# DEWATERING IMPROVEMENTS STUDY

FOR THE

Town of
Gilcrest, Colorado

OCTOBER 2016

# DEWATERING IMPROVEMENTS STUDY

FOR THE

# TOWN OF GILCREST, COLORADO

**JVA, Inc.**

**1319 Spruce Street**
**Boulder, CO 80302**
phone: 303-444-1951

JVA Project No. 2278.4c

OCTOBER 2016

# TABLE OF CONTENTS


SECTION 1 – INTRODUCTION ........ 1
SECTION 2 – PREVIOUS STUDIES, DATA COLLECTION, AND MAPPING OF DEPTH TO GROUNDWATER 3
PREVIOUS STUDIES AND DATA COLLECTION ........ 3
MAPPING OF DEPTH TO GROUNDWATER ........ 3
HISTORICAL LOW GROUNDWATER CONDITIONS ........ 6
RECENT HIGH GROUNDWATER CONDITIONS ........ 6
DEPTH TO GROUNDWATER TARGETS ........ 10
GROUNDWATER LEVEL DRAWDOWN TARGETS ........ 10
SECTION 3 – GROUNDWATER RECHARGE ........ 12
EXISTING SOURCES OF GROUNDWATER RECHARGE ........ 12
SECTION 4 – DEWATERING SYSTEMS ........ 15
EXISTING DEWATERING STRUCTURES ........ 15
NEW DEWATERING STRUCTURES ........ 15
600-FOOT SPACING ........ 15
PRACTICAL PUMPING RATES ........ 17
SECTION 5 – INITIAL DEWATERING RATE ESTIMATES ........ 18
AQUIFER CHARACTERISTICS ........ 18
GROUNDWATER UNDERFLOW CALCULATION ........ 21
WATER BUDGET MASS BALANCE CALCULATIONS ........ 21
SUPERPOSITION WELL DRAWDOWN ANALYSIS ........ 21
SECTION 6 – GROUNDWATER MODEL ANALYSIS ........ 22
GROUNDWATER MODEL OVERVIEW ........ 22
SIMULATION A: OPTIMIZED WELLS SCENARIO ........ 24
SIMULATION B: EXISTING WELLS SCENARIO ........ 24
SUMMARY OF MODEL RESULTS AND INTERIM ALTERNATIVE ........ 39
SECTION 7 – IMPACT ON CONTAMINANT PLUMES AND RECOMMENDED GROUNDWATER MONITORING SYSTEM ........ 42
IMPACT ON CONTAMINANT PLUMES ........ 42
TREATMENT OF GROUNDWATER ........ 42
GROUNDWATER MONITORING SYSTEM ........ 43
SECTION 8 – STORMWATER EVALUATION ........ 46
STORMWATER AND AGRICULTURAL RUNOFF ........ 46
SECTION 9 – CONVEYANCE ALTERNATIVES ........ 51
DEWATERING WELL CONVEYANCE ........ 51
GRAVITY DEWATERING ........ 51
ALTERNATIVE 1: PRESSURIZED PIPELINE TO SOUTH PLATTE RIVER ........ 54
ALTERNATIVE 2: PRESSURIZED/GRAVITY FLOW COMBINATION TO SOUTH PLATTE RIVER ........ 58

Alternative 3: Multi-Use Multi-Benefit Pipeline to South Platte River ............ 59
Selected Conveyance Alternative ............ 60
Design Criteria for Selected Alternative ............ 61
Summary of Recommendations ............ 61
Implementation ............ 62

# List of Figures

Figure 2.1 – Groundwater Measurement Locations ............ 4
Figure 2.2 – April 2016 Ground Water Level Elevation ............ 7
Figure 2.3 – Ground Surface Elevation ............ 8
Figure 2.4 – Target Ground Water Depths and Dewatering Well Locations ............ 9
Figure 2.5 – Drawdown Targets ............ 11
Figure 3.1 – Recharge Facilities in Vicinity of Town ............ 13
Figure 4.1 – Proposed Dewatering Well Locations ............ 16
Figure 5.1 – Aquifer Characteristics ............ 20
Figure 6.1 – SPDSS MODFLOW Model ............ 23
Figure 6.2 – Summary of Modeled Drawdown and Pumping Rates (Optimized) ............ 25
Figure 6.3 – Projected Water Level Drawdown After One Year of Pumping (Optimized) ............ 26
Figure 6.4 - Projected Water Level Drawdown After Five Years of Pumping (Optimized) ............ 27
Figure 6.5 – Depth to Groundwater After One Year of Pumping (Optimized) ............ 28
Figure 6.6 – Depth to Groundwater After Five Years of Pumping (Optimized) ............ 29
Figure 6.7 – Projected Groundwater Elevation Contour After One Year of Pumping (Optimized) ............ 30
Figure 6.8 – Projected Groundwater Elevation Contour After Five Years of Pumping (Optimized) ............ 31
Figure 6.9 – Summary of Modeled Drawdown and Pumping Rates (Existing) ............ 32
Figure 6.10 – Projected Groundwater Level Drawdown After One Year of Pumping (Existing) ............ 33
Figure 6.11 – Projected Groundwater Level Drawdown After Five Years of Pumping (Existing) ............ 34
Figure 6.12 – Projected Depth to Groundwater After One Year of Pumping (Existing) 35
Figure 6.13 – Projected Depth to Groundwater After Five Years of Pumping (Existing) 36
Figure 6.14 – Projected Groundwater Elevation Contour After One Year of Pumping (Existing) ............ 37
Figure 6.15 – Projected Groundwater Elevation Contour After Five Years of Pumping (Existing) ............ 38
Figure 6.16 – Projected Groundwater Drawdown After 1 Year of Pumping ............ 40
Figure 6.17 – Projected Groundwater Drawdown After 5 Years of Pumping ............ 41
Figure 7.1 – Recommended Monitoring Well Network ............ 44

FIGURE 7.2 – EXAMPLE MONITORING WELL DESIGN ........ 45
FIGURE 8.1 – STORMWATER BASINS ........ 48
FIGURE 9.1 – DEWATERING WELLS AND CONVEYANCE ALTERNATIVES ........ 52
FIGURE 9.2 – SOUTH PLATTE RIVER CONVEYANCE ALTERNATIVES ........ 53
FIGURE 9.3 – SITE PLAN OF PROPOSED DEWATERING AND STORMWATER PUMP STATION ........ 55
FIGURE 9.4 – PRELIMINARY PLAN AND SECTION OF PROPOSED PUMP STATION ........ 56

# LIST OF TABLES

TABLE 2.1 – WELL INFORMATION IN VICINITY OF TOWN ........ 5
TABLE 3.1 – ACTIVE RECHARGE FACILITIES IN VICINITY OF TOWN ........ 14
TABLE 4.1 – SUMMARY OF TOWN WELLS ........ 15
TABLE 4.2 – WELLS WITHIN 600 FEET OF PROPOSED DEWATERING WELLS ........ 17
TABLE 4.3 – SUMMARY OF ESTIMATED MAXIMUM PUMPING RATES ........ 17
TABLE 5.1 – UNDERFLOW CALCULATION ........ 19
TABLE 5.2 – SUMMARY OF THEIS EQUATION SUPERPOSITION MODEL RESULTS ........ 19
TABLE 7.1 – RECOMMENDED MONITORING WELL NETWORK ........ 43
TABLE 8.1 – 100-YEAR PEAK RUNOFF COMPARISON (FUTURE BUILDOUT CONDITIONS) ........ 47
TABLE 8.2 - 100-YEAR RETENTION VOLUME COMPARISON (FUTURE BUILDOUT CONDITIONS) ........ 49
TABLE 9.1 – ALTERNATIVE 1 COST SUMMARY ........ 58
TABLE 9.3 – ALTERNATIVE 3 COST SUMMARY ........ 60
TABLE 9.4 – SUMMARY OF COST FOR ALTERNATIVES ........ 60
TABLE 9.5 – DECISION CRITERIA MATRIX ........ 61
TABLE 9.6 – DESIGN CRITERIA FOR PRESSURIZED PIPELINE TO SOUTH PLATTE ........ 61

# APPENDICES

APPENDIX A – SUPPORTING DOCUMENTATION
APPENDIX B – CALCULATIONS
APPENDIX C – COST ESTIMATES
APPENDIX D – DRAWINGS

# SECTION 1 – INTRODUCTION

This report presents the results of a study for the Town of Gilcrest (Town) to implement a permanent solution for high groundwater conditions throughout the Town. The study includes the evaluation of dewatering alternatives and permanent solutions for the Town's high groundwater problem. The study was approved and funded pursuant to the Emergency Dewatering Grant Program (HB15-1178), with a grant from the Colorado Water Conservation Board (CWCB).

The Town of Gilcrest has experienced elevated groundwater levels since 2006 after the shutdown of approximately 440 irrigation wells by the Colorado Division of Water Resources and changes in water management. The irrigation wells were shutdown to satisfy senior surface water rights along the South Platte River following litigation. After the well shutdown, the Town saw approximately a 10 to 25-foot increase in groundwater levels leading to increased Inflow and Infiltration (I&I) in the sewer system, flooding of basements, and damage to the Town's wastewater treatment facility lagoon liners. Ideally, water management could be modified to lower groundwater levels throughout the area. However, this approach may take many more years and may never come to fruition. Therefore, the following report outlines alternative approaches to protect the Town from the high regional groundwater. Of the alternatives assessed, reinstating a portion or all of the irrigation wells was not included, as this is assumed to be a political, non-engineered solution with a much longer time-frame than those which could provide "immediate" relief to the Town.

Previous studies of the regional groundwater conditions include the Colorado Geological Survey report, *Gilcrest/LaSalle Pilot Project, Hydrogeologic Characterization Report, September 30, 2014* and *Addendum, June 15, 2015* ("2014 CGS Report" and "2015 CGS Report," respectively), and Brown and Caldwell reports, *Sterling and Gilcrest/LaSalle High Groundwater Analysis, July, 2015* ("2015 Brown and Caldwell Report") and *Gilcrest Modeling Scenario Evaluation Results, February, 2016*. This has study focused specifically on evaluating permanent solutions using dewatering pumping and conveyance to lower groundwater levels beneath the Town to acceptable target depths. Funding for this study was approved in early 2016 and the project was initiated with a kick-off meeting with the Town on March 16, 2016.

To complete this effort, JVA, Inc. (JVA) and Bishop-Brogden Associates, Inc. (BBA) relied upon information provided by the Town, along with work by others including Palmetto Environmental, Tetra Tech, Central Colorado Water Conservancy District (Central), Colorado State University (CSU), and information available from public agencies, including the Colorado Division of Water Resources (DWR), Colorado Geological Survey ("CGS"), and the USGS.

This report includes a description of the evaluations and analyses undertaken and the results of those efforts, including:

- Maps and tables of groundwater level data; maps of dewatering structures and 600-foot spacing to water supply wells; maps and tables of groundwater recharge areas in and around the Town;
- Analytical and numerical groundwater model descriptions and results
- Discussion of dewatering alternatives
- Discussion of stormwater collection systems and recommended upgrades
- Discussion of anticipated impacts of dewatering alternatives on existing contaminant plumes
- Description of recommended measurable outcomes for successful dewatering solution
- Description of conveyance alternatives for dewatering discharge and stormwater
- Initial estimated costs to implement and operate a preferred dewatering alternative

# Section 2 – Previous Studies, Data Collection, and Mapping of Depth to Groundwater

## Previous Studies and Data Collection

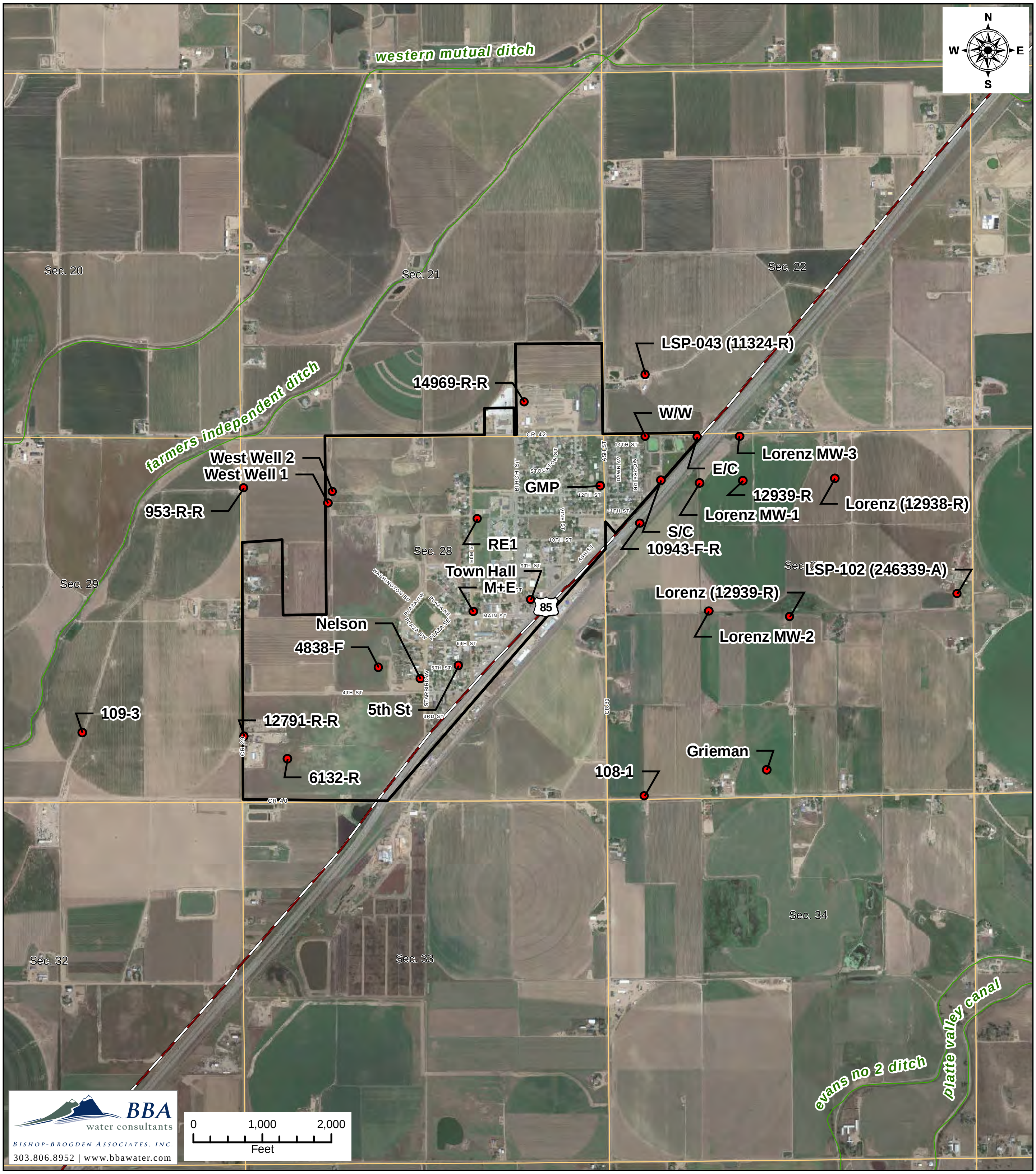
Previous studies by CGS, Brown and Caldwell, and others were reviewed to identify aquifer and groundwater data available in the vicinity of the Town. The information and data included: groundwater level data, groundwater recharge data, groundwater model input data, mapping, geologic characterizations and other information pertaining to the shallow groundwater conditions within the Town. In addition, we completed a review of groundwater information available from Town staff, Palmetto Environmental, Tetra Tech, Central Colorado Water Conservancy District, the Division of Water Resources, Colorado State University and input from residents and individuals with relevant knowledge.

## Mapping of Depth to Groundwater

The starting point for developing the Town dewatering plan is to identify the areas within the Town that are affected by shallow groundwater conditions. Water level data were compiled to prepare maps of groundwater elevation and depth to groundwater for the recent high-groundwater condition. Previous studies have included depth to groundwater mapping, but those studies did not include water level data from the Town's wells and were of a regional as opposed to local scale.

Ground level elevation data from the USGS and CDWR, were supplemented with new survey information developed from our own survey and data available from Tetra Tech and Palmetto for monitoring wells located adjacent to the Town. Additional water level data were collected from the Town's existing wells and new monitoring points in and around the Town, identified in Figure 2.1 and summarized in Table 2.1 on the following pages. Water level hydrographs were prepared for each of the monitoring wells relied upon in this study, as presented in Appendix A.

Historically, the depth to water within the Town limits has been as deep as 45 feet below ground surface, based on information reported on well permit forms and information provided by the Town regarding sewer line and pump station conditions. The current depth to water within the Town ranges from 9 to 17 feet below ground surface. During 2015, groundwater levels were as shallow as 6 feet below ground surface near Main Street and 11$^{th}$ Street (RE-1 Well) and 4 feet below ground surface near the wastewater treatment plant (located in the northeast corner of Town). The hydrographs indicate that water levels have generally trended down since the monitoring was initiated in 2015; with the water level decline for the approximate 16-month period ranging from 0.4 to 6.4 feet, and averaging approximately 2.5 feet.




# FIGURE 2.1
# Town of Gilcrest Ground Water Measurement Locations

Date: 9/20/2016 | Job No. 1606.00

*Legend*

- Ground Water Measurement Location
- Town Boundary
- PLSS Section

Aerial Photo Date: 6/19/14
Data Source: Town of Gilcrest, BBA, CDSS, CDOT, USGS, BLM

Colorado

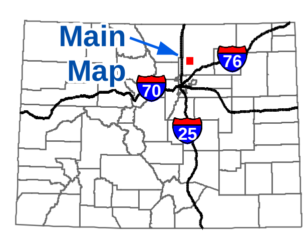


**Table 2.1**
**Town of Gilcrest**
**Well Information in Vicinity of Town**

| Name | Permit No. | Registered Owner | Depth (ft) | Pumping Rate (gpm) | Static Water Level (ft) | Static Water Level Date | Q40 | Q160 | Section | Township | Range |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Town of Gilcrest Pumping and Monitoring Wells | | | | | | | | | | | |
| Nelson | 16163-F-R | Henry Keiser | 92 | 1000 | 30 | 5/12/1955 | NE | SW | 28 | 4 N | 66 W |
| 5th St. | 12390-R | Western Wholesale Produce | 90 | 500 | 15 | 10/23/1945 | NW | SE | 28 | 4 N | 66 W |
| M+E | 13119-F-R | Town of Gilcrest | 95 | 800 | 22 | 3/23/1970 | SW | NE | 28 | 4 N | 66 W |
| RE1 | 59739-F | Weld County School District RE1 | 60 | - | - | 5/1/1951 | SE | NE | 28 | 4 N | 66 W |
| TH | 13118-F | Town of Gilcrest | 90 | 800 | 35 | 5/1/1956 | SE | NE | 28 | 4 N | 66 W |
| GMP | 47041-F | Town of Gilcrest | 92 | 700 | 20 | 9/26/1996 | NE | NE | 28 | 4 N | 66 W |
| W/W | 117787 | Gilcrest Sanitation District | 31 | - | 13 | 12/12/1980 | NW | NW | 27 | 4 N | 66 W |
| E/C | 117788 | Gilcrest Sanitation District | 31 | - | 13 | 12/12/1980 | NE | NW | 27 | 4 N | 66 W |
| S/C | 117789 | Gilcrest Sanitation District | 31 | - | 14 | 12/9/1980 | NW | NW | 27 | 4 N | 66 W |
| Lorenz MW-1 | 297252 | Town of Gilcrest | 16 | - | 7.75 | 4/22/2015 | NE | NW | 27 | 4 N | 66 W |
| Lorenz MW-2 | 297254 | Town of Gilcrest | 16 | - | 5.67 | 4/22/2015 | NE | NW | 27 | 4 N | 66 W |
| Lorenz MW-3 | 297253 | Town of Gilcrest | 16.5 | - | 6.17 | 4/22/2015 | SE | NW | 27 | 4 N | 66 W |
| State Monitoring Wells | | | | | | | | | | | |
| 108-1 | 19468-R | Nelson Hans | 66 | 550 | 12 | 5/6/1951 | SW | SW | 27 | 4 N | 66 W |
| 109-3 | 10987-R | Wiedman Terry | 97 | 800 | 32 | 11/1/1968 | SE | SW | 29 | 4 N | 66 W |
| LSP-043 | 11324-R | Kaveny A J Sr | 102 | 1000 | 22 | 7/1/1954 | NW | SW | 22 | 4 N | 66 W |
| LSP-102 | 246339--A | J Oliver Lorenz | 45 | 10 | 20 | 1/7/2004 | SE | NE | 27 | 4 N | 66 W |
| Greiman | 11224-R | Greiman Grant | 56 | 600 | 9 | 8/1/1943 | SE | SW | 27 | 4 N | 66 W |
| Lorenz | 12938-R | J Oliver Lorenz | 34 | 450 | 8 | 7/1/1938 | NW | NE | 27 | 4 N | 66 W |
| Nearby Wells | | | | | | | | | | | |
| West Well 1 | 630-R | Nelson Thyra | 90 | 800 | 35 | 5/1/1934 | NE | NW | 28 | 4 N | 66 W |
| West Well 2 | 19957-R | Western Equipment & Truck, Inc | 75 | 1500 | 30 | 6/1/1917 | NE | NW | 28 | 4 N | 66 W |
| | 12939-R | J Oliver Lorenz | 92 | 2500 | - | 7/1/1938 | NE | NW | 27 | 4 N | 66 W |
| | 12791-R-R | Benman Scott & Wendy | 74 | 800 | 45 | 7/9/1994 | SW | SW | 28 | 4 N | 66 W |
| | 953-R-R | Wiedman Terry | 80 | 1200 | 39 | 4/4/1997 | NW | NW | 28 | 4 N | 66 W |
| | 6132-R | Nelson Thyra | 101 | 700 | 32 | 8/18/1958 | SW | SW | 28 | 4 N | 66 W |
| | 4838-F | Hunt David & Kayleen | 98 | 2000 | 18 | 5/12/1964 | NE | SW | 28 | 4 N | 66 W |
| | 14969-R-R | Weld County Reorg School District | 80 | 1200 | 25 | 9/3/1973 | SE | SE | 21 | 4 N | 66 W |
| | 10943-F-R | Scaefer Carl & Venice | 94 | 800 | 21 | 3/20/1967 | NW | NW | 27 | 4 N | 66 W |

Notes:
Depth, pumping rate and static water levels from Division of Water Resources well permit files.

The depth to groundwater maps were used to identify and confirm those areas within the Town where water levels need to be lowered and to quantify the amount the water levels need to be lowered beneath the Town to achieve target water level depths. Utilizing the depth to groundwater maps and the Town's input regarding required water levels within Town, target depths for lowering the groundwater table were established, as discussed in more detail below.

## HISTORICAL LOW GROUNDWATER CONDITIONS

Historical (1950 to 1980) groundwater levels beneath Town are reported to be 13 to 35 feet below ground surface, based upon well construction reports. Regular water level measurements begin in August 2014 for the three wells located within the wastewater treatment plant. However, regular water level measurements were not recorded in a majority of the Town's wells until April 2015. Historical pre-2015 reported groundwater level measurements for Town wells are summarized above in Table 2.1. The nearest well with a long-term continuous water level record (LSP-102) does not indicate substantial long-term water level change (see Appendix A). However, a well located less than 1,000 feet north of the wastewater treatment plant (LSP-43) has continuous water level data from 1956 through 1976 and 2013 through present (see Appendix A). That well shows an approximately 10-foot water level rise, which is consistent with anecdotal reports from the Town.

## RECENT HIGH GROUNDWATER CONDITIONS

A recent depth to groundwater map was prepared for the Town based upon April 2016 water level data from the Town, CSU, Tetra Tech, CDWR and BBA's field visit. April 2016 was chosen for depth to water mapping because the greatest number of water level measurements were available, including recent measurements toward the west of Town collected as part of this study. To prepare the April 2016 depth to water level map, groundwater elevations were contoured and subtracted from land surface elevations within Town. April 2016 water levels are approximately 3.5 feet deeper than the peak depth to water measured near the wastewater treatment plant during September 2014.

The groundwater table was mapped at a 1-foot contour interval, as shown in Figure 2.2. The groundwater table beneath the Town shows groundwater flow direction from the southeast on the east side of the Town and from south-southwest through Town. The USGS 10-meter ground surface digital elevation model (DEM) was verified based upon surveyed elevation of the points within the Town, shown in Figure 2.3. The groundwater elevations from Figure 2.2 and ground surface elevations from Figure 2.3 were converted to a 15-meter grid spacing to determine April 2016 depth to groundwater, shown in Figure 2.4.

April 2016 depth to groundwater mapping shows extremely shallow groundwater five to ten feet beneath ground surface generally isolated to areas east of Birch Street and immediately north of Liberty Park (baseball field).

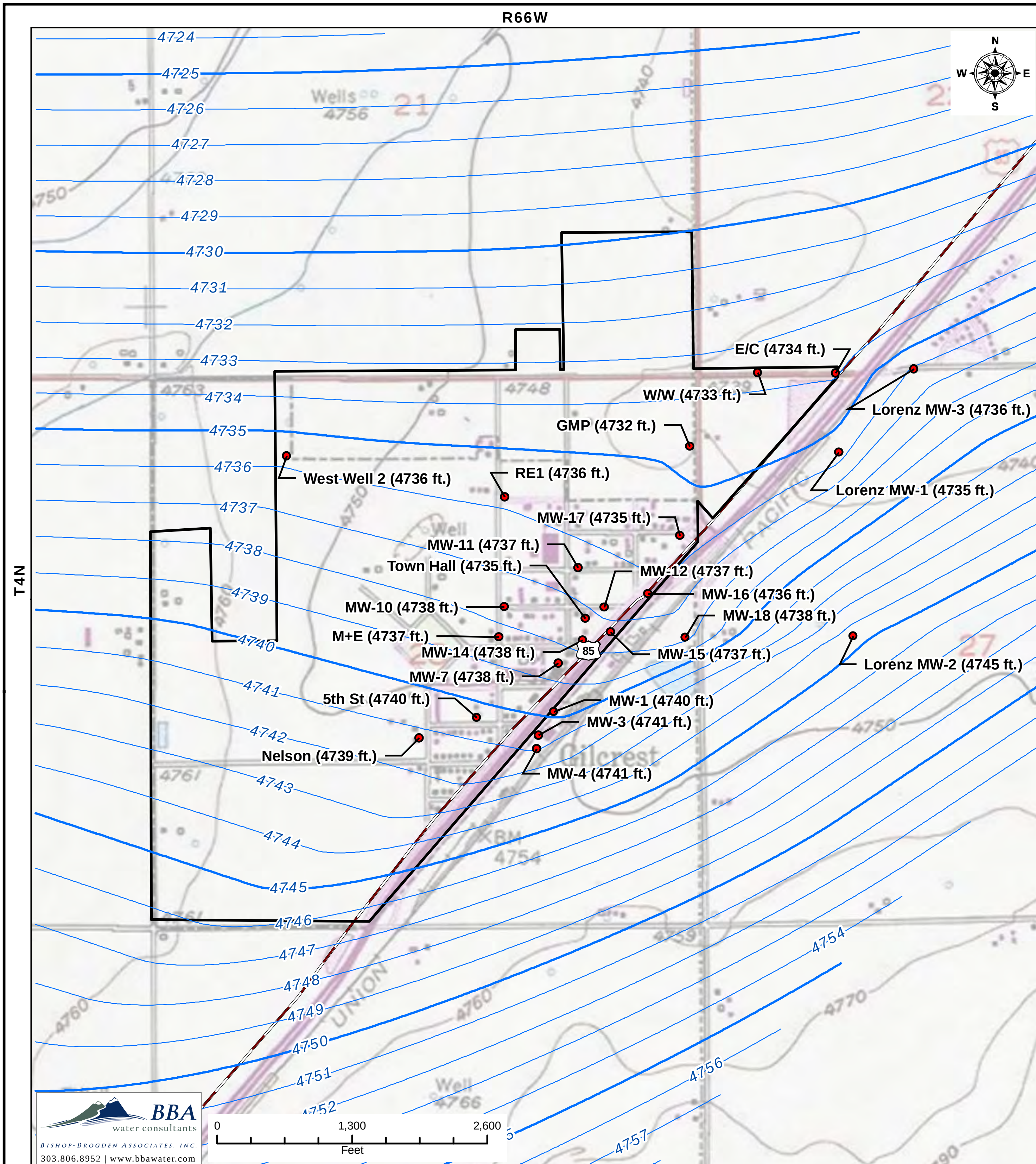
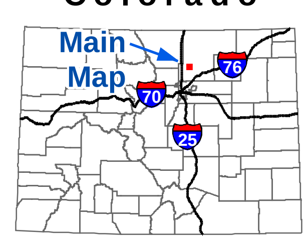



**FIGURE 2.2**
**Town of Gilcrest**
**April 2016 Ground Water Level Elevation**

Date: 8/25/2016 | Job No. 1606.00

Data Source: Town of Gilcrest, BBA, TetraTech, CDSS, CDOT, USGS, BLM

**Colorado**



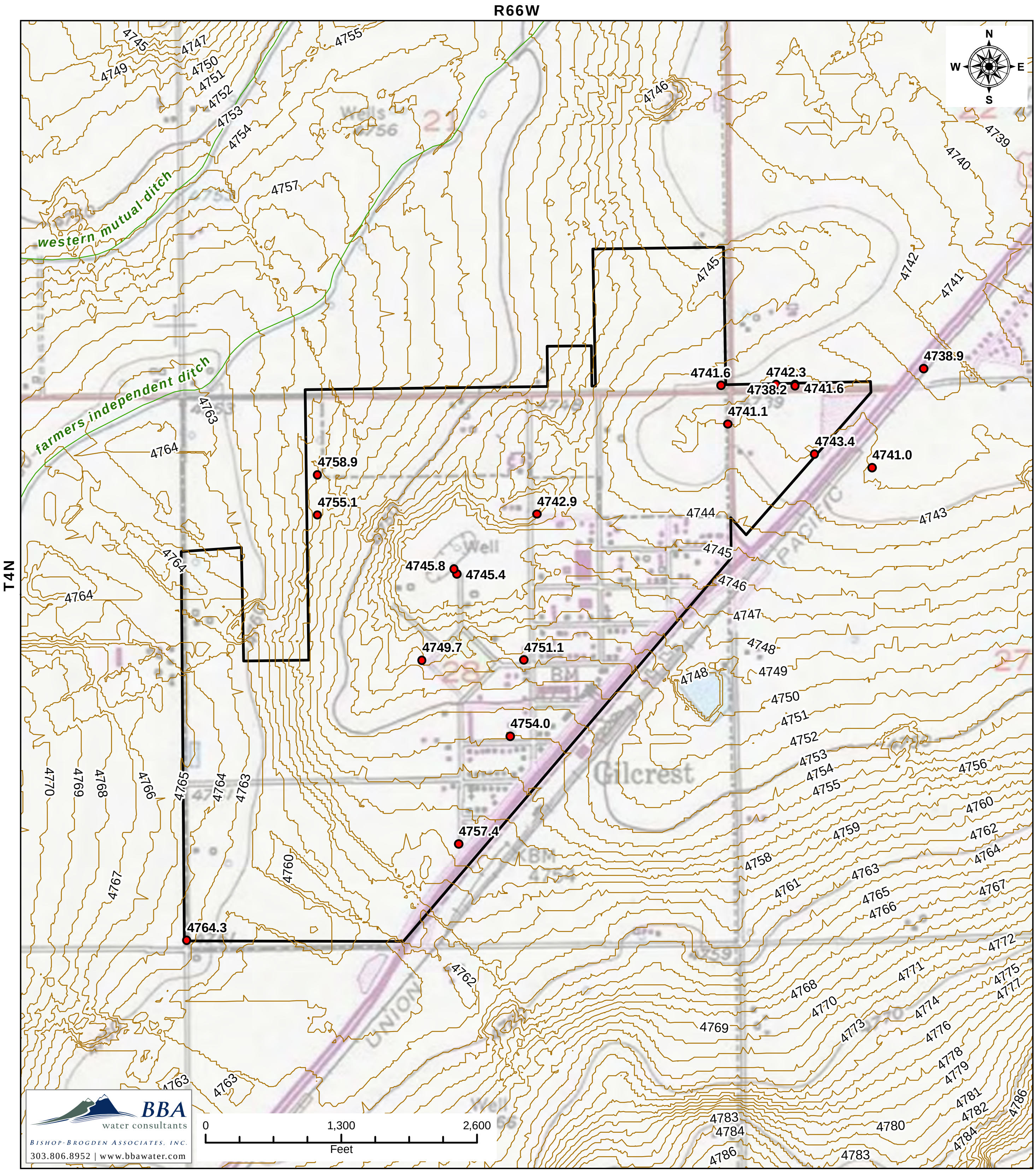



**FIGURE 2.3**
**Town of Gilcrest**
**Ground Surface Elevation**

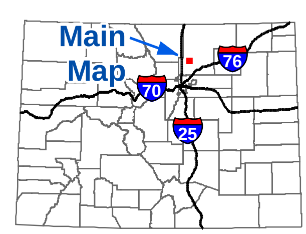
Date: 8/25/2016 | Job No. 1606.00

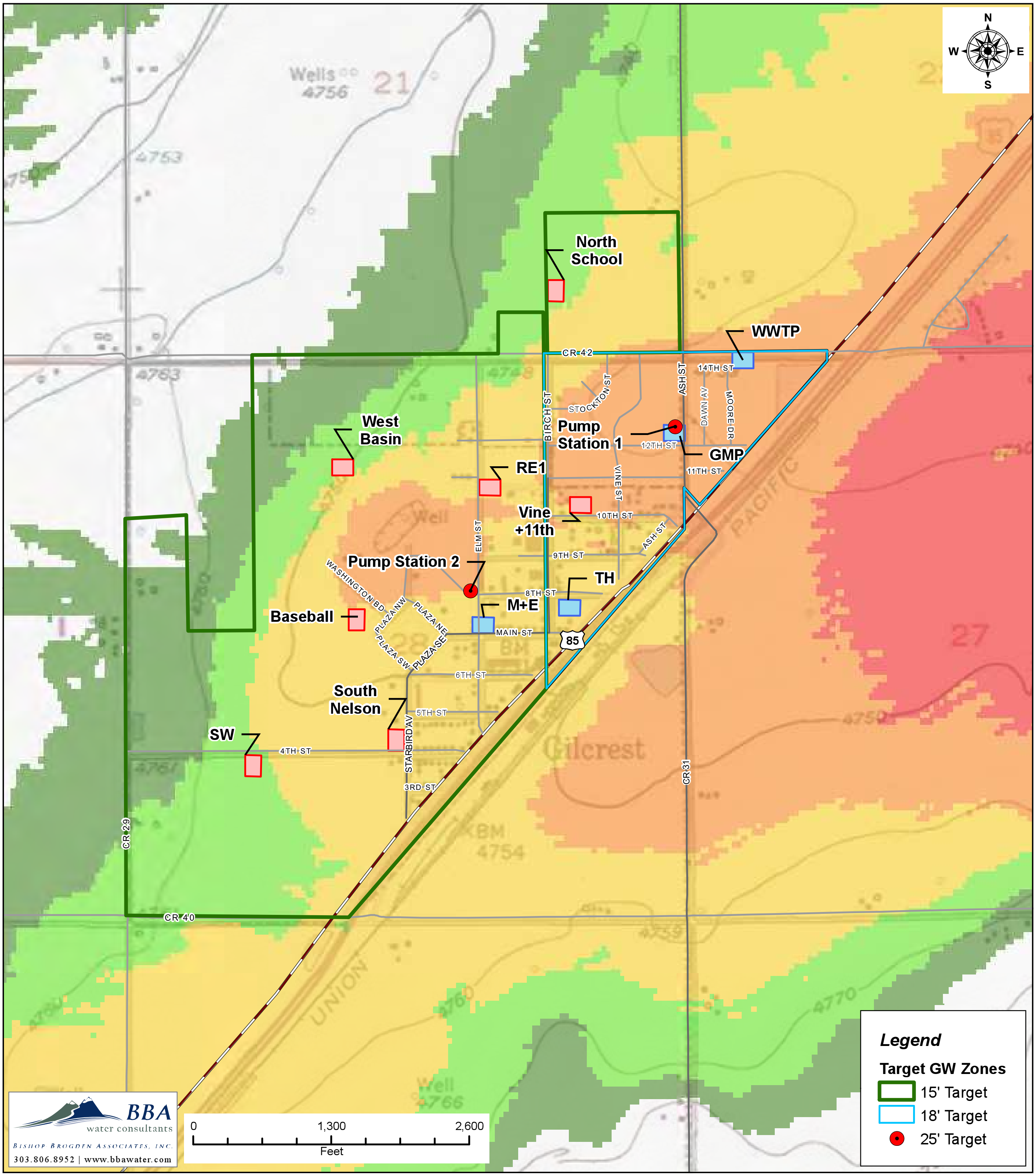
*Legend*

- Survey Points Relied Upon
- 1 Ft. Contours (From 10m DEM, USGS)
- Town Boundary

Data Source: CDSS, CDOT, USGS, BLM

**Colorado**






# FIGURE 2.4

**Town of Gilcrest**
**April 2016 Depth to Ground Water, Target Ground Water Depths and Dewatering Well Locations**

Date: 9/9/2016 | Job No. 1606.00

*Legend*

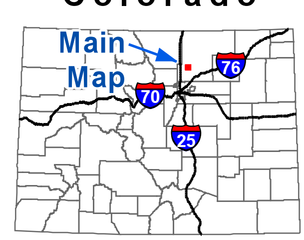
- Town Boundary
- Proposed Dewatering Well Site
- Alternative Dewatering Well Site

**Depth to Ground Water**

- < 5
- 5 - 10
- 10 - 15
- 15 - 20
- 20 - 25
- >25

Data Source: Town of Gilcrest, BBA, TetraTech, CDSS, CDOT, USGS, BLM

**Colorado**

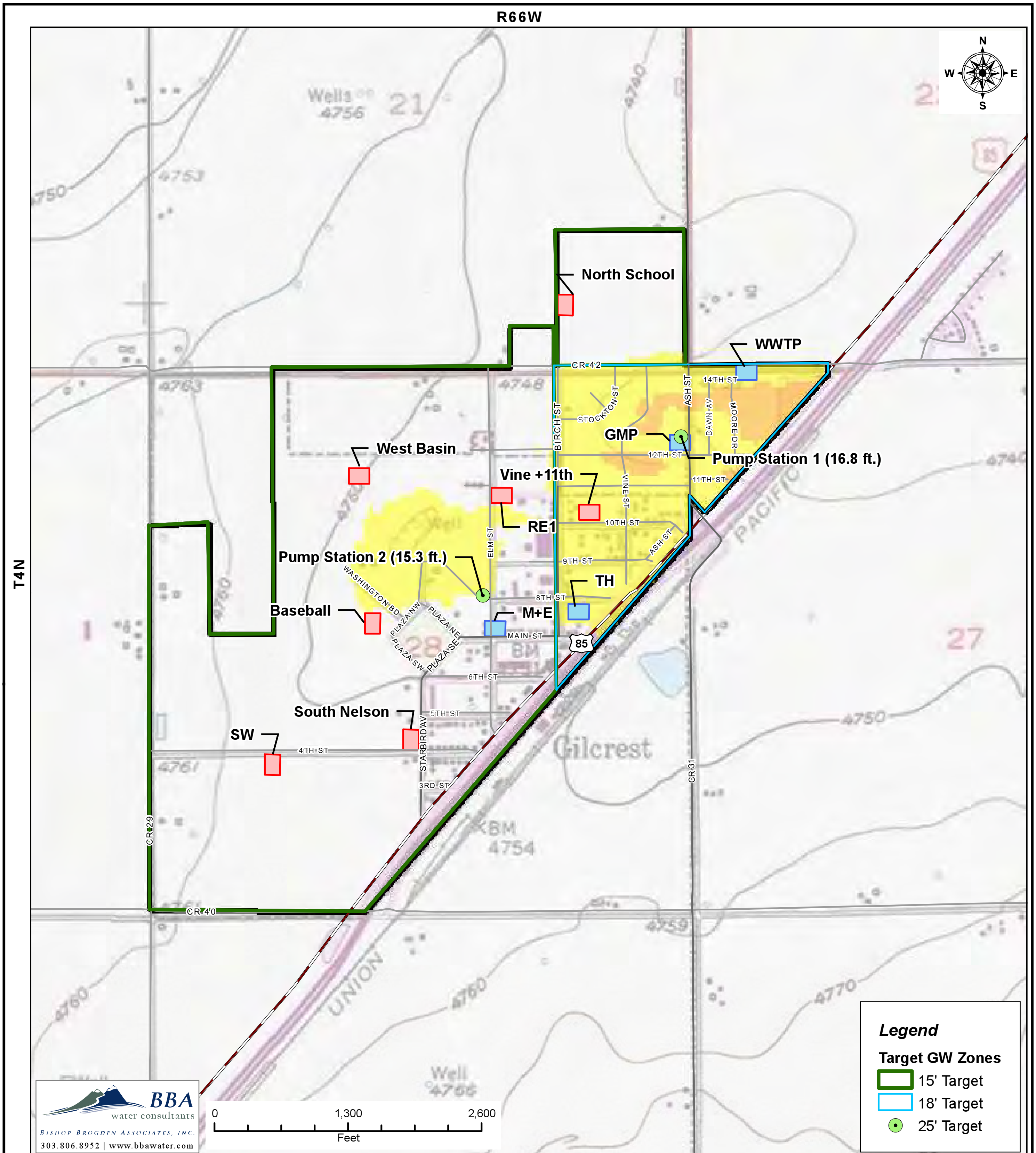


## Depth to Groundwater Targets

Depth to groundwater targets were developed based upon input from the Town for protection of critical infrastructure, shown in Figure 2.4. A depth to groundwater target of 18 feet was chosen for areas of the Town south of County Road 42 and east of Birch Street based upon a 12-foot depth of wastewater basins at the Town's wastewater treatment plant, depth of sewer lines and existence of basements in the historically developed portion of Town. The Town has two wastewater pump stations, one located at 12$^{th}$ Street and Ash Street and a second located at 8$^{th}$ Street and Elm Street identified in Figure 2.4. A depth to groundwater target of 25 feet was chosen at these locations to protect pump station infrastructure and minimize I&I. For all other areas of Town, a 15-foot depth to water target was chosen to prevent I&I to existing and future wastewater collection facilities. The depth to groundwater targets are conservatively deep and accommodate the relatively small water level decline observed in some wells between 2014 and 2016.

