quercus gambelli	Scrub Oak	FACU	T
15	Chrysothamnus viscidiflorus	Douglas Rabbitbrush	FACU	S
16	Symphoricarpos oreophilus	Mountain Snowberry	FACU	H
17	Achillea millefolium	Yarrow	FACU	H
18	Chrysothamnus nauseosus	Rubber Rabbitbrush	FACU	S
19	Cynoglossum officinale	Houndstongue	NI	H
20	Cirsium arvense	Canada Thistle	FACU	H
21	Poa pratensis	Kentucky Bluegrass	FACU	H
22	Agropyron smithii	Western Wheatgrass	FACU	H
23	Trifolium hybridum	Alsike Clover	FAC	H
24	Bromus tectorum	Cheatgrass	FACU	H
25	Festuca idahoensis	Idaho Fescue	FACU	H
26	Stipa lettermanii	Letterman Needlegrass	NI	H
27	Actaea rubra	Baneberry	NI	H

ID	Scientific Name	Common Name	Indicator Status	Stratum
28	Poa compressa	Canadian bluegrass	FACU	H
29	Arctostaphylos uva-ursi	Bearberry	NI	H
30	Thalictrum occidentale	Western Meadowrue	FACU	H
31	Osmorhiza berteros	Mtn Sweet Cicely	NI	H
32	Agrostis stolonifera	Redtop Bentgrass	FACW	H
33	Salix lasiandra	Whiplash Willow	OBL	S
34	Salix exigua	Coyote Willow	OBL	S
35	Delphinium geyeri	Low Larkspur	NI	H
36	Ribes inerme	White Stem Gooseberry	FAC	S
37	Juncus longistylus	Longstyl Rush	FACW	H
38	Juncus coloradoensis	Colorado Rush	FACW	H
39	Carex utriculata	Beaked Sedge	OBL	H
40	Carex aquatilis	Water Sedge	OBL	H
41	Scirpus acutus	Hard-Stem Bulrush	OBL	H
42	Rumex crispus	Curly Dock	FACW	H
43	Conyza canadensis	Horseweed	NI	H
44	Sisymbrium altissimum	Tumble Mustard	FACU	H
45	Grindelia squarrosa	Curlycup Gumweed	FACU	H

Appendix

SP-1, typical of wetland fringe soils around entire Reservoir. Soil wetland indicators such as redox features were almost entirely lacking other than the presence of hydrology.

SP-2, typical of the Shanley-Davoty upland soils surrounding the reservoir.

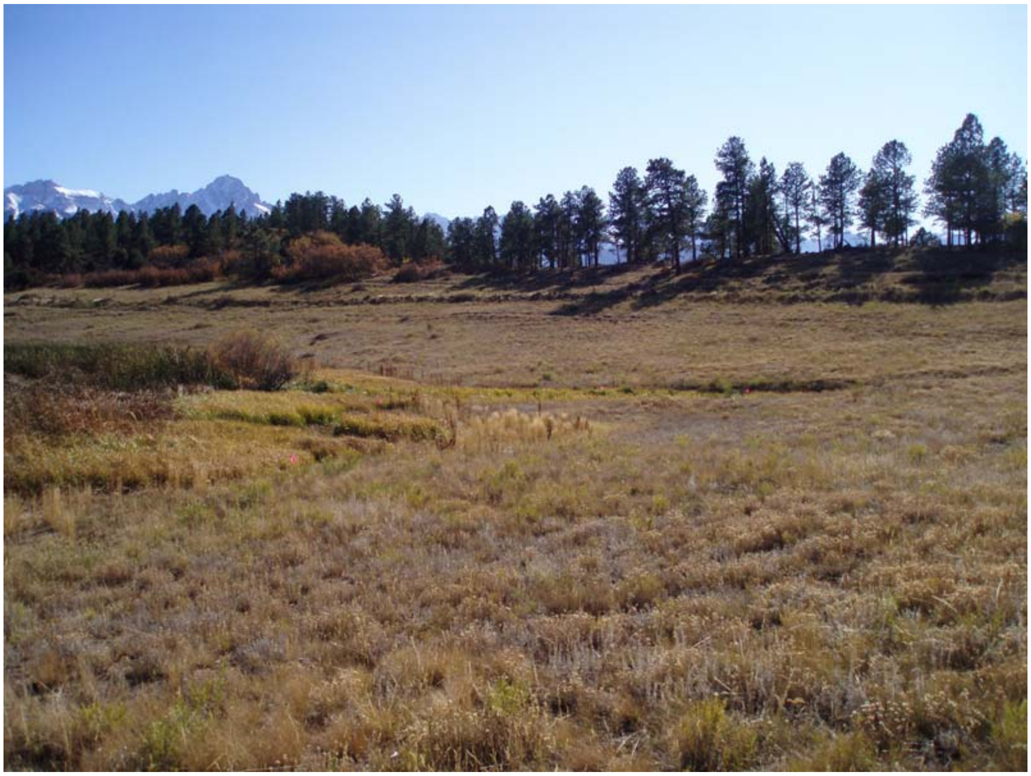
Typical wetland fringe along south edge of reservoir. Photo taken from Ridway Ditch at diversion structure facing Northeast. Note diversion channel in lower left photo actively flowing water to the reservoir. Control box is just out of photo to left.

Typical shoreline along dike on east side of reservoir. Note wave action has eroded the dike and only patches of wetland fringe can persist here.

Existing dam/dike for the Reservoir. The intent was to contain water on Town property and not allow flooding onto adjacent private land. The dam is in need of repairs.

Photo of one of four breach locations along the existing dam. The breaches allow water to spill onto the adjacent property when the reservoir is full or during storms with wave action, extending the created wetland area beyond the Town property.

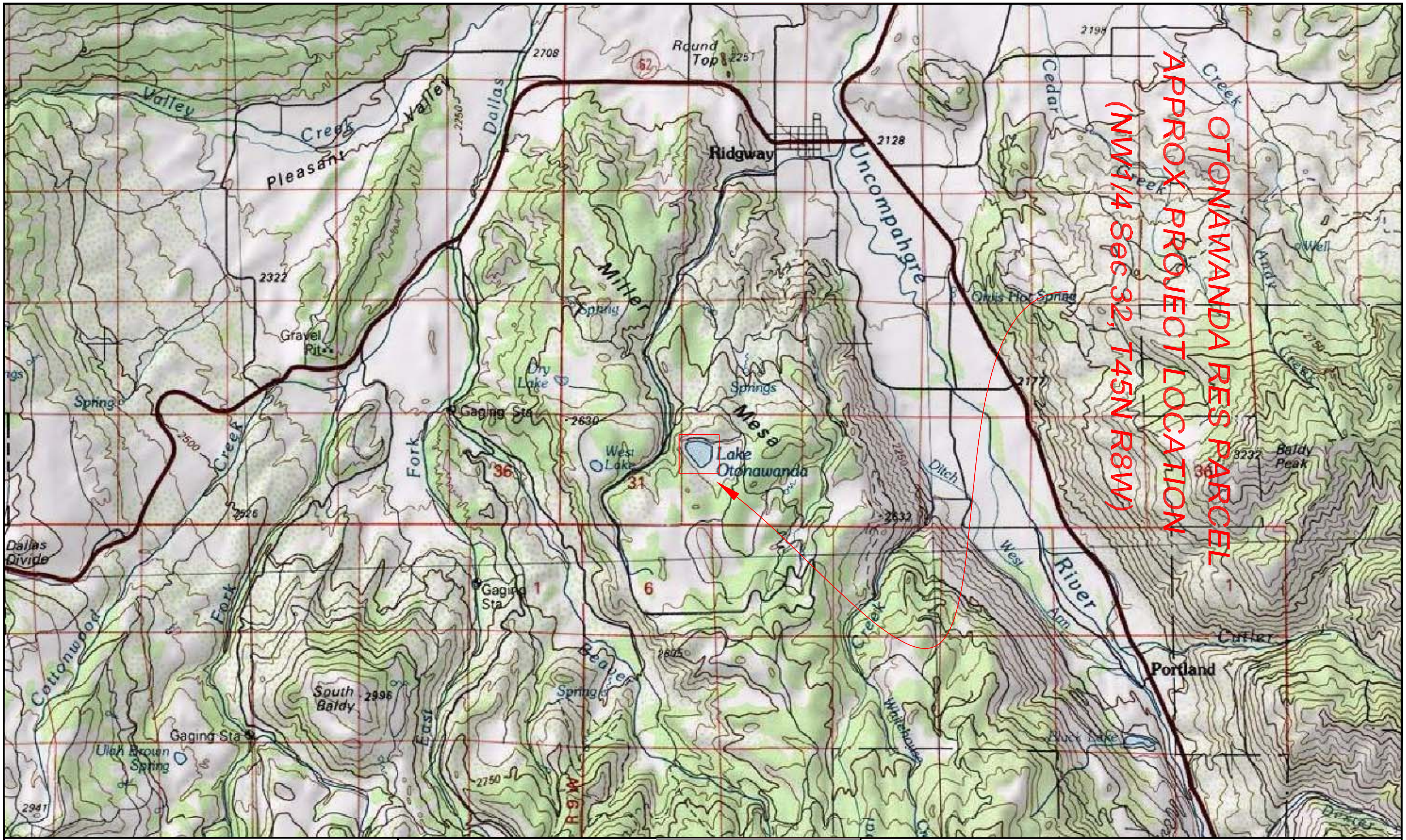
View below dike onto adjacent private land. Note flooded corrals and pasture due to dam breaches and leaks.



View of west side of reservoir looking south toward the original outlet works. Note the rim of the natural topographical depression in which the reservoir is located, and the Ridgway Ditch just under the rim, which feeds the reservoir.

View of original outlet tunnel location that drains the reservoir into the Town’s pipeline directly to water treatment facility.

Study pit SP-3, typical of the wettest portions of the reservoir perimeter. Note lack of oxidized roots and other redox features.



PROJECT LOCATION

SITE MAP

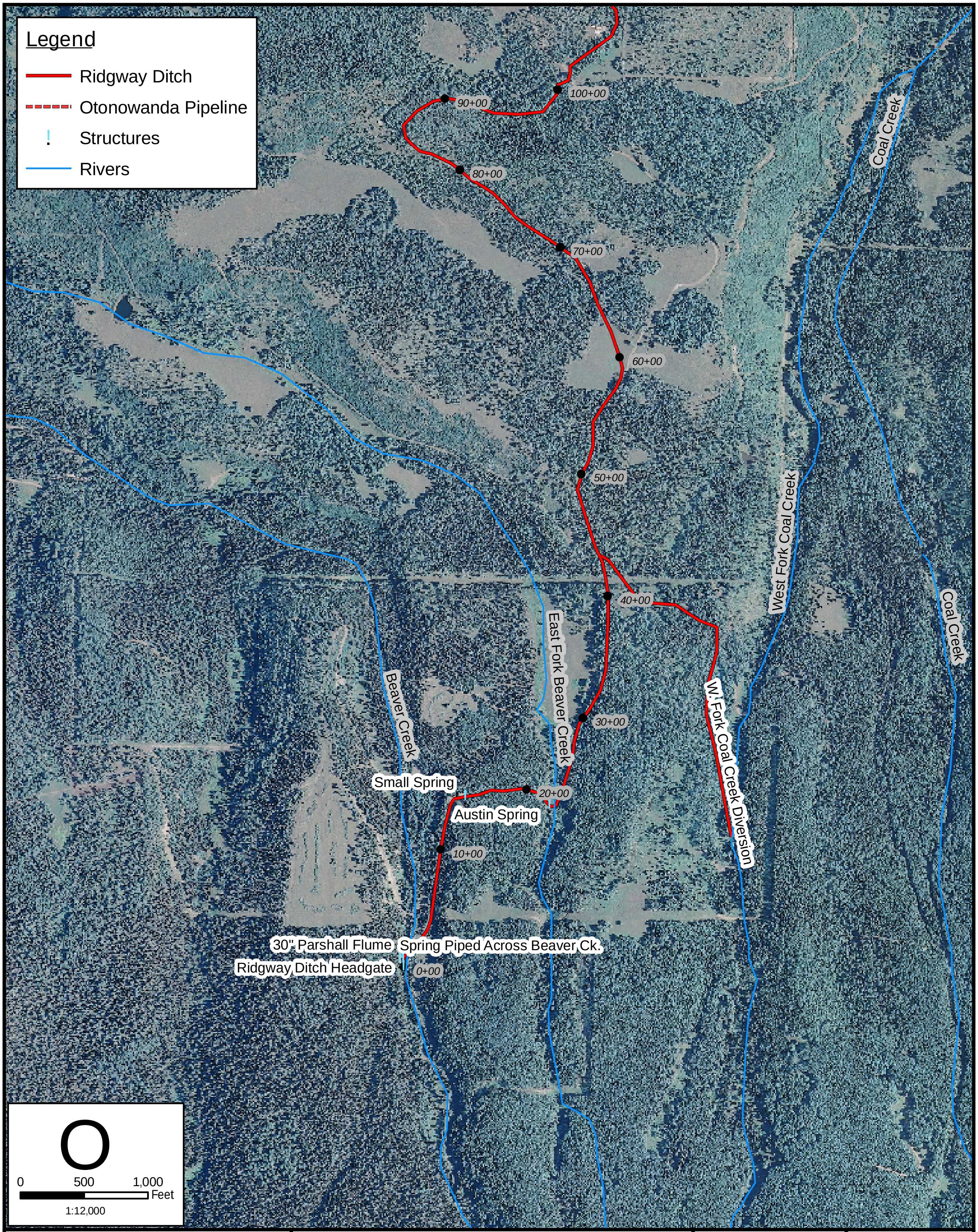
Environmental Solutions, Inc.
600 CR 216
Rifle, CO 81650
Phone: (970) 618-6841
Fax: (970) 625-1673
sdahmer@mailcw.com

INVENTORY PLANNING PERMITTING COMPLIANCE

OTONAWANDA PARCEL
OURAY COUNTY

FIGURE NO.
1

File Name: Otonawanda
Job No: 199
Drawn by: SDD
Aprvd by: SDD
Date: NOV 09
CLIENT: Applegate Group, Inc.



Applegate Group, Inc.

Water Resource Advisors for the West

1499 West 120th Ave., Ste 200
Denver, CO 80234-2728
www.ApplegateGroup.com

Phone: (303) 452-6611
Fax: (303) 452-2759
e-mail: info@applegategroup.com

Town of Ridgway

Ridgway Ditch

Reach 1

Date: 20 Jan 2010

Job #: 09-139

Drawn By: CMU

Figure: 2

Of: 5

Printed: February 6, 2008 by JMD
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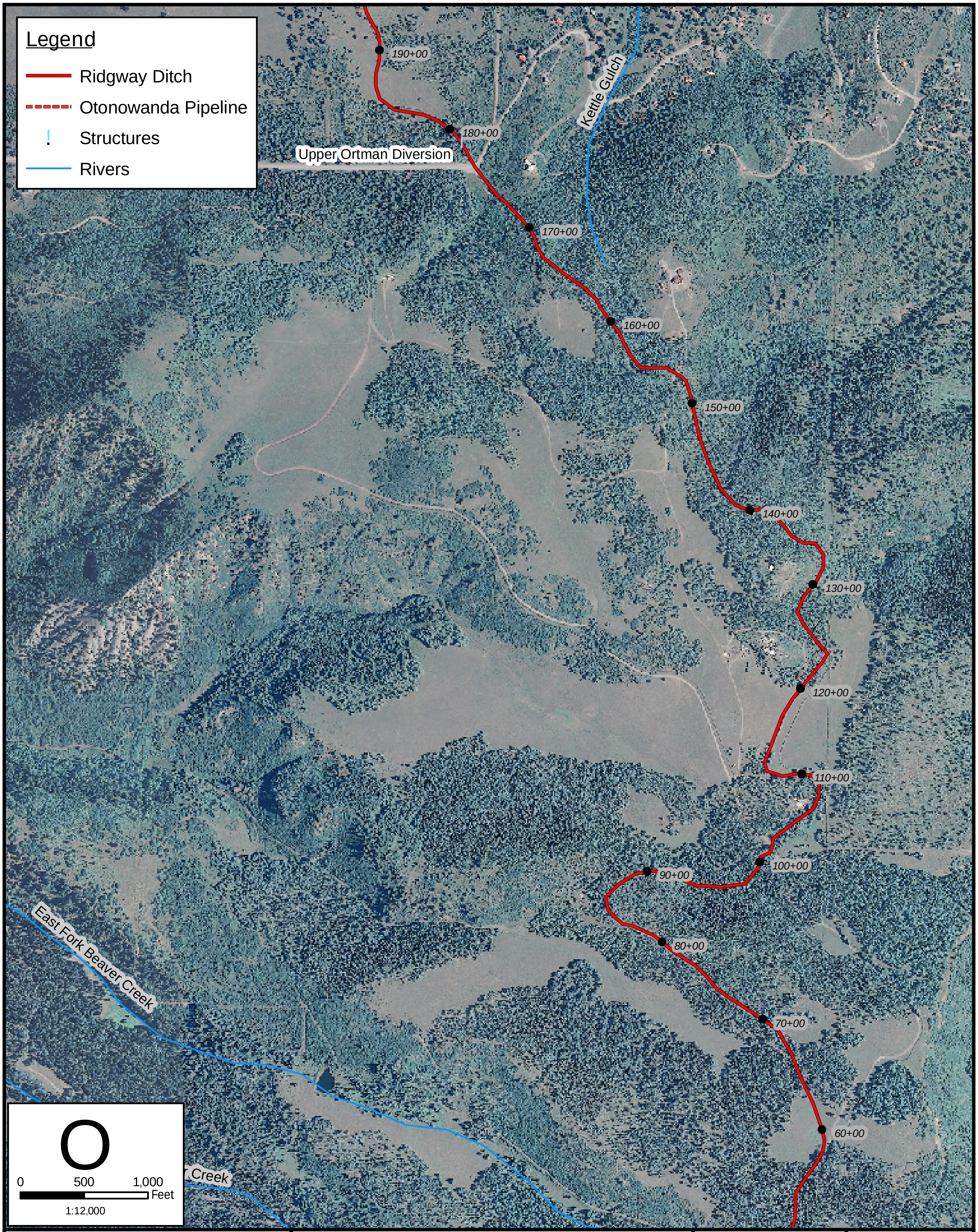

Printed: February 6, 2008 by JMD
G:\Templates\8.5 x 11 (Profile).mxd

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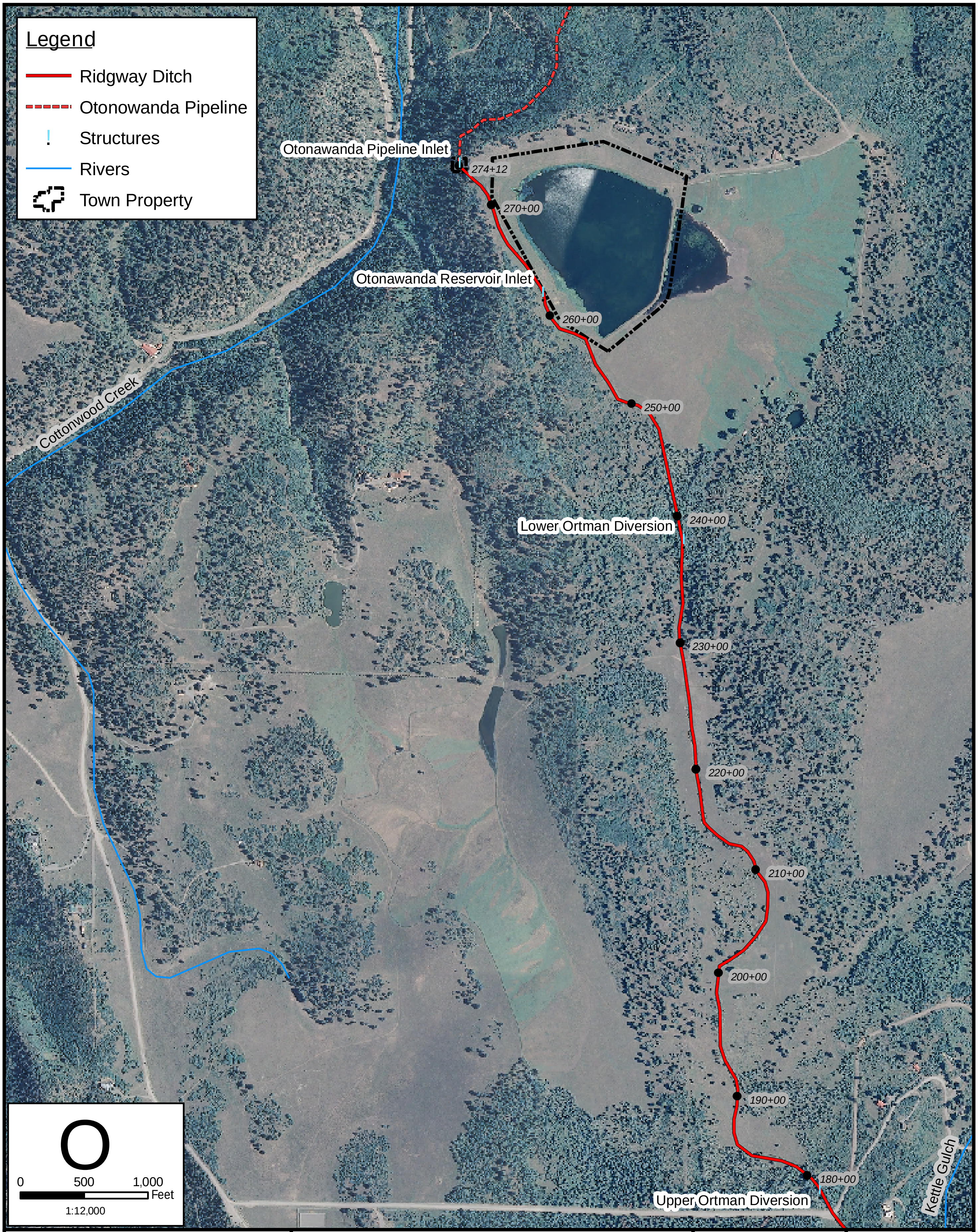
e-mail: info@applegategroup.com

Town of Ridgway
Ridgway Ditch
Reach 2

Date: 20 Jan 2010
Job #: 09-139
Drawn By: CMU

Figure: 3
Of: 5



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Phone: (303) 452-6611
Fax: (303) 452-2759
e-mail: info@applegategroup.com

Town of Ridgway

Ridgway Ditch
Reach 3

Date: 20 Jan 2010

Job #: 09-139

Drawn By: CMU

Figure: 4

Of: 5

Printed: February 6, 2008 by JMD
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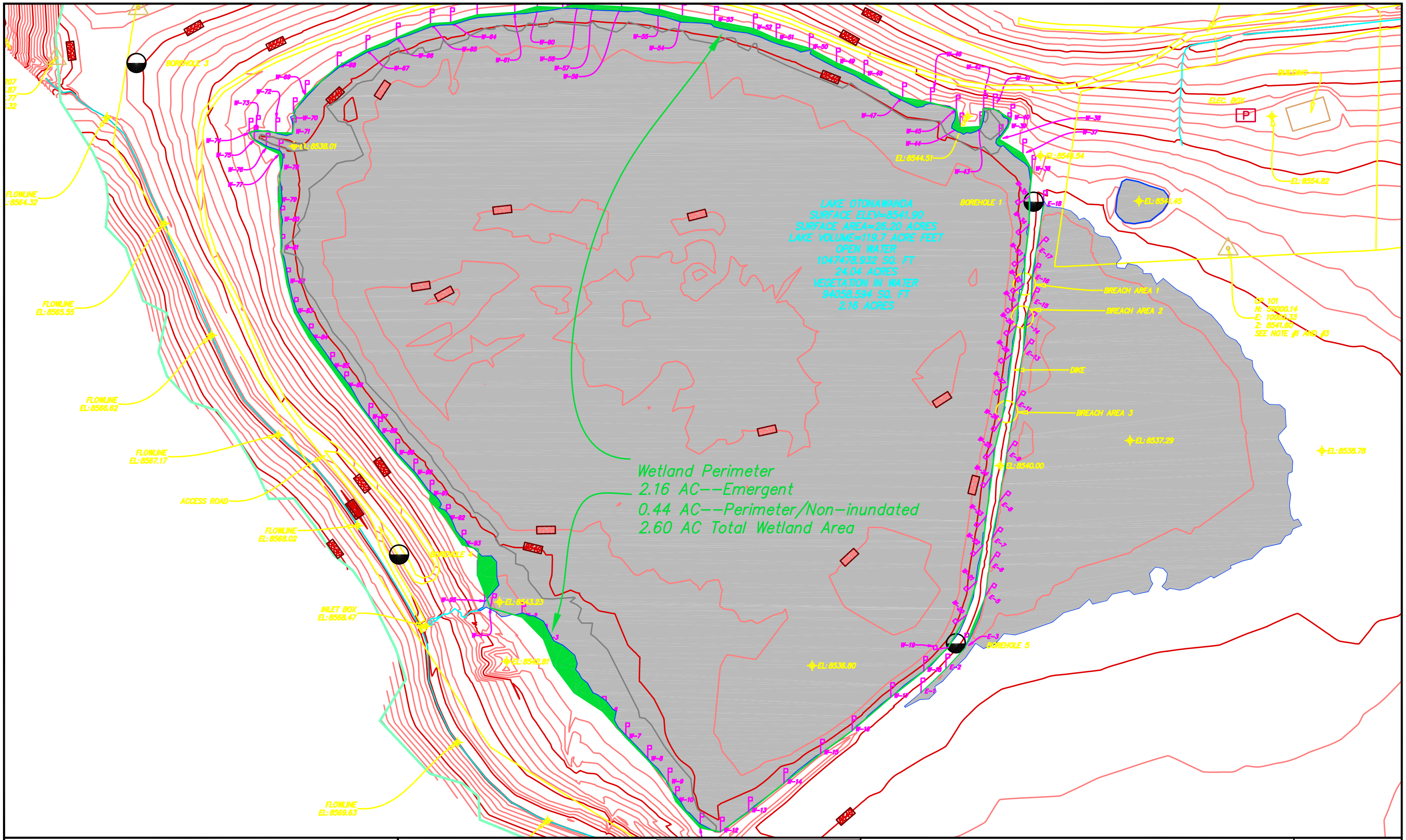


WETLAND DELINEATION DETAIL MAP

Environmental Solutions, Inc.
600 CR 216
Rifle, CO 81650
Phone: (970) 618-6841
Fax: (970) 625-1673
sdahmer@mailcw.com

INVENTORY PLANNING ES PERMITTING COMPLIANCE

OTONAWANDA PARCEL
OURAY COUNTY

FIGURE NO. 5

File Name: Otanawanda		Job No: 199	CLIENT: Applegate Group, Inc.
Drawn by: SDD	Aprvd by: SDD	Date: NOV 09	

INVENTORY PLANNING PERMITTING COMPLIANCE
ES
Environmental Solutions, Inc.
600 CR 216 Rifle, Colorado 81650
(970) 618-6841 phone ~ (970) 625-1673 fax
email sdahmer@mailcw.com

January 11, 2010

Attn: Craig Ullmann, Applegate Group, Inc.

SUBJ: Threatened, Endangered and Sensitive Species Potential: Otonawanda Res.

Craig,

Per our agreement, the following memo constitutes the proposed work plan for addressing potential Threatened and Endangered species that may occur on the Lake Otonawanda property owned by the Town of Ridgway. Preliminary review of existing records maintained by both the U.S. Fish and Wildlife Service (USFWS) and the Colorado Natural Heritage Program (CNHP) would indicate the Otonawanda Reservoir enlargement may have direct, indirect or cumulative impacts to species that are currently listed as Threatened, Endangered or Candidates for such listing under the Endangered Species Act. Those species are shown in Table 1.

Table 1: Federally-Listed Threatened, Endangered or Candidate Species potentially occurring on, or affected by, the proposed Otonawanda Reservoir enlargement.