## Groundwater Level Drawdown Targets

The depth to groundwater targets were subtracted from the April 2016 depth to groundwater mapping to determine the amount of groundwater level drawdown required throughout the Town, shown in Figure 2.5. The greatest drawdown targets are at the Town's two wastewater pump stations and in the northeast portion of Town near the wastewater treatment facility.




# FIGURE 2.5
## Town of Gilcrest Drawdown Targets

Date: 9/9/2016 | Job No. 1606.00

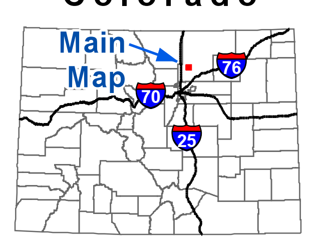
*Legend*

- Town Boundary
- Proposed Dewatering Well Site
- Alternative Dewatering Well Site

**Drawdown Targets**

- < 5
- 5 - 10
- 10 - 15
- 15 - 20

Data Source: Town of Gilcrest, BBA, TetraTech, CDSS, CDOT, USGS, BLM



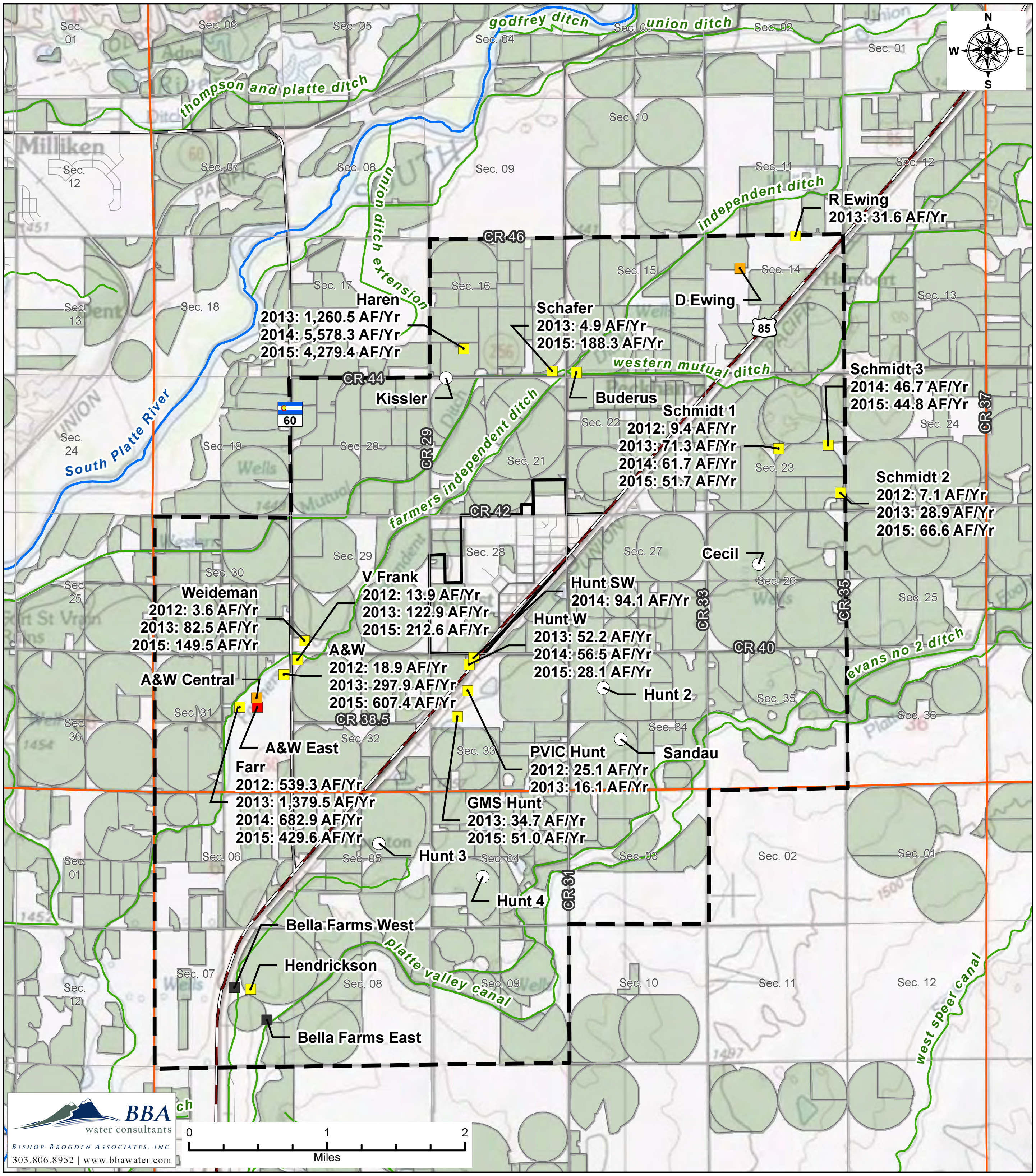
# SECTION 3 – GROUNDWATER RECHARGE

## EXISTING SOURCES OF GROUNDWATER RECHARGE

The purpose of this task was to identify sources of groundwater recharge that may most directly impact groundwater levels beneath the Town. The locations of recharge ponds, irrigation ditches and irrigated lands located near the Town were inventoried, as shown in Figure 3.1. We reviewed the DWR database and water rights decrees to identify the proposed location of additional structures and future potential infiltration (recharge) facilities and recharge rates for all structures. A summary of existing and proposed groundwater recharge facilities is presented in Table 3.1, including recharge facility location, historical recharge amount and maximum recharge rate.

Recharge operations at the GMS Hunt, PVIC Hunt, Hunt W and Hunt SW recharge facilities are located immediately up-gradient from the Town and are likely to have the greatest impact on groundwater conditions. However, the historical amount of recharge at these facilities has been relatively small (25.1 to 162.1 acre-foot per year [af/yr]), compared to calculated groundwater underflow and the estimated rates necessary for dewatering described in later sections of this report. Although it is desirable to minimize recharge operations at these sites, they are not the sole cause for shallow groundwater conditions at the Town.

The Evans No. 2 Ditch is located southeast of Town and previous groundwater modeling simulations have indicated that up to 5,614 af/yr of seepage from this ditch is a substantial source of groundwater flow beneath the Town. Anecdotal reports from irrigators that use the Evans No. 2 Ditch indicate that modeled ditch seepage may be overstated. The Evans No. 2 Ditch directly up-gradient from the Town can be used for recharge operations, (see decree in Case No. 05CW331, Water Division 1). However, irrigators report that this up gradient reach is not currently used for recharge.




# FIGURE 3.1
# Town of Gilcrest Recharge Facilities in Vicinity of Town

Date: 9/1/2016 | Job No. 1606.00

*Legend*

**Recharge Facilities**

- Active with diversion records
- Active no diversion records
- Historical with diversion records
- Inactive no diversion records
- Non-existent no diversion records

Town Boundary

Area of Interest

Irrigated Area - 2010

Data Source: B&C, CDSS, CDOT, USGS, BLM

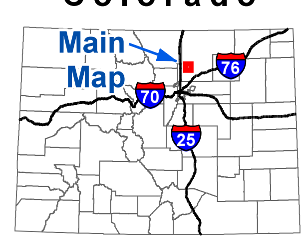


**Table 3.1**
**Town of Gilcrest**
**Active Recharge Facilities in Vicinity of Town**

| Facility | Recharge Deliveries (af/yr) | | | | Maximum Delivery Rate (cfs) |
|---|---|---|---|---|---|
| | 2012 | 2013 | 2014 | 2015 | |
| Farmer's Independent Ditch | | | | | |
| Weidman | 3.6 | 82.5 | 0.0 | 149.5 | 2.4 |
| V Frank | 13.9 | 122.9 | 0.0 | 212.6 | 9.9 |
| A & W | 18.9 | 297.9 | 0.0 | 607.4 | 54.7 |
| A & W Central | - | - | - | - | - |
| Farr | 539.3 | 1379.5 | 682.9 | 429.6 | 16.6 |
| R Ewing | 0.0 | 31.6 | 0.0 | 0.0 | 2.2 |
| D Ewing | - | - | - | - | - |
| Subtotal | 575.7 | 1914.4 | 682.9 | 1399.1 | - |
| Evans No. 2 Ditch | | | | | |
| Hunt SW | 0.0 | 0.0 | 94.1 | 0.0 | 1.3 |
| Hunt W | 0.0 | 52.2 | 56.5 | 28.1 | 0.9 |
| PVIC Hunt | 25.1 | 16.1 | 0.0 | 0.0 | 1.1 |
| GMS Hunt | 0.0 | 34.7 | 11.5 | 51.0 | 3.5 |
| Schmidt 1 | 9.4 | 71.3 | 61.7 | 51.7 | 1.0 |
| Schmidt 2 | 7.1 | 28.9 | 0.0 | 66.6 | 1.2 |
| Schmidt 3 | 0.0 | 0.0 | 46.7 | 44.8 | 1.1 |
| Subtotal | 41.6 | 203.0 | 270.5 | 242.1 | - |
| Western Mutual Ditch | | | | | |
| Haren | 0.0 | 1260.5 | 5578.3 | 4279.4 | 32.8 |
| Schafer | 0.0 | 4.9 | 0.0 | 188.3 | 1.3 |
| Subtotal | 0.0 | 1265.4 | 5578.3 | 4467.6 | - |
| Other | | | | | |
| Buderus [3] | - | - | - | - | - |
| Hendrickson [3] | - | - | - | - | - |
| Total Recharge | 617.3 | 3382.8 | 6531.8 | 6108.9 | |

Notes:
1. All recharge structures identified as "active" in CDSS records.
2. Recharge calculated utilizing daily diversion records from CDSS.
3. Structures are classified as active with diversion records in CDSS, but have no diversion records.

# SECTION 4 – DEWATERING SYSTEMS

## EXISTING DEWATERING STRUCTURES

An inventory was made of the existing wells and stormwater ponds that could potentially be used for dewatering. The Town's GMP, Main Street and Elm Street (M+E), Town Hall (TH) and 5$^{th}$ Street wells were identified as potential dewatering wells. These wells have estimated historical pumping capacities of approximately 300 gpm to 800 gpm. Details about these and other wells in Town are presented in Table 4.1. The GMP and M+E wells located at ideal dewatering sites, but are approximately 20 and 46 years old, respectively, and would need to be replaced to achieve maximum well yields.

## NEW DEWATERING STRUCTURES

Additional potential dewatering well sites were identified based upon current and expected future land use within Town. Figure 4.1 presents a total of 11 potential well sites that were considered for dewatering. Of those sites, only the WWTP site was included in the conceptual dewatering alternatives.

**Table 4.1 – Summary of Town Wells**

| Name | Permit No. | Year Constructed | Reported Rate | Status |
|---|---|---|---|---|
| 5th St. | 12390-R | 1945 | 300 | Active |
| M+E | 13119-F-R | 1970 | 800 | Operational, Inactive |
| TH | 13118-F | 1956 | 800* | Not Operational |
| GMP | 47041-F | 1996 | 700 | Active |

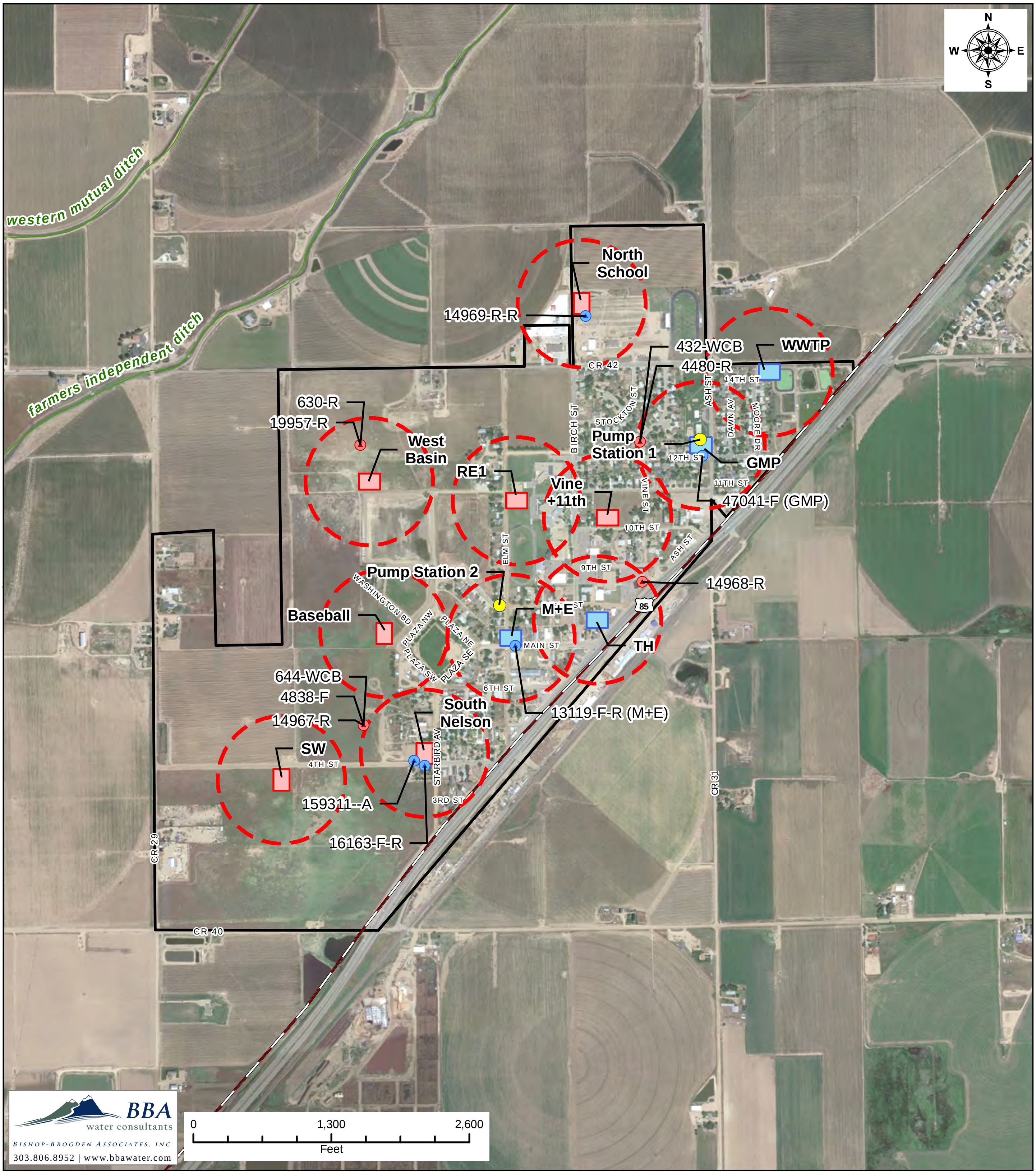
***Notes: Rates and statuses reported by Town of Gilcrest. *Rate reported on Permit No. 13118. Rates and statuses reported by Town of Gilcrest***

## 600-FOOT SPACING

Water supply wells located within 600-feet of each potential dewatering well sites were identified based upon records available from the Division of Water Resources and communications with Town staff. Figure 4.1 identifies water supply wells located within 600-feet of potential dewatering well sites and information regarding those wells is summarized in Table 4.2. Pursuant to C.R.S. 37-90-137(2)(b) dewatering wells must be located more than 600-feet from any water supply well unless a waiver has been obtained from the well owner.

R66W

T4N




BBA water consultants

Bishop-Brogden Associates, Inc.

303.806.8952 | www.bbawater.com

# FIGURE 4.1
# Town of Gilcrest Proposed Dewatering Well Locations

Date: 8/26/2016 | Job No. 1606.00

*Legend*

**Wells within 600 ft. of Dewatering Sites - Permit No.**

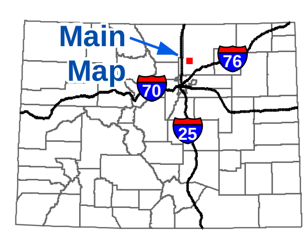
- Spotted From Quarters
- Spotted From Section Lines
- Town Sewer Pump Station

- Proposed Dewatering Well Site
- Alternative Dewatering Well Site
- 600 ft. Buffer
- Town Boundary

Aerial Photo Date: 6/19/2014

Data Source: Google Earth, CDSS, CDOT, USGS, BLM

Colorado



## PRACTICAL PUMPING RATES

The maximum practical pumping rate for newly constructed dewatering wells was calculated based upon a range of assumed aquifer characteristics and two-thirds well drawdown. Table 4.3 summarizes estimated maximum pumping rates for various length pumping periods and aquifer characteristics discussed later in this report. Based upon a 1-year pumping period, the range of expected maximum well pumping rates is 1,849 gpm to more than 2,500 gpm.

**Table 4.2 – Wells Within 600 Feet of Proposed Dewatering Wells**

| Permit No. | Owner | Depth (ft) | Pumping Rate (gpm) | Proposed Well Within 600 feet |
|---|---|---|---|---|
| 16163-F-R | Keiser Henry | 98 | 1000 | South Nelson |
| 159311--A | Keiser Henry | 95 | 15 | South Nelson |
| 14968-R | Cogburn Earl | 73 | 1200 | Vine + 11th |
| 14967-R | Cogburn Earl | 80 | 1000 | SW |
| 4838-F | Hunt David W | 98 | 2000 | SW |
| 630-R | Nelson Thyra | 90 | 800 | West Basin |
| 19957-R | Western Equipment | 70 | 1500 | West Basin |
| 14969-R-R | Weld School District | 80 | 1200 | North School |
| 432-WB | Mcleod Royal | 70 | - | GMP |
| 47041-F | Town of Gilrest | 91 | 700 | GMP |
| 13119-F-R | Town of Gilcrest | 95 | 800 | M+E |

*Notes: Depth and pumping rate from Division of Water Resources well permit files.*

**Table 4.3 – Summary of Estimated Maximum Pumping Rates**

| Transmissivity (gpd/ft) | Storage | Radius (ft) | Saturated Thickness (ft) | Maximum Rate (gpm) / Pumping Period (days) | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | 1 | 7 | 30 | 180 | 365 | 1826 |
| 100,000 | 0.2 | 1 | 90 | > 2500 | > 2500 | 2277 | 2039 | 1958 | 1796 |
| 300,000 | 0.2 | 1 | 90 | > 2500 | > 2500 | > 2500 | > 2500 | > 2500 | > 2500 |

*Notes: Maximum drawdown is 2/3 of saturated thickness. Jacob equation is used for maximum pumping rate calculations. Transmissivity is adjusted for declining saturated thickness.*

# SECTION 5 – INITIAL DEWATERING RATE ESTIMATES

Quantitative evaluation of groundwater flow conditions was evaluated using three methods: (1) groundwater underflow calculation based upon groundwater mapping prepared in the 2014 CGS Report and 2015 CGS Report, (2) calculations based upon the water budget mass balance presented in the 2015 Brown and Caldwell Report and (3) superposition well drawdown analysis. These analytical methods allowed the study team to estimate the magnitude of dewatering rates required to achieve the Town's target depths and identify optimal dewatering sites. Estimated dewatering rates were refined through groundwater model analysis, summarized in Section 6. Aquifer characteristics were evaluated to support the dewatering rate estimates. The results of the analyses are presented in Tables 5.1 and 5.2, and discussed further below.

## AQUIFER CHARACTERISTICS

Aquifer characteristics of saturated thickness, hydraulic conductivity and specific yield strongly influence groundwater flow and dewatering rates required to achieve the Town's depth to groundwater targets. Long-term aquifer pumping tests are the best source for data regarding aquifer characteristics, however there are limited aquifer pumping test data available in the immediate vicinity of the Town.

Based upon reported depth to bedrock and mapped groundwater levels in the vicinity of the Town, the average aquifer saturated thickness ranges from approximately 75 to 95 feet. Aquifer hydraulic conductivity was estimated to range from 400 to 800 ft/d in the vicinity of the Town based upon the 2014 CGS Report. The nearest controlled long-term aquifer pumping was completed 3 miles north of Town near the intersection of County Road 35 and U.S. Highway 85 in 1957, identified in Figure 5.1. That test is documented in Circular 11 (Pumping Test in Colorado, USGS, 1965) as B4-66-11adc, and shows a hydraulic conductivity of 1,270 ft/d, a transmissivity of 370,000 gpd/ft and storage coefficient of 0.03. During September 17 through November 1, 2015 water levels and pumping rates monitored near-continuous operation of the Lorenz well. Analysis of water level data from observation wells located near the Lorenz well indicate a hydraulic conductivity of approximately 150 ft/d and a transmissivity of approximately 100,000 gpd/ft.

Based upon published values and analysis of well pumping test data, a transmissivity (product of hydraulic conductivity and saturated thickness) ranging from 100,000 to 300,000 gpd/ft was used for groundwater analyses for the Town. That range is based upon an average saturated thickness of approximately 90 feet and a hydraulic conductivity ranging from 150 to 470 ft/d. Specific yield was assumed to be 20%, which is the same value used in regional groundwater models and in support of numerous water court decrees.

### Table 5.1 – Underflow Calculation

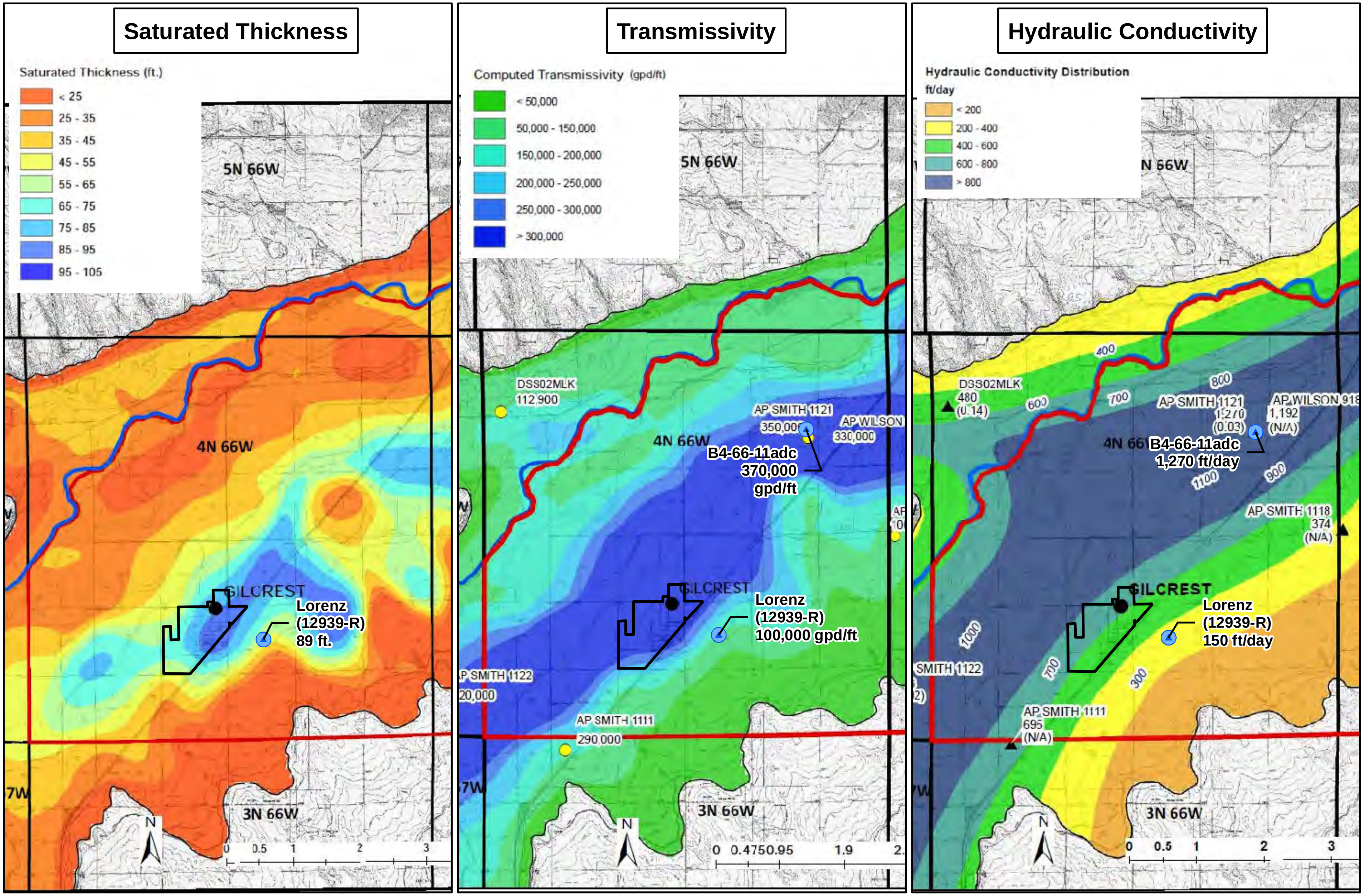
| Underflow Rate (cfs) | Underflow Rate per foot of Saturated Thickness (cfs/ft) | Hydraulic Conductivity (ft/d) | Saturated Thickness (ft) | Transmissivity (gpd/ft) | Hydraulic Gradient (ft/ft) |
|---|---|---|---|---|---|
| 2.55 | 0.03 | 149 | 90 | 100,000 | 2.6E-03 |
| 3.37 | 0.04 | 149 | 90 | 100,000 | 3.4E-03 |
| 7.65 | 0.08 | 446 | 90 | 300,000 | 2.6E-03 |
| 10.10 | 0.11 | 446 | 90 | 300,000 | 3.4E-03 |

*Notes: Hydraulic gradient from Appendix C, 2013 Time-Series Historic Groundwater Elevation Contour Map, CGS Report. Transmissivity from CGS Report and Lorenz monitoring well calculations. Town width is 6,375 feet. Saturated thickness estimated at 90 feet based on nearby well depths and water levels. Underflow rate = transmissivity * hydraulic gradient * town width.*

### Table 5.2 – Summary of Theis Equation Superposition Model Results

| Location | Target Drawdown (ft) | T = 100,000 gpd/ft | | T = 300,000 gpd/ft | |
|---|---|---|---|---|---|
| | | Rate (gpm) | Simulated Drawdown (ft) | Rate (gpm) | Simulated Drawdown (ft) |
| **Proposed Wells** | | | | | |
| WWTP | 9.4 | 785 | 27.1 | 2200 | 25.2 |
| GMP / Pump St. 1 | 15.3 | 1500 | 45.5 | 2200 | 25.9 |
| West Basin | 4.0 | - | 7.5 | - | 8.8 |
| RE1 | 10.6 | - | 10.6 | 695 | 14.8 |
| Vine + 11th | 8.3 | - | 13.2 | - | 12.5 |
| Baseball Field | 0.5 | - | 8.2 | - | 9.1 |
| M+E | 5.8 | 1022 | 29.5 | 2200 | 23.8 |
| South Nelson | 5.4 | - | 7.8 | - | 8.7 |
| SW | 0.0 | - | 5.2 | - | 6.6 |
| **Observation Points** | | | | | |
| SW-Town Boundary | 0.0 | - | 2.9 | - | 4.6 |
| NW-Town Boundary | 0.0 | - | 5.4 | - | 7.0 |
| SE-Town Boundary | 1.5 | - | 4.5 | - | 6.0 |
| E/C | 10.4 | - | 10.4 | - | 10.5 |
| Pump Station 2 | 12.6 | - | 12.8 | - | 12.6 |
| South Birch | 4.1 | - | 6.2 | - | 7.4 |
| North Birch | 1.3 | - | 9.3 | - | 10.4 |
| | **Total (gpm):** | **3307** | | **7295** | |
| | **Total (cfs):** | **7.4** | | **16.3** | |

*Note: Target drawdown calculated as the target water level at location less April 2015 water level.*




# Figure 5.1
# Town of Gilcrest
# Aquifer Characteristics

Date: 8/25/2016 | Job No. 0403.25

*Legend*

Well Location

Town Boundary

Base map from 2014 CGS Report

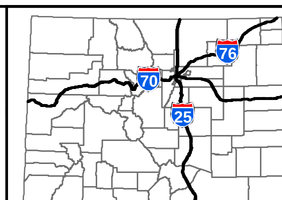
N W E S

0 1 2 Miles

BBA water consultants

BISHOP-BROGDEN ASSOCIATES, INC.

303.806.8952 | www.bbawater.com



## Groundwater Underflow Calculation

The estimated groundwater underflow beneath the Town was determined using a groundwater gradient based upon regional groundwater elevation mapping from the 2014 CGS Report and a range of transmissivity from 100,000 gpd/ft to 300,000 gpd/ft. As shown in Table 5.1, the total estimated groundwater underflow beneath the Town is approximately 2.55 to 10.10 cfs, or an average flow rate of approximately 1,846 to 7,312 af/yr. Groundwater underflow through the Town is estimated at 0.03 to 0.11 cfs per foot of saturated aquifer thickness. For example, 15 feet of decreased saturated thickness equates to 0.45 to 1.65 cfs of reduced underflow. Groundwater underflow calculations provide a reference for the effect of order-of-magnitude changes in the groundwater budget on groundwater levels, but underestimate actual dewatering rates due to time-dependent dewatering effects and changes in groundwater gradient during dewatering.

## Water Budget Mass Balance Calculations

Based upon the water budget analysis presented in the 2015 Brown and Caldwell Report, 5,000 af/yr of reduced groundwater inflows or increased groundwater outflows will result in a 1-foot regional water table change. Based upon the Town area of 517 acres, compared to Brown and Caldwell's study area of 33,799 acres and 5,000 af/yr for 1-foot of water level change, a 76 af/yr or 0.11 cfs change in the water budget at the Town would result in a 1-foot water level change. For example, 15 feet water level change would equate to a 1,140 af/yr or 1.65 cfs change in the water budget at Town. Like the underflow calculations, the water budget calculation provides a reference for the effect of order-of-magnitude changes in the groundwater budget on groundwater levels, but underestimate actual dewatering rates.

## Superposition Well Drawdown Analysis

An analytical model was used to evaluate pumping drawdown at locations throughout the Town, based on various aquifer properties, dewatering well locations and configurations and dewatering rates. Multiple model iterations were run to evaluate different pumping scenarios and target levels at selected locations in Town, identified in Figure 2.5. The model is based upon the Theis equation, adjusts aquifer drawdown due to changes in aquifer thickness and accounts for well-to-well interference. After refinement of the location and magnitude of the Town's desired water level targets, the model was used to examine the well configuration and dewatering rates needed to achieve the target goals. The results from the superposition well drawdown analysis indicate a total rate of 7.4 to 16.3 cfs to achieve the drawdown targets. The results of the superposition well drawdown analysis are presented in Table 5.2.

# SECTION 6 – GROUNDWATER MODEL ANALYSIS

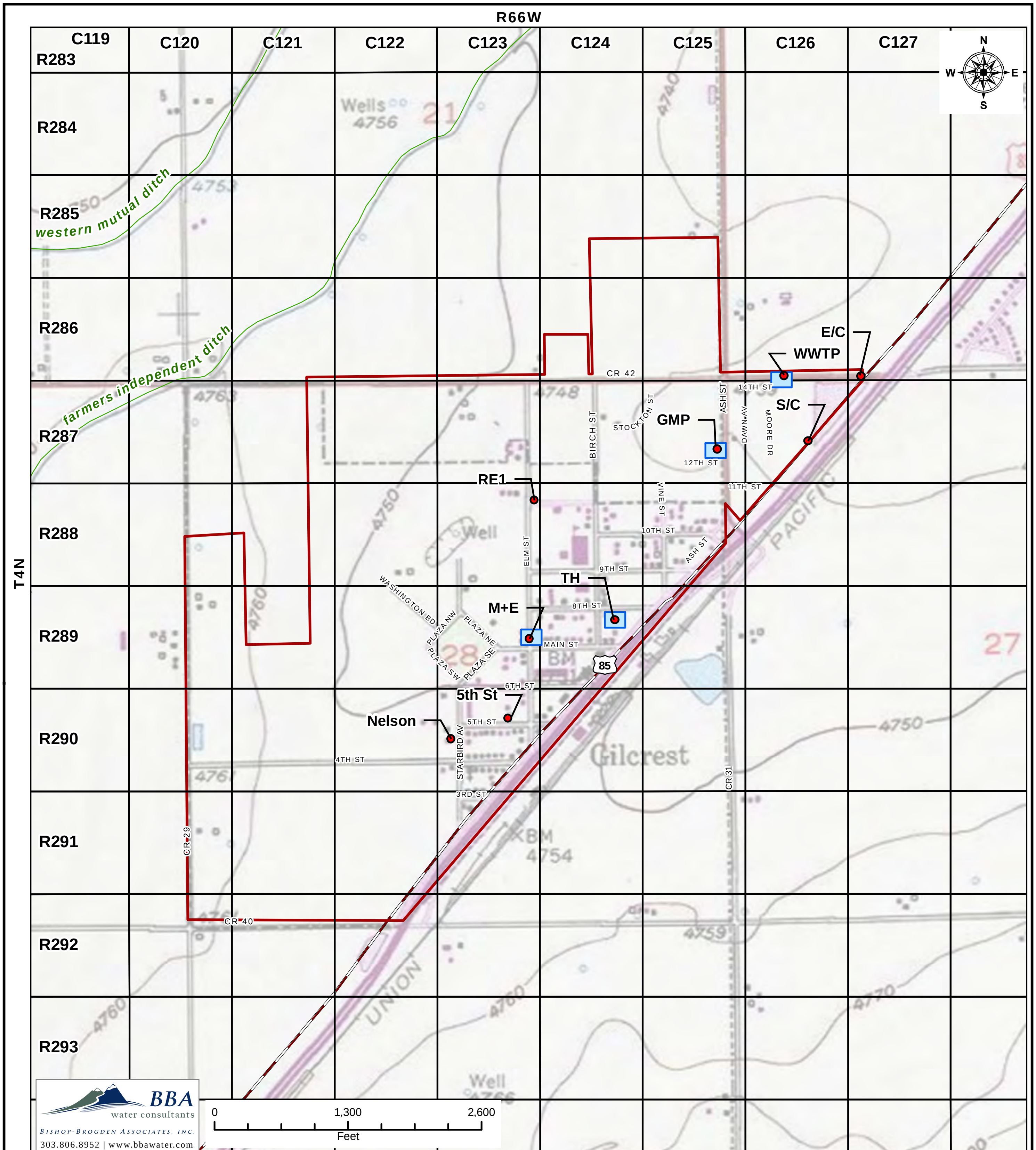
## GROUNDWATER MODEL OVERVIEW

Building on information from previous tasks, a numerical groundwater modeling analysis was used to evaluate pumping rates of dewatering wells to achieve the Town's target water levels and to evaluate various dewatering structure configurations. Results from groundwater modeling provide recommended dewatering well locations and expected timing of water level changes occurring within the Town. Hydrogeologic conditions can be substantially variable over relatively short distances and limited aquifer test data is available in the vicinity of the Town. Accordingly, model results are initial and the actual dewatering rates needed to achieve the Town's water level targets will not be known until long-term high-capacity dewatering is initiated and monitored.

The SPDSS MODFLOW groundwater model of the South Platte alluvial aquifer system updated by Brown and Caldwell in 2016 was used for the dewatering analysis. That groundwater model extends from Chatfield Reservoir to the Nebraska Stateline, has 1,000-foot grid spacing and a January 1950 through December 2012 simulation period divided into monthly stress periods. For the scope of this study, it was not practical to refine the grid spacing to less than 1000-feet in Town. However, the 1000-foot grid spacing is adequate to evaluate water level changes for different pumping rates and well configurations. Figure 6.1 identifies the row and column references from the SPDSS MODFLOW model in the vicinity of the Town.

The SPDSS MODFLOW model uses aquifer characteristics similar to those reported in the 2014 CGS Report. In the vicinity of Town, the modeled aquifer transmissivity is approximately 300,000 gpd/ft and the specific yield is 0.20. Attempts were made to modify the SPDSS MODFLOW model to include a transmissivity of 100,000 gpd/ft in the vicinity of Town, however this change caused the model to lose stability and report unrealistic ground water levels.