Major Group	Scientific Name	Common Name	Listing Status
Bird	*Coccyzus americanus*	Yellow-Billed Cuckoo	Federal Candidate
Insect	*Boloria acrocnema*	Uncompahgre Fritillary Butterfly	Federal Endangered
Fish	*Gila elegans*	Bonytail	Federal Endangered
Fish	*Ptychocheilus lucius*	Colorado Pikeminnow	Federal Endangered
Fish	*Gila cypha*	Humpback Chub	Federal Endangered
Fish	*Xyrauchen texanus*	Razorback Sucker	Federal Endangered
Mammal	*Lynx canadensis*	Canada Lynx	Federal Threatened

In addition to those federally-listed species in Table 1, other plant and animal species are listed as either State Threatened, Endangered or Sensitive or otherwise listed on either the U.S. Forest Service (USFS) or Bureau of Land Management (BLM) sensitive species lists in the region, and may also occur on, or be affected by, the Otonawanda Reservoir enlargement. Those species are designated in Table 2 on the following page.

Most county planning departments in Colorado require an analysis of potential effects to Threatened, Endangered or Sensitive species, regardless of the listing entity. As such, an appropriate report format is fairly standard for these assessments. The proposed report for the Otonawanda Reservoir project would contain the following elements for each of the species listed in the tables:

1. Detailed habitat requirements of each species
2. Research record review and interviews with appropriate authorities to determine known occurrences of any of the species on or near the Otonawanda property.
3. Determination of whether potential habitat exists on the Otonawanda parcel suitable for the listed species, or whether proposed work on the reservoir property could have impacts off the property.

4. Analysis of potential direct, indirect and cumulative impacts of the proposed project on those species for which habitat exists on the site or which are known to inhabit the area.
5. Potential mitigation steps to eliminate or reduce potential impacts of the proposed project as appropriate.

Table 2: State-Listed Threatened, Endangered or Sensitive Species potentially occurring on, or affected by, the proposed Otonawanda Reservoir enlargement.

Major Group	**Scientific Name**	**Common Name**	**Listing Status**
Insect	*Speyeria Nokomis nokomis*	Great Basin Silverspot Butterfly	BLM Sensitive
Plant	*Astragalus wetherillii*	Wetherill's Milkvetch	USFS Sensitive
Plant	*Lesquerella vicina*	Good-Neighbor Bladderpod	BLM Sensitive
Plant	*Cirsium perplexans*	Adobe Thistle	BLM Sensitive
Plant	*Ranunculus karelinii*	Tundra Buttercup	USFS Sensitive
Plant	*Lomatium concinnum*	Colorado Desert-Parsley	BLM Sensitive
Plant	*Cryptogramma stelleri*	Slender Rock-Brake	BLM Sensitive
Plant			
Bird	*Cupseloides niger*	Black Swift	USFS Sensitive
Bird	*Falco peregrinus anatum*	American Peregrine Falcon	USFS Sensitive; State Special Concern
Mammal	*Mustela nigripes*	Black-Footed Ferret	State Endangered
Mammal	*Gulo Gulo*	Wolverine	USFS Sensitive; State Endangered
Fish	*Oncorhunchus clarkii pleuriticus*	Colorado River Cutthroat Trout	UFFS Sensitive; State Special Concern

Based on the initial research and potential list of species contained in the tables above, I estimate an appropriate report can be developed within the original budget I provided you ($9725.00). This work will require at least 2 weeks to prepare, so please allow ample notice should you decide to move forward with this phase of the project.

If you have any questions, please don't hesitate to call.

Sincerely,

Steve D. Dahmer

U.S. ARMY CORPS OF ENGINEERS
COLORADO REGULATORY OFFICES GUIDANCE

MAINTENANCE OF IRRIGATION DITCHES

I. Background. In the past, there has been some disparity between the Corps Districts operating in Colorado on application of the Clean Water Act Section 404 (f)(1) exemption for maintenance of irrigation ditches. Albuquerque District, in its role as the lead District in Colorado, provides the following clarifications for activities related to the maintenance of irrigation ditches that are not prohibited by, or otherwise subject to, regulation under Section 404 of the CWA.

The Code of Federal Regulations at 33 CFR part 323.4(a) (Discharges Not Requiring Permits) states:

> "*... (A)ny discharge of dredged or fill material that may result from any of the following activities is not prohibited by or otherwise subject to regulation under section 404:*
>
> (3) Construction or maintenance of farm or stock ponds or irrigation ditches, or the maintenance (but not construction) of drainage ditches. Discharges associated with siphons, pumps, headgates, wingwalls, weirs, diversion structures, and such other facilities as are appurtenant and functionally related to irrigation ditches are included in this exemption."

II. Colorado Application. To qualify for this exemption, the maintenance to be performed on the jurisdictional ditch, including work on its appurtenant and functionally related facilities, must be for the purpose of delivering water used for agricultural irrigation, not for the delivery of municipal and industrial (M&I) water or water used to irrigate golf courses, lawns, etc. Therefore, if an irrigation ditch conveys both agricultural and M&I water and the agricultural water rights holder needs to maintain the ditch for delivery of their water rights, the work would qualify for this exemption. Conversely, if the M&I water rights holder needs to maintain the ditch for delivery of their water rights, the work would not qualify for this exemption.

In certain instances, irrigation ditches are owned by mutual ditch companies. If the mutual ditch company proposes maintenance of a ditch, the work will be exempt if greater than 50 percent of the water conveyed in the ditch is used, under normal circumstances, for agricultural purposes.

In order for appurtenant facilities, such as diversion structures, jetties, etc., to be exempt, they must be located within close proximity to the irrigation ditch, generally within 200 feet. The Colorado offices will apply RGL 07-02, as well as any other applicable National guidance, to maintenance of irrigation ditches.

III. 323.4(c) "Recapture Provision". Any discharge of dredged or fill material into waters of the United States incidental to activities identified in paragraph (a)(3) must have a permit if it is part of an activity whose purpose is to convert an area of the waters of the United States into a use to which it was not previously subject, where the flow or circulation of waters of the United States may be impaired or the reach of such waters reduced. Where the proposed discharge will result in significant discernible alterations to flow or circulation, the presumption is that flow or circulation may be impaired by such alteration. For example, a permit will be required for the conversion of a cypress swamp to some other use or the conversion of a wetland from silvicultural to agricultural use when there is a discharge of dredged or fill material into waters of the United States in conjunction with construction of dikes, drainage ditches or other works or structures used to effect such conversion. A conversion of a section 404 wetland to a non-wetland is a change in use of an area of waters of the United States. A discharge which elevates the bottom of waters of the United States without converting it to dry land does not thereby reduce the reach of, but may alter the flow or circulation of, waters of the United States.

IV. Colorado Application. To trigger recapture, the discharge in question need only be "incidental to" or "part of" an activity that is intended to or will foreseeably bring about the requisite change in use and impairment or reduction in flow, circulation or reach. In other words, the discharge need not be the sole cause of that result. Thus, in applying section 404(f)(2) [323.4(c)], the regulator should consider the discharge in context, not in isolation. Furthermore, where the change in use converts an area of waters of the United States to uplands, such conversion triggers both parts of the two-part recapture test.

DEPARTMENT OF THE ARMY
U.S. ARMY ENGINEER DISTRICT, SACRAMENTO
CORPS OF ENGINEERS
COLORADO WEST REGULATORY BRANCH
400 ROOD AVENUE, ROOM 134
GRAND JUNCTION, COLORADO 81501-2563

REPLY TO
ATTENTION OF

October 29, 2010

Regulatory Division (SPK-2010-00971)

Ms. Jo Anne Fagen
Town of Ridgway
Post Office Box 10
Ridgway, Colorado 81432

Dear Ms. Fagen:

We are responding to a request, submitted on your behalf by Environmental Solutions, Inc., for an approved jurisdictional determination for Otonawanda Reservoir. The reservoir is located approximately 2.5 miles south of the Town of Ridgway on Millard Mesa, NW ¼ of Section 32, Township 45 North, Range 8 West, Ouray County, Colorado.

Based on available information, we concur that no waters of the United States, as depicted in the Applegate Group map titled *Town of Ridgway, Ridgway Ditch, Reach 3, Figure 4 of* 5 (attached). All water features identified in the submitted delineation, including wetlands, were determined to derive hydrology solely from man-made, agricultural irrigation systems created in uplands. Additionally, none of these waters are tributary, either directly or indirectly, to a Traditional Navigable Waterway. Accordingly, these water features are isolated and are not considered jurisdictional under Section 404 of the Clean Water Act. The Environmental Protection Agency concurred with this finding on October 21, 2010. You should provide a copy of this letter and notice to all other affected parties, including any individual who has an identifiable and substantial legal interest in the property.

Please refer to identification number SPK-2010-00971 in any correspondence concerning this project. If you have any questions, please contact Stephen Moore at Colorado West Regulatory Branch, 400 Rood Avenue, Room 134, Grand Junction, Colorado 81501, email *Stephen.A.Moore@usace.army.mil*, or telephone 970-243-1199 x13. We appreciate your feedback. At your earliest convenience, please tell us how we are doing by completing the customer survey on our website under *Customer Service Survey*.