The SPDSS MODFLOW model was updated to include dewatering wells as a separate well package to simulate pumping during a 5-year period. Effects on groundwater levels from pumping in the Town were evaluated as the change in modeled water table elevation (drawdown) between historical simulated conditions and dewatered simulated conditions. First, the SPDSS MODFLOW model was run without the additional dewatering well pumping to simulate historical groundwater levels. Second, the model was run with dewatering pumping in Town to meet the drawdown targets identified in Figure 2.5. As shown in Appendix A 6.1, 6.2, the SPDSS MODFLOW model simulates approximately 10 feet of water level rise between 2003 and 2010 in Town. This relative groundwater level rise is similar to water level rise observed in LSP-43 and reports from Town staff.




# FIGURE 6.1
# Town of Gilcrest
# SPDSS MODFLOW Model

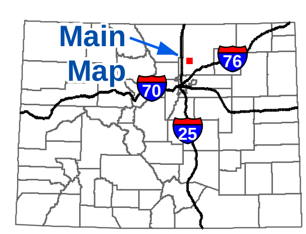
Date: 8/26/2016 | Job No. 1606.00

*Legend*

- Wells of Town's Monitoring Program
- Simulated Dewatering Well Site
- Town Boundary
- SPDSS MODFLOW Model Grid Row and Column (R###, C###)

Data Source: CDSS, CDOT, USGS, BLM

Colorado



Two model runs were completed: (1) an "optimized" scenario based on new dewatering wells located at the WWTP, M+E and GMP and (2) an "existing" scenario based on utilizing the existing M+E, GMP and TH wells at their respective pumping capacities, with the addition of one new WWTP well. In the "optimized" dewatering simulation, dewatering rates were adjusted at all three well sites to achieve groundwater level targets after one year of pumping and then reduced annually to maintain dewatering targets through the end of the 5-year simulation. In the "existing" dewatering simulation, dewatering rates were adjusted only at the new well site to achieve groundwater level targets at the end of the five-year dewatering simulation.

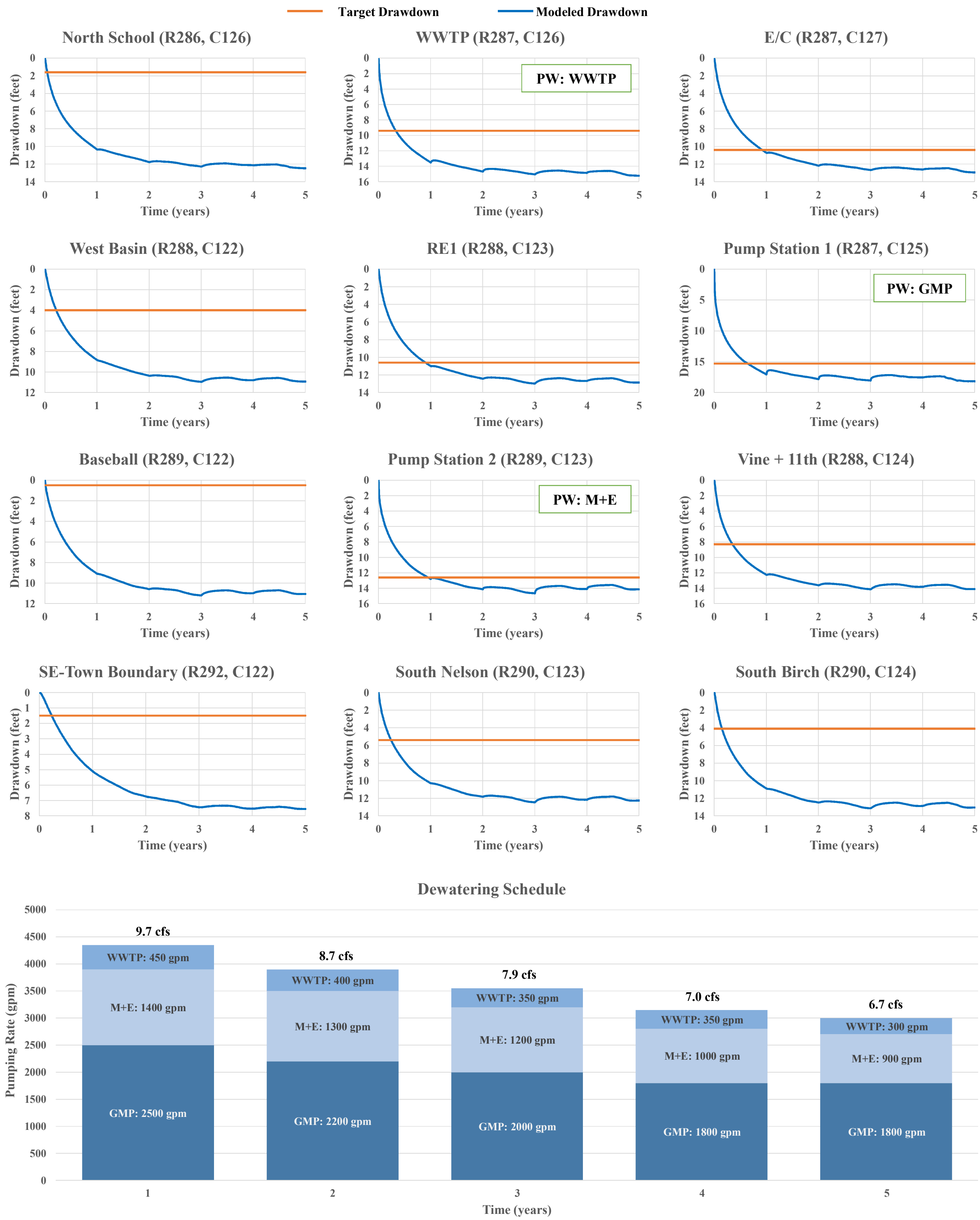
### SIMULATION A: OPTIMIZED WELLS SCENARIO

The optimized wells scenario includes pumping at new wells constructed at the GMP, M+E and WWTP locations. Total Town dewatering is estimated to require up to 9.7 cfs during the first year and decrease to 6.7 cfs by the end of the 5-year simulation. Figure 6.2 presents hydrographs of the projected drawdown at targeted locations throughout the Town, along with the projected well pumping rates. Figures 6.3 and 6.4 present projected drawdown during dewatering after one year and five years of pumping. Figures 6.5 and 6.6 present projected depth to groundwater, which is equal to the April 2016 depth to groundwater mapping plus the projected drawdown after one year and five years of pumping. Figures 6.7 and 6.8 present projected groundwater level elevation after one year and five years of pumping. The projected groundwater level elevation shown is equal to the April 2016 groundwater level elevation mapping minus the projected drawdown after one year and five years of respective pumping.

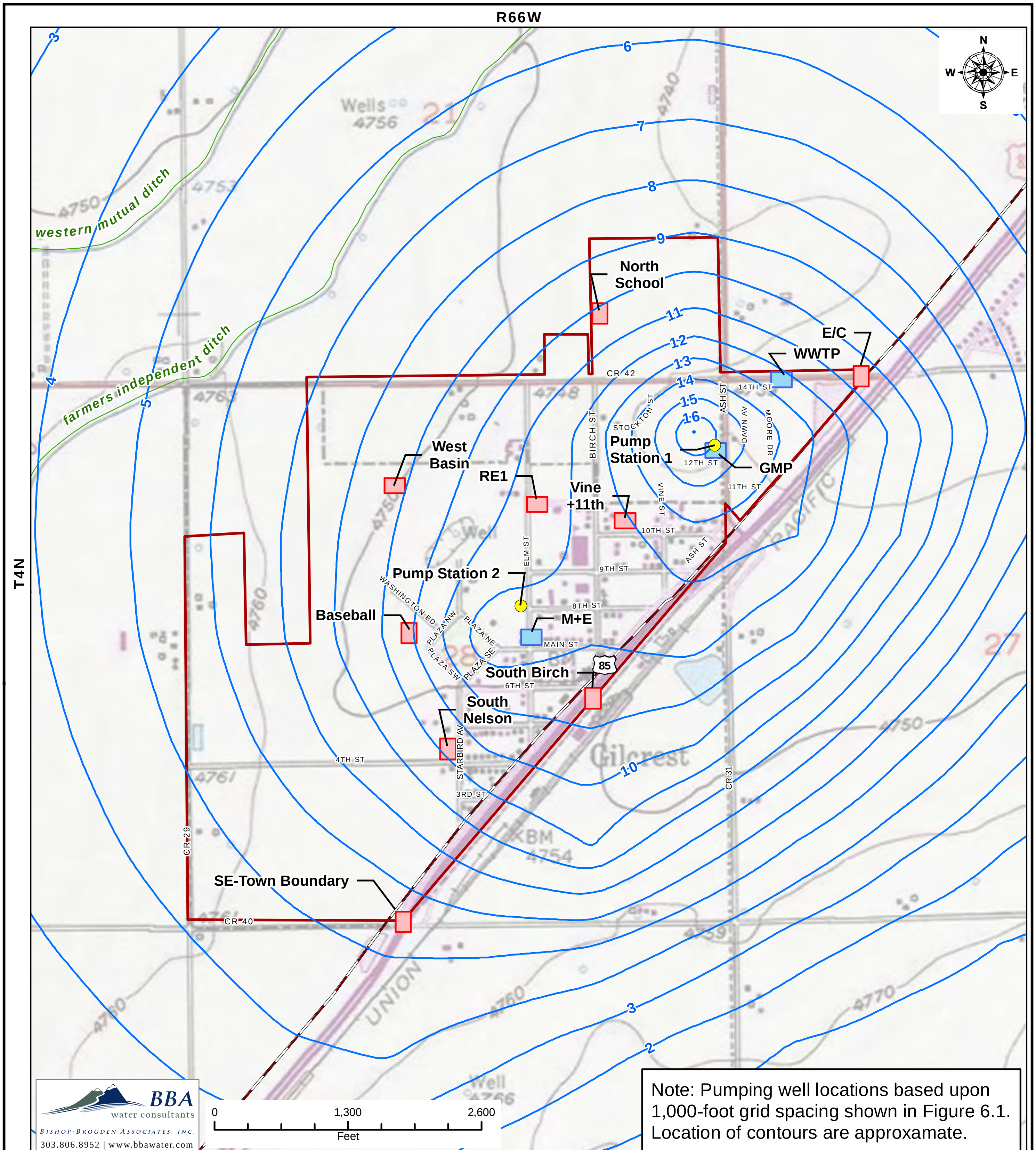
### SIMULATION B: EXISTING WELLS SCENARIO

The existing wells scenario includes pumping the Town's existing GMP, TH and M+E wells along with a new WWTP well. Due to the age of the existing wells, this scenario is intended to be a temporary, as opposed to permanent solution. Total Town dewatering is estimated to require 7 cfs of steady pumping for five years using four dewatering well locations. Figure 6.9 presents hydrographs of the projected drawdown at targeted locations throughout the Town, along with the projected pumping rates. Figures 6.10 and 6.11 present projected drawdown during dewatering after one year and five years of pumping. Figures 6.12 and 6.13 present projected depth to groundwater, which is equal to the April 2016 depth to groundwater mapping plus the projected drawdown after one year and five years of pumping. Figures 6.14 and 6.15 present projected groundwater level elevation after one year and five years of pumping. The projected groundwater level elevation shown is equal to the April 2016 groundwater level elevation mapping minus the projected drawdown after one year and five years of respective pumping. Figures 6.14 and 6.15 also show the approximate locations of the nitrate and benzene plumes within Town limits.

**Figure 6.2**
**Town of Gilcrest**
**Summary of Modeled Drawdown and Pumping Rates (Optimized)**




**Notes:**
1) Drawdown is calculated as the difference between model simulated head elevations for historical (without dewatering) conditions and with dewatering conditions.
2) The modeled dewatering schedule is held at constant rates for one-year blocks.
3) Values presented above each column on the Dewatering Schedule graph represent the combined total dewatering rate for all three wells in cubic feet per second (cfs).
4) For spatial relationships between targeted locations, see map in Figure 6.1.



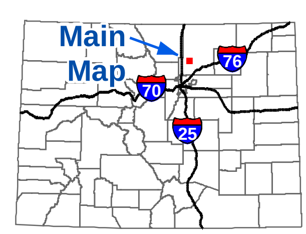

# FIGURE 6.3
## Town of Gilcrest Projected Ground Water Level Drawdown After 1 Year of Pumping (Optimized)

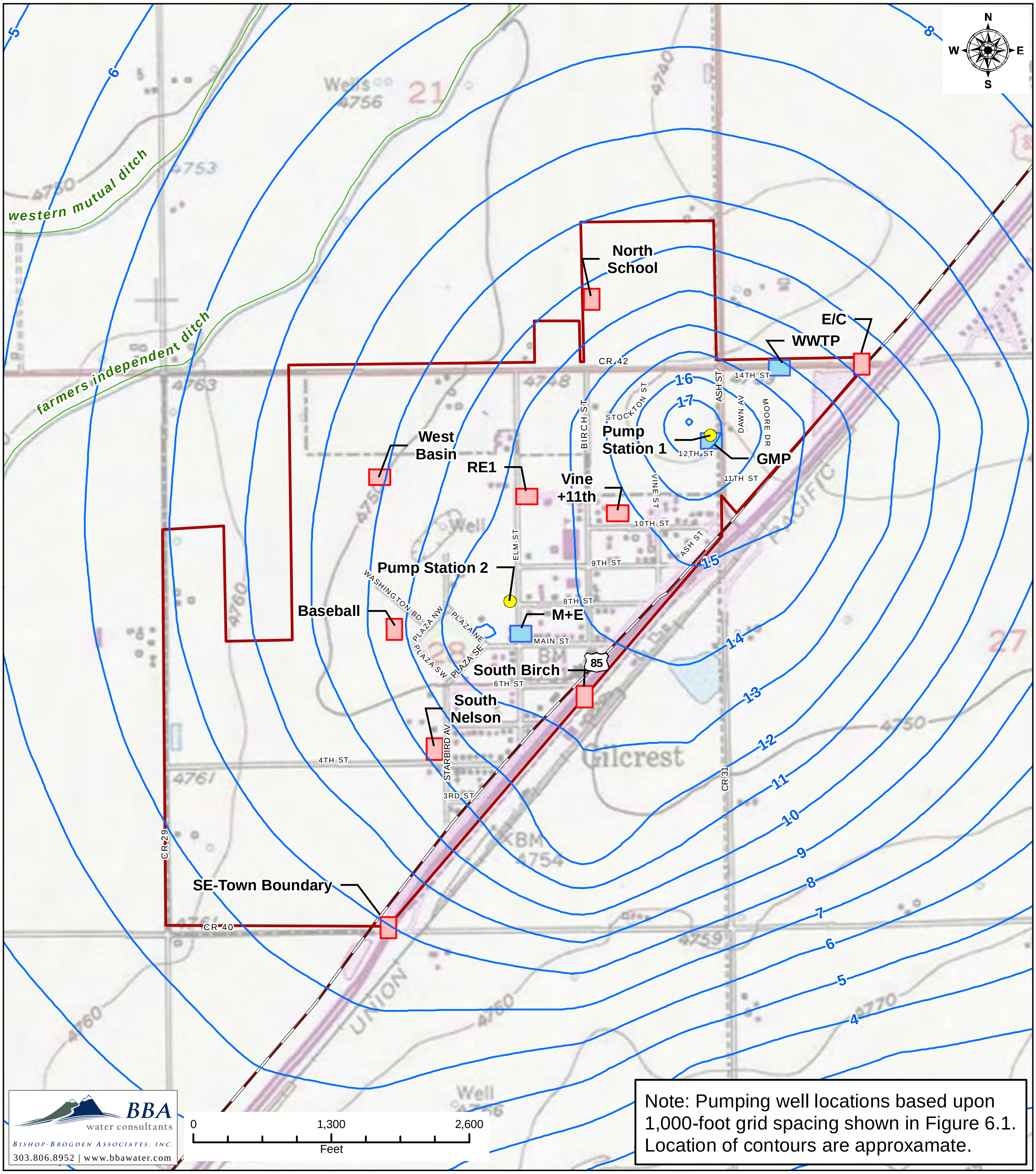
Date: 8/25/2016 | Job No. 1606.00

*Legend*

- Town Sewer Pump Station
- Drawdown After 1 Year of Pumping (1 ft. Contour)
- Simulated Dewatering Well Site
- Target Drawdown Location
- Town Boundary

Data Source: CDSS, CDOT, USGS, BLM






# FIGURE 6.4
## Town of Gilcrest Projected Ground Water Level Drawdown After 5 Years of Pumping (Optimized)

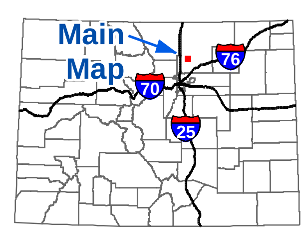
Date: 8/25/2016 | Job No. 1606.00

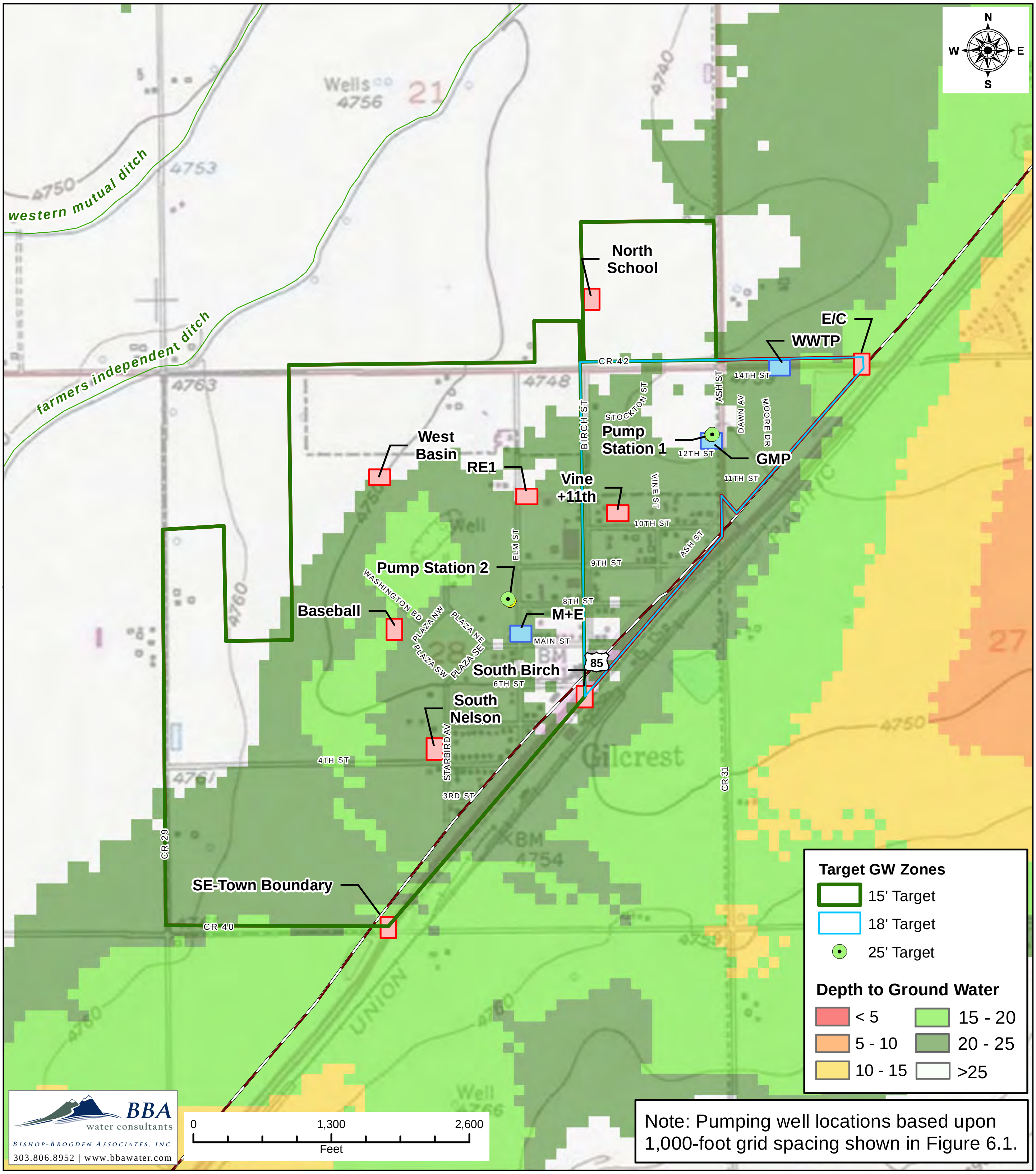
*Legend*

- Town Sewer Pump Station
- Drawdown After 5 Years of Pumping (1 ft. Contour)
- Simulated Dewatering Well Site
- Target Drawdown Location
- Town Boundary

Data Source: CDSS, CDOT, USGS, BLM

Colorado






# FIGURE 6.5
## Town of Gilcrest Projected Depth to Ground Water After 1 Year of Pumping (Optimized)

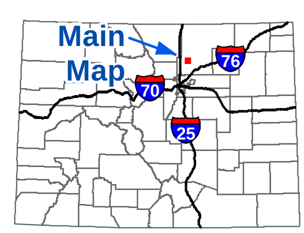
Date: 8/25/2016 | Job No. 1606.00

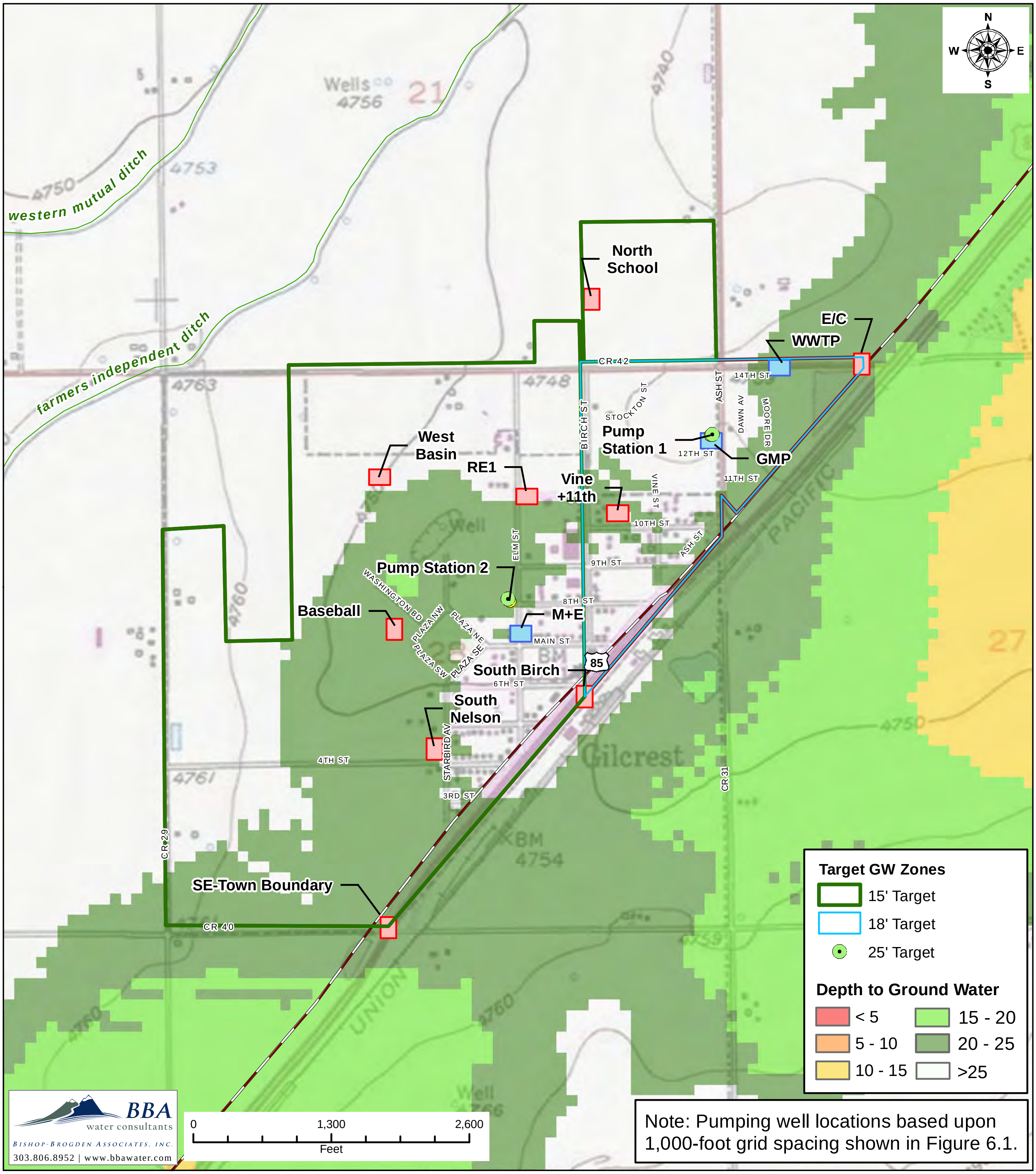
*Legend*

- Town Sewer Pump Station
- Simulated Dewatering Well Site
- Target Drawdown Location
- Town Boundary

Data Source: CDSS, CDOT, USGS, BLM

Colorado





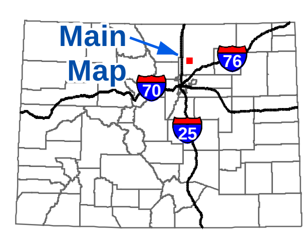

# FIGURE 6.6
## Town of Gilcrest Projected Depth to Ground Water After 5 Years of Pumping (Optimized)

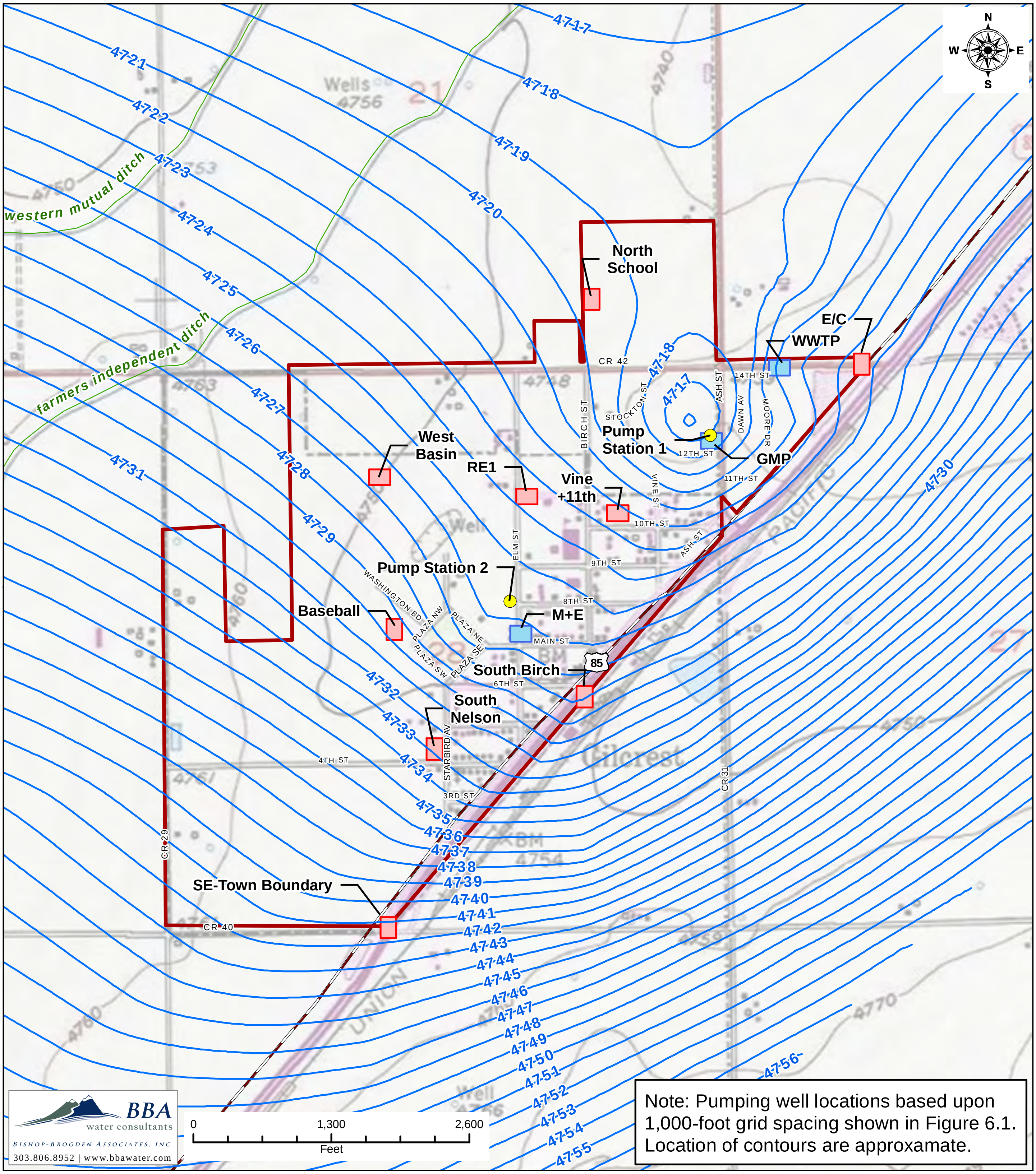
Date: 8/25/2016 | Job No. 1606.00

*Legend*

- Town Sewer Pump Station
- Simulated Dewatering Well Site
- Target Drawdown Location
- Town Boundary

Data Source: CDSS, CDOT, USGS, BLM






# FIGURE 6.7
## Town of Gilcrest Projected Ground Water Elevation Contour After 1 Year of Pumping (Optimized)

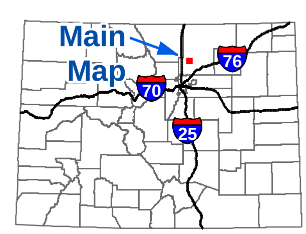
Date: 9/20/2016 | Job No. 1606.00

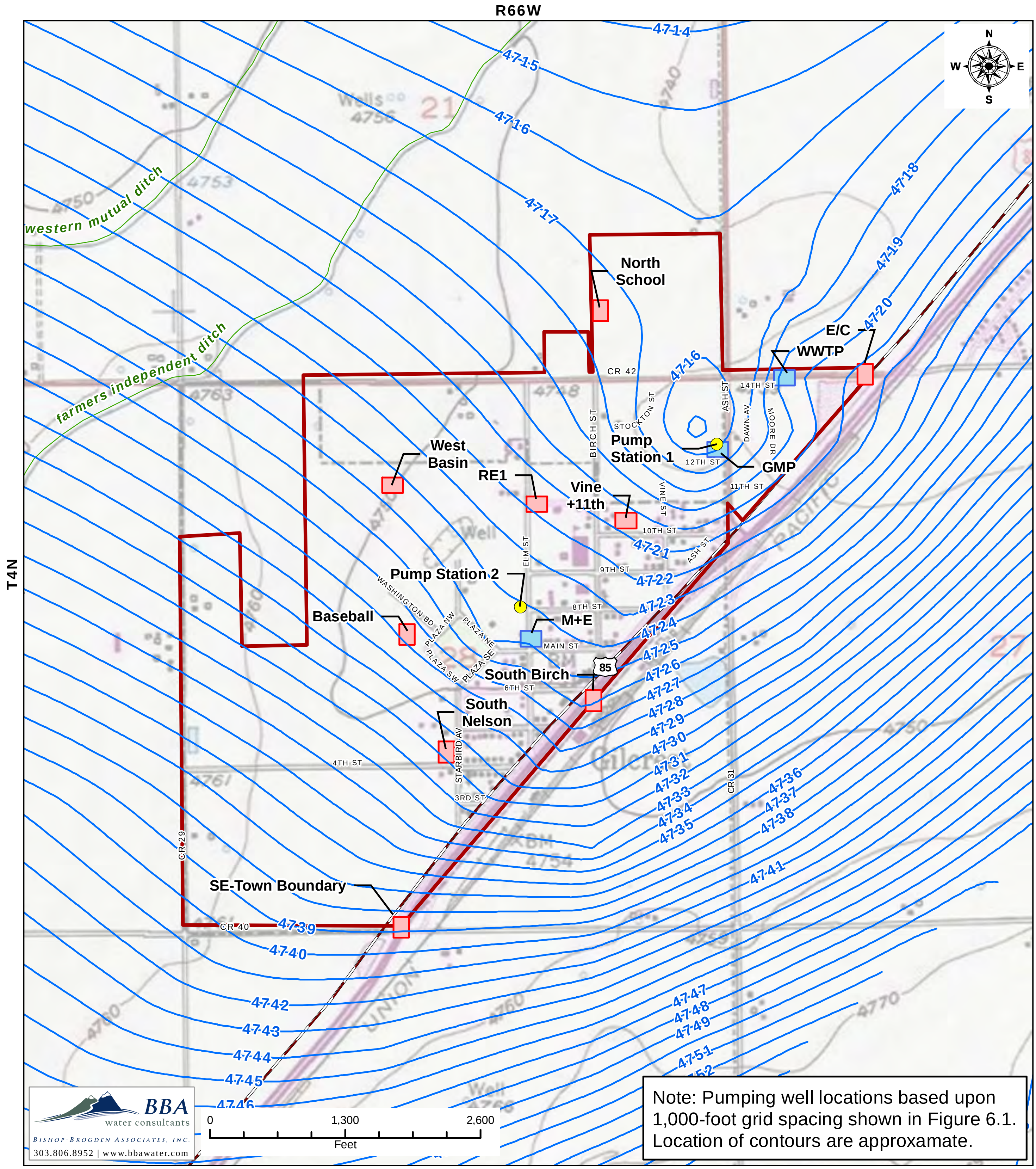
*Legend*

- Town Sewer Pump Station
- Ground Water Elevation (1 ft. Contour)
- Simulated Dewatering Well Site
- Target Drawdown Location
- Town Boundary

Data Source: CDSS, CDOT, USGS, BLM

**Colorado**






# FIGURE 6.8

## Town of Gilcrest Projected Ground Water Elevation Contour After 5 Years of Pumping (Optimized)

Date: 9/20/2016 | Job No. 1606.00

*Legend*

- Town Sewer Pump Station
- Ground Water Elevation (1 ft. Contour)
- Simulated Dewatering Well Site
- Target Drawdown Location
- Town Boundary

Data Source: CDSS, CDOT, USGS, BLM

**Colorado**

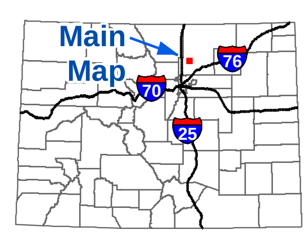
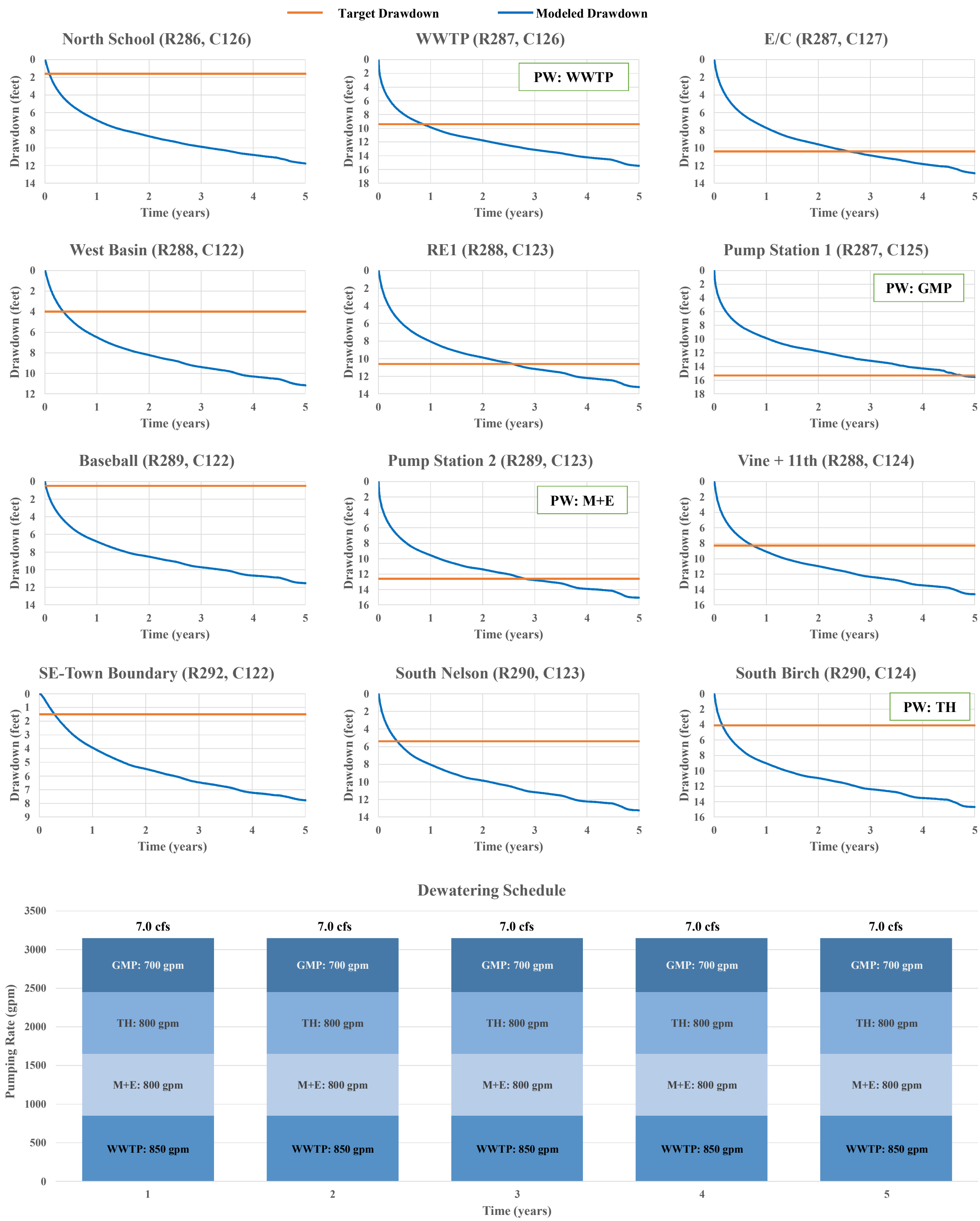


**Figure 6.9**
**Town of Gilcrest**
**Summary of Modeled Drawdown and Pumping Rates (Existing)**




**Notes:**
1) Drawdown is calculated as the difference between model simulated head elevations for historical (without dewatering) conditions and with dewatering conditions.
2) The modeled dewatering schedule is held at constant rates for one-year blocks.
3) Values presented above each column on the Dewatering Schedule graph represent the combined total dewatering rate for all three wells in cubic feet per second (cfs).
4) For spatial relationships between targeted locations, see map in Figure 6.1.