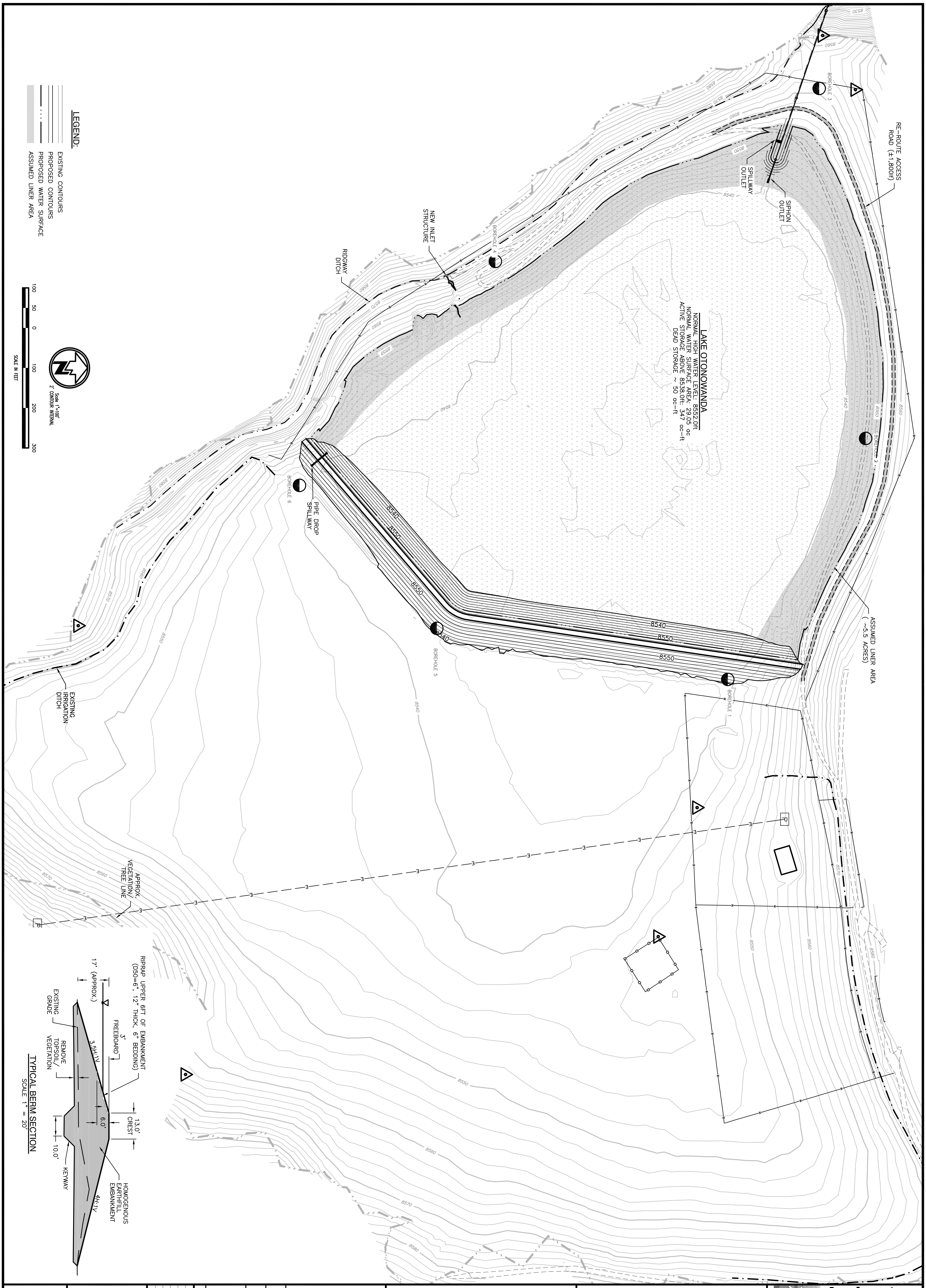
Sincerely,

Susan B. Nall

Susan Bachini Nall
Chief, Colorado West Regulatory Branch

APPENDIX E

REDUCED PRINTS ARE ISSUED FOR CONVENIENCE ONLY –
CONSULT FULLSIZE DRAWINGS IN CASE OF CONFLICTS



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REVISIONS

NO	DATE	BY	CHK'D	DESCRIPTION

TOWN OF RIDGWAY

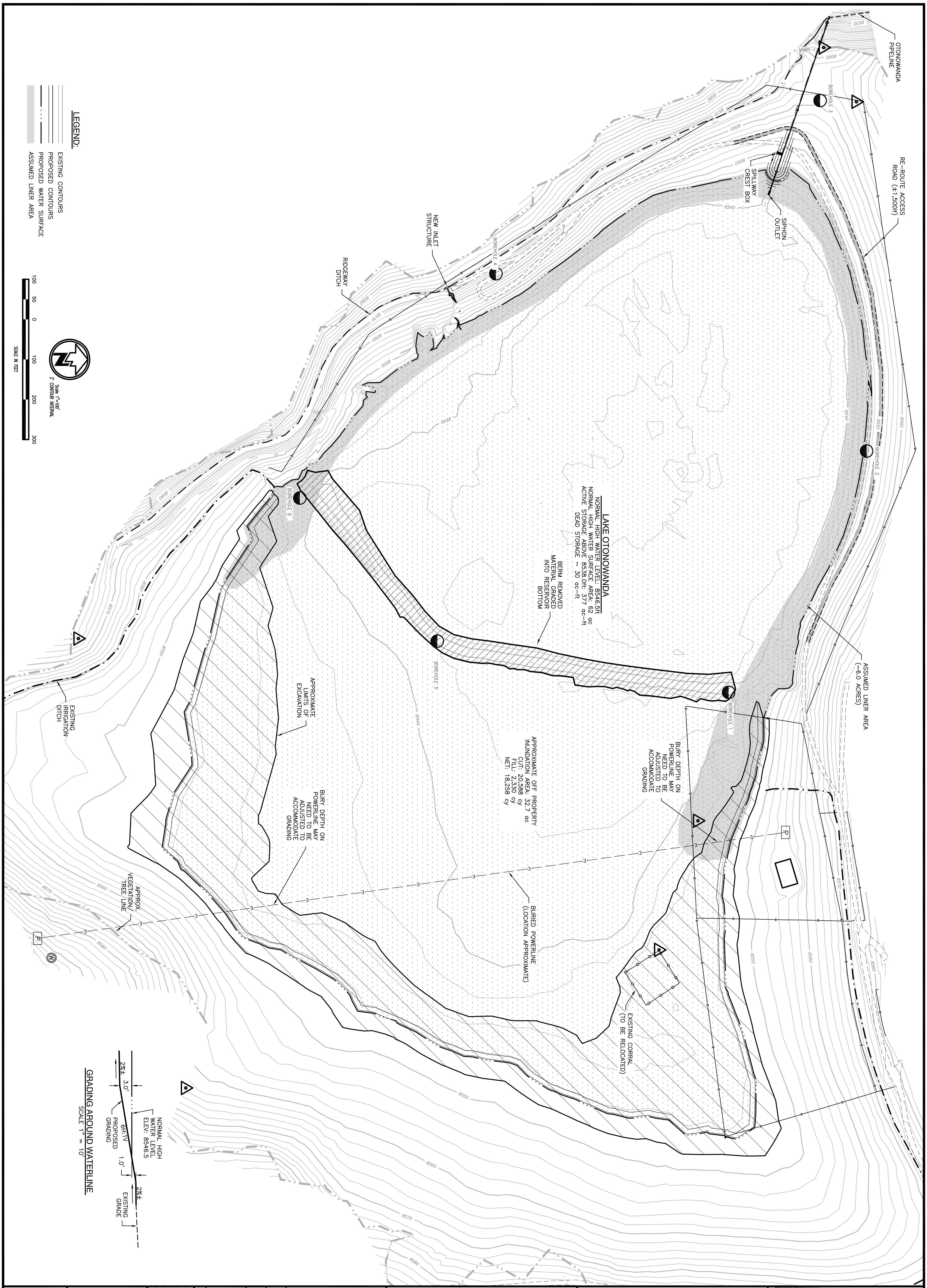
LAKE OTONONWANDA

RESERVOIR OPTION 1
ON PROPERTY

Applegate Group, Inc.
Water Resource Advisors for the West
1499 West 120th Ave., Ste. 200
Denver, CO 80234-2728
(303) 452-6611
Fax: (303) 452-2759
email: info@applegategroup.com Website: www.applegategroup.com

Plot Date: 01/24/11–6:04pm, Plotted by:LineDesign, Drawing Path: N:\09139 Ridgway\Drawings\Figures\Drawing Name:Ridgeway Op 1 Raised to 8554.dwg

REDUCED PRINTS ARE ISSUED FOR CONVENIENCE ONLY –
CONSULT FULLSIZE DRAWINGS IN CASE OF CONFLICTS



NO	DATE	BY	CHK'D	DESCRIPTION

REVISIONS

Date: 01/24/11
Job No: 09-139
Drawn: LD
Design: LG/CMU
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Scale: 1"=100'

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Of:

TOWN OF RIDGWAY

LAKE OTONOWANDA

RESERVOIR OPTION 2
OFF-PROPERTY

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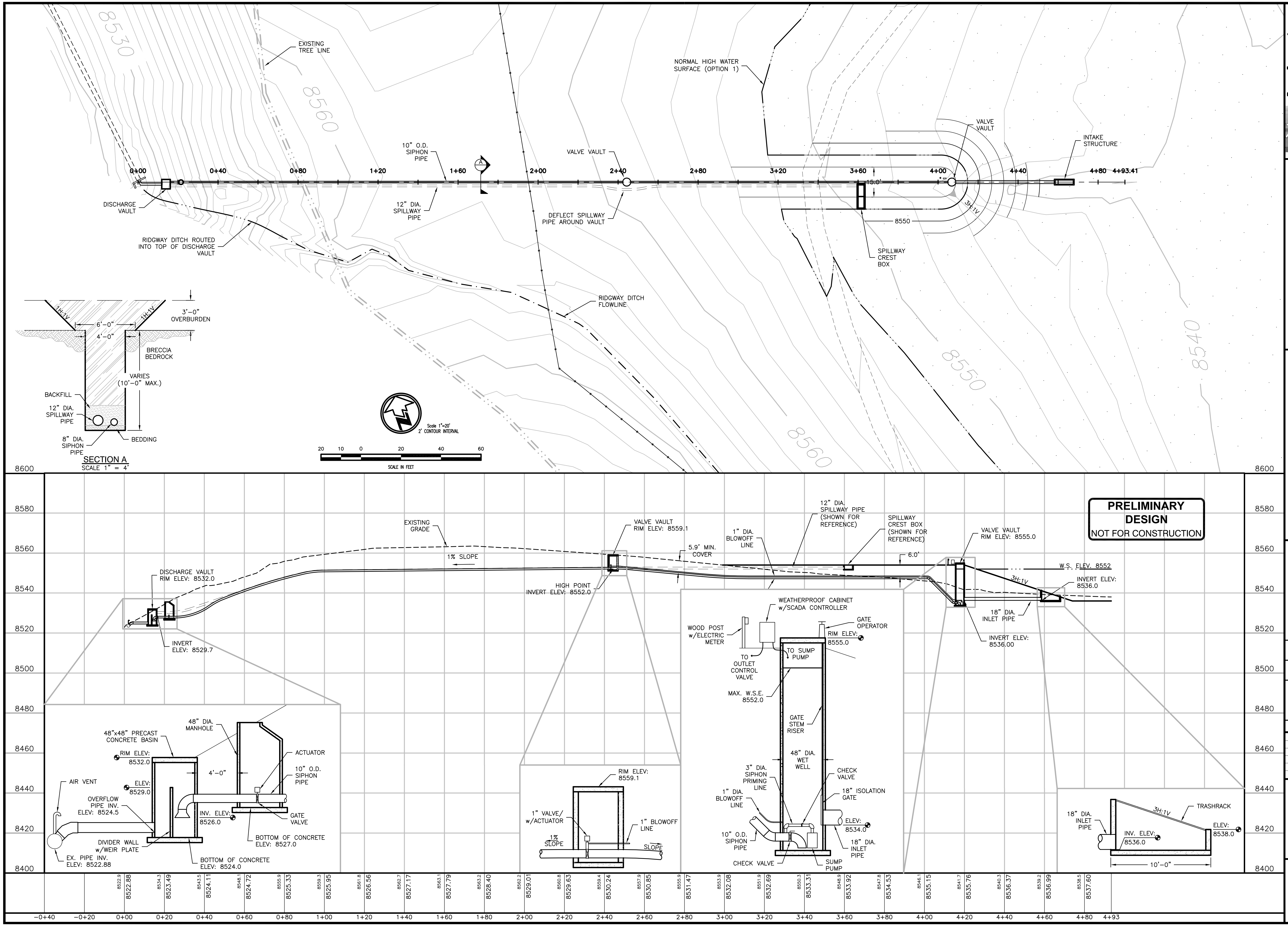
Plot Date: 01/25/11-4:40pm, Plotted by:LineDesign, Drawing Path: N:\09139 Ridgway\Drawings\Figures\Drawing Name:Ridgway Op 2 Berm Removed.dwg

REDUCED PRINTS ARE ISSUED FOR CONVENIENCE ONLY — CONSULT FULLSIZE DRAWINGS IN CASE OF CONFLICTS



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LAKE OTONOWANDA

SIPHON OUTLET

TOWN of RIDGWAY

REVISIONS

NO	DATE	BY	CHK'D	DESCRIPTION

Date: 01/20/11
Job No: 09-139
Drawn: LD
Design: CMU
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Scale: 1"=20' H&V

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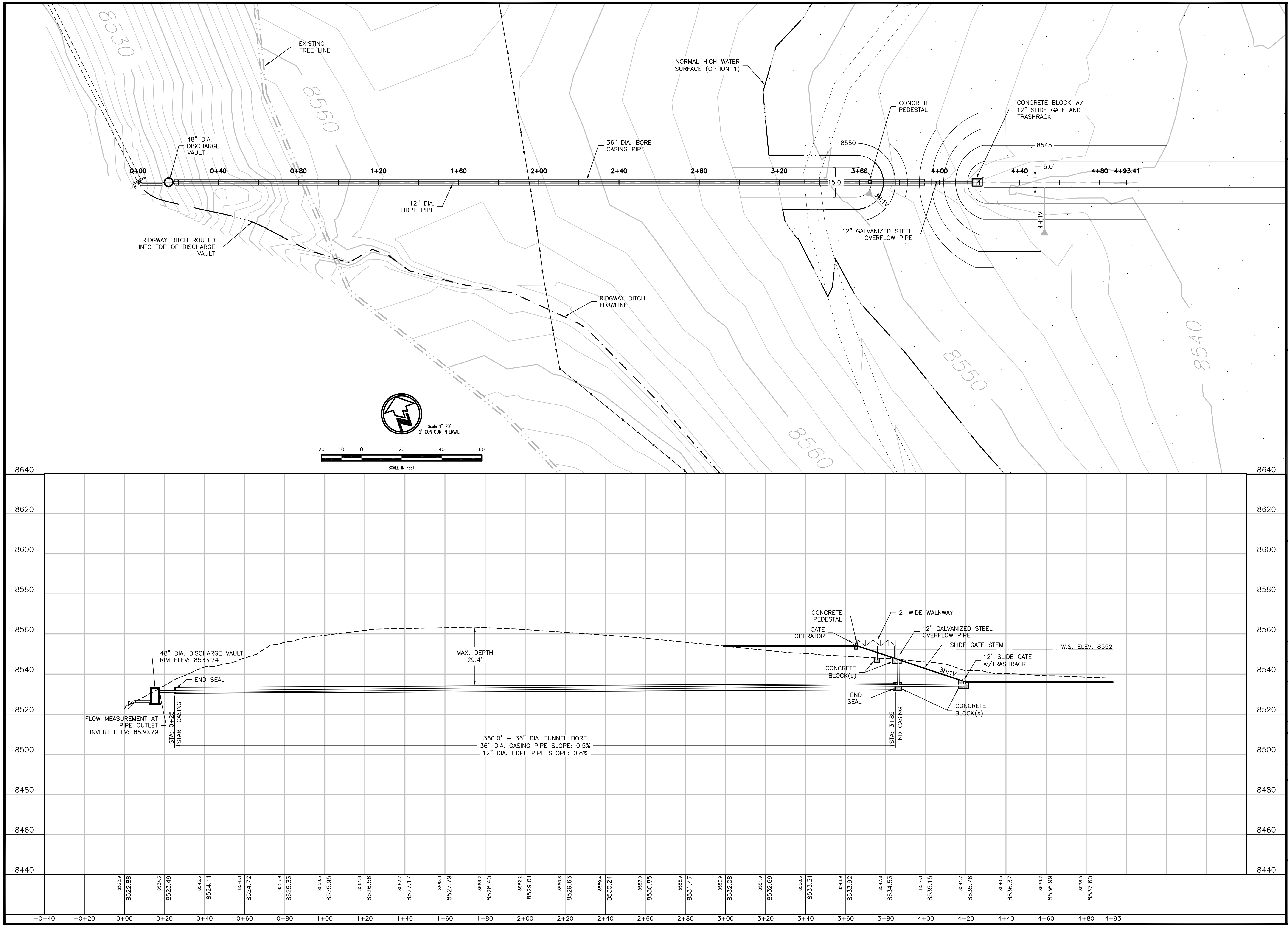

Station	Ground	Pipe
−0+40		
−0+20		
0+00	8522.9	8522.88
0+20	8534.3	8523.49
0+40	8543.5	8524.11
0+60	8548.1	8524.72
0+80	8555.9	8525.33
1+00	8559.3	8525.95
1+20	8561.8	8526.56
1+40	8562.7	8527.17
1+60	8563.1	8527.79
1+80	8563.2	8528.40
2+00	8562.2	8529.01
2+20	8560.8	8529.63
2+40	8559.4	8530.24
2+60	8557.9	8530.85
2+80	8555.9	8531.47
3+00	8553.9	8532.08
3+20	8551.9	8532.69
3+40	8550.3	8533.31
3+60	8548.9	8533.92
3+80	8547.8	8534.53
4+00	8546.1	8535.15
4+20	8541.7	8535.76
4+40	8540.3	8536.37
4+60	8539.2	8536.99
4+80	8538.5	8537.60
4+93		

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LAKE OTONOWANDA

TUNNEL OUTLET

TOWN of RIDGWAY

REVISIONS

NO	DATE	BY	CHK'D	DESCRIPTION

Date: 01/25/11
Job No: 09–139
Drawn: LD
Design: CMU
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Scale: 1"=20' H&V