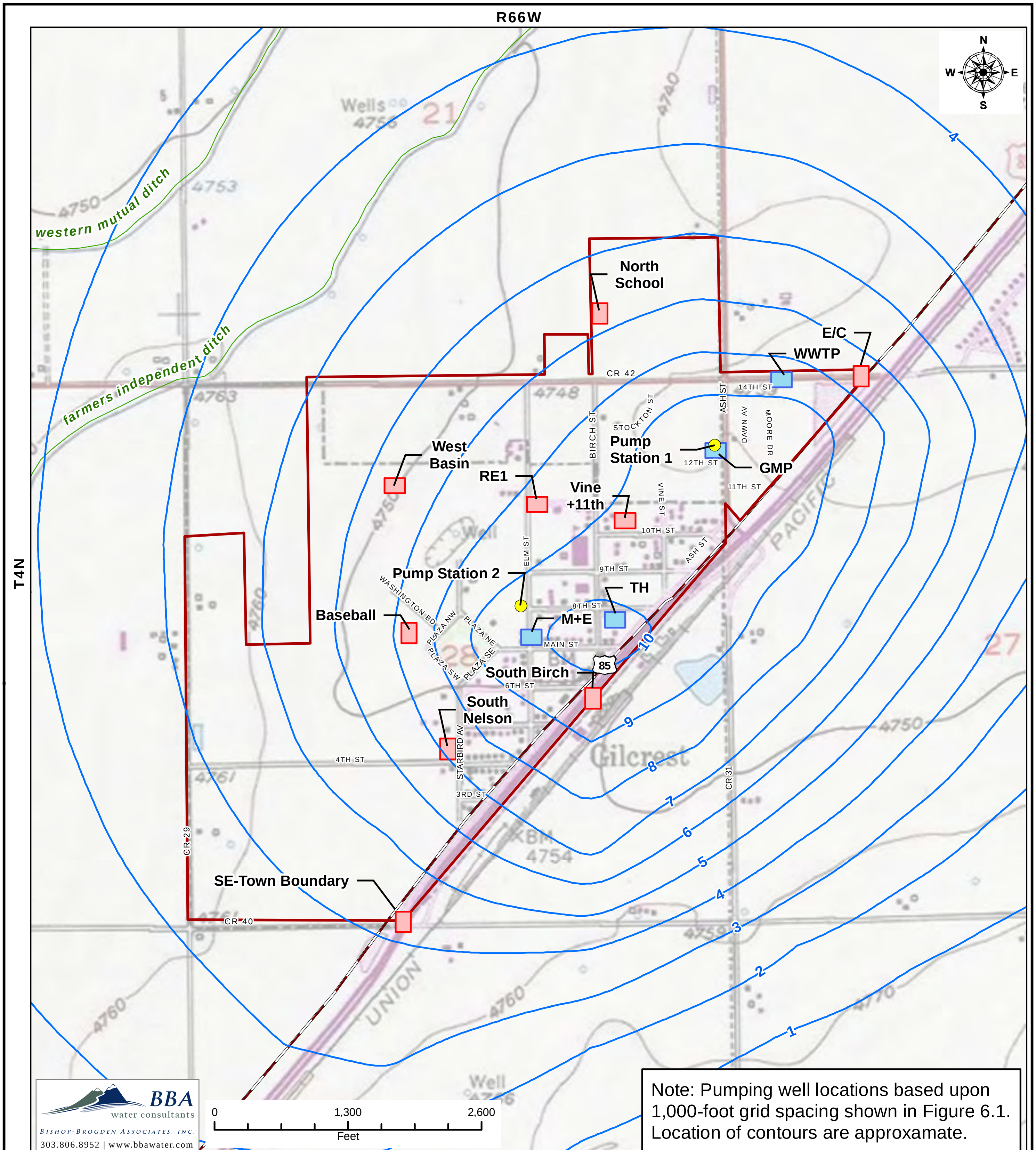


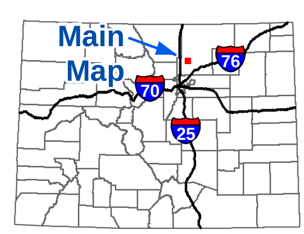

**FIGURE 6.10**
**Town of Gilcrest Projected Ground Water Level Drawdown After 1 Year of Pumping (Existing)**

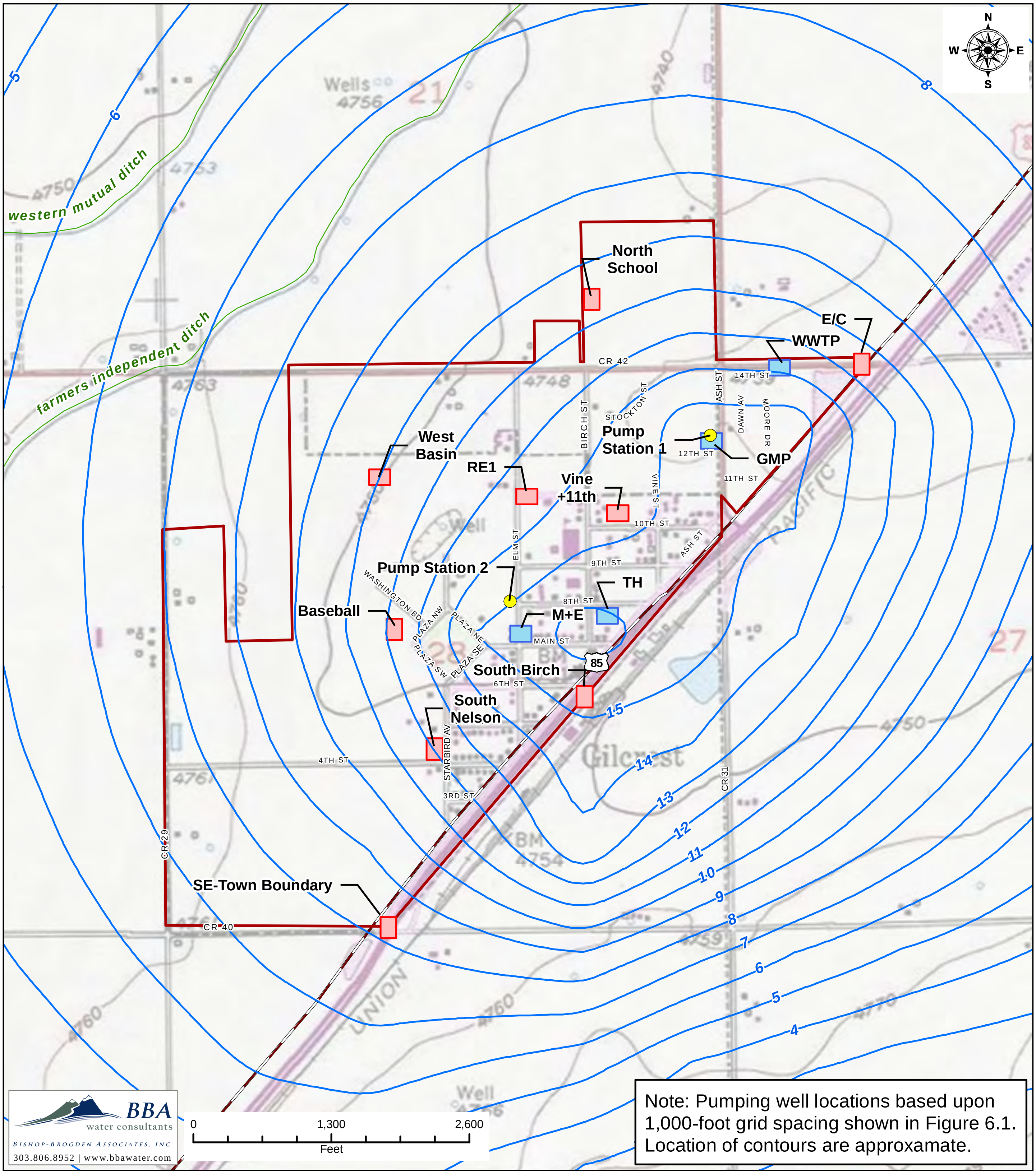
Date: 8/25/2016 | Job No. 1606.00

*Legend*

- Town Sewer Pump Station
- Drawdown After 1 Year of Pumping (1 ft. Contour)
- Simulated Dewatering Well Site
- Target Drawdown Location
- Town Boundary

Data Source: CDSS, CDOT, USGS, BLM






# FIGURE 6.11

## Town of Gilcrest Projected Ground Water Level Drawdown After 5 Years of Pumping (Existing)

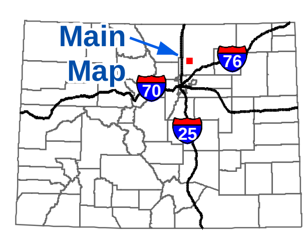
Date: 8/25/2016 | Job No. 1606.00

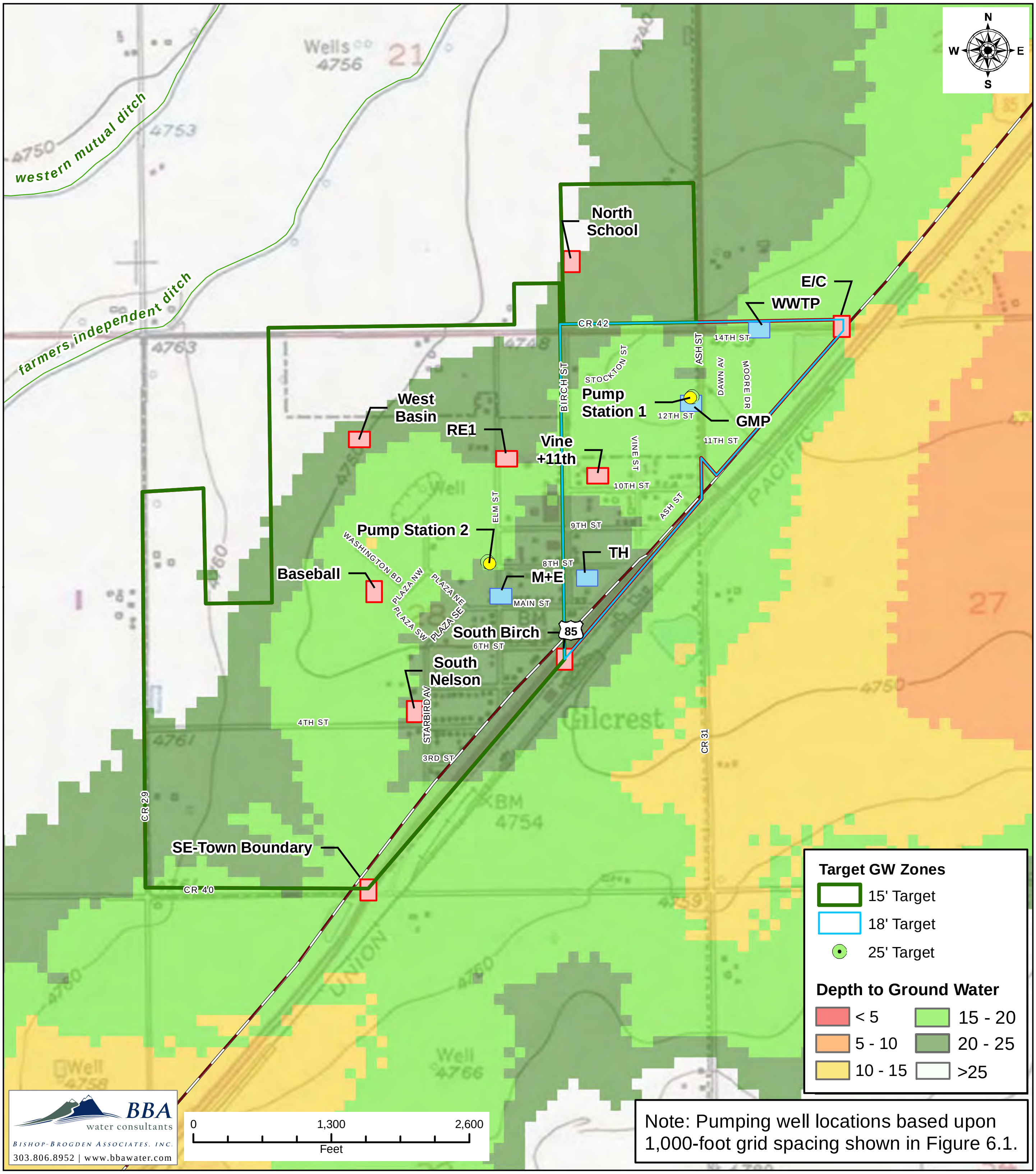
*Legend*

- Town Sewer Pump Station
- Drawdown After 5 Years of Pumping (1 ft. Contour)
- Simulated Dewatering Well Site
- Target Drawdown Location
- Town Boundary

Data Source: CDSS, CDOT, USGS, BLM

## Colorado






# FIGURE 6.12
## Town of Gilcrest
## Projected
## Depth to Ground Water
## After 1 Year of Pumping
## (Existing)

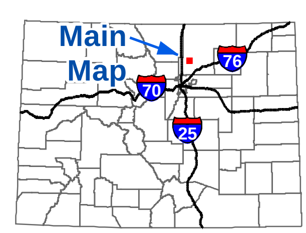
Date: 9/9/2016 | Job No. 1606.00

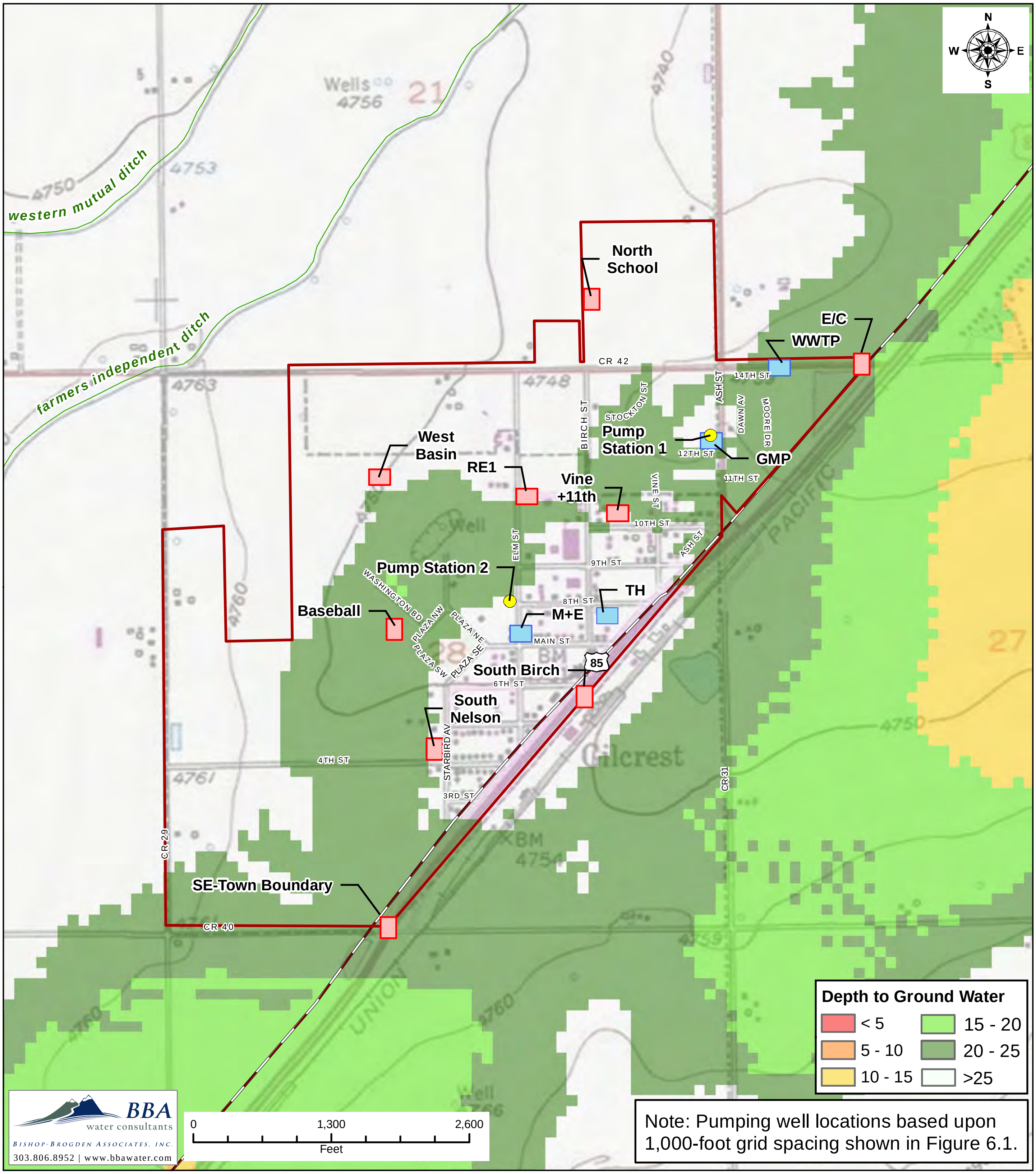
*Legend*

- Town Sewer Pump Station
- Simulated Dewatering Well Site
- Target Drawdown Location
- Town Boundary

Data Source: CDSS, CDOT, USGS, BLM

**Colorado**





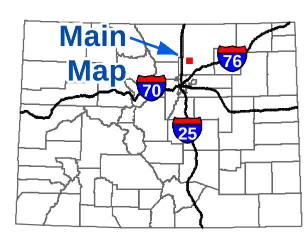

# FIGURE 6.13
## Town of Gilcrest Projected Depth to Ground Water After 5 Years of Pumping (Existing)

Date: 9/9/2016 | Job No. 1606.00

*Legend*

- Town Sewer Pump Station
- Simulated Dewatering Well Site
- Target Drawdown Location
- Town Boundary

Data Source: CDSS, CDOT, USGS, BLM



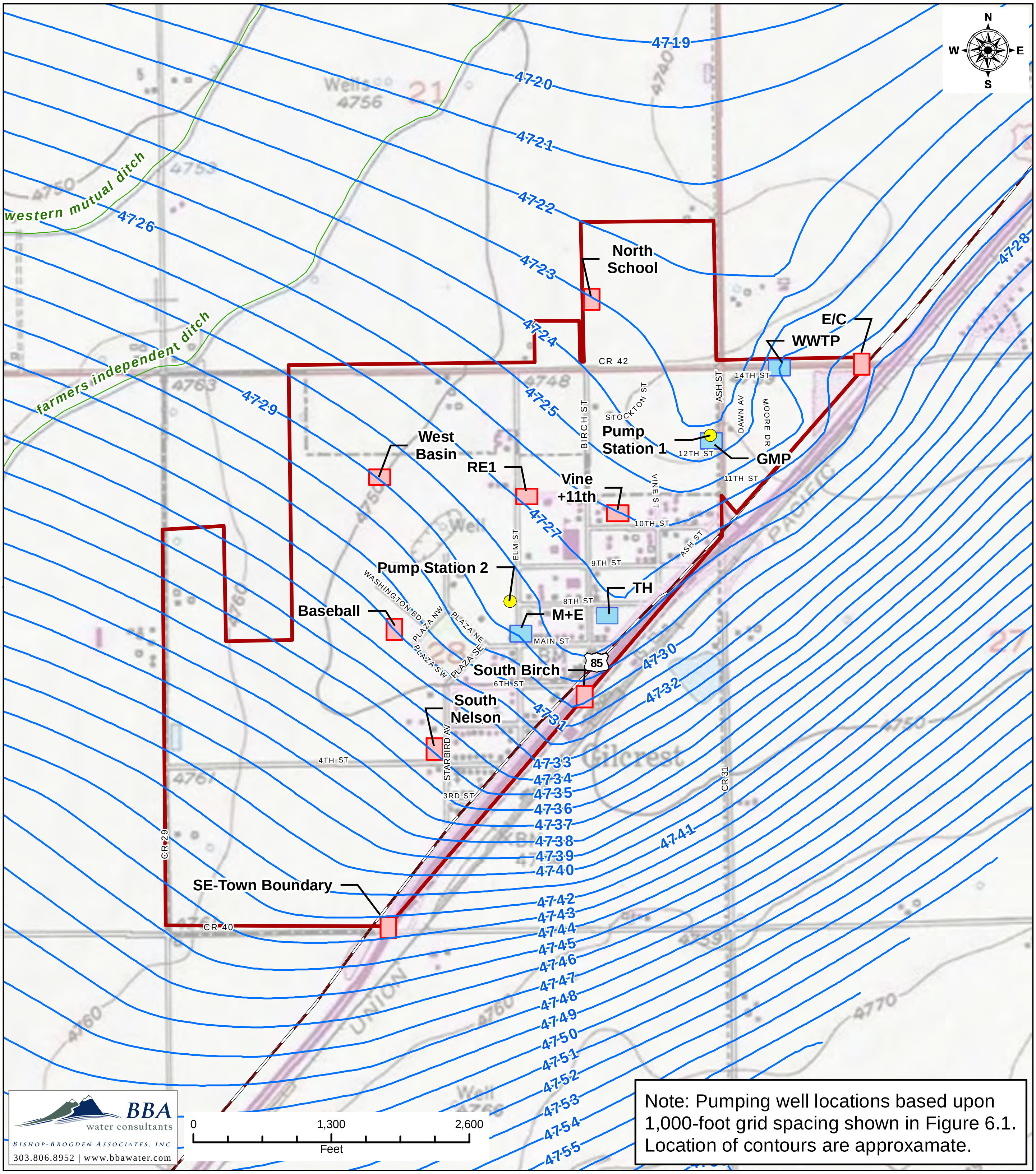



**FIGURE 6.14**
**Town of Gilcrest**
**Projected Ground Water Elevation Contour After 1 Year of Pumping (Existing)**

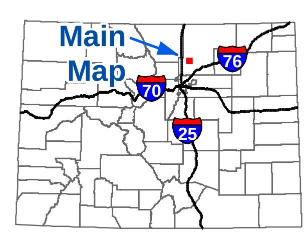
Date: 9/20/2016 | Job No. 1606.00

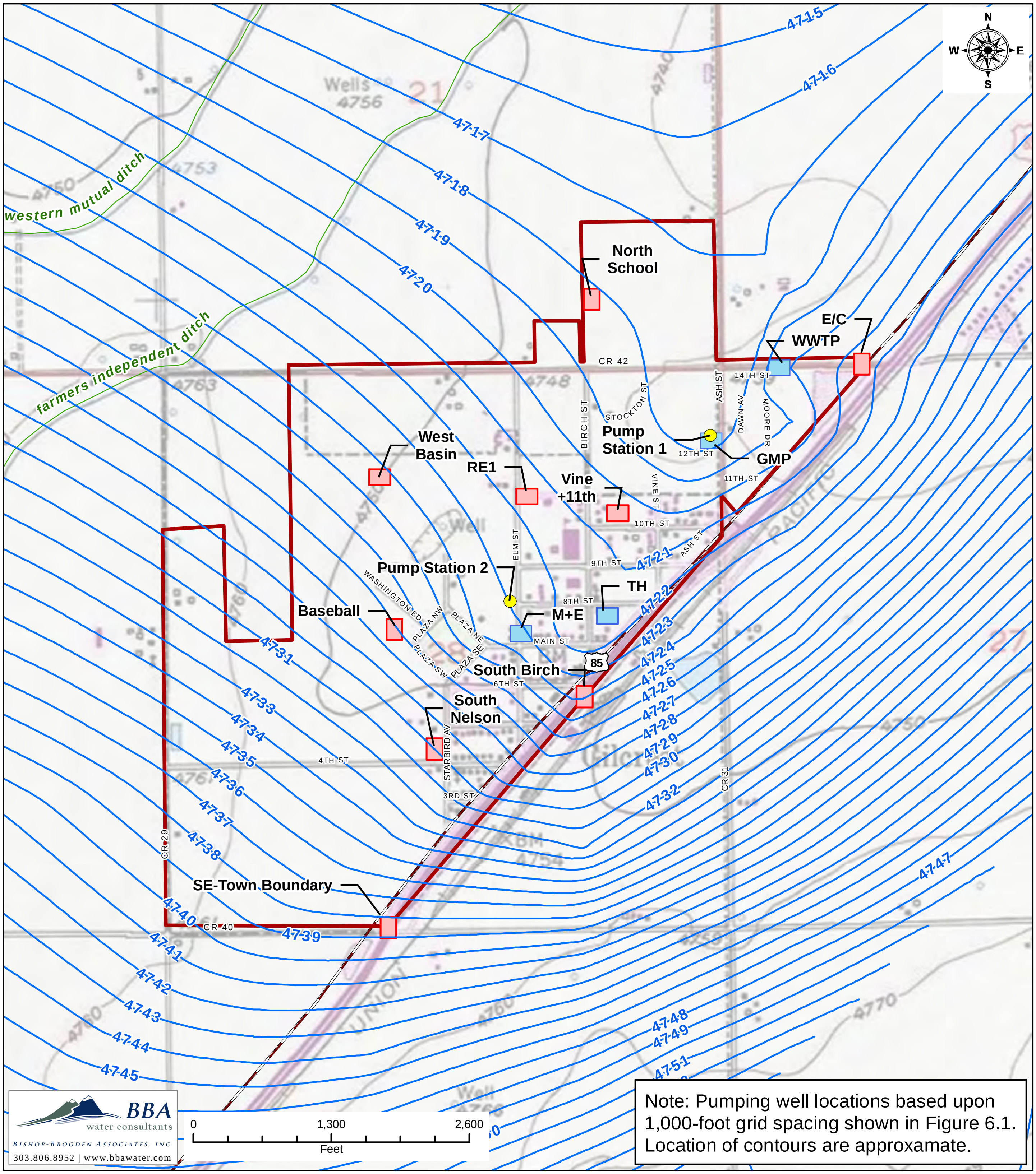
*Legend*

- Town Sewer Pump Station
- Ground Water Elevation (1 ft. Contour)
- Simulated Dewatering Well Site
- Target Drawdown Location
- Town Boundary

Data Source: CDSS, CDOT, USGS, BLM

**Colorado**





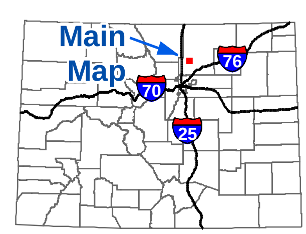

# FIGURE 6.15
## Town of Gilcrest Projected Ground Water Elevation Contour After 5 Years of Pumping (Existing)

Date: 9/20/2016 | Job No. 1606.00

*Legend*

- Town Sewer Pump Station
- Ground Water Elevation (1 ft. Contour)
- Simulated Dewatering Well Site
- Target Drawdown Location
- Town Boundary

Data Source: CDSS, CDOT, USGS, BLM



## Summary of Model Results and Interim Alternative

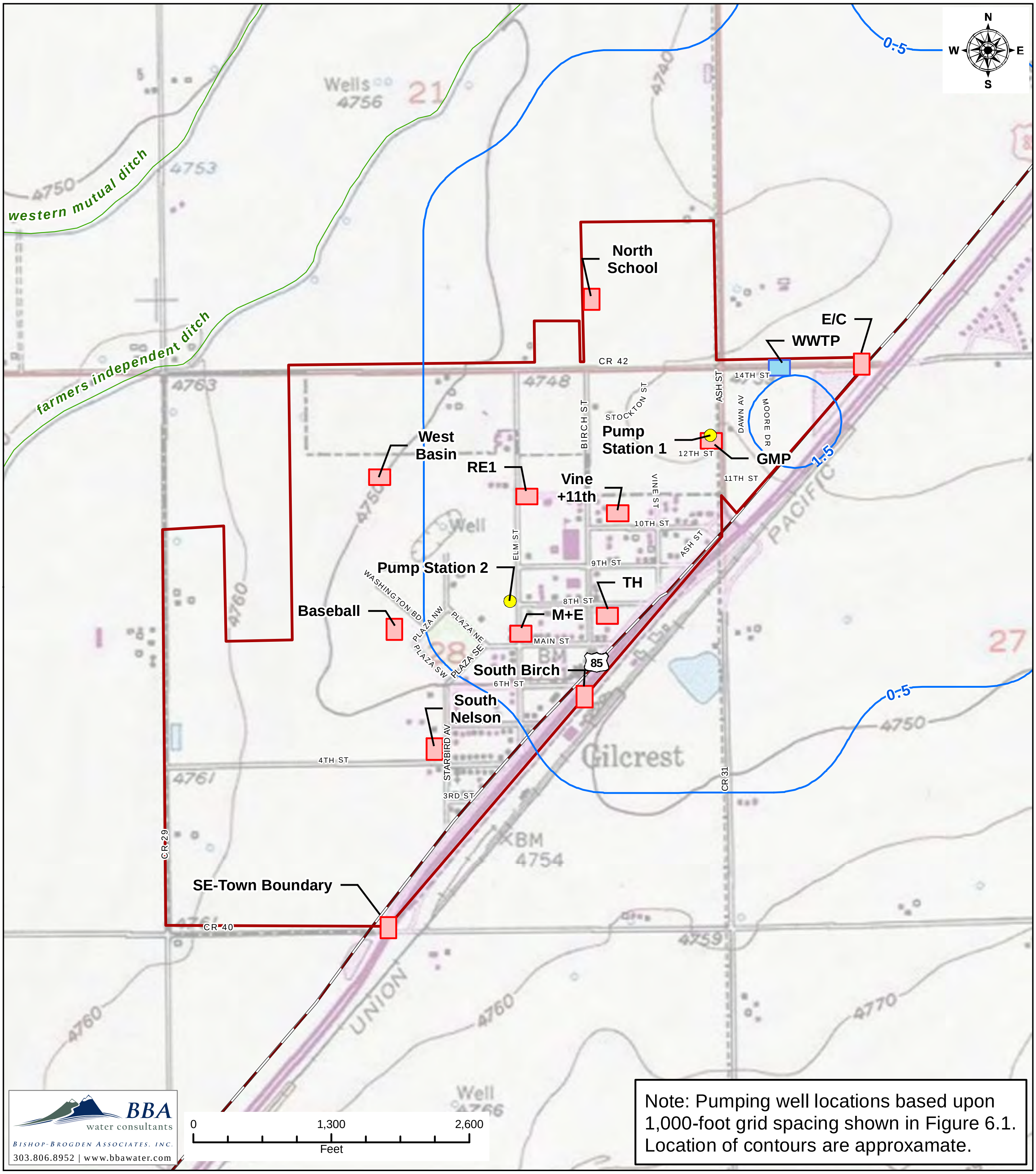
The SPDSS MODFLOW model results indicate that that the ideal network of dewatering wells is located near the eastern Town boundary. Both the "Optimized" and "Existing" well scenarios indicate that a long-term pumping rate of approximately 7 cfs is needed to achieve the Town's dewatering targets. The "Optimized" well configuration is preferable because the Town's existing wells are aging and would likely require replacement in the near-term.

We note that there is substantial uncertainty in the aquifer characteristics in Town. If the alluvial aquifer in Town has a transmissivity lower than 300,000 gpd/ft, lower pumping rates will be needed to achieve the drawdown targets. Conversely, if the alluvial aquifer in Town has a transmissivity higher than 300,000 gpd/ft, higher pumping rates will be needed to achieve drawdown targets. The fact that the Town suffers from shallow ground water conditions is an indication that the aquifer may be less transmissive than down-gradient areas. Ideally, controlled pumping tests would be performed on Town wells to confirm aquifer characteristics, however with no place to discharge and dispose of pumping test water, such tests may not yield useful drawdown results.

Recently, the Town has depended on pumping at the Lorenz well to alleviate the worst of shallow ground water impacts at the WWTP. Even one or two feet of water level drawdown can reduce problems with floating WWTP lagoon liners.

An analysis was completed to determine the amount of ground water level drawdown at the WWTP that could be achieved with a new dewatering well and use of excess capacity in the Town's existing 6-inch wastewater/stormwater pipeline. There is currently an annual average of approximately 420 gpm of excess capacity in the wastewater/stormwater pipeline; although at times 100% of capacity is used. Applying a 20% safety factor, an average of 336 gpm of ground water could be discharged through that pipeline to the South Platte River from a new well located near the western boundary of the WWTP. The controls for such pumping would need to be sophisticated, such that the well pumping rate would be adjusted by a variable frequency drive depending on pressure in the pipeline.

The results of the WWTP well analysis are summarized in Figures 6.16 and 6.17 and show that ground water levels could be lowered approximately 1 to 2 feet at the WWTP, which is a similar impact to Lorenz well pumping. Greater drawdown is expected in the immediate vicinity of the dewatering well. Advantages of installing a dewatering well at the WWTP include: (1) the WWTP well is part of the long-term dewatering solution, (2) data gathered during initial operation will allow the Town to evaluate aquifer characteristics and refine the permanent dewatering well configuration and (3) the WWTP well would provide a means to dewater using Town-owned infrastructure.




# FIGURE 6.16
## Town of Gilcrest Projected Ground Water Level Drawdown After 1 Year of Pumping (WWTP 336)

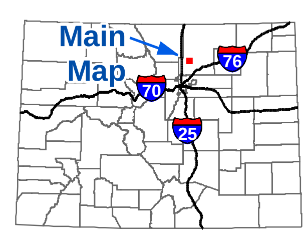
Date: 9/20/2016 | Job No. 1606.00

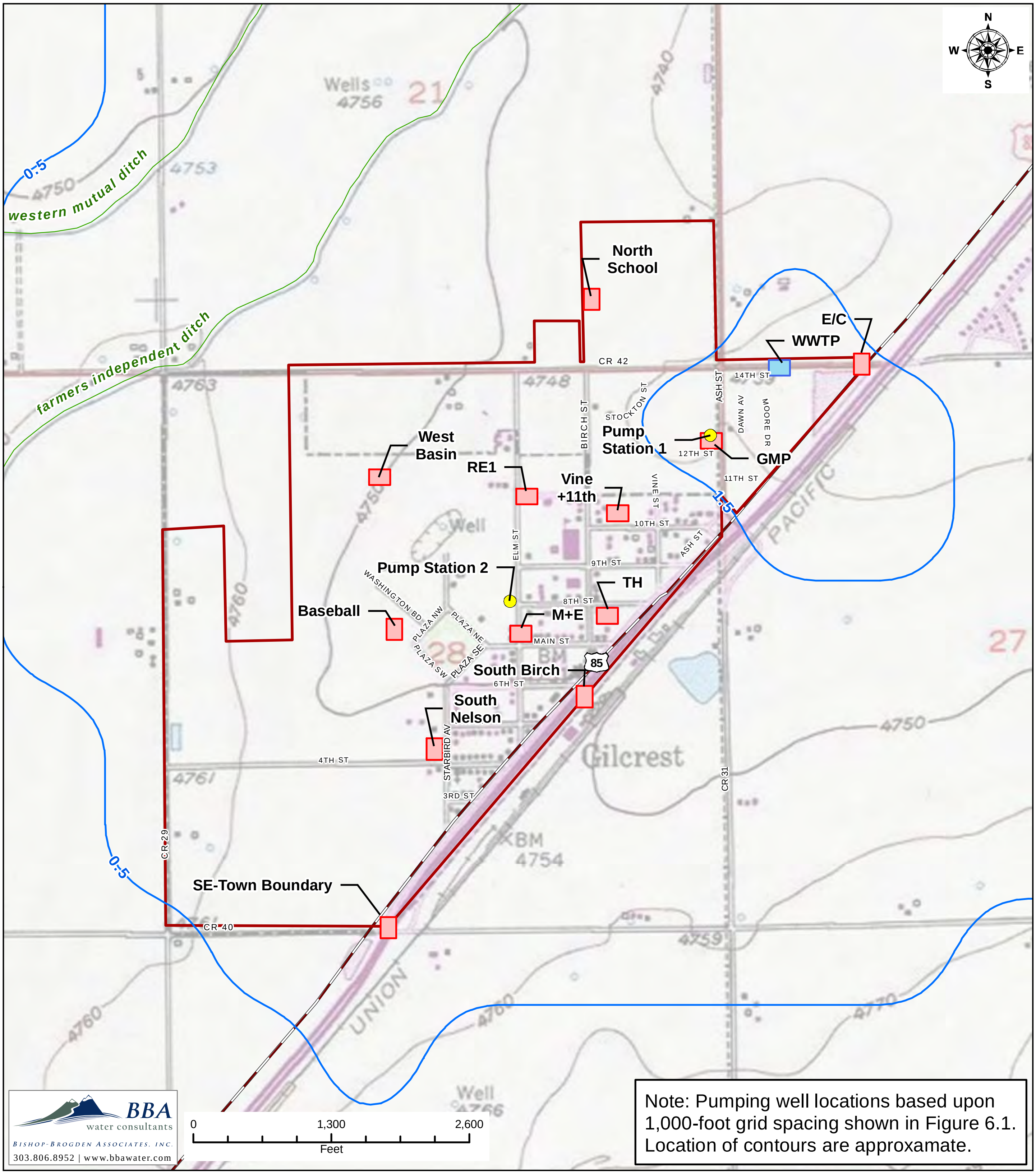
*Legend*

- Town Sewer Pump Station
- Drawdown After 1 Year of Pumping (1 ft. Contour)
- Simulated Dewatering Well Site
- Target Drawdown Location
- Town Boundary

Data Source: CDSS, CDOT, USGS, BLM

Colorado





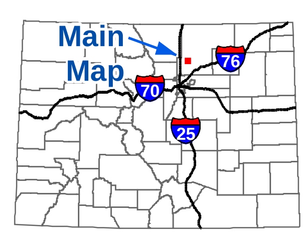

# FIGURE 6.17
## Town of Gilcrest Projected Ground Water Level Drawdown After 5 Years of Pumping (WWTP 336)

Date: 9/20/2016 | Job No. 1606.00

*Legend*

- Town Sewer Pump Station
- Drawdown After 5 Years of Pumping (1 ft. Contour)
- Simulated Dewatering Well Site
- Target Drawdown Location
- Town Boundary

Data Source: CDSS, CDOT, USGS, BLM



# SECTION 7 – IMPACT ON CONTAMINANT PLUMES AND RECOMMENDED GROUNDWATER MONITORING SYSTEM

## IMPACT ON CONTAMINANT PLUMES

To evaluate the potential impact of the dewatering alternatives on the existing Benzene and Nitrate contaminant plumes underlying portions of the Town, we communicated with Michael Critchley of Palmetto and April Hussey of Tetra Tech that are completing remediation of the contaminant plumes to present conceptual dewatering systems and projected effects on groundwater gradient from the groundwater modeling results. The contaminant plumes are generally located near Main Street on the east side of Town. No figure has been included of the containment plumes' location, as ongoing studies are currently in progress and final figures were not available from the respective consultants studying these flumes. Prior to this study, the initial indication from these individual was that lowering the groundwater table may enhance contaminant remediation efforts. Following these initial communications, recommendations are as follows.

- Generally, dewatering that is concentrated down-gradient of contaminant plumes is not expected to cause new contamination concerns. Since the majority of dewatering pumping is concentrated at the GMP and WWTP locations, there is limited concern for impact to remediation efforts.
- Pumping at the Main and Elm well may cause unwanted plume migration and will need further, future review. However, dewatering at the Main and Elm location is critical to achieve water level targets at the Wastewater Pump Station No. 2.
- Prior to implementing a final design, we recommend collecting additional local aquifer characteristic data in the vicinity of the plumes and completing a focused contaminant-transport modeling evaluation.
- Due to presence of contaminant plumes, water quality monitoring of dewatering discharge is recommended and may be required by the CDPHE discharge permit.

## TREATMENT OF GROUNDWATER

Aside from the contaminant plumes, naturally occurring groundwater may require treatment to surface water quality before it is discharged to the South Platte. Treatment of groundwater could incur a substantial cost to the Town. Typical treatments may include reduction of metals and nitrates, depending on permit requirements. While excluded from this evaluation, the capital cost of treatment could range from zero dollars to over ten million dollars depending on the level of treatment required (if any). Prior to implementing a final design, it is recommended that the Town collect additional water quality data in the vicinity of the dewatering wells. Due to presence of contaminant plumes, water quality monitoring of dewatering water discharge is recommended and may be required by the CDPHE discharge permit.

Regardless of the option chosen, the groundwater level monitoring system described below was used to develop specific, measurable groundwater level targets at which the project will be "successful." The 18 and 25 feet targets identified by the Town for the specific locations of the pump stations and certain sewer line area are the key drivers of the location and amount of dewatering pumping necessary for a successful dewatering system. The need for dewatering will be permanent due to the groundwater budget and hydrogeological setting of the Town, unless there are dramatic changes in local water use (e.g. increased well pumping, decreased recharge, decreased irrigation).

The recommended network of groundwater level monitoring wells within the Town to measure success of the dewatering system is shown in Figure 7.1 and summarized in Table 7.1. The groundwater level monitoring network utilizes wells already being monitoring in the Town's monitoring system and includes several proposed new monitoring well locations, as shown in Figure 7.1. The critical monitoring wells to measure success of the dewatering system include E/C, S/C, RE1, Town Hall and new monitoring wells located near Sewer Pump Station Nos. 1 and 2. These critical monitoring wells are located near target depth to water boundaries and represent areas in which the water table is closest to ground surface. Other wells in the Town's monitoring program should continue to be monitored to track the change in hydraulic gradient through the town.

New monitoring wells near Sewer Pump Station Nos. 1 and 2 will be useful for future groundwater monitoring efforts, in addition to transducer data from a recently drilled CSU monitoring well ("MW-7") monitored by Dr. Ryan Bailey and located in the northwest corner of Section 28. The estimated cost for a new monitoring well used in the monitoring system is approximately $2,600, based on an average well depth of 90 feet and 2-inch PVC casing. An example design for a new monitoring well is presented in Figure 7.2.

**Table 7.1 – Recommended Monitoring Well Network**

| Location | Frequency of Measurement |
|---|---|
| **Existing Wells** | |
| E/C | Daily (Transducer), Weekly (Manual) |
| TH | Daily |
| M+E | Weekly |
| GMP | Weekly |
| RE1 | Weekly |
| 5th Street | Weekly |
| Nelson | Weekly |
| W/W | Weekly |
| S/C | Weekly |
| **Proposed New Wells** | |
| Sewer Pump Station 1 | Weekly |
| Sewer Pump Station 2 | Weekly |


# FIGURE 7.1
# Town of Gilcrest Recommended Monitoring Well Network

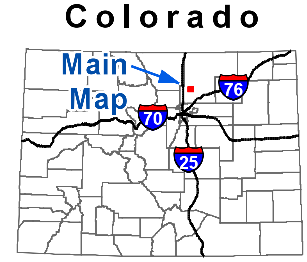
Date: 9/20/2016 | Job No. 1606.00

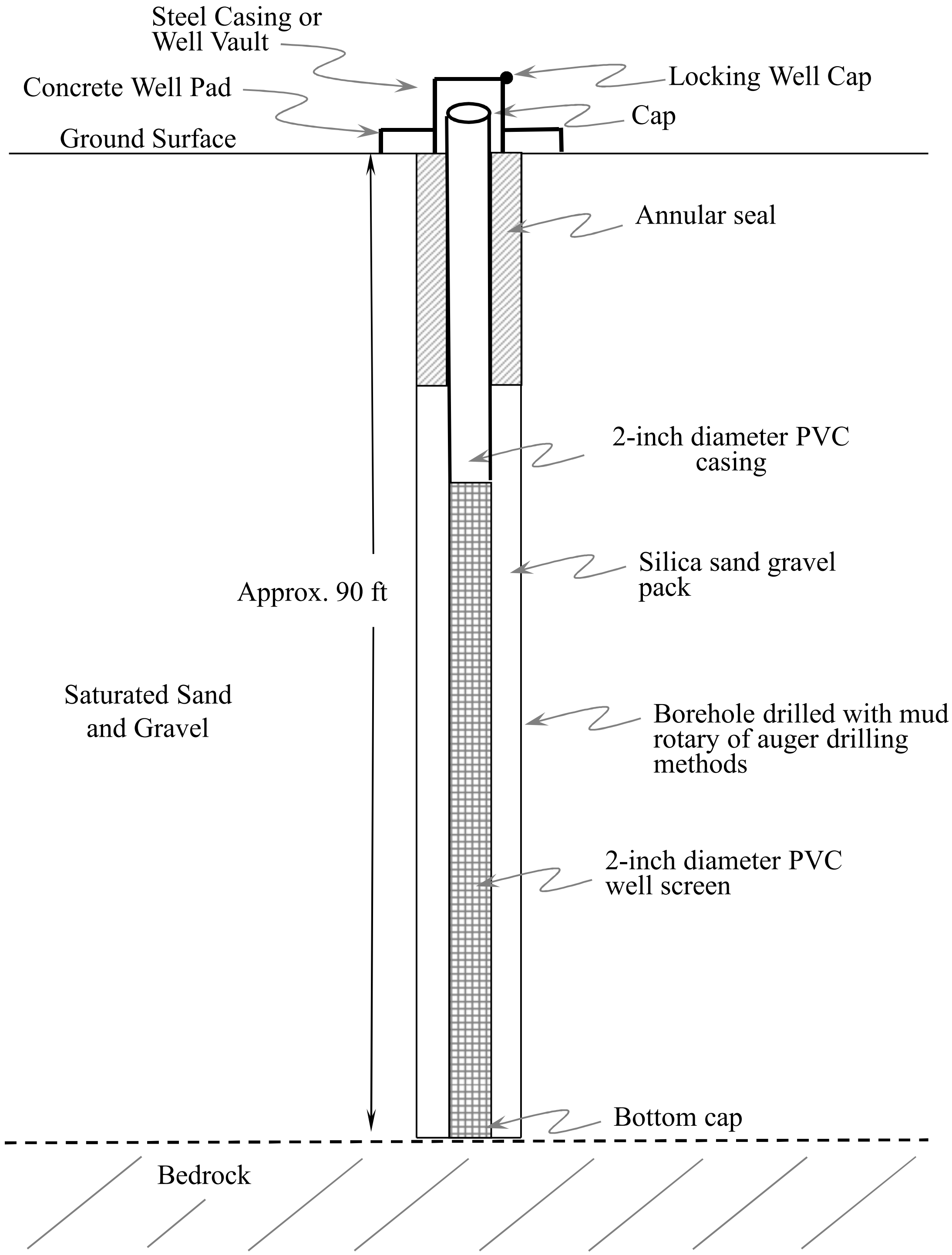
*Legend*

- New Proposed Monitoring Wells
- Existing Town Monitoring Wells
- Town Boundary

Aerial Photo Date: 6/19/14
Data Source: Town of Gilcrest, BBA, CDSS, CDOT, USGS, BLM

Colorado






Notes: See Colorado Water Well Construction Rule 14.

| | | |
|---|---|---|
| Date: September 19, 2016 | Figure 7.2<br>Monitoring Well Construction | BBA water consultants<br>Bishop-Brogden Associates, Inc.<br>333 W. Hampden Ave., Suite. 1050<br>Englewood, CO 80110<br>(303) 806-8952<br>www.bbawater.com |
| Job No.: 1606.00 | | |
| By: KE | | |

# SECTION 8 – STORMWATER EVALUATION

## STORMWATER AND AGRICULTURAL RUNOFF

### BACKGROUND

In addition to  groundwater issues, the Town experiences periodic surface flooding from storm events. There is currently minimal underground stormwater infrastructure throughout Town with concentrations of systems around the wastewater treatment plant in the northeast part of Town. Three stormwater detention ponds are located near the wastewater plant. A separate stormwater pond lies a block to the west in the Town park with no discernable gravity outfall and typically acts as a retention pond. Stormwater from the Town park is pumped to a roadside ditch on County Road 42 which flows east into the storm ponds located at the wastewater treatment facility. A large stormwater retention pond at the southwest side of Town was constructed in 2010 and expanded in 2011.

### PREVIOUS STUDIES

There were several previous drainage studies completed for the Town that have been referenced as part of this current stormwater evaluation. The 2003 Comprehensive Plan provided several recommendations for addressing stormwater detention within Town and potential offsite flows from the southeast. The plan also indicated the need for improved stormwater conveyance systems throughout the Town to improve flooding concerns during major storm events. However, until there is a detention pond or other location to receive these minor storm flows, a conveyance system will provide minimal flooding relief. The drainage related information in the 2003 Comprehensive Plan is detailed within the Master Drainage Plan prepared by RG Consulting Engineers, Inc., dated December 2003.

The 2003 Comprehensive Plan suggested the need for a 138 ac-ft pond located near Liberty Park that would alleviate some of the flooding potential caused by runoff entering the Town from the southwest and southeast. A design summary memo from 2010 prepared by Ketterling, Butherus & Norton Engineers, LLC indicated that a 32 ac-ft pond would be constructed with a sloped bottom and confirmed with as-built contours in late 2010. The pond was modified in 2011 by Tetra Tech, but the record drawings don't indicate a volume achieved with the modification. Digitizing the contours from the record drawings yields a storage volume of approximately 49 ac-ft at a water surface elevation of 4745.5, leaving a foot to the spillway for freeboard. There is no recommendation in the previous studies of detaining for stormwater flows crossing U.S. Highway 85 from the southeast.

### EXISTING STORMWATER SYSTEM EVALUATION

The various stormwater ponds around Town and associated proposed conveyance systems were hydraulically modeled using Autodesk Storm and Sanitary Analysis 2016 that is based on the EPA SWMM software. Due to the size of the basins, stormwater runoff was calculated using

Colorado Urban Hydrograph Procedure (CUHP). As the input hydrographs were missing from the Master Drainage Plan, the existing input parameters outlined in the Master Drainage Plan were used to recreate the input hydrographs for the stormwater/groundwater model. Existing topography used for basin delineation was taken from the latest USGS digital elevation model available. Groundwater flows of approximately 9.7 cfs used in the hydraulic model are based on discussions in Section 6.

The same basin designations as those noted in the 2003 Comprehensive Plan are being used in this stormwater evaluation. Basin 300 was broken into two separate basins to differentiate between outfall points. Basin 301 represents flows on the west side of previous basin 300 that drain to the pond in the Town park adjacent to 14th Street. Basin 302 represents flows on the east side of basin 300 that drain to the three stormwater ponds near the wastewater treatment plant. Basin 200 remains the same and drains to the recently constructed retention pond on the southwest side of Town. The basins are shown in Figure 8.1.

The CUHP input parameters noted in Table 2 of the 2003 Master Drainage Plan were used to recreate the input hydrographs for this updated stormwater/groundwater model. The area for Basin 200 was increased approximately 3% based on current USGS topography and Basin 300 remains the same size as listed in the Master Drainage Plan. All of the remaining basin characteristics were kept the same. Based on this data, 100-year peak runoff for the basins is consistent with what is listed in the Master Drainage Plan and summarized below. A summary of 100-year peak runoff is provided below in Table 8.1.

**Table 8.1 – 100-Year Peak Runoff Comparison (Future Buildout Conditions)**

| Basin Name | JVA Modeling | 2003 Comprehensive Plan |
|---|---|---|
| Basin 200 | 803 cfs | 825 cfs |
| Basin 301 | 217 cfs | - |
| Basin 302 | 108 cfs | - |
| Basin 300 (Total) | 325 cfs | 328 cfs |

Based on the updated hydraulic modeling in this report, the current retention pond for Basin 200 is approximately 3 times too small to contain the 100-year storm event. Excess flows are modelled as overtopping the north side of the pond after approximately 110 minutes into the storm and spilling into open fields south of Hwy 42. Although runoff data between the updated modeling and 2003 reports are similar, there appears to be a 17% increase in volume needed for the existing pond. As the appendices including the detailed model data are missing in the available 2003 report, it is difficult to determine the cause for the increase. When the historic runoff, not accounting for future growth, was modeled, the detention volume is reduced to 135 ac-ft and is more in line with the Master Drainage Plan. A summary of modeled retention volume required versus actual volume available is included in Table 8.2.


FIG 8.1 - STORMWATER BASINS
TOWN OF GILCREST
OCTOBER 2016

CONSULTING ENGINEERS

JVA, Incorporated
1319 Spruce Street
Boulder, CO 80302
Phone: 303.444.1951
E-mail: info@jvajva.com

**Table 8.2 - 100-Year Retention Volume Comparison (Future Buildout Conditions)**

| Basin Name | JVA Modeling | 2003 Comprehensive Plan | Existing Capacity |
|---|---|---|---|
| Basin 200 | 161 ac-ft | 138 ac-ft | 49 ac-ft |
| Basin 301 | 12 ac-ft | - | 4 ac-ft |
| Basin 302 | 5 ac-ft | - | 6 ac-ft |
| Basin 300 (Total) | 17 ac-ft | 23 ac-ft | 10 ac-ft |

The volumes in the updated model do not include the 1.5x multiplier typically recommended by regulators and consultants for retention ponds nor the potential U.S. Highway 85 overflow from the southeast. This 1.5x factor of safety accounts for runoff being infiltrated into the ground at a much slower rate in a retention pond than a typical detention pond that relies upon gravity to empty the pond. As outlined in the next section, it is recommended that the retention ponds be pumped to empty the entire volume in 72 hours to mimic a standard detention basin: as a result, the safety factor may not be required. As the appendices with detailed calculations were missing in the record copy of the 2003 Master Drainage Plan, it is not possible to determine if the previously calculated volumes included the 1.5x safety factor. It is assumed that flows do not cross U.S. Highway 85 from east to west due to the train tracks, although there may be an unidentified culvert which allows some flows to travel across the highway. For the purposes of this report, it is anticipated that the offsite runoff from the southeast that overtops U.S. Highway 85 will be addressed at a later date by Weld County, Colorado Department of Transportation, and Union Pacific Railroad as it is outside of Town limits.

## Stormwater System Recommendations

The 2003 Comprehensive Plan and Master Drainage Plan include an option to drain the western detention pond by an open channel. The open channel would theoretically discharge to a larger detention/retention facility to the east of the High School football field or continue approximately 6 miles to the South Platte River along U.S. Highway 85. This alignment could be problematic with several easements required as well as an approximate 20-foot climb to a ridgeline between U.S. Highway 85 and the South Platte River at County Road 29. Due to this high point, a channel would need to extend north towards La Salle in order to drain by gravity. A gravity alternative appears conceptually possible by connecting under U.S. Highway 85 East to the Big Bend Ditch. This ditch is currently receiving approximately 200 gpm of periodic pumped flows from the Lorenz well immediately east of the Town’s Wastewater Lagoons. However, there are numerous bottlenecks which prohibit the use of this ditch at the flow rates anticipated.

Alternatively, a portion of this alignment would still need to be pumped and possibly for a longer distance to achieve a positive slope in the channel bottom. Underground siphons would likely need to be installed at each of the two existing irrigation ditch crossings. Additional right-of-way may also need to be acquired depending on the limits of the proposed channel grading as it ties back into the existing grades.

The Town park’s detention pond in Basin 301 is currently pumped into an adjacent roadside ditch and flows to the ponds at the wastewater treatment plant serving Basin 302. The pumped discharge from Basin 302 ditch is planned to continue discharging through the shared wastewater treatment plant outfall pipe. If a new drainage alternative were constructed to convey stormwater to the South Platte River, pumped discharge from Basins 301 and 302 could be redirected to the new drainage alternative.

In addition to addressing stormwater concerns, this updated report also provides recommendations to lower groundwater within the Town. As groundwater dewatering will require several pumps spaced around Town, a dual use stormwater/groundwater pumping facility is recommended at the western detention pond to reduce the length of the conveyance system necessary to discharge into the South Platte River.

In order to drain by gravity, a manhole would be installed at the pumped system high point and convert to gravity flow for approximately 2 miles within a 24-inch concrete pipe installed within the right-of-way. Once more detailed topography is provided, the pumped line may need to be extended to avoid the inverts of the two existing irrigation ditch crossings and to reduce the approximately 20 feet of cut necessary at local high points in the proposed alignment required to provide a gravity line to the river. As long as the conveyance system remains within CR 42, limited right-of-way or easement acquisition is anticipated.

In order to drain the existing ponds on the east side of Town, a separate conveyance system would be required to the combined groundwater/stormwater pump facility at the western pond. Based on limited field survey, the existing pond at the Town park adjacent to CR 42 and Ash Street could drain by gravity to the ponds at the wastewater plant. A 24-inch concrete pipe could be installed within the County Road 42 right or way to hydraulically link the eastern ponds. There is currently no gravity outfall pipe for the westernmost detention pond at the wastewater plant. The existing pump at the easternmost stormwater pond may need to be upgraded to an approximate 2 cfs (900 gpm) pump with a six-inch diameter forcemain to drain all hydraulically connected ponds to the new proposed groundwater/stormwater pump facility at the western pond.

All pump systems that drain stormwater ponds should be sized to empty the entire detained volume within 72 hours to comply with water rights regulations. As the existing soils are very porous and with the installation of the groundwater pump facilities, the proposed hydraulic model shows limited impact to stormwater detention volumes when discharging groundwater flows directly into the stormwater ponds. However, in order to avoid recharging pumped groundwater within the Town, it is recommended that any pumped groundwater be prohibited from infiltrating back into the ground within Town Limits. This is discussed further under the dewatering conveyance alternatives section.

# SECTION 9 – CONVEYANCE ALTERNATIVES

## DEWATERING WELL CONVEYANCE

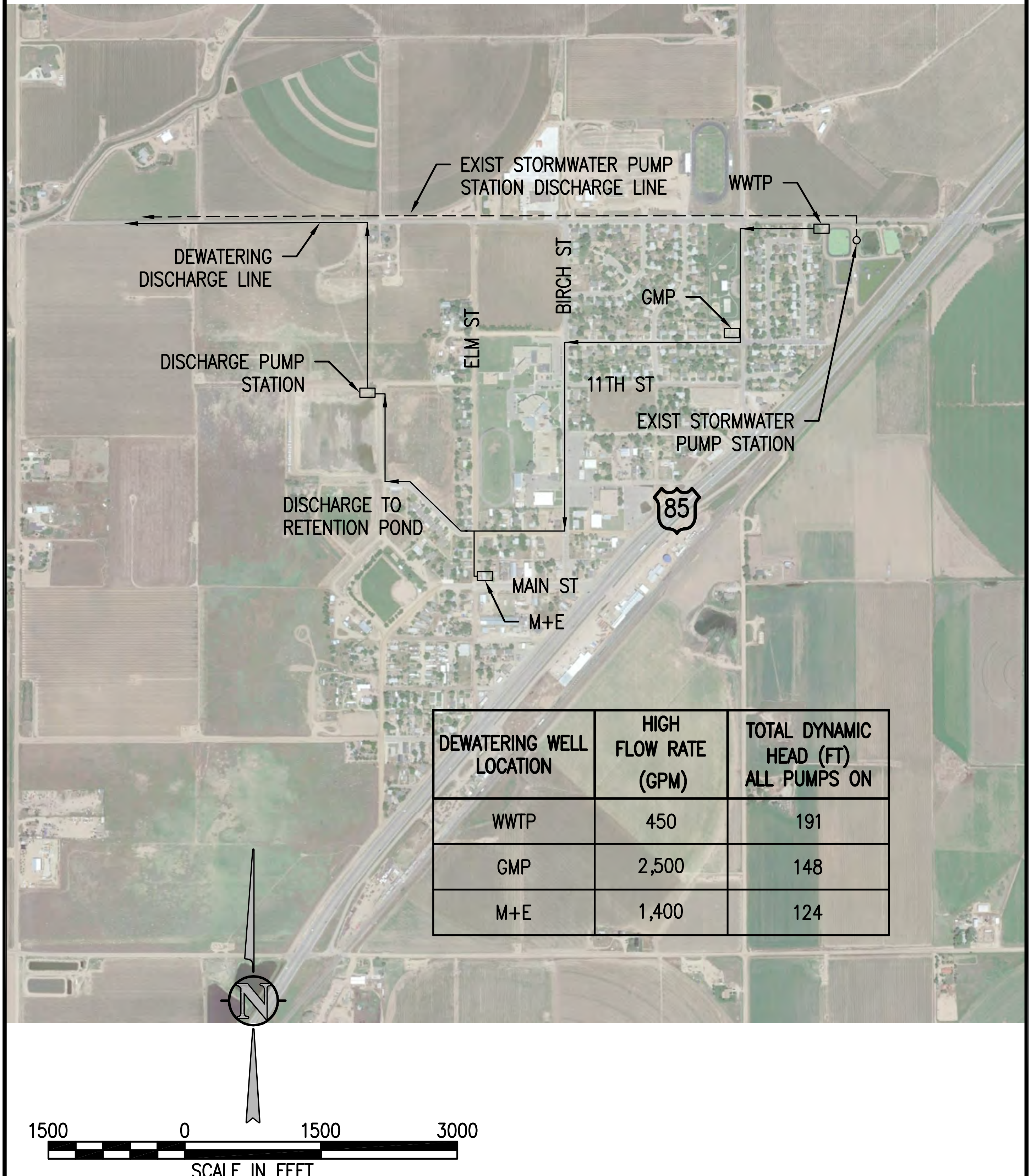
The proposed dewatering well sites, M+E, GMP, and WWTP were assumed to be the three chosen dewatering locations that supplied water to the dewatering pump station for all alternatives assessed. Figure 9.1 shows the proposed wells and pipelines that deliver water to the stormwater retention pond on the west side of Town. The initial dewatering rates were used to size the dewatering pumps as well as the pipeline diameters for each well. It was determined that each well should have its own dedicated pipeline to avoid periods of low velocities in a combined force main when not all wells are running. In Figure 8.1, the initial dewatering flowrates and associated total dynamic head is included for each well. Figure 9.2 shows a map of the dewatering conveyance alternatives discussed in this section.

## GRAVITY DEWATERING

This task was intended to identify practical discharge points for a gravity drainage dewatering system. Although we understand the Town has experienced limited success related to use of existing drainage ditches for conveyance, we identified gravity dewatering as a potentially reliable and low operational cost alternative for a dewatering system. Conceptually, gravity dewatering would operate with horizontal drains constructed at sufficient depth to achieve the dewatering targets. Groundwater collected by such drains would be discharged by gravity to a point where the land surface elevation is lower than the deepest target groundwater elevation beneath the Town, after accounting for head losses in the dewatering and conveyance system.

A gravity alternative appears conceptually possible by connecting under U.S. Highway 85 east to the Big Bend Ditch. This ditch is currently receiving approximately 1,300 gpm of periodic pumped flows from the Lorenz well immediately east of the Town's Wastewater Lagoons. However, there are numerous bottlenecks which prohibit the use of this ditch at the flow rates anticipated.

The advantage of gravity dewatering is that it would not require ongoing well pumping costs. Unfortunately, even before accounting for head losses, the nearest location where the land surface is below the deepest target groundwater level elevation of 4718 feet (at the $12^{th}$ and Ash Wastewater Pump Station) is approximately 6,920 feet northwest of Town near the intersection of County Roads 44 and 29. Even if a shallower target groundwater elevation of 4726 feet was used (at the Town wastewater treatment facility), the nearest location where the land surface is below that elevation is 6,850 feet northwest of Town near the intersection of County Roads 44 and 29. Due to the substantial distance required for gravity dewatering conveyance, deep burial depth, hydraulic constraints with the existing ditches, and disturbance caused by construction of horizontal drains through Town, gravity dewatering was not considered as practical solution to lowering the groundwater table.




| DEWATERING WELL LOCATION | HIGH FLOW RATE (GPM) | TOTAL DYNAMIC HEAD (FT) ALL PUMPS ON |
|---|---|---|
| WWTP | 450 | 191 |
| GMP | 2,500 | 148 |
| M+E | 1,400 | 124 |

**FIG 9.1 - DEWATERING & CONVEYANCE**

TOWN OF GILCREST

OCTOBER 2016

CONSULTING ENGINEERS

JVA, Incorporated
1319 Spruce Street
Boulder, CO 80302
Phone: 303.444.1951
E-mail: info@jvajva.com

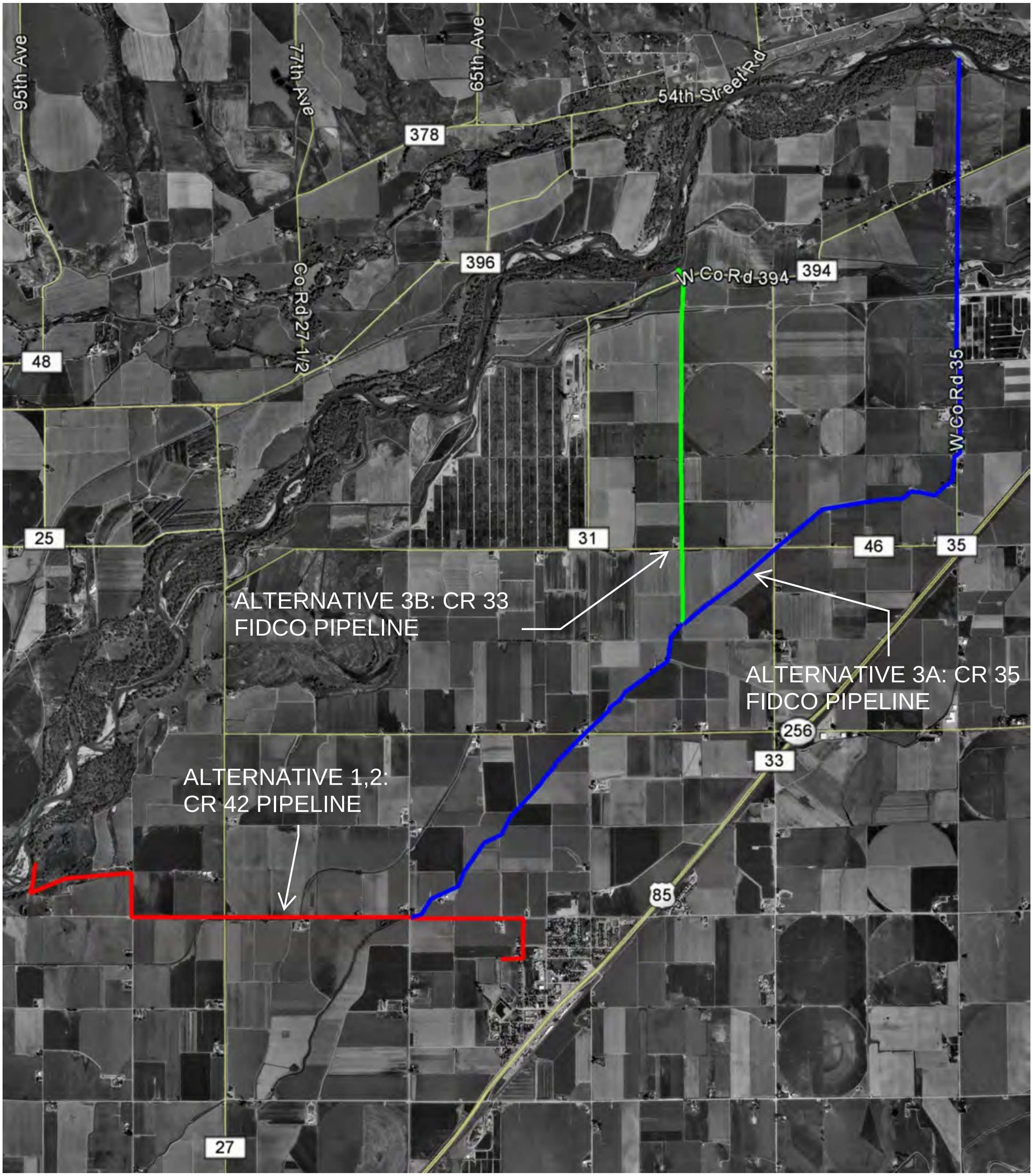



**FIG 9.2 - DISCHARGE CONVEYANCE**

TOWN OF GILCREST

OCTOBER 2016

CONSULTING ENGINEERS

JVA, Incorporated
1319 Spruce Street
Boulder, CO 80302
Phone: 303.444.1951
E-mail: info@jvajva.com

## Alternative 1: Pressurized Pipeline to South Platte River

A dedicated 16” diameter HDPE or PVC pipeline would convey water from the Stormwater Retention Pump Station (SRPS) and terminate at the South Platte River. For this option, the pipeline would follow County Road (CR) 42 west for approximately 2.14 miles, then north and to the west where it will discharge to the South Platte River. At approximately 800 feet west of CR 29, the highest elevation for the pipeline will occur with an invert elevation of approximately 4758 ft. From there, the pipeline will descend until it discharges to the Platte at roughly 4731 ft. At 10 cfs, water would flow at a rate of 7.43 feet per second, assuming an internal pipe diameter of 15.47 inches.

The SRPS will be located in the lowest point of the stormwater retention pond near the northeast corner. Water from the three dewatering wells will discharge into an approximate 100-foot by 100-foot depression which will be lined with an impermeable liner to prevent further groundwater infiltration. The water will then flow by gravity to the vertical turbine pump intake. The SRPS would consist of three 5 cfs vertical turbine pumps operating in a lead, lag, and standby configuration. Figures 9.3 and 9.4 show the SRPS site plan and pump station layout.

Pump startup conditions require pumping to overcome approximately 28-ft of static head between the turbine pump intake and the pipeline high point. Once the system is discharging and in normal operating conditions, the discharge location significantly reduces static head through a siphoning effect, but frictional forces increase as the length of pipeline increases. The total dynamic head required to pump at 10 cfs is 124-ft TDH at pump startup and 101.23-ft TDY during normal operating conditions. Detailed hydraulic calculations are included in Appendix B.

Each SRPS pump would be 150 hp, which will require a significant power draw, approximately 170 amps. Startup could require as much as six times full load amps, resulting in 1,020 amps. To reduce the spike electrical demand on the Town’s infrastructure, a Variable Frequency Drive (VFD) or soft start may help in distributing the demand out over a greater period of time. A VFD could also slow the pumps down before shutoff, reducing the need for a surge tank.

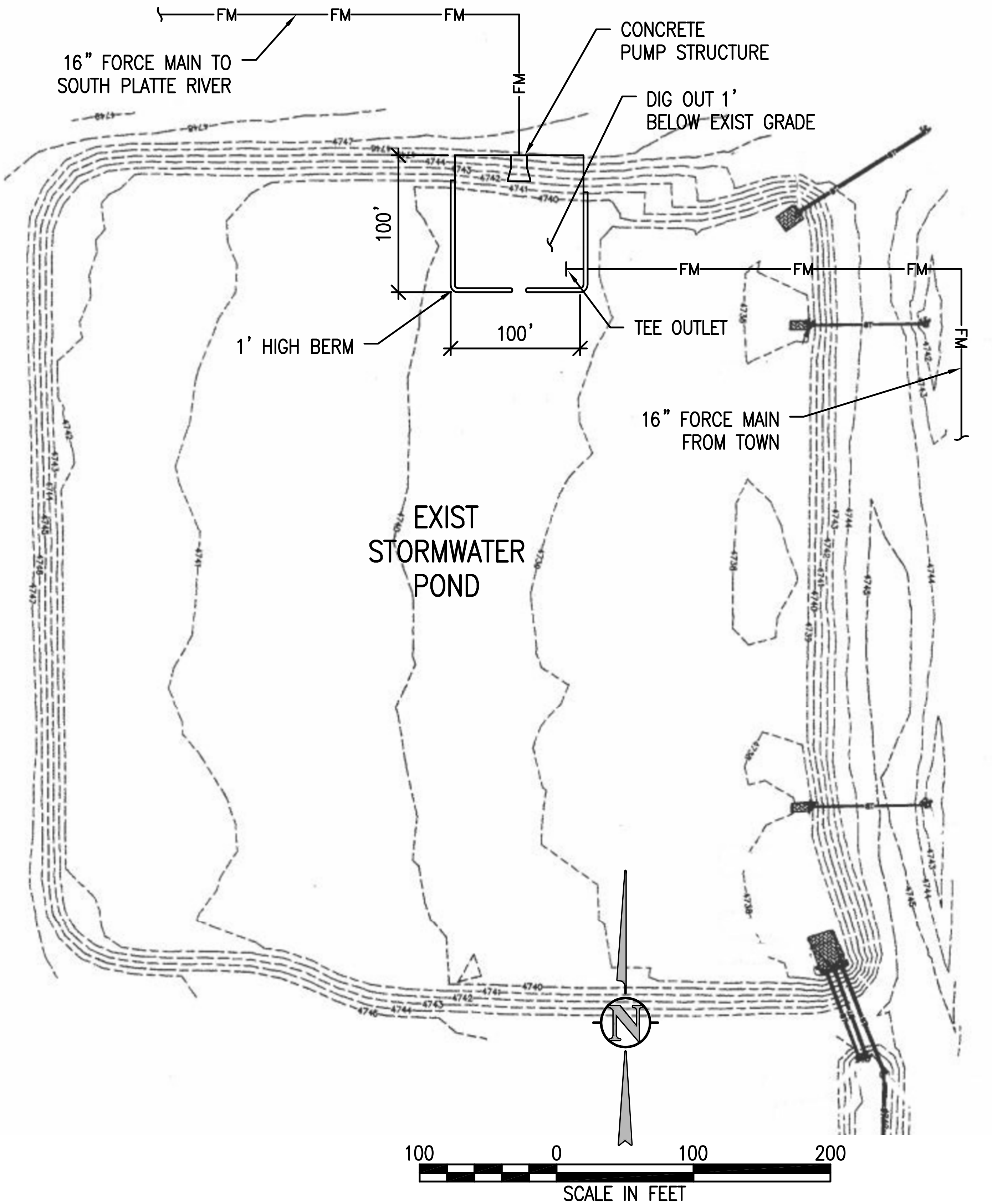



**FIG 9.3 - SITE PLAN OF WEST STORMWATER POND MODIFICATIONS**
TOWN OF GILCREST
OCTOBER 2016

CONSULTING ENGINEERS

JVA, Incorporated
1319 Spruce Street
Boulder, CO 80302
Phone: 303.444.1951
E-mail: info@jvajva.com

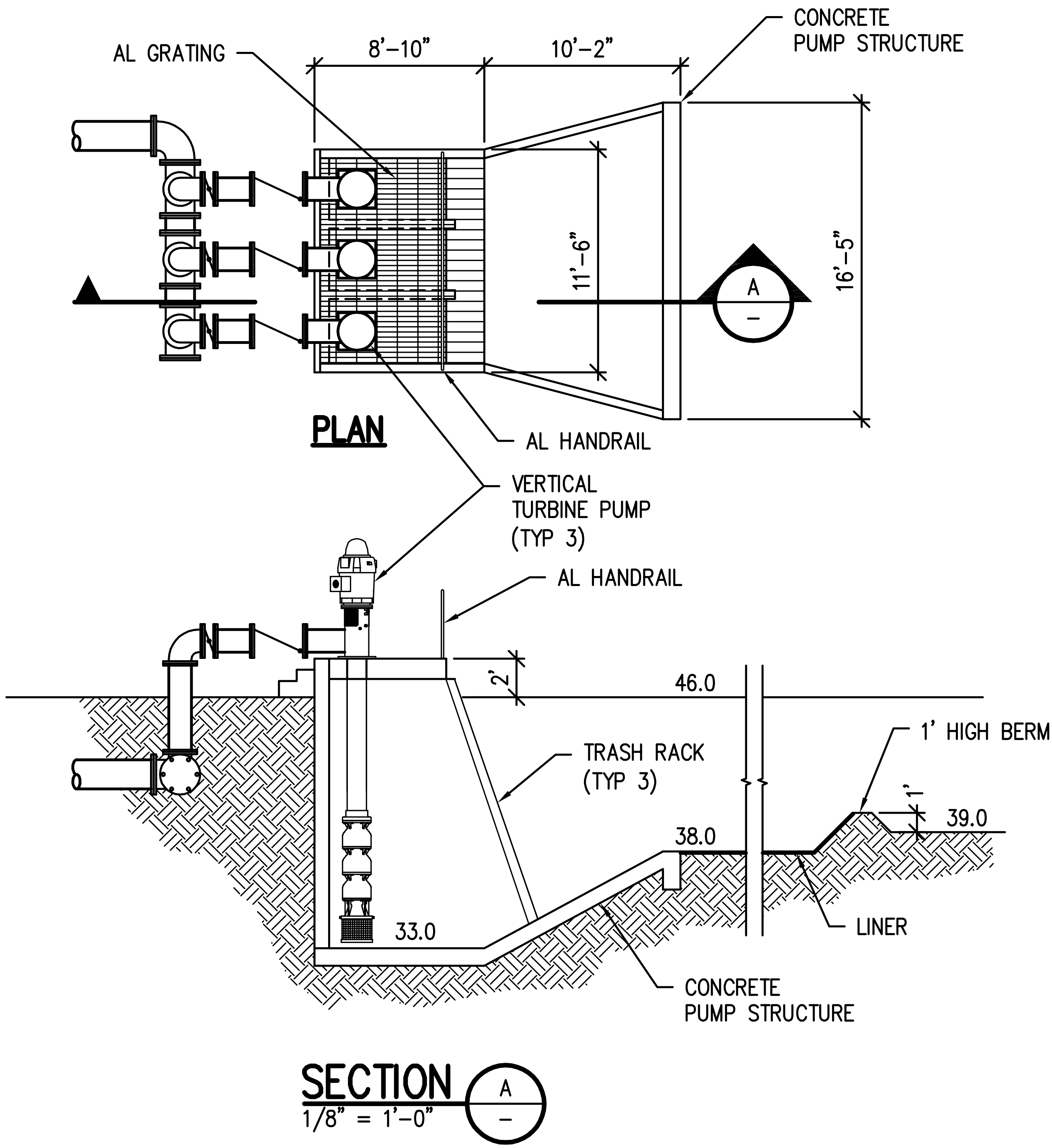



**FIG 9.4 - PUMP STATION PLAN AND ELEVATION**

TOWN OF GILCREST

OCTOBER 2016

JVA CONSULTING ENGINEERS

JVA, Incorporated
1319 Spruce Street
Boulder, CO 80302
Phone: 303.444.1951
E-mail: info@jvajva.com

## Opinion of Probable Cost / Life Cycle Costs

The assumptions and methodology below have been applied to all costs discussions throughout the conveyance alternatives analysis. The present worth costs are estimated at plant buildout capacity for comparison. The actual payback on equipment may be longer than assumed if the facility operates at less than buildout capacity longer than the 20-year projection.