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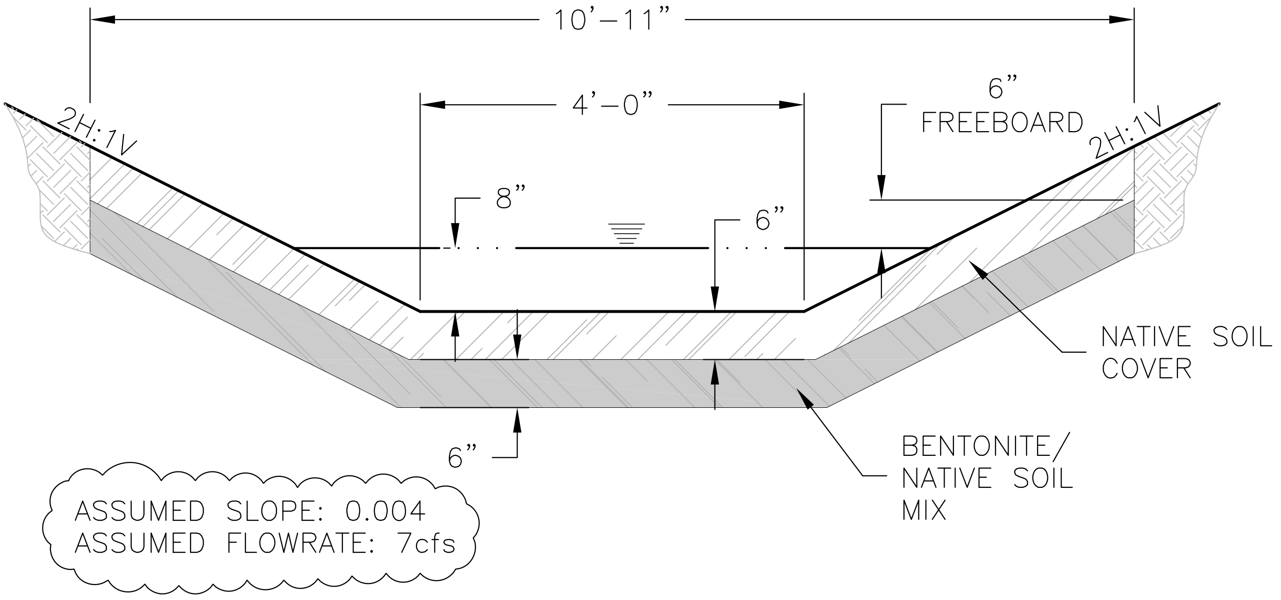

BENTONITE LINER DETAIL

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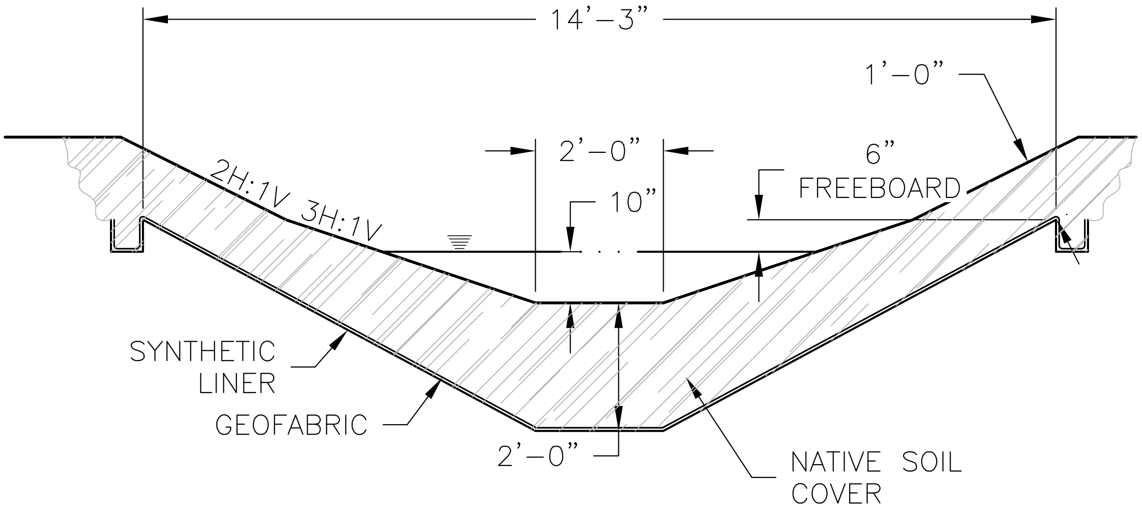

SYNTHETIC LINER DETAIL

SCALE 1" = 3'

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Job No:	09–139
Drawn:	LD
Design:	CMU
Checked:	
File:	Liner
Scale:	As Noted

TOWN of RIDGWAY

LAKE OTONWANDA

LINER DETAILS

Applegate Group, Inc.

Water Resource Advisors for the West

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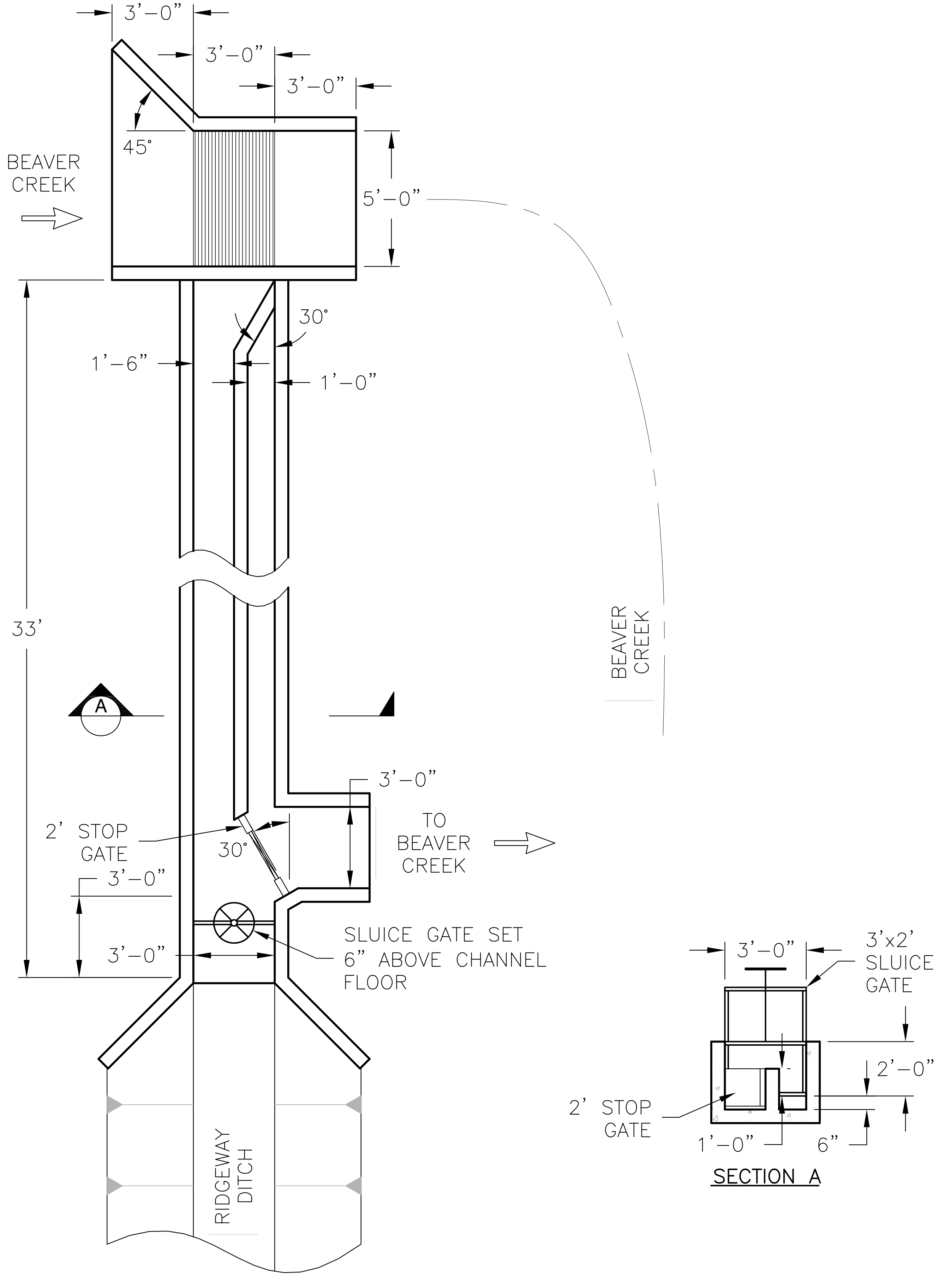

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Job No:	09–139
Drawn:	LD
Design:	CMU
Checked:	
File:	
Scale:	Not to Scale

TOWN of RIDGWAY

LAKE OTONWANDA

DITCH HEADGATE STRUCTURE

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Water Resource Advisors for the West
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Denver, CO 80234-2728
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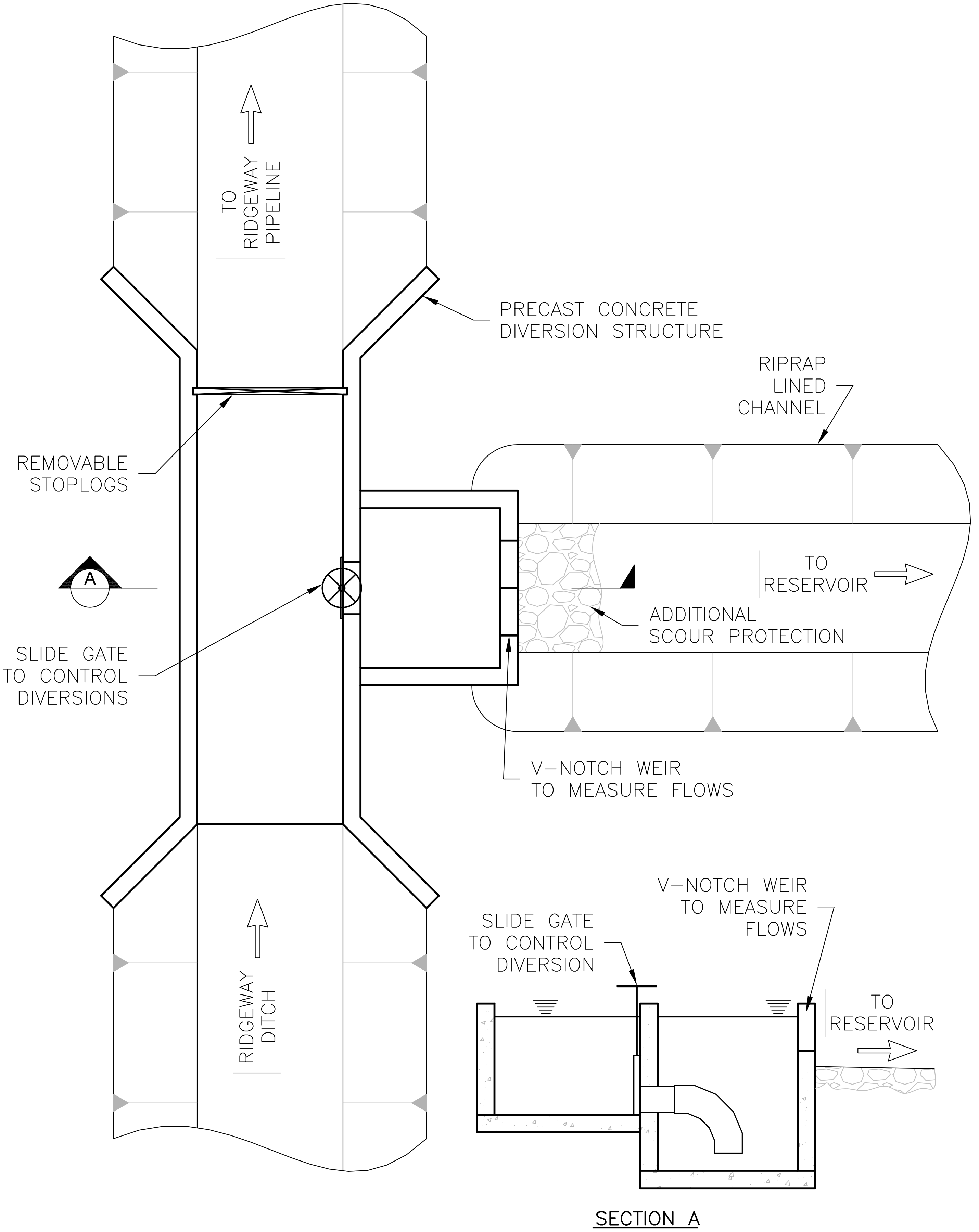
email: info@applegategroup.com Website: ApplegateGroup.com



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Date:	07/06/10
Job No:	09–139
Drawn:	LD
Design:	LG
Checked:	
File:	
Scale:	Not to Scale

TOWN of RIDGWAY

LAKE OTONWANDA

RESERVOIR INTAKE STRUCTURE

Applegate Group, Inc.
Water Resource Advisors for the West
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APPENDIX F

Engineers Opinion of Probable Construction Cost

Applegate Group, Inc.

1499 W. 120th Ave. Suite 200
Denver, CO 80234

Phone: (303) 452-6611
Fax: (303) 452-2759

Lake Otonowanda and Ridgway Ditch Feasibility Study
Reservoir Cost Estimate

Job No. : 09-139
By: LAG/CMU
Date: 1/24/2011

Client: Town of Ridgway

Description of Work	Item	Units	Quantity	Unit Cost	Total Cost
Reservoir Option 1 - On Property, With Dam	Remove Old Embankment	CY	6,783	$ 5.00	$ 33,915
	Bottom Grading	CY	1,000	$ 5.00	$ 5,000
	Foundation Preparation	SY	5,000	$ 2.50	$ 12,500
	Embankment Placement	CY	67,903	$ 7.50	$ 509,273
	Dam Embankment Riprap, upper 6 ft only	CY	1,050	$ 70	$ 73,500
	Solar Pump System for Irrigation Tailwater (Optional)	LS	1	$ 25,000	
	New Access Road	LF	1,800	$ 5.00	$ 9,000
	Siphon Outlet Works, Spillway	LS	1	$ 204,430	$ 204,430
	Reservoir Liner	ACRE	5	$ 20,000	$ 100,000
	Reservoir Inlet Structure	LS	1	$ 15,000	$ 15,000
	Seeding	ACRE	2	$ 2,500	$ 5,000
	Construction Subtotal				**$ 967,618**
	Mobilization	%		8%	$ 77,409
	Construction Contengency	%		20%	$ 193,524
	Construction Survey, Testing, Construction Observation	%		10%	$ 96,762
	Construction Total				**$ 1,335,312**
	Final Engineering Design, Geotechnical Investigation	%		10%	$ 133,531
				Project Total	**$ 1,468,843**
				Storage (ac-ft)	**347**
				Cost/Ac-ft	**$ 4,232.98**

Reservoir Option 2 - Off Property, Without Dam	Remove Old Embankment	CY	6,783	$ 5.00	$ 33,915
	Grading Work Around Waters Edge	CY	20,588	$ 5.00	$ 102,940
	Disposal of Excess Fill	CY	18,258	$ 15.00	$ 273,870
	Siphon Outlet Works, Spillway	LS	1	$ 204,430	$ 204,430
	Reservoir Liner	ACRE	6	$ 20,000	$ 120,000
	Agreement with Landowner	LS	1		$ -
	Seeding	ACRE	4	$ 2,500	$ 10,000
	Construction Subtotal				**$ 745,155**
	Mobilization	%		8%	$ 59,612
	Construction Contengency	%		20%	$ 149,031
	Construction Survey, Testing, Construction Observation	%		8%	$ 59,612
	Construction Total				**$ 1,013,411**
	Final Engineering Design, Geotechnical Investigation	%		10%	$ 101,341
				Project Total	**$ 1,114,752**
				Storage (ac-ft)	**377**
				Cost/Ac-ft	**$ 2,956.90**

Engineers Opinion of Probable Construction Cost

Applegate Group, Inc.