The scope of each improvement has been quantified as much as possible within the level of design completed to date so as to identify the magnitude of the required work and to enable the application of pricing. All quantities have been measured or estimated and are intended to be representative of the final installed scope, based on the information available at the time of the estimate development. Depending on the level of information available, quantities may have been developed based on the following:

- Take-off from engineered preliminary design
- Estimated take-offs from preliminary plans, sketches, general arrangement drawings or previous experience
- Factored from previous projects based on capacity
- Order-of-magnitude allowance

Pricing has been developed using information from RS Means Online, previous project experience, and vendor information on major equipment items. Indirect costs include total project contingency, contractors overhead and profit, and professional design fees (engineering, geotechnical, and surveying). Project contingency is based on the level of confidence in the scope of work, quantities, and complexity of the project. Contingency is intended to cover anticipated variances between the direct costs in the base estimates and the final actual project cost in order for the total estimated values to represent the most likely outcomes. The contingency sum does not cover changes to the stated design (scope changes) or the listed qualifications and exclusions. It is expected that the most likely outcome is that all contingency monies would be spent in the execution of the project. Contractor overhead and profit has been set at 15 percent of the capital costs. Engineering fees have been estimated at 10 percent. Because of previous project challenges with unsupervised constructors on Town projects, bidding and construction administration has been set at 7 percent due to the assumption that some full time observation of pipe installation would occur. All indirect costs are applied to the subtotal of capital costs to give a total cost.

The estimates are qualified by the following assumptions:

- All project improvements are to be developed in a single stage of construction.
- Estimates are based on reasonable construction time frames with no element of acceleration introduced to the construction schedule.
- Clear area is available for machinery access.
- Utilities such as water, sewerage and power are available to the contractors.
- Adequate lay-down area would be available for material storage and for minor field fabrication work.
- Costs do not include tax

Present worth evaluations were performed for selected alternatives. Key assumptions in the present worth evaluations include:

- Electricity costs of $0.10 / kW-hr
- Inflation at 2.3%
- Interest at 2.7%
- Present worth calculated for 20-year project life
- 10-year replacement costs were assumed to be 15% of each pumps capital cost

Cost estimates have been prepared to a nominal accuracy of +/- 30 percent. The engineering documentation used in the preparation of the estimates has been the existing drawings, photos from site visits, previous project experience, input from the Town, and vendor information on major equipment items.

Table 9.1 below summarizes the opinions of probable cost for Alternative 1 including mobilization, dewatering well construction, SRPS, pipelines, and all pertinent sitework, valves, and appurtenances. Operation and maintenance (O&M) cost considerations include electricity and annual maintenance and repairs. Detailed cost estimates are included in Appendix C.

**Table 9.1 – Alternative 1 Cost Summary**

| Alternative 1 | Project Cost | 20-Yr Present Worth O&M Cost |
|---|---|---|
| Pressurized Pipeline to South Platte River | $7,161,000 | $4,841,700 |

## ALTERNATIVE 2: PRESSURIZED/GRAVITY FLOW COMBINATION TO SOUTH PLATTE RIVER

Groundwater could also be pumped via pressurized pipe to the high point along CR 42, then gravity flow to the South Platte River. At CR 29, the pipe would increase in size to allow for gravity flow to the discharge location.

An important issue to consider with this alternative is the ground surface elevation only decreases 8 feet over a 9,000-foot distance. A minimum slope is required in the discharge channel to maintain gravity flow of 10 cfs. The lack of ground surface elevation change and minimum slope requirements results in a trench approximately 18 feet deep by the time the pipe terminates at the outfall. With excavated trench side slopes of 1:1, there is insufficient space in the Right of Way or potential easements to excavate an open channel for gravity flow, rendering this alternative practically unfeasible (and not considered in further evaluations).

If construction was performed with trench boxes or sheeting/shoring, costs are anticipated to considerably exceed Alternative 1. Therefore, capital costs are not included for this alternative. O&M costs are anticipated to be similar to Alternative 1. Appendix D includes the Plan and Profile from pumping to discharge of Alternative 2.

## ALTERNATIVE 3: MULTI-USE MULTI-BENEFIT PIPELINE TO SOUTH PLATTE RIVER

An alternative to a dedicated force main to the South Platte is a shared use pipeline. Groundwater could be pumped via pressurized pipeline to CR 29 where it will travel an additional 2.3 miles northeast along the existing ditch to connect to the Central Colorado Water Conservancy District's (CCWCD) proposed FIDCO augmentation pipeline. This pipeline would continue north along either CR 33 (Alternative 3A) or along CR 35 (Alternative 3B) and discharge to the South Platte. Only Alternative 3B was included for further analysis, as it is the longest pipeline distance, and is therefore the worst-case scenario. There are a number of existing augmentation wells along the FIDCO ditch which could potentially utilize the forcemain to augment water by pumping directly to the South Platte River.

This alternative assumes the pipeline increases to a 20-inch diameter pipe at the intersection of CR 29 and CR 42, which runs approximately 11,750 feet northeast to the FIDCO Augmentation Station. The pipeline is assumed to increase to a 24-inch diameter pipe at the FIDCO Augmentation Station and continues approximately 20,620 feet northeast and north along CR 35 (Alternative 2B in Figure 9.2) to discharge to the South Platte River. This assumed alignment is approximately 10,450 feet longer than the FIDCO pipeline along CR 33 as shown in Figure 9.2, and is assumed the preferred alternative due to the presence of additional augmentation wells in close proximity to the alignment.

All pump sizing and headloss calculations for this alternative only included flows expected from the Town's dewatering efforts with a 16-inch forcemain. The OPC includes costs for increasing pipe diameters. Increasing the pipe diameters along the FIDCO alignment will allow for additional flow to be pumped into the forcemain without increasing the modeled pumping/electricity requirements at the Town's pump station. Any additional flows from other stakeholders would need to be assessed and coordinated in the future if this alternative is pursued.

The perceived disadvantages of this alternative include additional pumping costs at the Towns SRPS due to the longer pipeline, higher capital costs due to the additional pipeline to get flow to the South Platte River. A multi-stakeholder project would also require considerable commitments, coordination, and effort to coordinate investments, easements, and right of ways from landowners and stakeholders.

The perceived benefits of this alternative includes as many as ten possible augmentation wells which may benefit from the ability to augment directly to the river. The pipeline may provide assistance in augmentation, thereby increasing Groundwater Management Subdistrict and Well Augmentation Subdistrict quotas. Each of the wells may represent additional stakeholders who may be able to contribute to the overall capital and operational costs to install and operate the pipeline and pump stations. Given that the high groundwater is a regional issue affecting the Town as well as surrounding properties, a shared pipeline gives an improved opportunity for nearby individuals and facilities to address issues with high groundwater or augmentation water outside of Town limits.

Another benefit of this alternative may include the ability for the pipeline to serve a dual purpose. Should the damaging groundwater abate over time due to other factors, the Town may be able to shut off their dewatering wells. The pipeline could then possibly serve an alternative function to deliver water from multiple storage areas near the South Platte River (WCR 35 and WCR 394 north of Town) back into the nearby farming communities. CCWCD has expressed interest in a pipeline capable of bi-directional flow for these reasons.

The pumping requirements for Alternative 3 are the highest of the three alternative. Startup and normal operating conditions would require similar pumping head (Although, depending on where the FIDCO pipeline terminates, pumping requirements may be more stringent than that of Alternative 1). Alternative 3 would utilize the same SRPS as Alternative 1.

### Opinion of Probable Cost / Life Cycle Costs

Table 9.3 below summarizes the opinions of probable cost for Alternative 3 including mobilization, dewatering well construction, SRPS, pipelines, and all pertinent sitework, valves, and appurtenances. Operation and maintenance (O&M) cost considerations include electricity and annual maintenance and repairs. Detailed cost estimates are included in Appendix C.

**Table 9.2 – Alternative 3 Cost Summary**

| Alternative 3 | Project Cost | 20-Yr Present Worth O&M Cost |
|---|---|---|
| FIDCO Shared Forcemain to South Platte along CR 35 | $11,233,000 | $7,051,000 |

### Selected Conveyance Alternative

Table 9.4 shows the cost summary for the two most feasible alternatives.

**Table 9.3 – Summary of Cost for Alternatives**

| Costs | Alternative 1 | Alternative 3 |
|---|---|---|
| Project Cost | $7,161,000 | $11,233,000 |
| 20 Year O&M (2016PW) | $4,841,700 | $7,051,000 |
| **Total Cost** | **$12,002,700** | **$18,284,000** |

The two feasible conveyance alternatives were evaluated with respect to the following criteria:

- Capital Costs
- Operation and Maintenance Costs
- Ease of Operations
- Ease of Implementation

Although Alternative 3 has a higher capital and operating cost, it is assumed to be cost competitive with Alternative 1 due to the number of potential stakeholders that could contribute to the project.

Table 9.5 shows a decision matrix developed for alternative selection. The matrix assigns a value (1-3) of rank to each category for each alternative with 1 being the most favorable and 3 being the least. The importance factor (higher number meaning more importance given to each category) is multiplied by the rank to get the total score with the lowest score winning.

**Table 9.4 – Decision Criteria Matrix**

| Comparison Criteria | Importance Factor | Alternative 1 Rank / Score | Alternative 3 Rank / Score |
|---|---|---|---|
| Capital Cost | 3 | 1 / 3 | 2 / 6 |
| O&M Cost | 2 | 1 / 2 | 2 / 4 |
| Ease of Operations | 1 | 2 / 2 | 2 / 2 |
| Ability to Address Regional Issues and Multi-functions | 2 | 2 / 4 | 1 / 2 |
| Suitability for Multiple Stakeholder Benefit | 3 | 2 / 6 | 1 / 3 |
| **Total Score** | | 17 | 17 |

Alternative 1 and Alternative 3 rank similarly based upon the selection criteria. Alternative 3 has advantages of addressing regional issues, multiple stakeholders and an economy of scale. Furthermore, Alternative 3 has greater opportunity for funding through multiple stakeholder contributions towards capital and O&M costs. However, those same benefits could make implementation and operation more complex. Alternative 1 has advantages on capital and O&M costs, and provides the Town with a robust solution that protects the Town without reliance on other stakeholders. Because of the potential to provide a regional solution, and benefit multiple stakeholders, Alternative 3 is recommended.

## Design Criteria for Selected Alternative

Table 9.6 includes the design criteria for the selected alternative.

**Table 9.5 – Design Criteria for Pressurized Pipeline to South Platte**

| Dewatering Infrastructure | Flow Rate (gpm) | Total Dynamic Head (ft) |
|---|---|---|
| WWTP | 450 | 191 |
| GMP | 2,500 | 148 |
| M+E | 1,400 | 124 |
| Stormwater Retention Pump Station | 4,350 | 168.28 |

## Summary of Recommendations

As stated in the introduction, ideally, water management could be modified to lower groundwater levels throughout the area. However, this approach may never come to fruition or at best may take many more years before becoming a realistic alternative to help alleviate damaging groundwater in the Gilcrest area. Therefore, this report provides a recommendation for an alternative approach to protect the Town from high groundwater through pumping and conveyance to the South Platte River.

The recommended dewatering and conveyance alternative includes the following key infrastructure:

- 450 gpm dewatering well and pump station (WWTP) with a 6-inch forcemain to the common GMP dewatering pump station forcemain
- 2,500 gpm dewatering well and pump station (GMP) with a 16-inch forcemain to carry WWTP and GMP dewatering to the Town's western stormwater pond
- 1,400 gpm dewatering well and pump station (M+E) with a 12-inch forcemain to the common GMP dewatering pump station forcemain
- 4,350 gpm, lead-lag-standby vertical turbine (150 hp each) pump station with a lined sump for a wet well and concrete structure at the Town's western stormwater pond to carry dewatered flows or stormwater flows to the South Platte River

The actual dewatering rates needed to achieve water level targets will not be known until wells are drilled, constructed and operated due to uncertainty in local aquifer characteristics.

The recommended alternative is not anticipated to detrimentally impact existing benzene and nitrate plumes within Town limits because, generally, dewatering that is concentrated down-gradient of contaminant plumes is not expected to cause new contamination concerns. Since the majority of dewatering pumping is concentrated at the GMP and WWTP locations, there is also limited concern for impact to currently ongoing remediation efforts. Pumping at the M+E well may cause unwanted plume migration and will need further future review. However, dewatering at the Main and Elm location is critical to achieve water level targets at the Wastewater Pump Station No. 2.

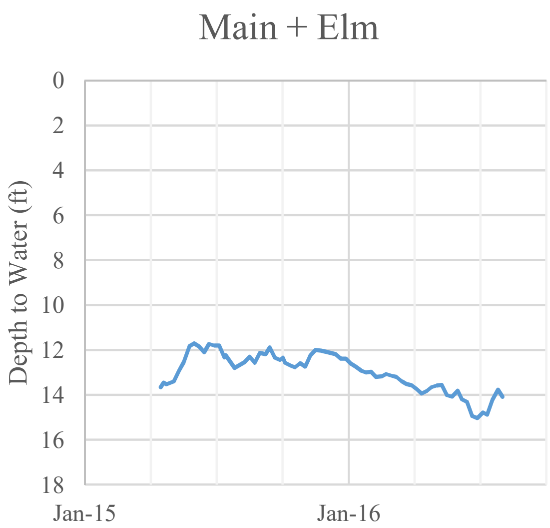
Prior to any further design, it is recommended the Town undertake initial water quality testing and discuss with CDPHE the likelihood for treatment of dewatering groundwater to the South Platte. If required, the cost of treatment could render the recommended alternative unfeasible.

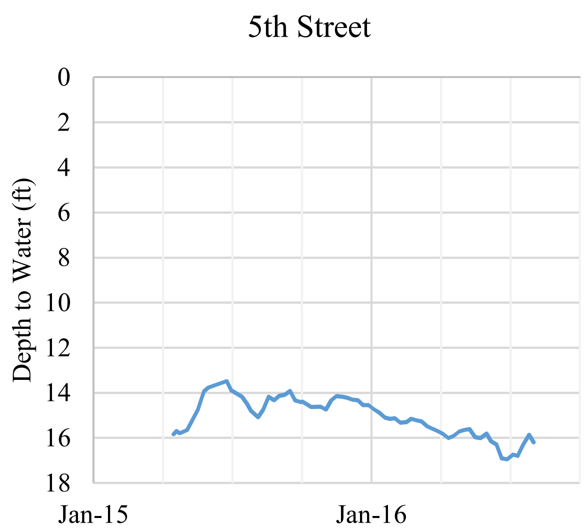
## IMPLEMENTATION

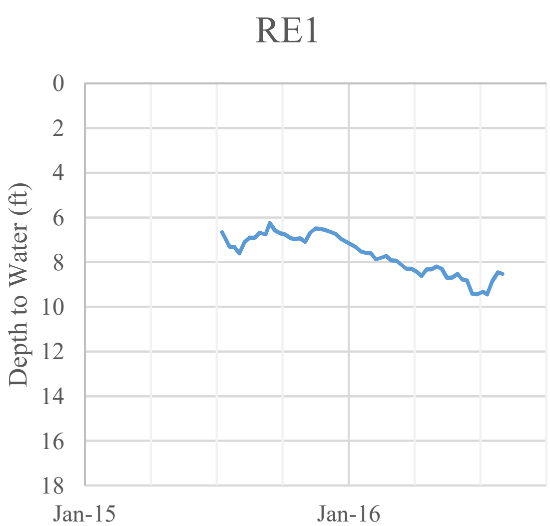
Only with the full financial involvement of all pipeline users will Alternative 3 be the most economic and beneficial solution. In order to accomplish the recommended alternative, substantial collaboration and financial assistance from all stakeholders will be required. Depending on the number of stakeholders and projected flow into the FIDCO pipeline, either upsizing the pipeline diameter or additional pumping/injection/withdrawal requirements will need to be fulfilled, and the specific needs of each stakeholder must be understood completely ahead of any final design. Coordination and buy-in from all stakeholders will be needed for this project's success, requiring significant administrative efforts. The Town will also need to investigate and pursue a number of funding alternatives. Finally, the State of Colorado will play an integral role in any solution moving forward, and close coordination and support from the State is critical.

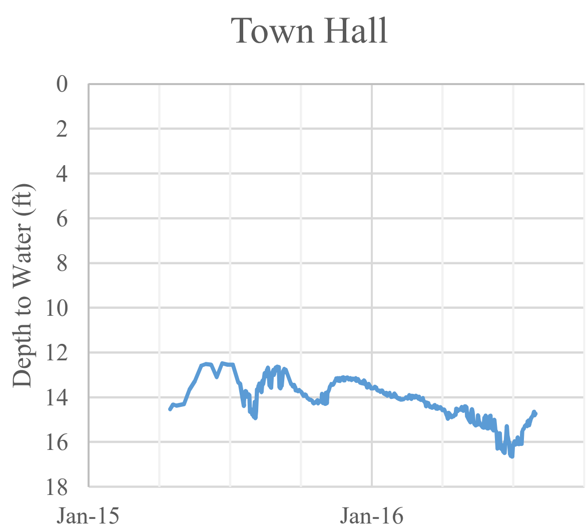
# APPENDIX A – SUPPORTING DOCUMENTATION

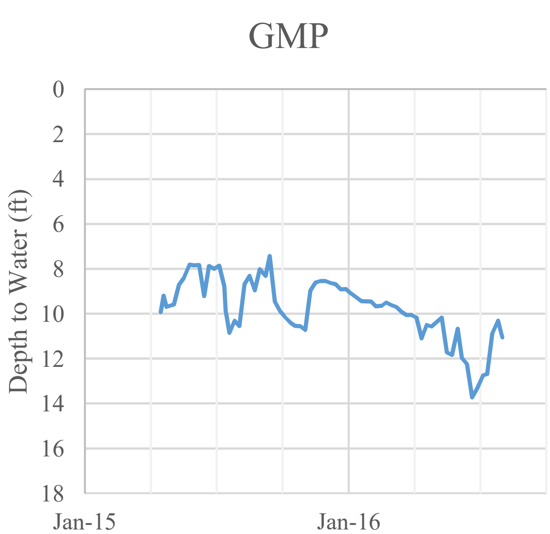
**Appendix A 2.1**
**Town of Gilcrest**
**Well Water Levels in Vicinity of Town**

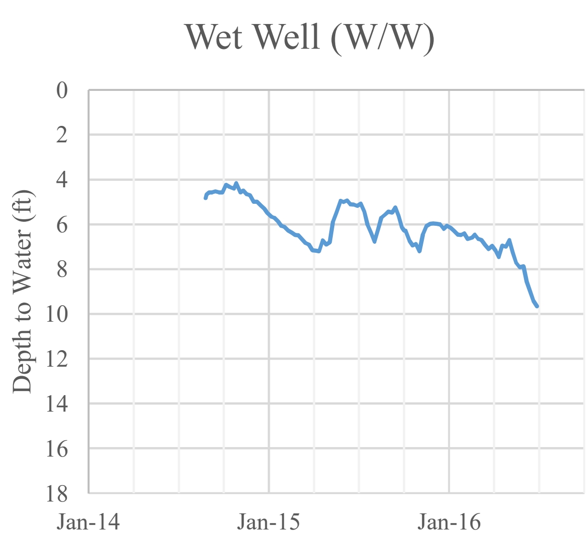


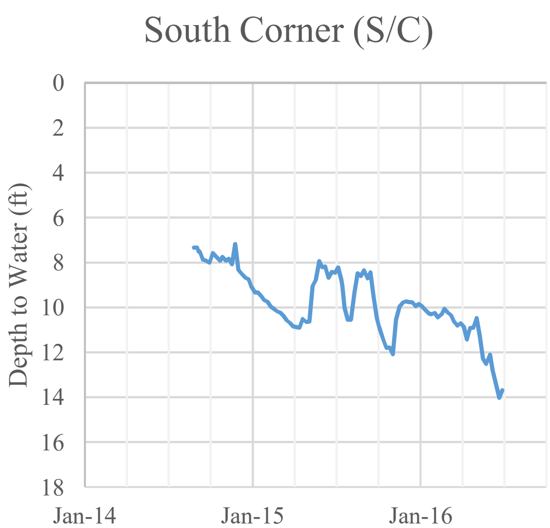



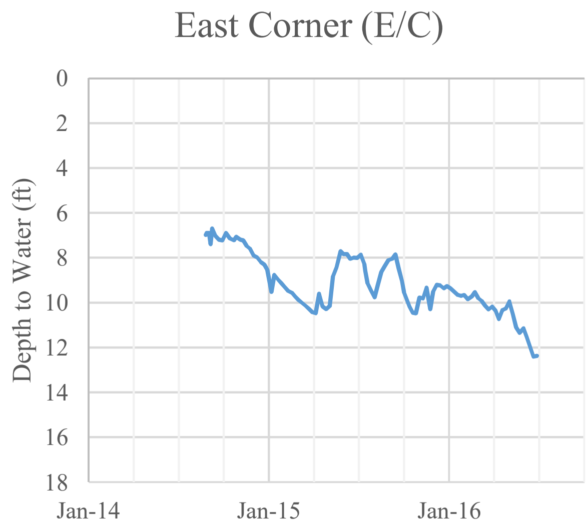



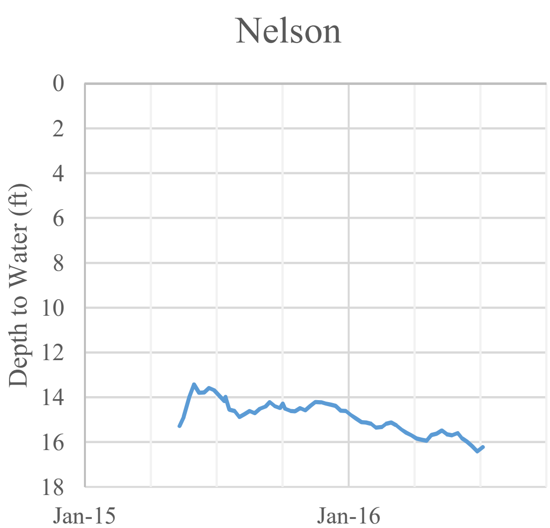



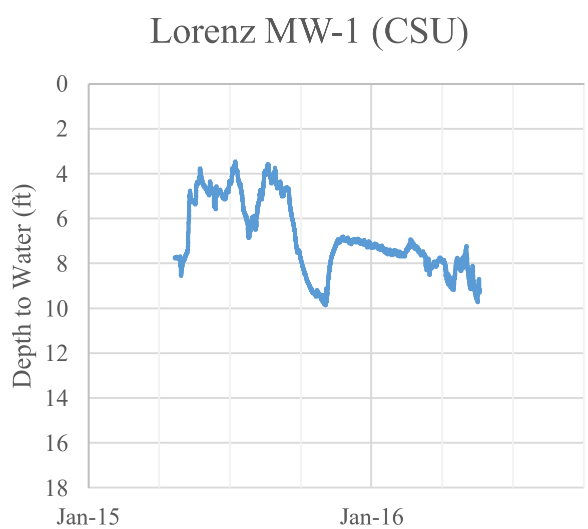



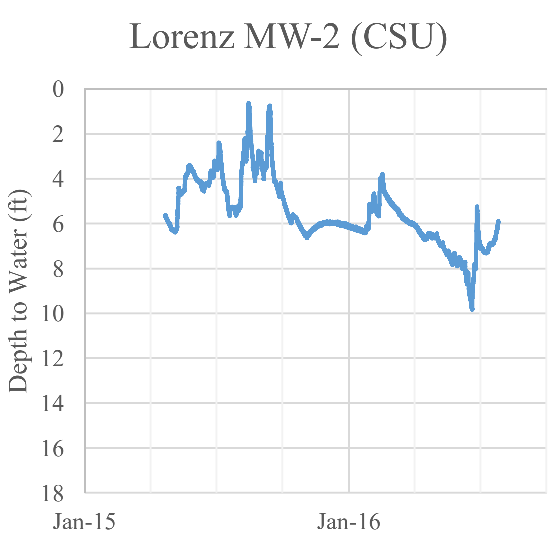
Appendix A 2.1 (continued)

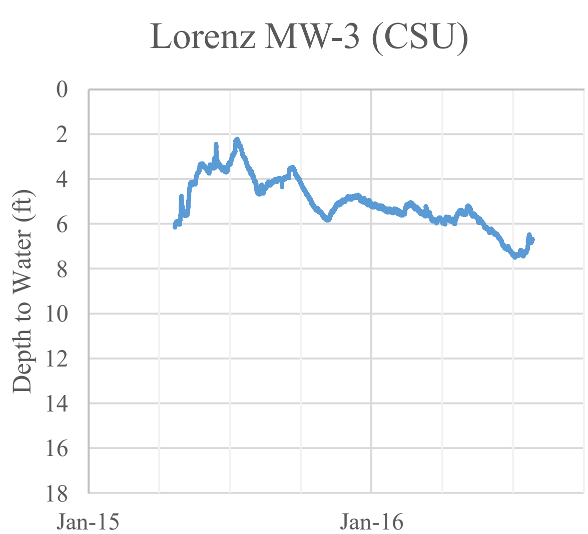


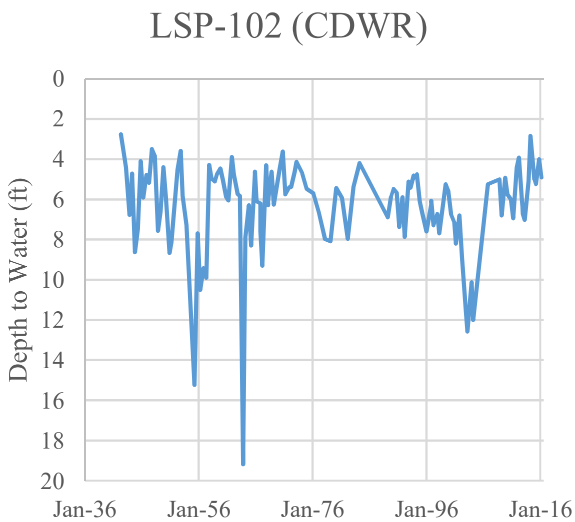



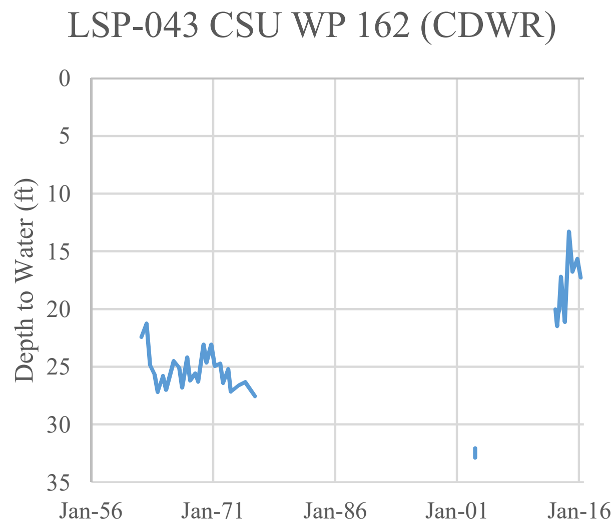



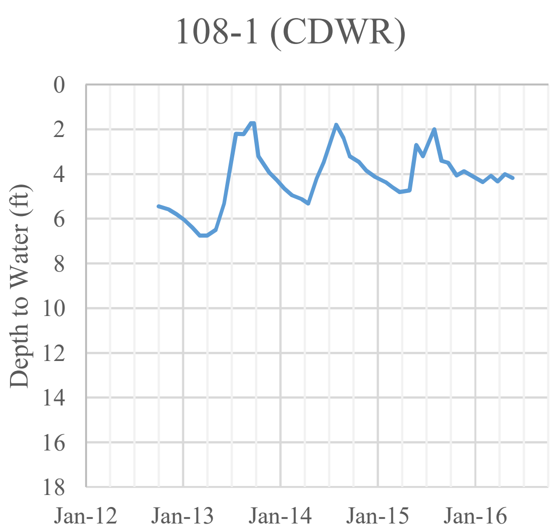



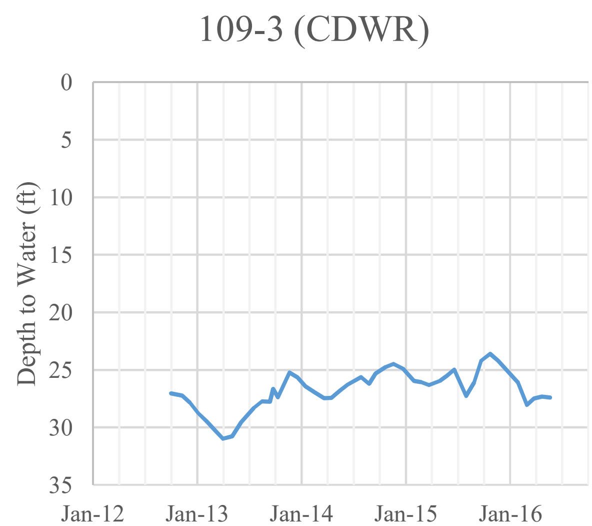



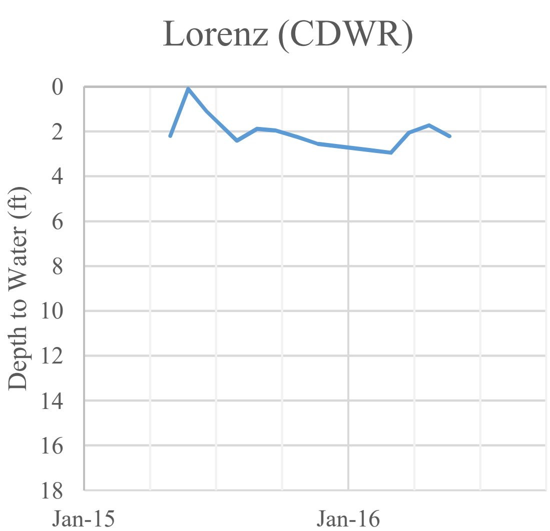



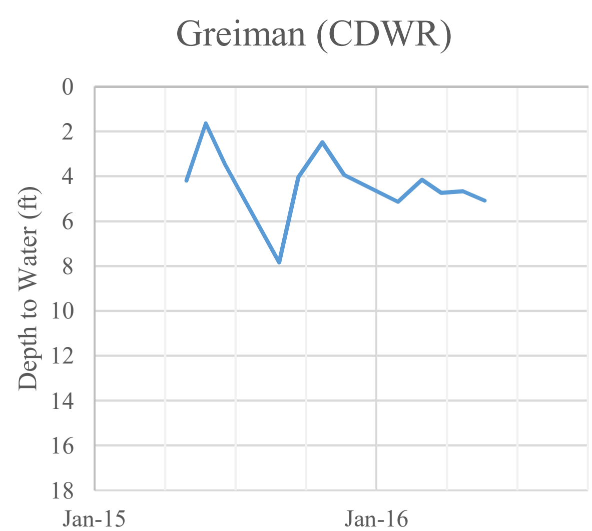

LSP-102 (CDWR)
Depth to Water (ft)
0
2
4
6
8
10
12
14
16
18
20
Jan-36
Jan-56
Jan-76
Jan-96
Jan-16


LSP-043 CSU WP 162 (CDWR)
Depth to Water (ft)
0
5
10
15
20
25
30
35
Jan-56
Jan-71
Jan-86
Jan-01
Jan-16


108-1 (CDWR)
Depth to Water (ft)
0
2
4
6
8
10
12
14
16
18
Jan-12
Jan-13
Jan-14
Jan-15
Jan-16


109-3 (CDWR)
Depth to Water (ft)
0
5
10
15
20
25
30
35
Jan-12
Jan-13
Jan-14
Jan-15
Jan-16


Lorenz (CDWR)
Depth to Water (ft)
0
2
4
6
8
10
12
14
16
18
Jan-15
Jan-16


Greiman (CDWR)
Depth to Water (ft)
0
2
4
6
8
10
12
14
16
18
Jan-15
Jan-16

**Appendix A 2.1 (continued)**
**Town of Gilcrest**
**Well Water Levels in Vicinity of Town**
All values shown as depth to water below ground surface in feet.

| Date | Wet Well | South Corner | East Corner | Main + Elm | 5th Street | RE1 | Town Hall | GMP | Nelson |
|---|---|---|---|---|---|---|---|---|---|
| 8/26/2014 | 4.83 | 7.33 | 6.98 | - | - | - | - | - | - |
| 9/2/2014 | 4.58 | 7.33 | 6.89 | - | - | - | - | - | - |
| 9/8/2014 | 4.58 | 7.5 | 6.69 | - | - | - | - | - | - |
| 9/15/2014 | 4.53 | 7.88 | 7.02 | - | - | - | - | - | - |
| 9/22/2014 | 4.58 | 7.92 | 7.19 | - | - | - | - | - | - |
| 9/29/2014 | 4.58 | 8 | 7.23 | - | - | - | - | - | - |
| 10/6/2014 | 4.24 | 7.58 | 6.89 | - | - | - | - | - | - |
| 10/14/2014 | 4.33 | 7.75 | 7.14 | - | - | - | - | - | - |
| 10/22/2014 | 4.41 | 7.92 | 7.23 | - | - | - | - | - | - |
| 10/27/2014 | 4.16 | 7.75 | 7.06 | - | - | - | - | - | - |
| 11/4/2014 | 4.58 | 7.92 | 7.19 | - | - | - | - | - | - |
| 11/10/2014 | 4.49 | 7.83 | 7.23 | - | - | - | - | - | - |
| 11/17/2014 | 4.66 | 8.08 | 7.48 | - | - | - | - | - | - |
| 11/24/2014 | 4.7 | 7.17 | 7.6 | - | - | - | - | - | - |
| 12/1/2014 | 4.99 | 8.33 | 7.89 | - | - | - | - | - | - |
| 12/8/2014 | 4.99 | 8.5 | 7.98 | - | - | - | - | - | - |
| 12/16/2014 | 5.16 | 8.67 | 8.19 | - | - | - | - | - | - |
| 12/23/2014 | 5.31 | 8.75 | 8.31 | - | - | - | - | - | - |
| 12/29/2014 | 5.49 | 9.06 | 8.54 | - | - | - | - | - | - |
| 1/6/2015 | 5.66 | 9.33 | 9.52 | - | - | - | - | - | - |
| 1/12/2015 | 5.7 | 9.33 | 8.77 | - | - | - | - | - | - |
| 1/20/2015 | 5.87 | 9.5 | 8.98 | - | - | - | - | - | - |
| 1/26/2015 | 6.08 | 9.67 | 9.14 | - | - | - | - | - | - |
| 2/2/2015 | 6.12 | 9.75 | 9.31 | - | - | - | - | - | - |
| 2/9/2015 | 6.28 | 9.96 | 9.48 | - | - | - | - | - | - |
| 2/17/2015 | 6.37 | 10.08 | 9.56 | - | - | - | - | - | - |
| 2/23/2015 | 6.47 | 10.17 | 9.73 | - | - | - | - | - | - |
| 3/2/2015 | 6.49 | 10.23 | 9.87 | - | - | - | - | - | - |
| 3/9/2015 | 6.66 | 10.38 | 10 | - | - | - | - | - | - |
| 3/16/2015 | 6.83 | 10.58 | 10.14 | - | - | - | - | - | - |
| 3/23/2015 | 6.91 | 10.69 | 10.27 | - | - | - | - | - | - |
| 3/30/2015 | 7.16 | 10.85 | 10.44 | - | - | - | - | - | - |
| 4/6/2015 | 7.18 | 10.88 | 10.48 | - | - | - | - | - | - |
| 4/13/2015 | 7.2 | 10.9 | 9.6 | - | - | - | - | - | - |
| 4/20/2015 | 6.72 | 10.52 | 10.18 | 13.46 | 15.7 | - | 14.32 | 9.2 | - |
| 4/27/2015 | 6.91 | 10.64 | 10.29 | - | - | - | - | - | - |
| 5/4/2015 | 6.81 | 10.63 | 10.15 | 13.41 | 15.65 | - | 14.3 | 9.6 | - |
| 5/11/2015 | 5.89 | 9.05 | 8.83 | 12.93 | 15.2 | - | 13.66 | 8.71 | - |
| 5/18/2015 | 5.48 | 8.77 | 8.43 | 12.55 | 14.74 | - | 13.28 | 8.42 | 14.89 |
| 5/26/2015 | 4.95 | 7.93 | 7.71 | 11.84 | 13.93 | - | 12.59 | 7.81 | 13.99 |
| 6/1/2015 | 5.01 | 8.21 | 7.84 | 11.71 | 13.77 | - | 12.51 | 7.85 | 13.43 |
| 6/8/2015 | 4.94 | 8.18 | 7.83 | 11.85 | - | - | 12.55 | 7.83 | 13.8 |
| 6/15/2015 | 5.11 | 8.67 | 8.05 | 12.11 | - | - | 13.1 | 9.21 | 13.78 |
| 6/22/2015 | 5.12 | 8.42 | 7.99 | 11.74 | - | - | 12.48 | 7.87 | 13.59 |
| 6/29/2015 | 5.18 | 8.47 | 8.01 | 11.81 | - | - | 12.55 | 8 | 13.69 |
| 7/6/2015 | 5.08 | 8.22 | 7.87 | 11.81 | - | - | 12.55 | 7.86 | - |
| 7/13/2015 | 5.44 | 8.77 | 8.28 | 12.33 | - | - | 13.35 | 8.78 | 14.17 |
| 7/15/2015 | 5.58 | 9.03 | 8.59 | 12.23 | 14.17 | 6.96 | 13.39 | 9.86 | 13.98 |

**Appendix 2.1 (continued)**
**Town of Gilcrest**
**Well Water Levels in Vicinity of Town**
All values shown as depth to water below ground surface in feet.