1499 W. 120th Ave. Suite 200
Denver, CO 80234

Phone: (303) 452-6611
Fax: (303) 452-2759

Lake Otonowanda and Ridgway Ditch Feasibility Study
Outlet/Spillway Cost Estimate

Job No. : 09-139
By: LAG/CMU
Date: 1/24/2011

Client: Town of Ridgway

Description of Work	Item	Units	Quantity	Unit Cost	Total Cost
Siphon Outlet	10" HDPE Siphon Pipe Including Backfill	LF	460	$ 56.0	$ 25,760
	1" Blowoff Line w/valve for air vent and fittings	LS	1	$ 1,000.0	$ 1,000
	Soil Excavation (top 3 ft, 4 ft wide)	CY	350	$ 15	$ 5,250
	Rock Excavation (Below 3 ft, 4 ft wide)	CY	230	$ 90	$ 20,700
	Inlet Structure - Trashrack	LS	1	$ 8,000	$ 8,000
	Valve Vault - 48" Manhole(21'), Isolation Valve	LS	1	$ 13,500	$ 13,500
	Sump Pump and Fittings	LS	1	$ 2,500	$ 2,500
	Flow Meter Vault - 48" Manhole(7'), Isolation Valve, Meter	LS	1	$ 12,500	$ 12,500
	Discharge Vault - 48" Manhole (7')	LS	1	$ 6,000	$ 6,000
	Control Valve Vault, Valve, and actuator	LS	1	$ 12,500	$ 12,500
	Compacted Fill for Penninsula	CY	2000	$ 12	$ 24,000
	Spillway Pipe, 12" PVC, Purchase and Assemble Only	LF	340	$ 8	$ 2,720
	Spillway Inlet Structure	LS	1	$ 7,000	$ 7,000
	Sealing Tunnel	LS	1	$ 10,000	$ 10,000
	SCADA Communication System	LS	1	$ 20,000	$ 20,000
	Power Line Extension	LF	2200	$ 15	$ 33,000
	Construction Subtotal				**$ 204,430**
	Mobilization	%		10%	$ 20,443
	Construction Contengency	%		30%	$ 61,329
	Construction Total				**$ 286,202**
	Final Engineering Design/Geotech Testing	%		10%	$ 28,620
	Total				**$ 314,822**
New Tunnel Outlet	36" Tunnel w/grouted steel casing, skids, bore pits	LF	360	$ 600	$ 216,000
	Outlet Pipe, 12" HDPE, purchase and fuse	LF	420	$ 10	$ 4,200
	Pipe Backfill - outside of boring	LF	60	$ 50	$ 3,000
	Inlet Structure - Concrete and Trashrack	LS	1	$ 18,000	$ 18,000
	Slide Gate, Pedestal, Manual Operator	LS	1	$ 8,000	$ 8,000
	Overflow Spillway	LS	1	$ 3,000	$ 3,000
	Discharge Vault with Flowmeter	LS	1	$ 8,500	$ 8,500
	Walkway	LS	1	$ 7,500	$ 7,500
	Sealing Old Tunnel	LS	1	$ 10,000	$ 10,000
	Control System	LS	1	$ 15,000	$ 15,000
	Construction Subtotal				**$ 293,200**
	Mobilization	%		10%	$ 20,443
	Construction Contengency	%		30%	$ 61,329
	Construction Total				**$ 374,972**
	Final Engineering Design/Geotech Testing	%		10%	$ 37,497
	Total				**$ 412,469**

Engineers Opinion of Probable Construction Cost

Applegate Group, Inc.

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Denver, CO 80234

Phone: (303) 452-6611
Fax: (303) 452-2759

Lake Otonowanda and Ridgway Ditch Feasibility Study
Ridgway Ditch Headgate Cost Estimate

Job No. : 09-139
By: CMU/LAG
Date: 1/24/2011

Client: Town of Ridgway

Description of Work	Item	Units	Quantity	Unit Cost	Total Cost
Modifications	Bar Screen	LS	1	$ 4,000	$ 4,000
	Slide Gate	LS	1	$ 3,500	$ 3,500
	Stop Gate	LS	1	$ 700	$ 700
	Gate Installation	LS	1	$ 1,500	$ 1,500
	Construction Subtotal				**$ 9,700**
	Mobilization	%		8%	$ 776
	Construction Contengency	%		20%	$ 1,940
	Construction Total				**$ 12,416**
	Engineering Design, Bar Screen	LS			$ 2,500
				Project Total	**$ 14,916**

Replacement with Concrete	Demolition of Wood Structure	LS	1	$ 1,500	$ 1,500
	Dewatering/Stream Management (during fall 1-2 cfs in B.C.)	LS	1	$ 10,000	$ 10,000
	Grading/Foundation Prep	LS	1	$ 3,000	$ 3,000
	Cast-In-Place Concrete (4 pours)	CY	17	$ 1,900	$ 32,300
	Structure Backfill/Compaction	CY	20	$ 75	$ 1,500
	Bar Screen	LS	1	$ 4,000	$ 4,000
	Slide Gate	LS	1	$ 4,500	$ 4,500
	Stop Gate	LS	1	$ 700	$ 700
	Gate Installation	LS	1	$ 1,500	$ 1,500
	Seeding	AC	0.25	$ 2,500	$ 625
	Construction Subtotal				**$ 59,625**
	Mobilization	%		8%	$ 4,770
	Construction Contengency	%		10%	$ 5,963
	Construction Total				**$ 70,358**
	Survey	%		7%	$ 4,925
	Engineering Design and Construction Observation	%		25%	$ 17,589
				Project Total	**$ 87,947**

Replacement with Wood	Demolition of Wood Structure	LS	1	$ 1,500	$ 1,500
	Dewatering/Stream Management (during fall 1-2 cfs in B.C.)	LS	1	$ 10,000	$ 10,000
	Grading/Foundation Prep	LS	1	$ 3,000	$ 3,000
	Replacement of wood elements	LS	1	$ 12,000	$ 12,000
	Small Precast Concrete Structure for Gates	LS	1	$ 7,000	$ 7,000
	Structure Backfill/Compaction	CY	20	$ 75	$ 1,500
	Bar Screen	LS	1	$ 4,000	$ 4,000
	Slide Gate	LS	1	$ 4,500	$ 4,500
	Stop Gate	LS	1	$ 700	$ 700
	Gate Installation	LS	1	$ 1,500	$ 1,500
	Seeding	AC	0.25	$ 2,500	$ 625
	Construction Subtotal				**$ 46,325**
	Mobilization	%		8%	$ 3,706
	Construction Contengency	%		10%	$ 4,633
	Construction Total				**$ 54,664**
	Survey	%		7%	$ 3,826
	Engineering Design and Construction Observation	%		25%	$ 13,666
				Project Total	**$ 68,329**

Engineers Opinion of Probable Construction Cost

Applegate Group, Inc.

1499 W. 120th Ave. Suite 200
Denver, CO 80234

Phone: (303) 452-6611
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Lake Otonowanda and Ridgway Ditch
Feasibility Study
Piping Ridgway Ditch

Job No. : 09-139
By: LAG/CMU
Date: 1/24/2011

Client: Town of Ridgway

Description of Work	Item	Units	Quantity	Unit Cost	Total Cost
Piping the Ridgway Ditch - Moderately High to High Permeability Soils	15" PIP PVC Materials	LF	3,300	$ 8	$ 26,400
	Pipe Assembly	LF	3,300	$ 4	$ 13,200
	Trench Excavation (33" Deep X 30" Wide)	CY	840	$ 12	$ 10,080
	Imported Backfill	CY	200	$ 115	$ 23,000
	Native Backfill	CY	500	$ 20	$ 10,000
	Spring Box Inlets	LS	2	$ 5,000	$ 10,000
	Construction Subtotal				**$ 92,680**
	Mobilization	%		10%	$ 9,268
	Construction Contengency	%		10%	$ 9,268
	Construction Total				**$ 111,216**
	Survey	%		5%	$ 5,561
	Final Engineering Design	%		12%	$ 13,346
	Total				**$ 130,123**
Piping the Ridgway Ditch - Moderately Slow to Slow Permeability Soils	15" PIP PVC Materials	LF	4,200	$ 8	$ 33,600
	Pipe Install	LF	4,200	$ 4	$ 16,800
	Trench Excavation (33" Deep X 30" Wide)	CY	1,070	$ 12	$ 12,840
	Imported Backfill	CY	252	$ 115	$ 28,980
	Native Backfill	CY	630	$ 20	$ 12,600
	Construction Subtotal				**$ 104,820**
	Mobilization	%		10%	$ 10,482
	Construction Contengency	%		10%	$ 10,482
	Construction Total				**$ 125,784**
	Survey	%		5%	$ 6,289
	Final Engineering Design	%		12%	$ 15,094
	Total				**$ 147,167**
Piping the Ridgway Ditch - Slow Permeability Soils	15" PIP PVC Materials	LF	16,650	$ 8	$ 133,200
	18" PIP PVC Materials	LF	2,050	$ 12	$ 24,600
	Pipe Installation	LF	18,700	$ 4	$ 74,800
	Trench Excavation (33" Deep X 30" Wide)	CY	4,760	$ 12	$ 57,120
	Imported Backfill	CY	1,122	$ 115	$ 129,030
	Native Backfill	CY	2,805	$ 20	$ 56,100
	Irrigation Diversions	LS	2	$ 5,000	$ 10,000
	Construction Subtotal				**$ 484,850**
	Mobilization	%		10%	$ 48,485
	Construction Contengency	%		10%	$ 48,485
	Construction Total				**$ 581,820**
	Survey	%		5%	$ 29,091
	Final Engineering Design	%		12%	$ 69,818
	Total				**$ 680,729**

Engineers Opinion of Probable Construction Cost

Applegate Group, Inc.

1499 W. 120th Ave. Suite 200
Denver, CO 80234

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Lake Otonowanda and Ridgway Ditch Feasibility Study
Bentonite Ditch Lining

Job No. : 09-139
By: LAG/CMU
Date: 1/24/2011

Client: Town of Ridgway

Description of Work	Item	Units	Quantity	Unit Cost	Total Cost
Bentonite Lining - Moderately High to High Permeability Soils	Bentonite (46 lb/ft, 0.625 lb/sq ft))	LF	3,300	$ 7.3	$ 24,090
	Spreading and Mixing	LF	3,300	$ 3.3	$ 10,890
	Excavation and Stockpile	LF	3,300	$ 2.3	$ 7,590
	Backfill and Compaction	LF	3,300	$ 3.5	$ 11,550
	Construction Subtotal				**$ 54,120**
	Mobilization	%		10%	$ 5,412
	Construction Contengency	%		10%	$ 5,412
	Construction Total				**$ 64,944**
	Survey	%		5%	$ 3,247
	Final Engineering Design/Geotech Testing	%		12%	$ 7,793
	Total				**$ 75,984**
Bentonite Lining - Moderately Slow to Slow Permeability Soils	Bentonite (46 lb/ft, 0.625 lb/sq ft))	LF	4,200	$ 5.8	$ 24,360
	Spreading and Mixing	LF	4,200	$ 3.3	$ 13,860
	Excavation and Stockpile	LF	4,200	$ 2.3	$ 9,660
	Backfill and Compaction	LF	4,200	$ 3.5	$ 14,700
	Construction Subtotal				**$ 62,580**
	Mobilization	%		10%	$ 6,258
	Construction Contengency	%		10%	$ 6,258
	Construction Total				**$ 75,096**
	Survey	%		5%	$ 3,755
	Final Engineering Design	%		12%	$ 9,012
	Total				**$ 87,862**

Engineers Opinion of Probable Construction Cost

Applegate Group, Inc.

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Lake Otonowanda and Ridgway Ditch Feasibility Study
Synthetic Ditch Lining

Job No. : 09-139
By: LAG/CMU
Date: 1/24/2011
Client: Town of Ridgway

Description of Work	Item	Units	Quantity	Unit Cost	Total Cost
Bentonite Lining - Moderately High to High Permeability Soils	Synthetic Liner (18' wide, 45 mil RPP) and Install	LF	3,300	$ 12.15	$ 40,095
	Geofabric (18' wide 8 oz Nonwoven) and Install	LF	3,300	$ 2.70	$ 8,910
	Excavation and Stockpile (0.75 CY/ft)	LF	3,300	$ 10.0	$ 33,000
	Backfill and Compaction (0.75 CY/ft)	LF	3,300	$ 15.0	$ 49,500
	Construction Subtotal				**$ 131,505**
	Mobilization	%		10%	$ 13,151
	Construction Contengency	%		10%	$ 13,151
	Construction Total				**$ 157,806**
	Survey	%		5%	$ 7,890
	Final Engineering Design/Geotech Testing	%		12%	$ 18,937
	Total				**$ 184,633**
Bentonite Lining - Moderately Slow to Slow Permeability Soils	Synthetic Liner (18' wide, 45 mil RPP) and Install	LF	4,200	$ 12.15	$ 51,030
	Geofabric (18' wide 8 oz Nonwoven) and Install	LF	4,200	$ 2.70	$ 11,340
	Excavation and Stockpile (0.75 CY/ft)	LF	4,200	$ 10.0	$ 42,000
	Backfill and Compaction (0.75 CY/ft)	LF	4,200	$ 15.0	$ 63,000
	Construction Subtotal				**$ 167,370**
	Mobilization	%		10%	$ 16,737
	Construction Contengency	%		10%	$ 16,737
	Construction Total				**$ 200,844**
	Survey	%		5%	$ 10,042
	Final Engineering Design	%		12%	$ 24,101
	Total				**$ 234,987**

APPENDIX G

Client: Town of Ridgway
Job No: 09-139
Project:
By: LAG Date: 7/12/10
Description: Lake Otonowanda PMP
Chk: CMU Date: 12/21/10
Page: of:

HMR 49 General Storm PMP (See Table 6.1) = 10.3 in (24-hr)

Option 1 on property

Drainage Area = 63 acres

Reservoir Area = 30 acres

Using Table 5.2 SEO Guidelines

Small, Significant hazard = 0.45 PMP

IDF = 4.7 inches

Reduction for elevation:
Using Table 5.3

Small Dam, General Storm West, above 8,000ft. High hazard

= 0.35 PMP

IDF = 0.35 (10.3 in) = 3.6 inches.