| Date | Wet Well | South Corner | East Corner | Main + Elm | 5th Street | RE1 | Town Hall | GMP | Nelson |
|---|---|---|---|---|---|---|---|---|---|
| 7/20/2015 | 6.02 | 10.02 | 9.13 | - | - | 7.3 | 14.39 | 10.85 | 14.56 |
| 7/27/2015 | 6.38 | 10.55 | 9.46 | 12.8 | 14.8 | 7.32 | 13.91 | 10.31 | 14.62 |
| 8/3/2015 | 6.77 | 10.54 | 9.76 | - | - | 7.61 | 14.21 | 10.55 | 14.88 |
| 8/10/2015 | 6.24 | 9.33 | 9.16 | 12.55 | - | 7.1 | 13.39 | 8.69 | 14.76 |
| 8/17/2015 | 5.7 | 8.48 | 8.64 | 12.31 | - | 6.91 | - | 8.31 | 14.61 |
| 8/24/2015 | 5.58 | 8.61 | 8.37 | 12.58 | - | 6.9 | 13.57 | 8.95 | 14.71 |
| 8/31/2015 | 5.44 | 8.35 | 8.11 | 12.13 | - | 6.67 | 12.79 | 8.02 | 14.52 |
| 9/8/2015 | 5.48 | 8.71 | 8.05 | 12.19 | - | 6.76 | 13.01 | 8.31 | 14.42 |
| 9/14/2015 | 5.25 | 8.43 | 7.85 | 11.89 | - | 6.25 | - | 7.43 | 14.21 |
| 9/21/2015 | 5.62 | 9.55 | 8.46 | 12.34 | - | 6.58 | 13.51 | 9.47 | 14.4 |
| 9/28/2015 | 6.15 | 10.47 | 9.03 | 12.45 | - | 6.71 | 13.74 | 9.88 | 14.48 |
| 10/2/2015 | 6.28 | 10.79 | 9.55 | 12.35 | 14.39 | 6.74 | 13.76 | 10.03 | 14.29 |
| 10/5/2015 | 6.29 | 11.01 | 9.74 | 12.57 | - | 6.76 | 13.94 | 10.15 | 14.51 |
| 10/13/2015 | 6.73 | 11.52 | 10.18 | 12.71 | - | 6.93 | 14.12 | 10.41 | 14.61 |
| 10/19/2015 | 6.95 | 11.8 | 10.45 | 12.77 | - | 6.97 | 14.23 | 10.54 | 14.63 |
| 10/26/2015 | 6.87 | 11.79 | 10.48 | 12.59 | 14.62 | 6.93 | 14.17 | 10.55 | 14.49 |
| 11/2/2015 | 7.21 | 12.09 | 9.78 | 12.74 | 14.75 | 7.09 | 14.3 | 10.73 | 14.59 |
| 11/9/2015 | 6.44 | 10.54 | 9.82 | 12.25 | 14.35 | 6.67 | 13.48 | 8.97 | 14.38 |
| 11/16/2015 | 6.08 | 9.96 | 9.34 | 12.01 | 14.14 | 6.49 | 13.16 | 8.61 | 14.21 |
| 11/24/2015 | 5.98 | 9.77 | 10.29 | 12.04 | 14.17 | 6.52 | 13.12 | 8.54 | 14.23 |
| 11/30/2015 | 5.96 | 9.74 | 9.51 | 12.07 | 14.22 | 6.56 | 13.12 | 8.54 | 14.28 |
| 12/7/2015 | 5.97 | 9.76 | 9.2 | 12.14 | 14.3 | 6.65 | 13.19 | 8.63 | 14.33 |
| 12/14/2015 | 6.01 | 9.77 | 9.23 | 12.19 | 14.33 | 6.73 | 13.19 | 8.68 | 14.39 |
| 12/21/2015 | 6.2 | 9.94 | 9.37 | 12.39 | 14.55 | 6.95 | 13.39 | 8.91 | 14.6 |
| 12/28/2015 | 6.06 | 9.85 | 9.27 | 12.39 | 14.55 | - | 13.4 | 8.9 | 14.62 |
| 1/4/2016 | 6.15 | 9.95 | 9.37 | 12.61 | 14.73 | - | 13.57 | 9.08 | 14.79 |
| 1/11/2016 | 6.29 | 10.11 | 9.51 | 12.74 | 14.87 | 7.32 | 13.7 | 9.25 | 14.94 |
| 1/19/2016 | 6.46 | 10.28 | 9.67 | 12.93 | 15.1 | 7.53 | 13.89 | 9.44 | 15.11 |
| 1/25/2016 | 6.47 | 10.31 | 9.71 | 13.01 | 15.16 | 7.6 | 13.94 | - | 15.13 |
| 2/1/2016 | 6.41 | 10.24 | 9.66 | 12.98 | 15.13 | 7.61 | 13.91 | 9.45 | 15.18 |
| 2/8/2016 | 6.66 | 10.45 | 9.85 | 13.21 | 15.32 | 7.88 | 14.1 | 9.67 | 15.35 |
| 2/16/2016 | 6.61 | 10.3 | 9.72 | 13.18 | 15.3 | 7.8 | 14.06 | 9.64 | 15.33 |
| 2/22/2016 | 6.46 | 10.06 | 9.53 | 13.07 | 15.16 | 7.72 | 13.93 | 9.5 | 15.19 |
| 2/29/2016 | 6.66 | 10.23 | 9.81 | 13.14 | 15.23 | 7.92 | 13.98 | 9.61 | 15.13 |
| 3/7/2016 | 6.7 | 10.35 | 9.92 | 13.2 | 15.27 | 7.93 | 14.03 | 9.7 | 15.26 |
| 3/14/2016 | 6.92 | 10.65 | 10.13 | 13.39 | 15.48 | 8.12 | 14.26 | 9.92 | 15.44 |
| 3/21/2016 | 7.1 | 10.82 | 10.3 | 13.51 | 15.6 | 8.3 | 14.38 | 10.05 | 15.58 |
| 3/28/2016 | 6.96 | 10.7 | 10.18 | 13.58 | 15.7 | 8.3 | 14.45 | 10.06 | 15.7 |
| 4/4/2016 | 7.17 | 10.86 | 10.36 | 13.75 | 15.83 | 8.42 | 14.59 | 10.17 | 15.84 |
| 4/11/2016 | 7.46 | 11.43 | 10.74 | 13.95 | 16.01 | 8.62 | 14.88 | 11.1 | 15.9 |
| 4/18/2016 | 6.95 | 10.92 | 10.33 | 13.83 | 15.91 | 8.32 | 14.79 | 10.5 | 15.94 |
| 4/25/2016 | 7.01 | 10.91 | 10.26 | 13.66 | 15.73 | 8.32 | 14.56 | 10.57 | 15.68 |
| 5/2/2016 | 6.71 | 10.48 | 9.95 | 13.59 | 15.65 | 8.19 | 14.45 | 10.37 | 15.62 |
| 5/9/2016 | 7.26 | 11.28 | 10.51 | 13.56 | 15.61 | 8.3 | 14.54 | 10.17 | 15.48 |
| 5/16/2016 | 7.71 | 12.29 | 11.11 | 14.02 | 15.96 | 8.69 | 15.24 | 11.73 | 15.66 |
| 5/23/2016 | 7.91 | 12.51 | 11.34 | 14.09 | 16.01 | 8.7 | 15.35 | 11.84 | 15.7 |
| 5/31/2016 | 7.87 | 12.1 | 11.14 | 13.82 | 15.81 | 8.53 | 14.93 | 10.67 | 15.6 |
| 6/6/2016 | 8.56 | 12.8 | 11.53 | 14.2 | 16.15 | 8.76 | 15.5 | 11.98 | 15.82 |
| 6/20/2016 | 9.41 | 14.03 | 12.41 | 14.95 | 16.9 | 9.41 | 16.49 | 13.73 | 16.18 |
| 6/27/2016 | 9.66 | 13.69 | 12.37 | 15.04 | 16.95 | 9.44 | 16.55 | 13.33 | 16.42 |

Notes:
All field measurements reduced by height of casing to reflect depth of water below ground surface.
Town Hall is measured daily, with selected values shown.

**Appendix A 2.1 (continued)**
**Town of Gilcrest**
**Well Water Levels in Vicinity of Town**
All values shown as depth to water below ground surface in feet.

| Date | Lorenz MW-1 (CSU) | Lorenz MW-2 (CSU) | Lorenz MW-3 (CSU) | LSP-102 (CDWR) | LSP-043 (CDWR) | 108-1 (CDWR) | 109-3 (CDWR) | Lorenz (CDWR) | Greiman (CDWR) |
|---|---|---|---|---|---|---|---|---|---|
| 10/1/2012 | - | - | - | 5.99 | - | 5.45 | 27.05 | - | - |
| 11/9/2012 | - | - | - | 6.35 | - | 5.59 | 27.24 | - | - |
| 12/6/2012 | - | - | - | - | - | 5.8 | 27.85 | - | - |
| 1/4/2013 | - | - | - | - | - | 6.05 | 28.76 | - | - |
| 2/4/2013 | - | - | - | - | - | 6.4 | 29.5 | - | - |
| 3/4/2013 | - | - | - | - | - | 6.75 | 30.25 | - | - |
| 4/1/2013 | - | - | - | 7.03 | - | 6.75 | 30.98 | - | - |
| 5/3/2013 | - | - | - | - | - | 6.5 | 30.77 | - | - |
| 6/3/2013 | - | - | - | - | 20.43 | 5.32 | 29.54 | - | - |
| 7/18/2013 | - | - | - | - | 20.1 | 2.2 | 28.3 | - | - |
| 8/15/2013 | - | - | - | - | - | 2.21 | 27.75 | - | - |
| 9/12/2013 | - | - | - | - | - | 1.73 | 27.76 | - | - |
| 9/23/2013 | - | - | - | - | - | 1.72 | 26.64 | - | - |
| 10/10/2013 | - | - | - | - | - | 3.2 | 27.38 | - | - |
| 11/19/2013 | - | - | - | - | - | 3.93 | 25.24 | - | - |
| 12/18/2013 | - | - | - | 5.1 | - | 4.27 | 25.66 | - | - |
| 1/15/2014 | - | - | - | - | - | 4.65 | 26.44 | - | - |
| 2/13/2014 | - | - | - | - | - | 4.95 | 26.9 | - | - |
| 3/21/2014 | - | - | - | - | - | 5.12 | 27.45 | - | - |
| 4/15/2014 | - | - | - | 2.85 | - | 5.32 | 27.42 | - | - |
| 5/16/2014 | - | - | - | - | - | 4.2 | 26.8 | - | - |
| 6/12/2014 | - | - | - | - | - | 3.47 | 26.29 | - | - |
| 7/28/2014 | - | - | - | - | - | 1.8 | 25.62 | - | - |
| 8/25/2014 | - | - | - | - | - | 2.4 | 26.2 | - | - |
| 9/17/2014 | - | - | - | - | - | 3.2 | 25.33 | - | - |
| 10/22/2014 | - | - | - | - | - | 3.46 | 24.75 | - | - |
| 11/19/2014 | - | - | - | - | - | 3.85 | 24.49 | - | - |
| 12/22/2014 | - | - | - | 5 | - | 4.13 | 24.9 | - | - |
| 1/29/2015 | - | - | - | - | - | 4.36 | 25.95 | - | - |
| 2/24/2015 | - | - | - | - | - | 4.59 | 26.07 | - | - |
| 3/23/2015 | - | - | - | 5.25 | 16.77 | 4.8 | 26.32 | - | - |
| 4/30/2015 | 8.323 | 6.136 | 5.355 | - | - | 4.74 | 25.96 | 2.19 | 4.19 |
| 5/25/2015 | 3.816 | 3.488 | 3.362 | - | - | 2.7 | 25.5 | 0.1 | 1.64 |
| 6/19/2015 | 4.765 | 4.228 | 3.416 | - | - | 3.2 | 24.97 | 1.1 | 3.46 |
| 7/31/2015 | 5.968 | 5.382 | 3.817 | - | - | 2 | 27.26 | 2.41 | - |
| 8/28/2015 | 4.171 | 3.418 | 4.116 | - | - | 3.42 | 26.1 | 1.88 | 7.84 |
| 9/22/2015 | 6.205 | 4.126 | 3.531 | - | - | 3.51 | 24.2 | 1.95 | 4.03 |
| 10/23/2015 | 9.329 | 5.941 | 5.316 | 4.01 | 15.64 | 4.06 | 23.6 | 2.25 | 2.48 |
| 11/20/2015 | 6.967 | 6.054 | 5.047 | - | - | 3.88 | 24.19 | 2.55 | 3.93 |
| 1/29/2016 | 7.478 | 5.962 | 5.403 | - | - | 4.36 | 26.11 | 2.83 | 5.13 |
| 2/29/2016 | 7.321 | 5.054 | 5.3 | - | - | 4.07 | 28.05 | 2.95 | 4.15 |
| 3/25/2016 | 7.988 | 5.931 | 5.863 | 4.91 | 17.25 | 4.33 | 27.48 | 2.05 | 4.73 |
| 4/22/2016 | 7.916 | 6.486 | 5.507 | - | - | 4 | 27.32 | 1.72 | 4.66 |
| 5/20/2016 | - | 7.497 | 5.789 | - | - | 4.17 | 27.41 | 2.21 | 5.08 |

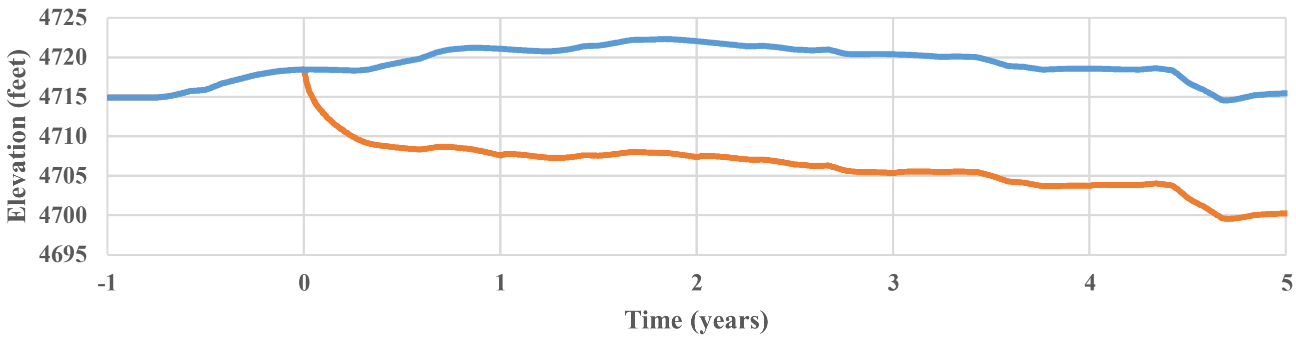
Notes:
All field measurements reduced by height of casing to reflect depth of water below ground surface.
Lorenz monitoring wells are measured in 15-minute intervals, with selected values shown.

**Appendix A 6.1**
**Town of Gilcrest**
**Modeled Effects of Dewatering**
**Optimized Wells Scenario**

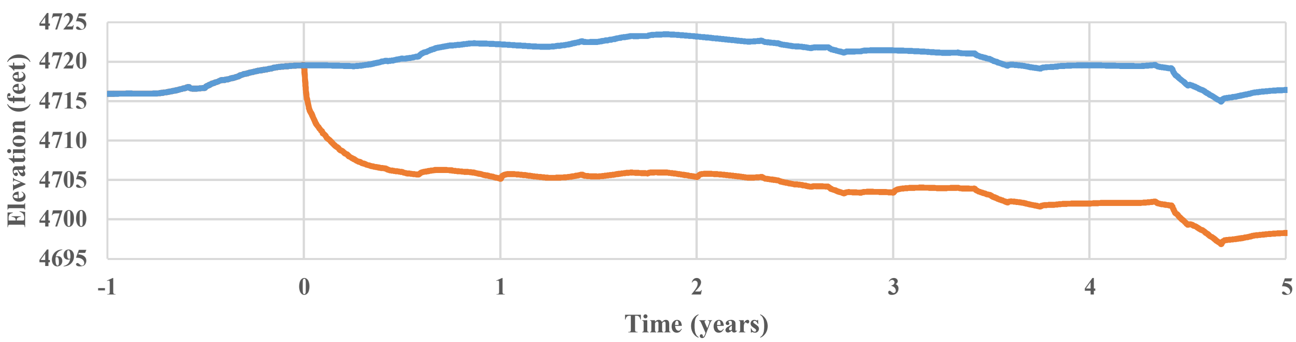
Groundwater Level Hydrographs

**Target Drawdown** **Modeled Drawdown**

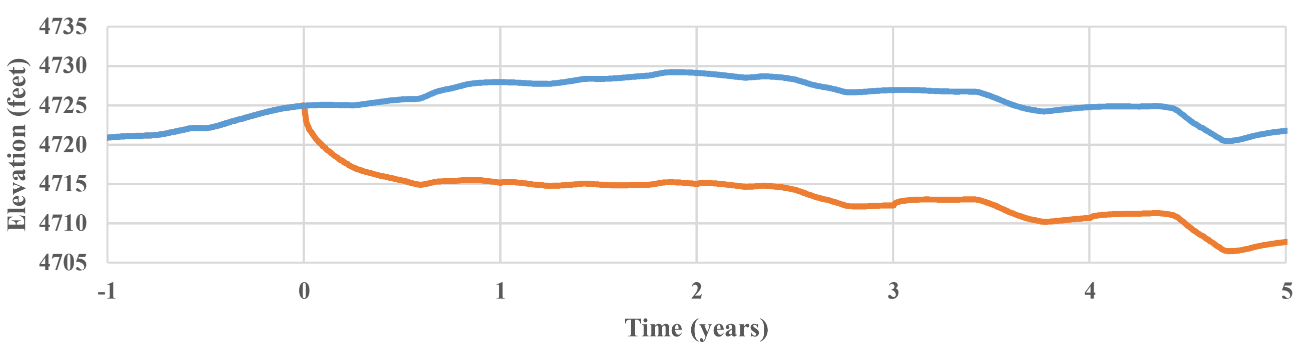
## WWTP (R287, C126)




## GMP (R287, C125)




## M+E (R289, C123)



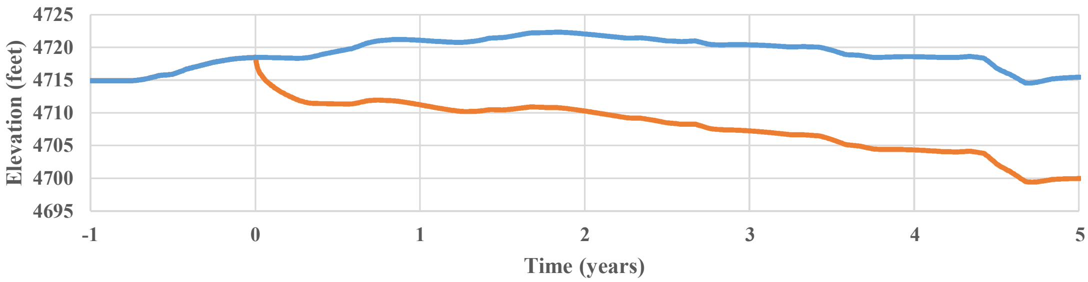

**Notes:**
Water level elevations from SPDSS MODFLOW model.

**Appendix A 6.2**
**Town of Gilcrest**
**Modeled Effects of Dewatering**
**Existing Wells Scenario**

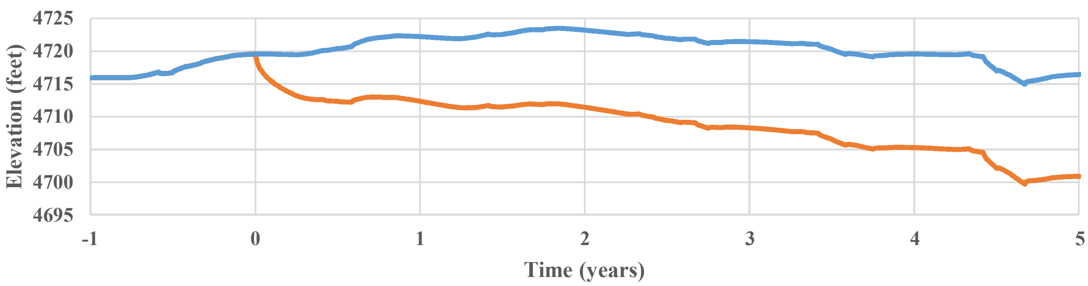
Groundwater Level Hydrographs

**Target Drawdown** **Modeled Drawdown**

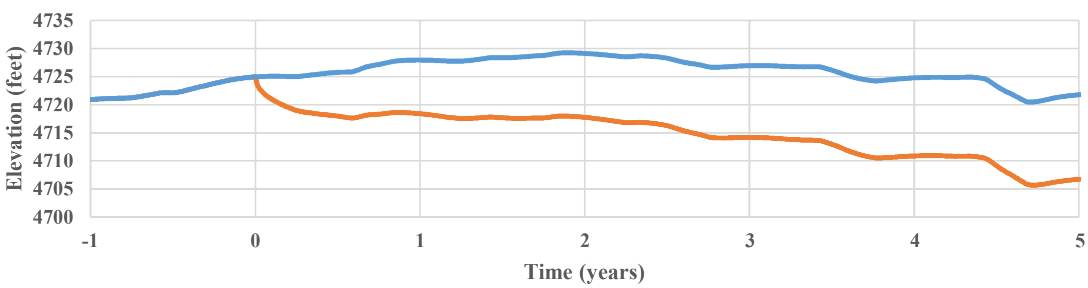
## WWTP (R287, C126)



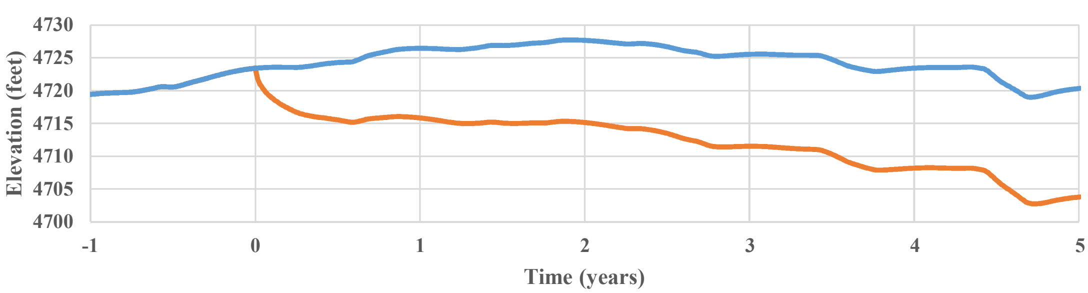

## GMP (R287, C125)




## M+E (R289, C123)




## TH (R289, C124)




**Notes:**
Water level elevations from SPDSS MODFLOW model.

# APPENDIX B – CALCULATIONS

## WWTP Pump Sizing Calculations Only Pump in Operation

| Parameter | Description | Value | Unit | Note |
|---|---|---|---|---|
| **Step 1: Define or determine process variables** | | | | |
| $Q_{DESIGN}$ | Design Flow From Well | 450 | gpm | |
| $N_P$ | Number of pumps | 1 | | |
| $N_{Pr}$ | Number of pumps running | 1 | | |
| $D_{DESIGN}$ | Pipe Design Diameter | 16.0 | in | |
| **Suction line** | | | | |
| $E_{SCL}$ | Pump suction centerline elevation | 4657.00 | ft | |
| $E_{SHW}$ | High water level - suction side | 4657.00 | ft | |
| $E_{SLW}$ | Low water level - suction side | 4657.00 | ft | Assumed Depth of Well 90 ft |
| $D_{SNOM}$ | Nominal suction diameter | 16.0 | inch | |
| **Discharge line** | | | | |
| $H_{MAX}$ | Discharge Max Height | 4747.00 | ft | |
| $D_{DNOM}$ | Nominal discharge diameter | 16.0 | inch | |
| **Step 2: Determine Friction Head ($H_L$-FRICTION) for pump system** | | | | |

| | | |
|---|---|---|
| Friction Head Loss for Pipes | $h_f$ = .002083 x (L) x (100/C)^1.85 x (Q^1.85/D^4.8655) | Hazen-Williams Equation |
| Friction Head Loss Fittings & Valves | $h_f = K \times (V^2/2g)$ | |
| Total Static Head | $h_l$ = Elevation Difference | |
| Velocity Head | $h_v = V^2/2g$ | |
| Total Dynamic Head | TDH = hl + hf + hv | |

**AT MAXIMUM FLOW RATE**

| Item Description | Pipe ID (inches) D | Length or Quantity (feet or number) L or n | Flow (gpm) Q | Velocity (fps) V=Q/A | Velocity Head $V^2/2g$ | Resistance Factor k | Resistance Factor Sum k | Resistance Factor C | Friction Head (feet) $h_f$ |
|---|---|---|---|---|---|---|---|---|---|
| **Pump Suction** | | | | | | | | | |
| Entrance Losses | 6.00 | 1 | 450 | 5.11 | 0.41 | 1.000 | 1.000 | | 0.406 |
| | | | | | Sum: | | 1.000 | | |
| **Pump Discharge** | | | | | | | | | |
| Pump No. 1 | | | | | | | | | |
| Pipe | 5.42 | 1,390.00 | 450 | 6.26 | 0.61 | | | 150 | 29.724 |
| Pipe + GMP | 13.09 | 3,400.00 | 450 | 1.07 | 0.02 | | | 150 | 0.996 |
| Pipe + GMP + M+E | 13.09 | 1,520.00 | 450 | 1.07 | 0.02 | | | 150 | 0.445 |
| | | | | | Sum: | | 0.000 | | |
| Discharge Piping | | | | | | | | | |
| Check Valve | 6.00 | 1 | 450 | 5.11 | 0.41 | 4.000 | 4.000 | | 1.623 |
| Butterfly Valve | 6.00 | 1 | 450 | 5.11 | 0.41 | 0.400 | 0.400 | | 0.162 |
| Elbow (90°) | 6.00 | 1 | 450 | 5.11 | 0.41 | 0.420 | 0.420 | | 0.170 |
| Through Tee to at GMP | 16.00 | 1 | 450 | 0.72 | 0.01 | 0.300 | 0.300 | | 0.002 |
| Elbow (90°) | 16.00 | 5 | 450 | 0.72 | 0.01 | 0.420 | 2.100 | | 0.017 |
| Elbow (45°) | 16.00 | 2 | 450 | 0.72 | 0.01 | 0.150 | 0.300 | | 0.002 |
| Through Tee at M+E | 16.00 | 1 | 450 | 0.72 | 0.01 | 0.300 | 0.300 | | 0.002 |
| Duckbill Valve (outlet) | 16.00 | 1 | 450 | 0.72 | 0.01 | 0.500 | 0.500 | | 0.004 |
| | | | | | Sum: | | 8.320 | | |

| | | | |
|---|---|---|---|
| Pump η | 70.00% | Total Discharge Friction Head | 33.15 |
| Motor η | 95.00% | Total Suction Friction Head | 0.41 |
| Hydraulic hp | 15.4 | Minimum Total Static Head | 90.00 |
| Bhp | 22.1 | Maximum Total Static Head | 90.00 |
| Motor hp | 23.2 | ***Minimum Total Dynamic Head*** | 123.55 |
| | | ***Maximum Total Dynamic Head*** | 123.55 |
| | | SF | 1.1 |
| | | ***Maximum Total Dynamic Head*** | 135.91 |

## GMP Pump Sizing Calculations Only Pump in Operation

| Parameter | Description | Value | Unit | Note |
|---|---|---|---|---|
| **Step 1: Define or determine process variables** | | | | |
| $Q_{DESIGN}$ | Design Flow From Well | 2500 | gpm | |
| $N_P$ | Number of pumps | 1 | | |
| $N_{Pr}$ | Number of pumps running | 1 | | |
| $D_{DESIGN}$ | Pipe Design Diameter | 16.0 | in | |
| **Suction line** | | | | |
| $E_{SCL}$ | Pump suction centerline elevation | 4657.00 | ft | |
| $E_{SHW}$ | High water level - suction side | 4657.00 | ft | |
| $E_{SLW}$ | Low water level - suction side | 4657.00 | ft | Assumed Depth of Well 90 ft |
| $D_{SNOM}$ | Nominal suction diameter | 16.0 | inch | |
| **Discharge line** | | | | |
| $H_{MAX}$ | Discharge Max Height | 4747.00 | ft | |
| $D_{DNOM}$ | Nominal discharge diameter | 16.0 | inch | |
| **Step 2: Determine Friction Head ($H_L$-FRICTION) for pump system** | | | | |

| | | |
|---|---|---|
| Friction Head Loss for Pipes | $h_f$ = .002083 x (L) x (100/C)^1.85 x (Q^1.85/D^4.8655) | Hazen-Williams Equation |
| Friction Head Loss Fittings & Valves | $h_f = K \times (V^2/2g)$ | |
| Total Static Head | $h_l$ = Elevation Difference | |
| Velocity Head | $h_v = V^2/2g$ | |
| Total Dynamic Head | $TDH = hl + hf + hv$ | |

**AT MAXIMUM FLOW RATE**

| Item Description | Pipe ID (inches) D | Length or Quantity (feet or number) L or n | Flow (gpm) Q | Velocity (fps) V=Q/A | Velocity Head $V^2/2g$ | Resistance Factor k | Resistance Factor Sum k | Resistance Factor C | Friction Head (feet) $h_f$ |
|---|---|---|---|---|---|---|---|---|---|
| **Pump Suction** | | | | | | | | | |
| Entrance Losses | 16.00 | 1 | 2,500 | 3.99 | 0.25 | 1.000 | 1.000 | | 0.248 |
| | | | | | **Sum:** | | **1.000** | | |
| **Pump Discharge** | | | | | | | | | |
| Pump No. 1 | | | | | | | | | |
| Pipe | 13.09 | 3,400.00 | 2,500 | 5.96 | 0.55 | | | 150 | 23.775 |
| Pipe + M+E | 13.09 | 1,520.00 | 2,500 | 5.96 | 0.55 | | | 150 | 10.629 |
| | | | | | **Sum:** | | **0.000** | | |
| Discharge Piping | | | | | | | | | |
| Check Valve | 16.00 | 1 | 2,500 | 3.99 | 0.25 | 4.000 | 4.000 | | 0.990 |
| Butterfly Valve | 16.00 | 1 | 2,500 | 3.99 | 0.25 | 0.400 | 0.400 | | 0.099 |
| Branch Tee to Force Main | 16.00 | 1 | 2,500 | 3.99 | 0.25 | 0.750 | 0.750 | | 0.186 |
| Elbow (90°) | 16.00 | 3 | 2,500 | 3.99 | 0.25 | 0.420 | 1.260 | | 0.312 |
| Through Tee at M+E | 16.00 | 1 | 2,500 | 3.99 | 0.25 | 0.300 | 0.300 | | 0.074 |
| Elbow (45°) | 16.00 | 2 | 2,500 | 3.99 | 0.25 | 0.150 | 0.300 | | 0.074 |
| Elbow (90°) | 16 | 2 | 2,500 | 3.99 | 0.25 | 0.420 | 0.840 | | 0.208 |
| Duckbill Valve (outlet) | 16.00 | 1 | 2,500 | 3.99 | 0.25 | 0.500 | 0.500 | | 0.124 |
| | | | | | **Sum:** | | **8.350** | | |

| | | | |
|---|---|---|---|
| Pump η | 70.00% | Total Discharge Friction Head | 36.47 |
| Motor η | 95.00% | Total Suction Friction Head | 0.25 |
| Hydraulic hp | 88.0 | Minimum Total Static Head | 90.00 |
| Bhp | 125.7 | Maximum Total Static Head | 90.00 |
| Motor hp | 132.3 | ***Minimum Total Dynamic Head*** | 126.72 |
| | | ***Maximum Total Dynamic Head*** | 126.72 |
| | | SF | 1.1 |
| | | ***Maximum Total Dynamic Head*** | 139.39 |

## M+E Pump Sizing Calculations Only Pump in Operation

| Parameter | Description | Value | Unit | Note |
|---|---|---|---|---|
| **Step 1: Define or determine process variables** | | | | |
| $Q_{DESIGN}$ | Design Flow From Well | 1400 | gpm | |
| $N_P$ | Number of pumps | 1 | | |
| $N_{Pr}$ | Number of pumps running | 1 | | |
| $D_{DESIGN}$ | Pipe Design Diameter | 16.0 | in | |
| **Suction line** | | | | |
| $E_{SCL}$ | Pump suction centerline elevation | 4657.00 | ft | |
| $E_{SHW}$ | High water level - suction side | 4657.00 | ft | |
| $E_{SLW}$ | Low water level - suction side | 4657.00 | ft | Assumed Depth of Well 90 ft |
| $D_{SNOM}$ | Nominal suction diameter | 16.0 | inch | |
| **Discharge line** | | | | |
| $H_{MAX}$ | Discharge Max Height | 4747.00 | ft | |
| $D_{DNOM}$ | Nominal discharge diameter | 16.0 | inch | |

**Step 2: Determine Friction Head ($H_L$-FRICTION) for pump system**

| | | |
|---|---|---|
| Friction Head Loss for Pipes | $h_f$ = .002083 x (L) x (100/C)^1.85 x (Q^1.85/D^4.8655) | Hazen-Williams Equation |
| Friction Head Loss Fittings & Valves | $h_f = K \times (V^2/2g)$ | |
| Total Static Head | $h_l$ = Elevation Difference | |
| Velocity Head | $h_v = V^2/2g$ | |
| Total Dynamic Head | $TDH = hl + hf + hv$ | |

**AT MAXIMUM FLOW RATE**

| Item Description | Pipe ID (inches) D | Length or Quantity (feet or number) L or n | Flow (gpm) Q | Velocity (fps) V=Q/A | Velocity Head $V^2/2g$ | Resistance Factor k | Resistance Factor Sum k | Resistance Factor C | Friction Head (feet) $h_f$ |
|---|---|---|---|---|---|---|---|---|---|
| **Pump Suction** | | | | | | | | | |
| Entrance Losses | 12.00 | 1 | 1,400 | 3.97 | 0.25 | 1.000 | 1.000 | | 0.245 |
| | | | | | **Sum:** | | **1.000** | | |
| **Pump Discharge** | | | | | | | | | |
| Pump No. 1 | | | | | | | | | |
| Pipe | 10.43 | 320.00 | 1,400 | 5.26 | 0.43 | | | 150 | 2.312 |
| Pipe + Forcemain | 13.09 | 1,520.00 | 1,400 | 3.34 | 0.17 | | | 150 | 3.636 |
| | | | | | **Sum:** | | **0.000** | | |
| Discharge Piping | | | | | | | | | |
| Check Valve | 12.00 | 1 | 1,400 | 3.97 | 0.25 | 4.000 | 4.000 | | 0.982 |
| Butterfly Valve | 12.00 | 1 | 1,400 | 3.97 | 0.25 | 0.400 | 0.400 | | 0.098 |
| Elbow (90°) | 12.00 | 1 | 1,400 | 3.97 | 0.25 | 0.420 | 0.420 | | 0.103 |
| Branch Tee to Force Main | 16.00 | 1 | 1,400 | 2.24 | 0.08 | 0.750 | 0.750 | | 0.058 |
| Elbow (90°) | 16.00 | 2 | 1,400 | 2.24 | 0.08 | 0.420 | 0.840 | | 0.065 |
| Elbow (45°) | 16.00 | 2 | 1,400 | 2.24 | 0.08 | 0.150 | 0.300 | | 0.023 |
| Duckbill Valve (outlet) | 16.00 | 1 | 1,400 | 2.24 | 0.08 | 0.500 | 0.500 | | 0.039 |
| | | | | | **Sum:** | | **7.210** | | |
| | | | Pump η | 70.00% | | | | Total Discharge Friction Head | 7.32 |
| | | | Motor η | 95.00% | | | | Total Suction Friction Head | 0.25 |
| | | | Hydraulic hp | 37.9 | | | | Minimum Total Static Head | 90.00 |
| | | | Bhp | 54.2 | | | | Maximum Total Static Head | 90.00 |
| | | | Motor hp | 57.1 | | | | ***Minimum Total Dynamic Head*** | 97.56 |
| | | | | | | | | ***Maximum Total Dynamic Head*** | 97.56 |
| | | | | | | | | SF | 1.1 |
| | | | | | | | | ***Maximum Total Dynamic Head*** | 107.32 |

## WWTP Pump Sizing Calculations All Pumps in Operation

| Parameter | Description | Value | Unit | Note |
|---|---|---|---|---|
| **Step 1: Define or determine process variables** | | | | |
| $Q_{DESIGN}$ | Design Flow From Well | 450 | gpm | |
| $N_P$ | Number of pumps | 1 | | |
| $N_{Pr}$ | Number of pumps running | 1 | | |
| $D_{DESIGN}$ | Pipe Design Diameter | 16.0 | in | |
| **Suction line** | | | | |
| $E_{SCL}$ | Pump suction centerline elevation | 4657.00 | ft | |
| $E_{SHW}$ | High water level - suction side | 4657.00 | ft | |
| $E_{SLW}$ | Low water level - suction side | 4657.00 | ft | Assumed Depth of Well 90 ft |
| $D_{SNOM}$ | Nominal suction diameter | 16.0 | inch | |
| **Discharge line** | | | | |
| $H_{MAX}$ | Discharge Max Height | 4747.00 | ft | |
| $D_{DNOM}$ | Nominal discharge diameter | 16.0 | inch | |

**Step 2: Determine Friction Head ($H_L$-FRICTION) for pump system**

| | | |
|---|---|---|
| Friction Head Loss for Pipes | $h_f = .002083 \times (L) \times (100/C)^{1.85} \times (Q^{1.85}/D^{4.8655})$ | Hazen-Williams Equation |
| Friction Head Loss Fittings & Valves | $h_f = K \times (V^2/2g)$ | |
| Total Static Head | $h_l$ = *Elevation Difference* | |
| Velocity Head | $h_v = V^2/2g$ | |
| Total Dynamic Head | $TDH = hl + hf + hv$ | |

**AT MAXIMUM FLOW RATE**

| Item Description | Pipe ID (inches) D | Length or Quantity (feet or number) L or n | Flow (gpm) Q | Velocity (fps) V=Q/A | Velocity Head $V^2/2g$ | Resistance Factor k | Resistance Factor Sum k | Resistance Factor C | Friction Head (feet) $h_f$ |
|---|---|---|---|---|---|---|---|---|---|
| **Pump Suction** | | | | | | | | | |
| Entrance Losses | 6.00 | 1 | 450 | 5.11 | 0.41 | 1.000 | 1.000 | | 0.406 |
| | | | | | **Sum:** | | **1.000** | | |
| **Pump Discharge** | | | | | | | | | |
| Pump No. 1 | | | | | | | | | |
| Pipe | 5.42 | 1,390.00 | 450 | 6.26 | 0.61 | | | 150 | 29.724 |
| Pipe + GMP | 13.09 | 3,400.00 | 2,950 | 7.04 | 0.77 | | | 150 | 32.292 |
| Pipe + GMP + M+E | 13.09 | 1,520.00 | 3,270 | 7.80 | 0.95 | | | 150 | 17.467 |
| | | | | | **Sum:** | | **0.000** | | |
| Discharge Piping | | | | | | | | | |
| Check Valve | 6.00 | 1 | 450 | 5.11 | 0.41 | 4.000 | 4.000 | | 1.623 |
| Butterfly Valve | 6.00 | 1 | 450 | 5.11 | 0.41 | 0.400 | 0.400 | | 0.162 |
| Elbow (90°) | 6.00 | 1 | 450 | 5.11 | 0.41 | 0.420 | 0.420 | | 0.170 |
| Through Tee to at GMP | 16.00 | 1 | 2,950 | 4.71 | 0.34 | 0.300 | 0.300 | | 0.103 |
| Elbow (90°) | 16.00 | 5 | 2,950 | 4.71 | 0.34 | 0.420 | 2.100 | | 0.724 |
| Elbow (45°) | 16.00 | 2 | 4,350 | 6.95 | 0.75 | 0.150 | 0.300 | | 0.225 |
| Through Tee at M+E | 16.00 | 1 | 4,350 | 6.95 | 0.75 | 0.300 | 0.300 | | 0.225 |
| Duckbill Valve (outlet) | 16.00 | 1 | 4,350 | 6.95 | 0.75 | 0.500 | 0.500 | | 0.375 |
| | | | | | **Sum:** | | **8.320** | | |
| | | | Pump η | 70.00% | | | | Total Discharge Friction Head | 83.09 |
| | | | Motor η | 95.00% | | | | Total Suction Friction Head | 0.41 |
| | | | Hydraulic hp | 21.7 | | | | Minimum Total Static Head | 90.00 |
| | | | Bhp | 31.0 | | | | Maximum Total Static Head | 90.00 |
| | | | Motor hp | 32.6 | | | | ***Minimum Total Dynamic Head*** | 173.50 |
| | | | | | | | | ***Maximum Total Dynamic Head*** | 173.50 |
| | | | | | | | | SF | 1.1 |
| | | | | | | | | ***Maximum Total Dynamic Head*** | 190.85 |