Assuming 100% Runoff, Results in 7.6 inches in Reservoir.

Local Storm = 9.4 inches (6 hour Storm)

Small, significant hazard = 0.45 PMP , IDF = 4.23 inches

Assuming 100% Runoff, Results in 8.9 inches in Reservoir

Table 6.1.--General-storm PMP computations for the Colorado River and Great basin

Drainage Lake Otonowanda   Area 63 acres mi^2 (km^2)

Latitude ________, Longitude ____ of basin center

Month September

Step   Duration (hrs)
6 12 18 24 48 72

A. Convergence PMP

1. Drainage average value from one of figures 2.5 to 2.16 13.7 in. (mm)
2. Reduction for barrier-elevation [fig. 2.18] 31 %
3. Barrier-elevation reduced PMP [step 1 X step 2] 4.3 in. (mm)
4. Durational variation [figs. 2.25 to 2.27 and table 2.7]. ___ ___ ___ 100% ___ ___ %
5. Convergence PMP for indicated durations [steps 3 X 4] ___ ___ ___ 4.3 ___ ___ in. (mm)
6. Incremental 10 mi^2 (26 km^2) PMP [successive subtraction in step 5] ___ ___ ___ 0 ___ ___ in. (mm)
7. Areal reduction [select from figs. 2.28 and 2.29] ___ ___ ___ 0 ___ ___ %
8. Areally reduced PMP [step 6 X step 7] ___ ___ ___ 4.3 ___ ___ in. (mm)
9. Drainage average PMP [accumulated values of step 8] ___ ___ ___ ___ ___ ___ in. (mm)

B. Orographic PMP

1. Drainage average orographic index from figure 3.11a to d. 6 in. (mm)
2. Areal reduction [figure 3.20] 100 %
3. Adjustment for month [one of figs. 3.12 to 3.17] 100 %
4. Areally and seasonally adjusted PMP [steps 1 X 2 X 3] 6 in. (mm)
5. Durational variation [table 3.6] ___ ___ ___ 100 ___ ___ %
6. Orographic PMP for given durations [steps 4 X 5] ___ ___ ___ 6 ___ ___ in. (mm)

C. Total PMP

1. Add steps A9 and B6 ___ ___ ___ 10.3 ___ ___ in. (mm)
2. PMP for other durations from smooth curve fitted to plot of computed data.
3. Comparison with local-storm PMP (see sec. 6.3).

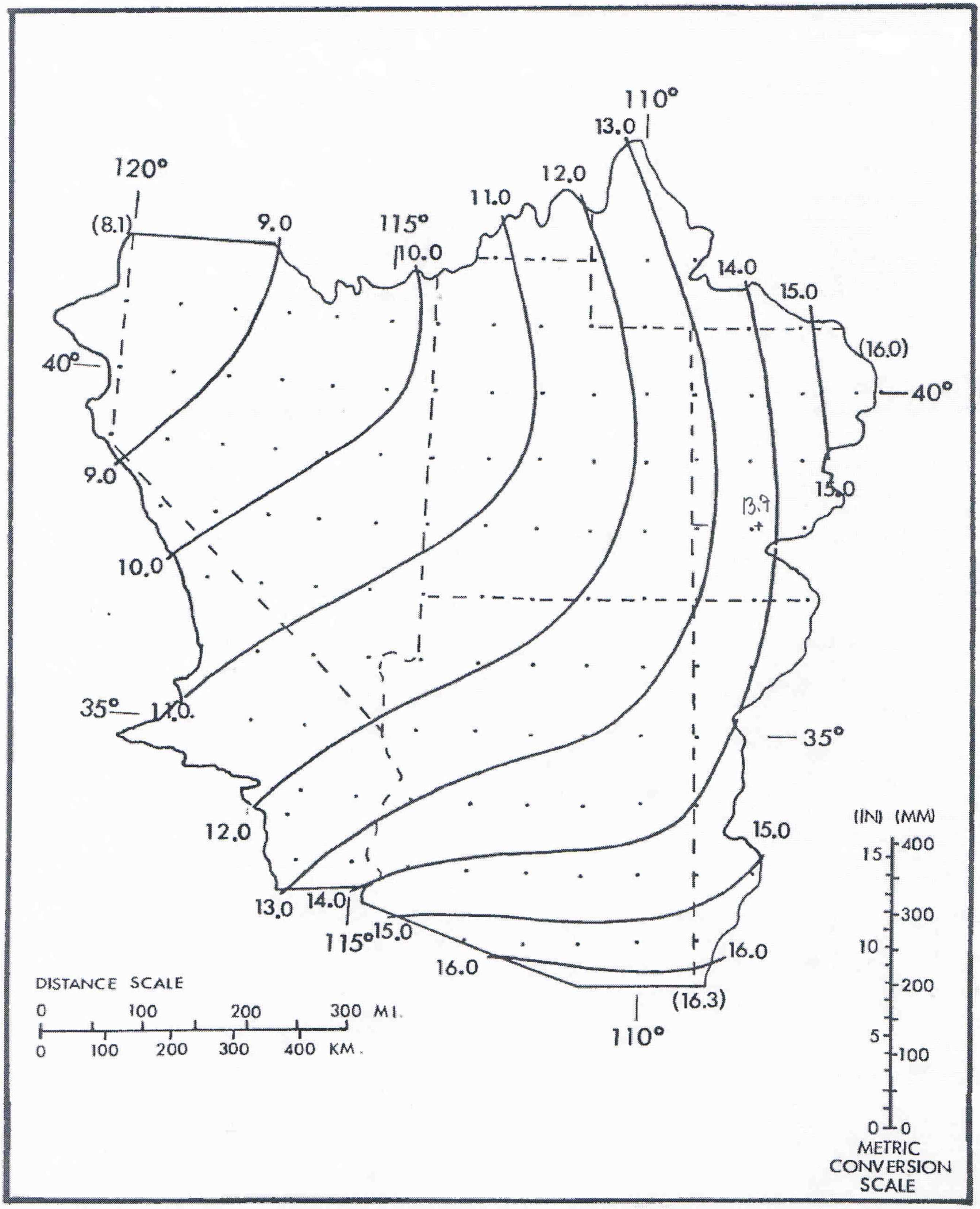


Figure 2.13.--1000-mb (100-kPa) 24-hr convergence PMP (inches) for 10 mi^2 (26 km^2) for September. Values in parentheses are limiting values and are to facilitate extrapolation beyond the indicated gradient.

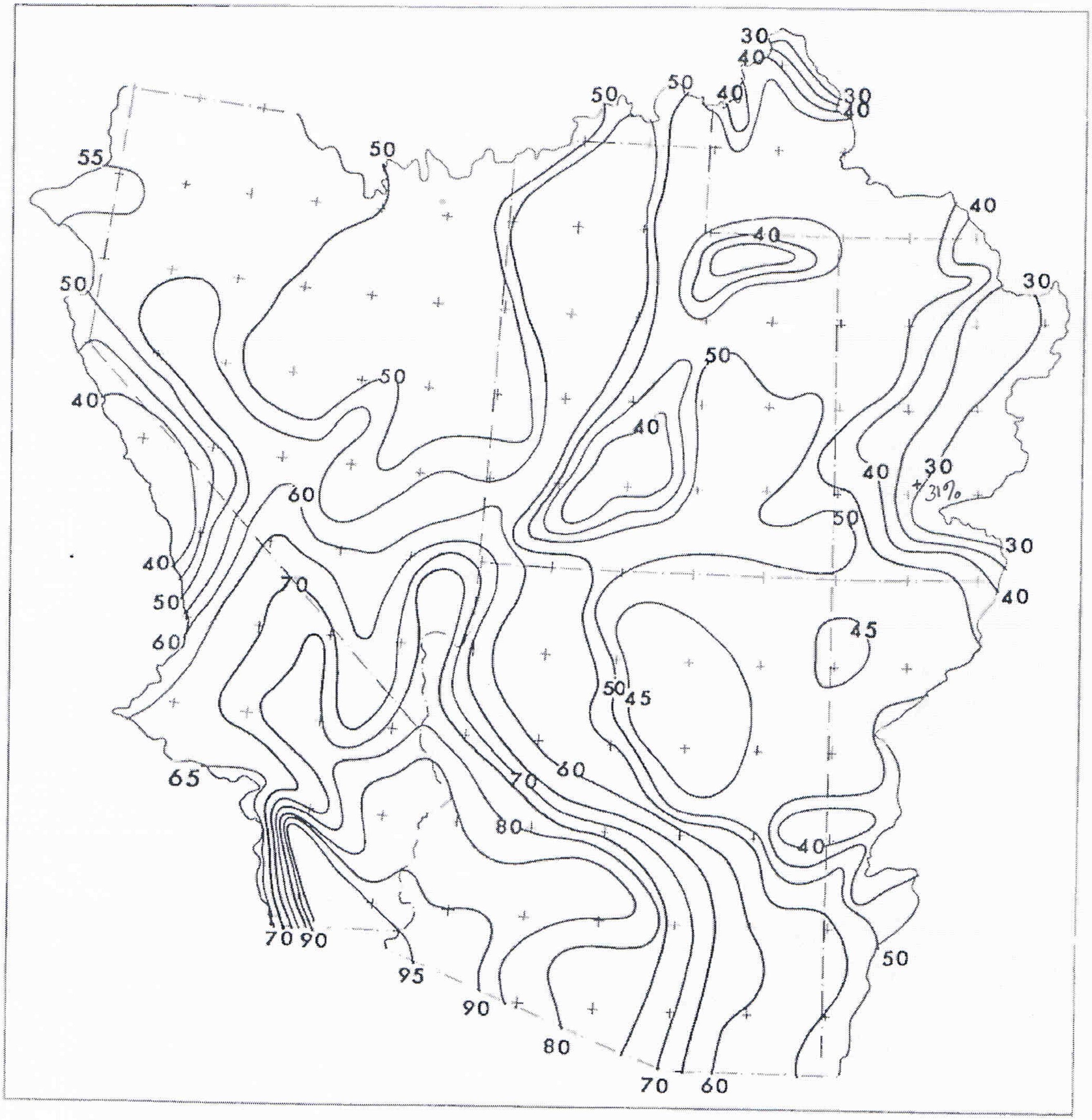


Figure 2.18.--Percent of 1000-mb (100-kPa) convergence PMP resulting from effective elevation and barrier considerations. Isolines drawn for every five percent.

Table 6.3A.--Local-storm PMP computation, Colorado River, Great Basin and California drainages. For drainage average depth PMP. Go to table 6.3B if areal variation is required.

Drainage Lake Otonowanda Area 63 acres mi^2 (km^2)
Latitude ______ Longitude ______ Minimum Elevation 8500 ft (m)

Steps correspond to those in sec. 6.3A.

1. Average 1-hr 1-mi^2 (2.6-km^2) PMP for drainage [fig. 4.5]. 8.7 in. (mm)

2. a. Reduction for elevation. [No adjustment for elevations up to 5,000 feet (1,524 m): 5% decrease per 1,000 feet (305 m) above 5,000 feet (1,524 m)]. − 17.5% %

   b. Multiply step 1 by step 2a. 7.2 in. (mm)

3. Average 6/1-hr ratio for drainage [fig. 4.7]. 1.30

	Duration (hr)									
	1/4	1/2	3/4	1	2	3	4	5	6	
4. Durational variation for 6/1-hr ratio of step 3 [table 4.4].	74	89	95	100	114	121	125	128	130	%
5. 1-mi^2 (2.6-km^2) PMP for indicated durations [step 2b X step 4].	5.3	6.4	6.8	7.2	8.2	8.7	9	9.2	9.4	in. (mm)
6. Areal reduction [fig. 4.9].	← 100% →									%
7. Areal reduced PMP [steps 5 X 6].										in. (mm)

8. Incremental PMP [successive subtraction in step 7]. ___ ___ ___ ___ ___ ___ in. (mm)

   ___ ___ ___ ___ } 15-min. increments

9. Time sequence of incremental PMP according to:

   Hourly increments [table 4.7]. ___ ___ ___ ___ ___ ___ in. (mm)

   Four largest 15-min. increments [table 4.8]. ___ ___ ___ ___ in. (mm)

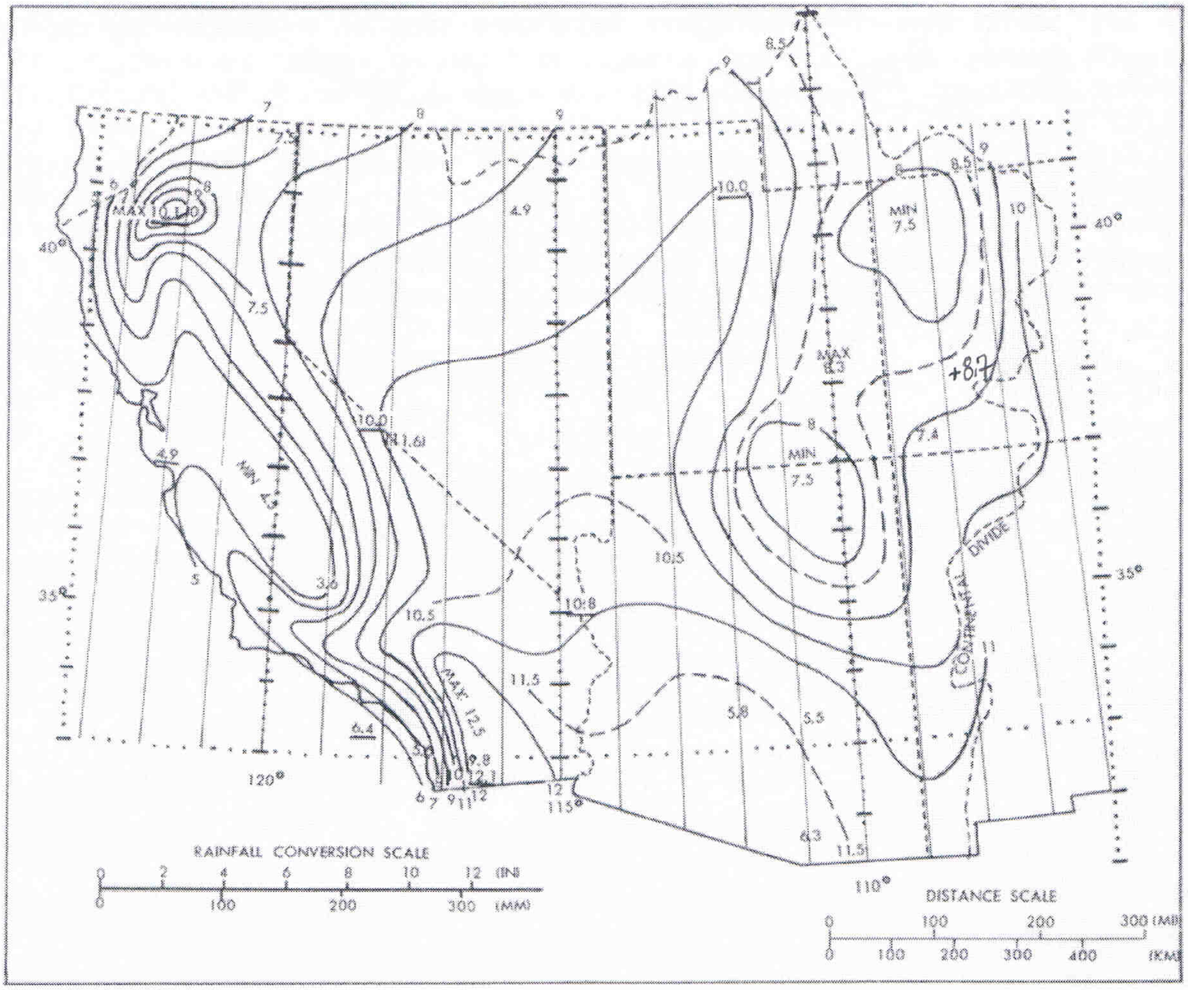


Figure 4.5--Local-storm PMP for 1 mi^2 (2.6 km^2) 1 hr. Directly applicable for locations between sea level and 5000 ft (1524 m). Elevation adjustment must be applied for locations above 5000 ft.

events. In contrast to figure 4.4, figure 4.5 maintains a maximum between these two locations. There is no known meteorological basis for a different solution. The analysis suggests that in the northern portion of the region maximum PMP occurs between the Sierra Nevada on the west and the Wasatch range on the east.

A discrete maximum (> 10 inches, 254 mm) occurs at the north end of the Sacramento Valley in northern California because the northward-flowing moist air is increasingly channeled and forced upslope. Support for this PMP center comes from the Newton, Kennett, and Red Bluff storms (fig. 4.1). Although the analysis in this region appears to be an extension of the broad maximum through the center of the Southwestern Region, it does not indicate the direction of moist inflow. The pattern has evolved primarily as a result of attempts to tie plotted maxima into a reasonable picture while considering inflow directions, terrain effects, and moisture potential.

Lake Otonowonda
Siphon Outlet Calculations

Reservoir WSE	8538	ft	Input
Pipe Diam., I.D.	8	in	
Flowrate	2.25	cfs	

Velocity	6.4	ft/s	Calculations
Velocity Head	0.65	ft	
Hazen Williams, C	140		
Atmospheric Pressure	24.7	ft	
Factor of Safety	1.25		
Allowable Min Pressure	-19.7	ft	

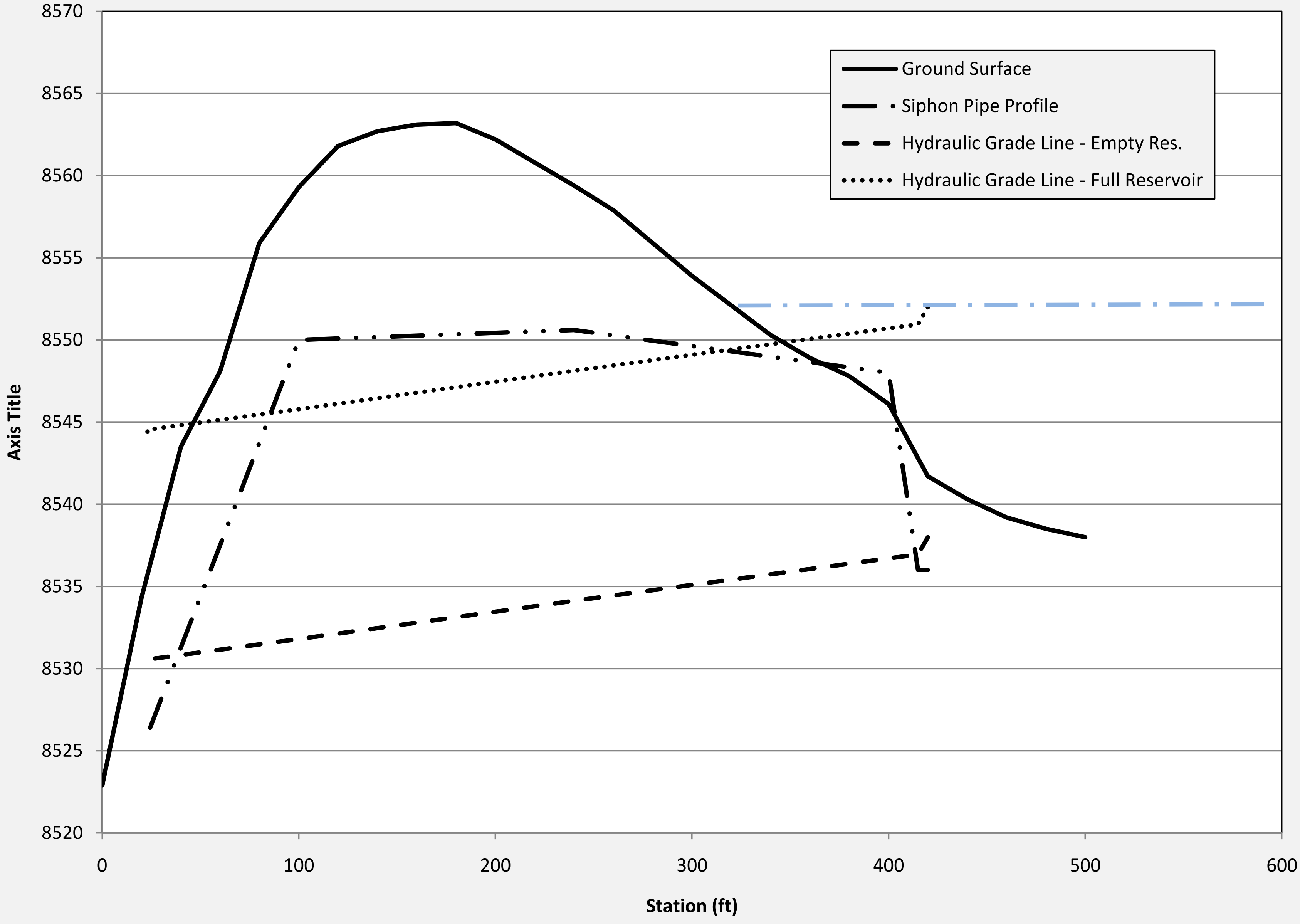
Minimum Reservoir

Station	Description	Pipe Invert	EGL	HGL	Pressure
420	Wet Well	8536	8538	8538	2.0
415	DS Wet Well	8536	8537.6	8537.0	1.0
400	DS Wet Well	8548	8537.4	8536.7	-11.3
240	High Point	8550.6	8534.8	8534.1	-16.5
100	Elbow	8550	8532.4	8531.8	-18.2
23.1	Exit, US	8526	8531.2	8530.5	4.5
23	Exit	8526	8530.9	8530.2	4.2
					OK

Minimum Reservoir

Station	Description	Pipe Invert	EGL	HGL	Pressure
420	Wet Well	8536	8552	8552	16.0
415	DS Wet Well	8536	8551.6	8551.0	15.0
400	DS Wet Well	8548	8551.4	8550.7	2.7
240	High Point	8550.6	8548.8	8548.1	-2.5
100	Elbow	8550	8546.4	8545.8	-4.2
23.1	Exit, US	8526	8545.2	8544.5	18.5
23	Exit	8526	8544.9	8544.2	18.2
					OK

Lake Otonowonda - Siphon Outlet Hydraulic Grade Line



Tyrolean Screen Calculations
Ridgway Ditch

Diverted Discharge	Q	7.00	ft3/s	Inputs
Rack Width	B	4.00	ft	
Rack Inclination	α	10	degrees	
Bar Width	b	3.13	in	
Gap Between Bars	e	0.58	in	
Diverted Discharge	Q	0.198	m3/s	Calculations
Rack Width	B	1.22	m	
Unit Discharge	q	0.16	m3/s/m	
Center-to-Center Bar Spacing	a	0.09	m	
Gap Between Bars	e	0.01	m	
Constructio Ratio	Ψ	0.16		
Critical Depth	hc	0.14	m	
reduction factor	x	0.95		
Water Depth US	h	0.13	m	
Contraction Coeff.	m	1.04		
Gravity Constant	g	9.81		
	λ	0.71		
	l	1.60	m	
Safety Factor	c	1.50		
Wetted Rack Length	**L**	**2.41**	**m**	Output
		7.89	**ft**	

*Calculations perfomed According to "Frank J., Fortschritte in der Hydraulik des Sohlenrechens; Der Bauingenieur 1959, Heft 1 "

Worksheet for Open Channel Pipe Flow

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.010	
Channel Slope	0.01400	ft/ft
Diameter	1.25	ft
Discharge	7.00	ft³/s

Results

Normal Depth	0.77	ft
Flow Area	0.80	ft²
Wetted Perimeter	2.26	ft
Hydraulic Radius	0.35	ft
Top Width	1.21	ft
Critical Depth	1.06	ft
Percent Full	61.9	%
Critical Slope	0.00657	ft/ft
Velocity	8.77	ft/s
Velocity Head	1.20	ft
Specific Energy	1.97	ft
Froude Number	1.91	
Maximum Discharge	10.69	ft³/s
Discharge Full	9.94	ft³/s
Slope Full	0.00695	ft/ft
Flow Type	SuperCritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Average End Depth Over Rise	0.00	%
Normal Depth Over Rise	61.93	%
Downstream Velocity	Infinity	ft/s

Worksheet for Open Channel Pipe Flow

GVF Output Data

Upstream Velocity	Infinity	ft/s
Normal Depth	0.77	ft
Critical Depth	1.06	ft
Channel Slope	0.01400	ft/ft
Critical Slope	0.00657	ft/ft

Worksheet for Pressure Pipe - 1

Project Description

Friction Method	Manning Formula
Solve For	Pipe Diameter

Input Data

Pressure 1	0.00	psi
Pressure 2	0.00	psi
Elevation 1	9286.00	ft
Elevation 2	9166.00	ft
Length	8507.00	ft
Roughness Coefficient	0.010	
Discharge	5.00	ft³/s

Results

Diameter	0.96	ft
Headloss	120.00	ft
Energy Grade 1	9286.73	ft
Energy Grade 2	9166.73	ft
Hydraulic Grade 1	9286.00	ft
Hydraulic Grade 2	9166.00	ft
Flow Area	0.73	ft²
Wetted Perimeter	3.03	ft
Velocity	6.84	ft/s
Velocity Head	0.73	ft
Friction Slope	0.01411	ft/ft

Worksheet for Trapezoidal Channel - 1

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.030	
Channel Slope	0.01400	ft/ft
Left Side Slope	3.00	ft/ft (H:V)
Right Side Slope	3.00	ft/ft (H:V)
Bottom Width	2.00	ft
Discharge	7.00	ft³/s

Results

Normal Depth	0.59	ft
Flow Area	2.24	ft²
Wetted Perimeter	5.75	ft
Hydraulic Radius	0.39	ft
Top Width	5.56	ft
Critical Depth	0.55	ft
Critical Slope	0.01895	ft/ft
Velocity	3.12	ft/s
Velocity Head	0.15	ft
Specific Energy	0.74	ft
Froude Number	0.87	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	0.59	ft
Critical Depth	0.55	ft
Channel Slope	0.01400	ft/ft

Worksheet for Trapezoidal Channel - 1

GVF Output Data

Critical Slope	0.01895	ft/ft

Pipe Size using slopes from profile - Calculations Performed in Flowmaster

Average Slopes				PVC Pipe Pipe Size				HDPE Pressure Pipe Pipe Size				Lined Channel 3:1 Side Slopes	
Station *(ft)*	Elevation *(ft)*	Length *(ft)*	Slope *(ft/ft)*	2cfs *(ft)*	normal depth *(ft)*	7cfs *(ft)*	normal depth *(ft)*	2cfs *(ft)*	3cfs *(ft)*	4cfs *(ft)*	5cfs *(ft)*	bottom width *(ft)*	depth *(ft)*
0	9285.9												
8507	9165.6	8507	0.014	0.83	0.46	1.25	0.77	0.68	0.80	0.89	0.96	2	1.1
16377	9063.0	7870	0.013	0.83	0.47	1.25	0.79	0.69	0.81	0.90	0.98	2	1.1
17531	8776.0	1153	0.249	0.83	0.21	0.67	0.49	0.40	0.47	0.52	0.56		
18959	8705.7	1428	0.049	0.67	0.36	1.00	0.61	0.54	0.63	0.70	0.76	2	1.0
21582	8682.9	2623	0.009	0.83	0.54	1.25	0.92	0.75	0.87	0.97	1.05	2	1.2
24145	8575.1	2563	0.042	0.67	0.38	1.00	0.64	0.56	0.65	0.72	0.79	2	1.0
26661	8564.6	2516	0.004	1.00	0.6	1.50	1.01	0.87	1.01	1.13	1.22	2	1.3

PVC Pipe	HDPE Pressure Pipe	Lined Channel
running ~75% full	running with zero pressure	Depth includes 6" of Freeboard
n=0.01	n=0.01	n=0.035