CONSULTING ENGINEERS

## GMP Pump Sizing Calculations All Pumps in Operation

| Parameter | Description | Value | Unit | Note |
|---|---|---|---|---|
| **Step 1: Define or determine process variables** | | | | |
| $Q_{DESIGN}$ | Design Flow From Well | 2500 | gpm | |
| $N_P$ | Number of pumps | 1 | | |
| $N_{Pr}$ | Number of pumps running | 1 | | |
| $D_{DESIGN}$ | Pipe Design Diameter | 16.0 | in | |
| **Suction line** | | | | |
| $E_{SCL}$ | Pump suction centerline elevation | 4657.00 | ft | |
| $E_{SHW}$ | High water level - suction side | 4657.00 | ft | |
| $E_{SLW}$ | Low water level - suction side | 4657.00 | ft | Assumed Depth of Well 90 ft |
| $D_{SNOM}$ | Nominal suction diameter | 16.0 | inch | |
| **Discharge line** | | | | |
| $H_{MAX}$ | Discharge Max Height | 4747.00 | ft | |
| $D_{DNOM}$ | Nominal discharge diameter | 16.0 | inch | |

**Step 2: Determine Friction Head ($H_L$-FRICTION) for pump system**

| | | |
|---|---|---|
| Friction Head Loss for Pipes | $h_f = .002083 \times (L) \times (100/C)^{1.85} \times (Q^{1.85}/D^{4.8655})$ | Hazen-Williams Equation |
| Friction Head Loss Fittings & Valves | $h_f = K \times (V^2/2g)$ | |
| Total Static Head | $h_l$ = *Elevation Difference* | |
| Velocity Head | $h_v = V^2/2g$ | |
| Total Dynamic Head | $TDH = hl + hf + hv$ | |

**AT MAXIMUM FLOW RATE**

| Item Description | Pipe ID (inches) D | Length or Quantity (feet or number) L or n | Flow (gpm) Q | Velocity (fps) V=Q/A | Velocity Head $V^2/2g$ | Resistance Factor k | Resistance Factor Sum k | Resistance Factor C | Friction Head (feet) $h_f$ |
|---|---|---|---|---|---|---|---|---|---|
| **Pump Suction** | | | | | | | | | |
| Entrance Losses | 16.00 | 1 | 2,500 | 3.99 | 0.25 | 1.000 | 1.000 | | 0.248 |
| | | | | | **Sum:** | | **1.000** | | |
| **Pump Discharge** | | | | | | | | | |
| Pump No. 1 | | | | | | | | | |
| Pipe | 13.09 | 3,400.00 | 2,500 | 5.96 | 0.55 | | | 150 | 23.775 |
| Pipe + M+E | 13.09 | 1,520.00 | 3,270 | 7.80 | 0.95 | | | 150 | 17.467 |
| | | | | | **Sum:** | | **0.000** | | |
| Discharge Piping | | | | | | | | | |
| Check Valve | 16.00 | 1 | 2,500 | 3.99 | 0.25 | 4.000 | 4.000 | | 0.990 |
| Butterfly Valve | 16.00 | 1 | 2,500 | 3.99 | 0.25 | 0.400 | 0.400 | | 0.099 |
| Branch Tee to Force Main | 16.00 | 1 | 2,950 | 4.71 | 0.34 | 0.750 | 0.750 | | 0.259 |
| Elbow (90°) | 16.00 | 3 | 2,500 | 3.99 | 0.25 | 0.420 | 1.260 | | 0.312 |
| Through Tee at M+E | 16.00 | 1 | 4,350 | 6.95 | 0.75 | 0.300 | 0.300 | | 0.225 |
| Elbow (45°) | 16.00 | 2 | 4,350 | 6.95 | 0.75 | 0.150 | 0.300 | | 0.225 |
| Elbow (90°) | 16 | 2 | 4,350 | 6.95 | 0.75 | 0.420 | 0.840 | | 0.630 |
| Duckbill Valve (outlet) | 16.00 | 1 | 4,350 | 6.95 | 0.75 | 0.500 | 0.500 | | 0.375 |
| | | | | | **Sum:** | | **8.350** | | |
| | | | Pump η | 70.00% | | | | Total Discharge Friction Head | 44.36 |
| | | | Motor η | 95.00% | | | | Total Suction Friction Head | 0.25 |
| | | | Hydraulic hp | 93.5 | | | | Minimum Total Static Head | 90.00 |
| | | | Bhp | 133.5 | | | | Maximum Total Static Head | 90.00 |
| | | | Motor hp | 140.6 | | | | ***Minimum Total Dynamic Head*** | 134.60 |
| | | | | | | | | ***Maximum Total Dynamic Head*** | 134.60 |
| | | | | | | | | SF | 1.1 |
| | | | | | | | | ***Maximum Total Dynamic Head*** | 148.06 |

## M+E Pump Sizing Calculations All Pumps in Operation

| Parameter | Description | Value | Unit | Note |
|---|---|---|---|---|
| **Step 1: Define or determine process variables** | | | | |
| $Q_{DESIGN}$ | Design Flow From Well | 1400 | gpm | |
| $N_P$ | Number of pumps | 1 | | |
| $N_{Pr}$ | Number of pumps running | 1 | | |
| $D_{DESIGN}$ | Pipe Design Diameter | 16.0 | in | |
| **Suction line** | | | | |
| $E_{SCL}$ | Pump suction centerline elevation | 4657.00 | ft | |
| $E_{SHW}$ | High water level - suction side | 4657.00 | ft | |
| $E_{SLW}$ | Low water level - suction side | 4657.00 | ft | Assumed Depth of Well 90 ft |
| $D_{SNOM}$ | Nominal suction diameter | 16.0 | inch | |
| **Discharge line** | | | | |
| $H_{MAX}$ | Discharge Max Height | 4747.00 | ft | |
| $D_{DNOM}$ | Nominal discharge diameter | 16.0 | inch | |
| **Step 2: Determine Friction Head ($H_L$-FRICTION) for pump system** | | | | |

Friction Head Loss for Pipes: $h_f = .002083 \times (L) \times (100/C)^{1.85} \times (Q^{1.85}/D^{4.8655})$ Hazen-Williams Equation

Friction Head Loss Fittings & Valves: $h_f = K \times (V^2/2g)$

Total Static Head: $h_l$ = *Elevation Difference*

Velocity Head: $h_v = V^2/2g$

Total Dynamic Head: $TDH = hl + hf + hv$

**AT MAXIMUM FLOW RATE**

| Item Description | Pipe ID (inches) D | Length or Quantity (feet or number) L or n | Flow (gpm) Q | Velocity (fps) V=Q/A | Velocity Head $V^2/2g$ | Resistance Factor k | Resistance Factor Sum k | Resistance Factor C | Friction Head (feet) $h_f$ |
|---|---|---|---|---|---|---|---|---|---|
| **Pump Suction** | | | | | | | | | |
| Entrance Losses | 12.00 | 1 | 1,400 | 3.97 | 0.25 | 1.000 | 1.000 | | 0.245 |
| | | | | | **Sum:** | | **1.000** | | |
| **Pump Discharge** | | | | | | | | | |
| Pump No. 1 | | | | | | | | | |
| Pipe | 10.43 | 320.00 | 1,400 | 5.26 | 0.43 | | | 150 | 2.312 |
| Pipe + Forcemain | 13.09 | 1,520.00 | 3,270 | 7.80 | 0.95 | | | 150 | 17.467 |
| | | | | | **Sum:** | | **0.000** | | |
| Discharge Piping | | | | | | | | | |
| Check Valve | 12.00 | 1 | 1,400 | 3.97 | 0.25 | 4.000 | 4.000 | | 0.982 |
| Butterfly Valve | 12.00 | 1 | 1,400 | 3.97 | 0.25 | 0.400 | 0.400 | | 0.098 |
| Elbow (90°) | 12.00 | 1 | 1,400 | 3.97 | 0.25 | 0.420 | 0.420 | | 0.103 |
| Branch Tee to Force Main | 16.00 | 1 | 4,350 | 6.95 | 0.75 | 0.750 | 0.750 | | 0.562 |
| Elbow (90°) | 16.00 | 2 | 4,350 | 6.95 | 0.75 | 0.420 | 0.840 | | 0.630 |
| Elbow (45°) | 16.00 | 2 | 4,350 | 6.95 | 0.75 | 0.150 | 0.300 | | 0.225 |
| Duckbill Valve (outlet) | 16.00 | 1 | 4,350 | 6.95 | 0.75 | 0.500 | 0.500 | | 0.375 |
| | | | | | **Sum:** | | **7.210** | | |
| | | | Pump η | 70.00% | | | Total Discharge Friction Head | | 22.75 |
| | | | Motor η | 95.00% | | | Total Suction Friction Head | | 0.25 |
| | | | Hydraulic hp | 43.9 | | | Minimum Total Static Head | | 90.00 |
| | | | Bhp | 62.8 | | | Maximum Total Static Head | | 90.00 |
| | | | Motor hp | 66.1 | | | ***Minimum Total Dynamic Head*** | | 113.00 |
| | | | | | | | ***Maximum Total Dynamic Head*** | | 113.00 |
| | | | | | | | SF | | 1.1 |
| | | | | | | | ***Maximum Total Dynamic Head*** | | 124.30 |

## Alternative 1 Pump Sizing Calculations Startup Head Required

| Parameter | Description | Value | Unit | Note |
|---|---|---|---|---|
| **Step 1: Define or determine process variables** | | | | |
| $Q_{DESIGN}$ | Design Flow | 4350 | gpm | |
| $N_P$ | Number of pumps | 3 | | |
| $N_{Pr}$ | Number of pumps running | 2 | | |
| $D_{DESIGN}$ | Design diameter | 16.0 | in | |
| **Suction line** | | | | |
| $E_{SCL}$ | Pump suction centerline elevation | 4733.00 | ft | |
| $E_{SHW}$ | High water level - suction side | 4738.00 | ft | |
| $E_{SLW}$ | Low water level - suction side | 4735.00 | ft | |
| $D_{SNOM}$ | Nominal suction diameter | 16.0 | inch | |
| **Discharge line** | | | | |
| $H_{MAX}$ | Discharge Max Height | 4761.00 | ft | Highest Elevation in Line |
| $D_{DNOM}$ | Nominal discharge diameter | 16.0 | inch | |
| **Step 2: Determine Friction Head ($H_L$-FRICTION) for pump system** | | | | |

| | | |
|---|---|---|
| Friction Head Loss for Pipes | $h_f = .002083 \times (L) \times (100/C)^{1.85} \times (Q^{1.85}/D^{4.8655})$ | Hazen-Williams Equation |
| Friction Head Loss Fittings & Valves | $h_f = K \times (V^2/2g)$ | |
| Total Static Head | $h_l$ = Elevation Difference | |
| Velocity Head | $h_v = V^2/2g$ | |
| Total Dynamic Head | $TDH = hl + hf + hv$ | |

**AT MAXIMUM FLOW RATE**

| Item Description | Pipe ID (inches) D | Length or Quantity (feet or number) L or n | Flow (gpm) Q | Velocity (fps) V=Q/A | Velocity Head $V^2/2g$ | Resistance Factor k | Resistance Factor Sum k | Resistance Factor C | Friction Head (feet) $h_f$ |
|---|---|---|---|---|---|---|---|---|---|
| **Pump Suction** | | | | | | | | | |
| Entrance Losses | 16.00 | 1 | 4,350 | 6.95 | 0.75 | 1.000 | 1.000 | | 0.750 |
| | | | | | **Sum:** | | **1.000** | | |
| **Pump Discharge** | | | | | | | | | |
| Check Valve | 16.00 | 3 | 1,450 | 2.32 | 0.08 | 4.000 | 12.000 | | 0.999 |
| Butterfly Valve | 16.00 | 3 | 1,450 | 2.32 | 0.08 | 0.4 | 1.200 | | 0.100 |
| Pipe | 15.47 | 5,491.00 | 4,350 | 7.43 | 0.86 | | | 150 | 47.458 |
| Elbow (90°) | 16.00 | 3 | 4,350 | 6.95 | 0.75 | 0.300 | 0.900 | | 0.675 |
| | | | | | **Sum:** | | **14.100** | | |

| | | | |
|---|---|---|---|
| Pump η | 70.00% | Total Discharge Friction Head | 48.13 |
| Motor η | 95.00% | Total Suction Friction Head | 0.75 |
| Hydraulic hp | 45.2 | Minimum Total Static Head | 23.00 |
| Bhp | 64.6 | Maximum Total Static Head | 26.00 |
| Motor hp | 68.0 | ***Minimum Total Dynamic Head*** | 71.88 |
| | | ***Maximum Total Dynamic Head*** | 74.88 |
| | | SF | 1.1 |
| | | | 82.37 |

## Alternative 1 Pump Sizing Calculations Normal Operation

| Parameter | Description | Value | Unit | Note |
|---|---|---|---|---|
| **Step 1: Define or determine process variables** | | | | |
| $Q_{DESIGN}$ | Design Flow | 4350 | gpm | |
| $N_P$ | Number of pumps | 3 | | |
| $N_{Pr}$ | Number of pumps running | 2 | | |
| $D_{DESIGN}$ | Design diameter | 16.0 | in | |
| **Suction line** | | | | |
| $E_{SCL}$ | Pump suction centerline elevation | 4733.00 | ft | |
| $E_{SHW}$ | High water level - suction side | 4738.00 | ft | |
| $E_{SLW}$ | Low water level - suction side | 4735.00 | ft | |
| $D_{SNOM}$ | Nominal suction diameter | 16.0 | inch | |
| **Discharge line** | | | | |
| $H_{MAX}$ | Discharge Max Height | 4720.00 | ft | |
| $D_{DNOM}$ | Nominal discharge diameter | 16.0 | inch | |
| **Step 2: Determine Friction Head ($H_L$-FRICTION) for pump system** | | | | |

| | | |
|---|---|---|
| Friction Head Loss for Pipes | $h_f = .002083 \times (L) \times (100/C)^{1.85} \times (Q^{1.85}/D^{4.8655})$ | Hazen-Williams Equation |
| Friction Head Loss Fittings & Valves | $h_f = K \times (V^2/2g)$ | |
| Total Static Head | $h_l$ = *Elevation Difference* | |
| Velocity Head | $h_v = V^2/2g$ | |
| Total Dynamic Head | $TDH = hl + hf + hv$ | |

**AT MAXIMUM FLOW RATE**

| Item Description | Pipe ID (inches) D | Length or Quantity (feet or number) L or n | Flow (gpm) Q | Velocity (fps) V=Q/A | Velocity Head $V^2/2g$ | Resistance Factor k | Resistance Factor Sum k | Resistance Factor C | Friction Head (feet) $h_f$ |
|---|---|---|---|---|---|---|---|---|---|
| **Pump Suction** | | | | | | | | | |
| Entrance Losses | 16.00 | 1 | 4,350 | 6.95 | 0.75 | 1.000 | 1.000 | | 0.750 |
| | | | | | **Sum:** | | **1.000** | | |
| **Pump Discharge** | | | | | | | | | |
| Check Valve | 16.00 | 3 | 1,450 | 2.32 | 0.08 | 4.000 | 12.000 | | 0.999 |
| Butterfly Valve | 16.00 | 3 | 1,450 | 2.32 | 0.08 | 0.4 | 1.200 | | 0.100 |
| Pipe | 15.47 | 19,219.00 | 4,350 | 7.43 | 0.86 | | | 150 | 166.108 |
| Elbow (90°) | 16.00 | 4 | 4,350 | 6.95 | 0.75 | 0.300 | 1.200 | | 0.900 |
| Elbow (45°) | 16.00 | 2 | 4,350 | 6.95 | 0.75 | 0.150 | 0.300 | | 0.225 |
| | | | | | **Sum:** | | **14.700** | | |
| | | | Pump η | 70.00% | | | Total Discharge Friction Head | | 167.23 |
| | | | Motor η | 95.00% | | | Total Suction Friction Head | | 0.75 |
| | | | Hydraulic hp | 92.4 | | | Minimum Total Static Head | | -18.00 |
| | | | Bhp | 132.0 | | | Maximum Total Static Head | | -15.00 |
| | | | Motor hp | 139.0 | | | ***Minimum Total Dynamic Head*** | | 149.98 |
| | | | | | | | ***Maximum Total Dynamic Head*** | | 152.98 |
| | | | | | | | SF | | 1.1 |
| | | | | | | | | | 168.28 |

## Alternative 3 Pump Sizing Calculations

| Parameter | Description | Value | Unit | Note |
|---|---|---|---|---|
| **Step 1: Define or determine process variables** | | | | |
| $Q_{DESIGN}$ | Design Flow | 4350 | gpm | Assume only Gilcrest dewatering flow rates and not additional stakeholder flows. |
| $N_P$ | Number of pumps | 3 | | |
| $N_{Pr}$ | Number of pumps running | 2 | | |
| $D_{DESIGN}$ | Design diameter | 16.0 | in | |
| **Suction line** | | | | |
| $E_{SCL}$ | Pump suction centerline elevation | 4733.00 | ft | |
| $E_{SHW}$ | High water level - suction side | 4738.00 | ft | |
| $E_{SLW}$ | Low water level - suction side | 4735.00 | ft | |
| $D_{SNOM}$ | Nominal suction diameter | 16.0 | inch | |
| **Discharge line** | | | | |
| $H_{MAX}$ | Discharge Max Height | 4666.00 | ft | |
| $D_{DNOM}$ | Nominal discharge diameter | 16.0 | inch | |
| **Step 2: Determine Friction Head ($H_L$-FRICTION) for pump system** | | | | |

Friction Head Loss for Pipes: *$h_f$ = .002083 x (L) x (100/C)^1.85 x (Q^1.85/D^4.8655)* Hazen-Williams Equation
Friction Head Loss Fittings & Valves: *$h_f = K \times (V^2/2g)$*
Total Static Head: *$h_l$ = Elevation Difference*
Velocity Head: *$h_v = V^2/2g$*
Total Dynamic Head: *TDH = hl + hf + hv*

**AT MAXIMUM FLOW RATE**

| Item Description | Pipe ID (inches) D | Length or Quantity (feet or number) L or n | Flow (gpm) Q | Velocity (fps) V=Q/A | Velocity Head $V^2/2g$ | Resistance Factor k | Resistance Factor Sum k | Resistance Factor C | Friction Head (feet) $h_f$ |
|---|---|---|---|---|---|---|---|---|---|
| **Pump Suction** | | | | | | | | | |
| Entrance Losses | 16.00 | 1 | 4,350 | 6.95 | 0.75 | 1.000 | 1.000 | | 0.750 |
| Check Valve | 16.00 | 1 | 4,350 | 6.95 | 0.75 | 4.000 | 4.000 | | 2.998 |
| | | | | | **Sum:** | | **5.000** | | |
| **Pump Discharge** | | | | | | | | | |
| Pump No. 1 | | | | | | | | | |
| Pipe | 15.47 | 38,519.00 | 4,350 | 7.43 | 0.86 | | | 150 | 332.917 |
| | | | | | **Sum:** | | **0.000** | | |
| Discharge Piping | | | | | | | | | |
| Elbow (90°) | 16.00 | 4 | 4,350 | 6.95 | 0.75 | 0.300 | 1.200 | | 0.900 |
| Elbow (45°) | 16.00 | 2 | 4,350 | 6.95 | 0.75 | 0.150 | 0.300 | | 0.225 |
| | | | | | **Sum:** | | **1.500** | | |

| | | | |
|---|---|---|---|
| Pump η | 70.00% | Total Discharge Friction Head | 334.04 |
| Motor η | 95.00% | Total Suction Friction Head | 3.75 |
| Hydraulic hp | 162.4 | Minimum Total Static Head | -72.00 |
| Bhp | 232.0 | Maximum Total Static Head | -69.00 |
| Motor hp | 244.2 | ***Minimum Total Dynamic Head*** | 265.79 |
| | | ***Maximum Total Dynamic Head*** | 268.79 |
| | | SF | 1.1 |
| | | | 295.67 |

# APPENDIX C – COST ESTIMATES

# OPINION OF PROBABLE COST
## FOR
# DEWATERING IMPROVEMENTS STUDY: ALTERNATIVE 1
# TOWN OF GILCREST

| Description | Quantity | Units | Unit Cost | Total Cost |
|---|---|---|---|---|
| **Division 00 and 01 - General Conditions and Requirements** | | | | |
| Mobilization/Demobilization | 1 | LS | $50,000 | $50,000 |
| | | | **General Requirements Subtotal** | **$50,000** |
| **Division 02 - Sitework** | | | | |
| Erosion Control | 1 | LS | $7,500 | $7,500 |
| Excavation / Fill | 464 | CY | $10 | $4,700 |
| Dewatering | 1 | LS | $25,000 | $25,000 |
| Prep and Impermeable Liner | 12100 | SF | $1.25 | $15,200 |
| Site Grading | 1 | LS | $2,500 | $2,500 |
| Chainlink Fence | 100 | LF | $40 | $4,000 |
| Dewatering Site Piping HDPE 6" | 1390 | LF | $34 | $47,600 |
| Dewatering Site Piping HDPE 12" | 320 | LF | $70 | $22,400 |
| Dewatering Site Piping HDPE 16" | 4920 | LF | $94 | $460,100 |
| Discharge Site Piping HDPE 16" | 19219 | LF | $94 | $1,797,000 |
| Seeding | 258490 | SF | $0.05 | $13,000 |
| Dewatering Well | 1 | LS | $80,000.00 | $80,000 |
| Dewatering Well | 1 | LS | $100,000.00 | $100,000 |
| Dewatering Well | 1 | LS | $120,000.00 | $120,000 |
| Monitoring Wells | 2 | EA | $2,600.00 | $5,200 |
| | | | **Sitework Subtotal** | **$2,704,200** |
| **Division 03 - Concrete** | | | | |
| Concrete | 23 | CY | $750 | $17,300 |
| Precast Vaults | 3 | EA | $8,000 | $24,000 |
| | | | **Concrete Subtotal** | **$41,300** |
| **Division 09 - Painting** | | | | |
| Pipe Coatings | 1 | LS | $15,000 | $15,000 |
| | | | **Painting Subtotal** | **$15,000** |
| **Division 11 - Equipment** | | | | |
| 6" Magmeter | 1 | EA | $6,000 | $6,000 |
| 12" Magmeter | 1 | EA | $12,000 | $12,000 |
| 16" Magmeter | 2 | EA | $20,000 | $40,000 |
| Vertical Turbine Pumps | 3 | EA | $101,428 | $304,300 |
| Dewatering Well Pump 500gpm | 1 | EA | $23,000 | $23,000 |
| Dewatering Well Pump 1,500gpm | 1 | EA | $37,950 | $38,000 |
| Dewatering Well Pump 2,500gpm | 1 | EA | $75,500 | $75,500 |
| Surge Tank | 1 | EA | $125,000 | $125,000 |
| | | | **Equipment Subtotal** | **$623,800** |
| **Division 15 - Mechanical** | | | | |
| Butterfly Valves 6" | 1 | EA | $975 | $1,000 |
| Butterfly Valves 12" | 1 | EA | $4,475 | $4,500 |
| Butterfly Valves 16" | 4 | EA | $8,357 | $33,500 |
| Check Valves 6" | 1 | EA | $1,688 | $1,700 |
| Check Valves 12" | 1 | EA | $6,216 | $6,300 |
| Check Valves 16" | 4 | EA | $16,136 | $64,600 |
| Guide Rails/Hatches | 3 | EA | $10,000 | $30,000 |
| | | | **Mechanical Subtotal** | **$141,600** |
| **Division 16 - Electrical** | | | | |
| Electrical | 1 | LS | $250,000 | $350,000 |
| Instrumentation and Controls | 1 | LS | $100,000 | $150,000 |
| | | | **Electrical Subtotal** | **$500,000** |

| | |
|---|---|
| **Subtotal** | **$4,075,900** |
| Contingency (30%) | $1,223,000 |
| Contractor's OH&P (15%) | $795,000 |
| Engineering, Permitting and Design (10% or fixed fee) | $408,000 |
| Bidding and Construction Administration (7% or fixed fee) | $455,133 |
| Administrative and Legal (5%) | $204,000 |
| **Project Total** | **$7,161,000** |

CONSULTING ENGINEERS

## OPINION OF PROBABLE COST
## FOR
## DEWATERING IMPROVEMENTS STUDY: ALTERNATIVE 3
## TOWN OF GILCREST

| Description | Quantity | Units | Unit Cost | Total Cost |
|---|---|---|---|---|
| **Division 00 and 01 - General Conditions and Requirements** | | | | |
| Mobilization/Demobilization | 1 | LS | $50,000 | $50,000 |
| | | | **General Requirements Subtotal** | **$50,000** |
| **Division 02 - Sitework** | | | | |
| Erosion Control | 1 | LS | $7,500 | $7,500 |
| Excavation / Fill | 464 | CY | $10 | $4,700 |
| Dewatering | 1 | LS | $25,000 | $25,000 |
| Prep and Impermeable Liner | 12100 | SF | $1.25 | $15,200 |
| Site Grading | 1 | LS | $2,500 | $2,500 |
| Chainlink Fence | 100 | LF | $40 | $4,000 |
| Dewatering Site Piping HDPE 6" | 1390 | LF | $34 | $47,600 |
| Dewatering Site Piping HDPE 12" | 320 | LF | $70 | $22,400 |
| Dewatering Site Piping HDPE 16" | 4920 | LF | $94 | $460,100 |
| Discharge Site Piping HDPE 16" | 4100 | LF | $94 | $383,400 |
| Discharge Site Piping HDPE 20" | 11750 | LF | $110 | $1,292,500 |
| Discharge Site Piping HDPE 24" | 20620 | LF | $120 | $2,474,400 |
| Seeding | 431000 | SF | $0.05 | $21,600 |
| Dewatering Well | 1 | LS | $80,000.00 | $80,000 |
| Dewatering Well | 1 | LS | $100,000.00 | $100,000 |
| Dewatering Well | 1 | LS | $120,000.00 | $120,000 |
| Monitoring Wells | 2 | EA | $2,600.00 | $5,200 |
| | | | **Sitework Subtotal** | **$5,066,100** |
| **Division 03 - Concrete** | | | | |
| Concrete | 23 | CY | $750 | $17,300 |
| Precast Vaults | 3 | EA | $8,000 | $24,000 |
| | | | **Concrete Subtotal** | **$41,300** |
| **Division 09 - Painting** | | | | |
| Pipe Coatings | 1 | LS | $15,000 | $15,000 |
| | | | **Painting Subtotal** | **$15,000** |
| **Division 11 - Equipment** | | | | |
| 6" Magmeter | 1 | EA | $6,000 | $6,000 |
| 12" Magmeter | 1 | EA | $12,000 | $12,000 |
| 16" Magmeter | 2 | EA | $20,000 | $40,000 |
| Vertical Turbine Pumps | 3 | EA | $126,428 | $379,300 |
| Dewatering Well Pump 500gpm | 1 | EA | $23,000 | $23,000 |
| Dewatering Well Pump 1,500gpm | 1 | EA | $37,950 | $38,000 |
| Dewatering Well Pump 2,500gpm | 1 | EA | $75,500 | $75,500 |
| Surge Tank | 1 | EA | $125,000 | $125,000 |
| | | | **Equipment Subtotal** | **$698,800** |
| **Division 15 - Mechanical** | | | | |
| Butterfly Valves 6" | 1 | EA | $975 | $1,000 |
| Butterfly Valves 12" | 1 | EA | $4,475 | $4,500 |
| Butterfly Valves 16" | 4 | EA | $8,357 | $33,500 |
| Check Valves 6" | 1 | EA | $1,688 | $1,700 |
| Check Valves 12" | 1 | EA | $6,216 | $6,300 |
| Check Valves 16" | 4 | EA | $16,136 | $64,600 |
| Guide Rails/Hatches | 3 | EA | $10,000 | $30,000 |
| | | | **Mechanical Subtotal** | **$141,600** |
| **Division 16 - Electrical** | | | | |
| Electrical | 1 | LS | $250,000 | $350,000 |
| Instrumentation and Controls | 1 | LS | $100,000 | $150,000 |
| | | | **Electrical Subtotal** | **$500,000** |

| | |
|---|---|
| **Subtotal** | **$6,512,800** |
| Contingency (30%) | $1,954,000 |
| Contractor's OH&P (15%) | $1,270,000 |
| Engineering, Permitting and Design (10% or fixed fee) | $651,000 |
| Bidding and Construction Administration (5% or fixed fee) | $519,390 |
| Administrative and Legal (5%) | $326,000 |
| **Project Total** | **$11,233,000** |

| | | Alternative 1 | | Alternative 3 | |
|---|---|---|---|---|---|
| Year | n | Annual Cost | 2015 PW | Annual Cost | 2015 PW |
| 2016 | 0 | $ 327,800 | $ 327,800 | $ 477,600 | $ 477,600 |
| 2017 | 1 | $ 303,300 | $ 295,326 | $ 441,300 | $ 429,698 |
| 2018 | 2 | $ 281,100 | $ 266,514 | $ 408,100 | $ 386,924 |
| 2019 | 3 | $ 254,500 | $ 234,951 | $ 370,000 | $ 341,578 |
| 2020 | 4 | $ 242,500 | $ 217,987 | $ 355,700 | $ 319,744 |
| 2021 | 5 | $ 248,100 | $ 217,157 | $ 363,900 | $ 318,515 |
| 2022 | 6 | $ 253,800 | $ 216,306 | $ 372,200 | $ 317,215 |
| 2023 | 7 | $ 259,600 | $ 215,433 | $ 380,800 | $ 316,012 |
| 2024 | 8 | $ 265,600 | $ 214,617 | $ 389,500 | $ 314,734 |
| 2025 | 9 | $ 271,700 | $ 213,774 | $ 398,500 | $ 313,541 |
| 2026 | 10 | $ 360,900 | $ 276,492 | $ 504,800 | $ 386,736 |
| 2027 | 11 | $ 284,300 | $ 212,081 | $ 417,000 | $ 311,072 |
| 2028 | 12 | $ 290,900 | $ 211,299 | $ 426,600 | $ 309,867 |
| 2029 | 13 | $ 297,500 | $ 210,412 | $ 436,500 | $ 308,723 |
| 2030 | 14 | $ 304,400 | $ 209,632 | $ 446,500 | $ 307,493 |
| 2031 | 15 | $ 311,400 | $ 208,815 | $ 456,800 | $ 306,316 |
| 2032 | 16 | $ 318,600 | $ 208,027 | $ 467,300 | $ 305,119 |
| 2033 | 17 | $ 325,900 | $ 207,199 | $ 478,000 | $ 303,900 |
| 2034 | 18 | $ 333,400 | $ 206,394 | $ 489,000 | $ 302,720 |
| 2035 | 19 | $ 341,000 | $ 205,549 | $ 500,300 | $ 301,573 |
| 2036 | 20 | $ 453,100 | $ 265,941 | $ 633,700 | $ 371,942 |
| | | **20 Year O&M (2016PW) =** | **$ 4,841,700** | | **$ 7,051,000** |

| Annual O&M Costs | Alternative 1 | Alternative 3 |
|---|---|---|
| Year 1 Electrical | $327,843 | $477,639 |
| Year 2 Electrical | $296,517 | $431,334 |
| Year 3 Electrical | $268,634 | $389,969 |
| Year 4 Electrical | $237,728 | $345,582 |
| Year 5-20 Electrical | $221,395 | $324,755 |
| | | |
| **Other O&M Costs** | | |
| 5 year Replacement Cost | $0 | $0 |
| 10 year Replacement Costs | $66,120 | $77,370 |

Given:
Energy = $ 0.10 /kwh
Inflation (I) = 2.3%
Interest (i) = 2.70%

FORMULAS

Annual Cost = (Sum of O&M items) x $(1 + I)^{n}$

Present Worth = (Annual Cost ) x $(1 + i)^{-n}$

NOTES

**Inflation Rate:** value as indicated at http://www.bls.gov/news.release/cpi.nr0.htm . "Over the last 12 months, the index increased 2.3 percent before seasonal adjustment"

**Interest Rate:** According to USDA The “real” federal discount rate from Appendix C of OMB Circular A-94 should be used for determining the present worth of the uniform series of O & M values ; see: www.whitehouse.gov/omb/circulars/a094/a94_appx-c.html. As of May 3, 2010 the 20-yr real discount rate was 2.7%

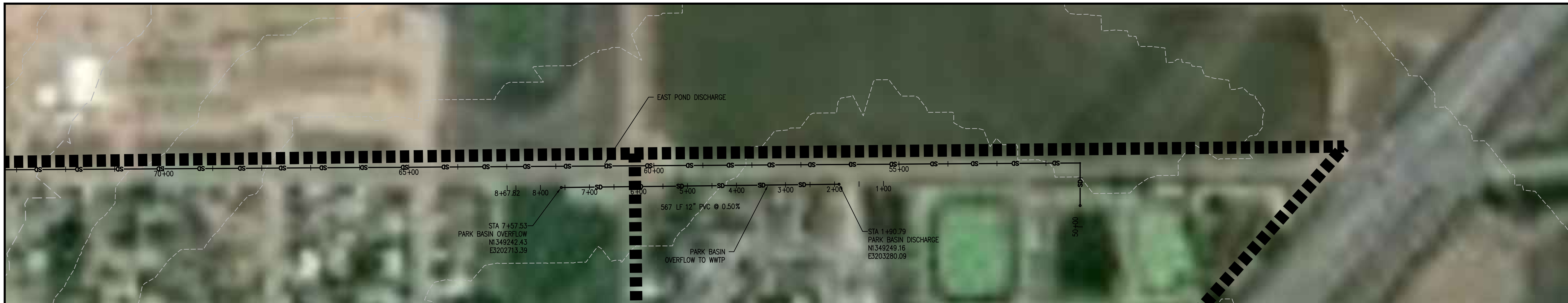
CONSULTING ENGINEERS

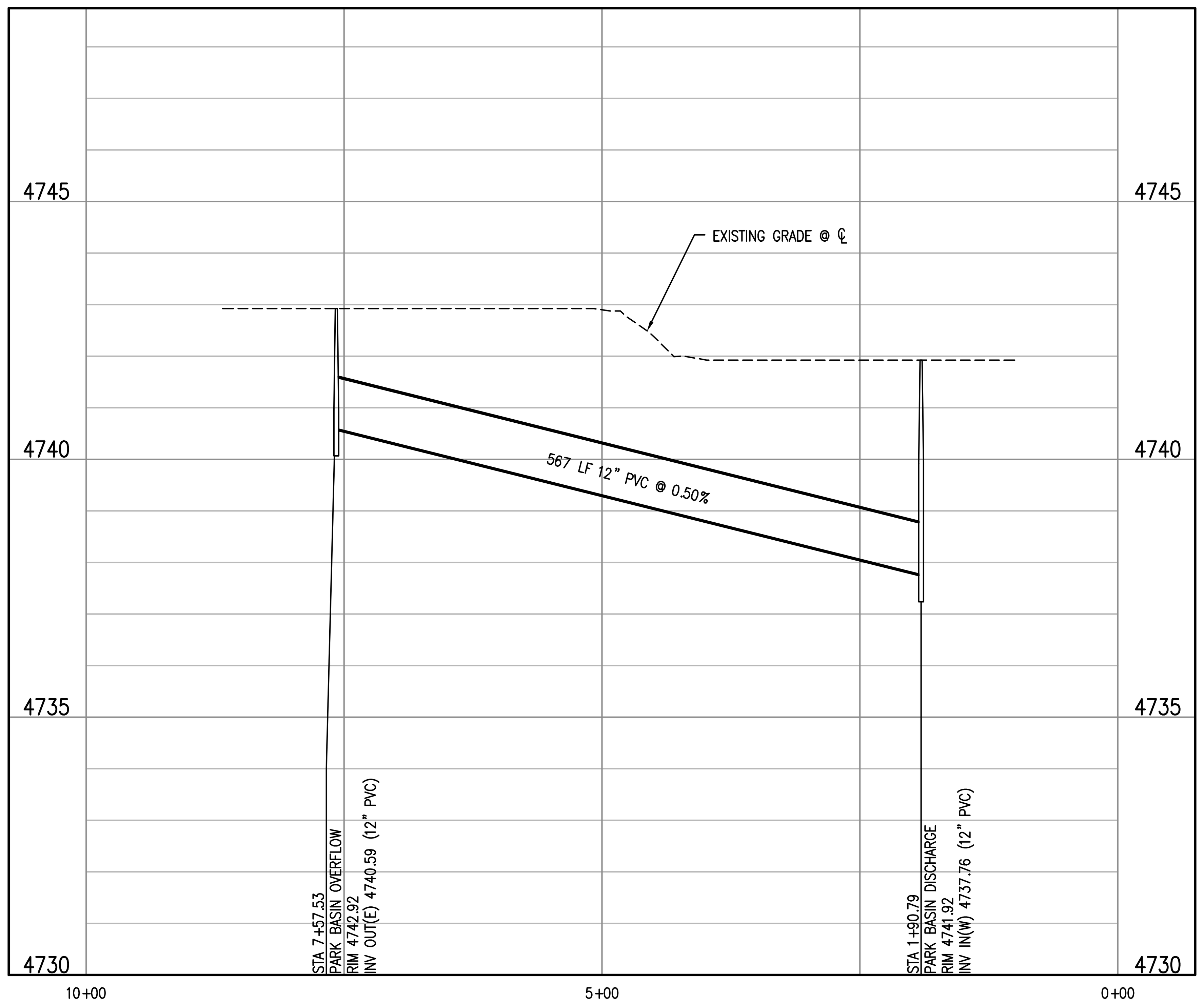
**ELECTRICAL ANNUAL COST**
**FOR**
**DEWATERING IMPROVEMENTS STUDY**
**TOWN OF GILCREST**

| Alternative 1 Electrical Cost | Motor hp | kwH | Year 1 Runtime | Total kwH | Year 2 Runtime | Total kwH | Year 3 Runtime | Total kwH | Year 4 Runtime | Total kwH | Year 5 Runtime | Total kwH |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| M+E | 66 | 49.5 | 100% | 433,620 | 88% | 381,586 | 78% | 338,224 | 78% | 338,224 | 67% | 290,525 |
| GMP | 140 | 105 | 100% | 919,800 | 93% | 855,414 | 86% | 791,028 | 71% | 653,058 | 64% | 588,672 |
| WWTP | 33 | 24.75 | 100% | 216,810 | 88% | 190,793 | 80% | 173,448 | 72% | 156,103 | 72% | 156,103 |
| Dewatering Pumps | 130 X 2 | 195 | 100% | 1,708,200 | 90% | 1,537,380 | 81% | 1,383,642 | 72% | 1,229,904 | 69% | 1,178,658 |
| | | | Totals | 3,278,430 | | 2,965,172 | | 2,686,342 | | 2,377,289 | | 2,213,959 |
| | | | Cost @ 10cents/kwH | $ 327,843.00 | | $ 296,517.24 | | $ 268,634.16 | | $ 237,728.88 | | $ 221,395.86 |

| Alternative 3 Electrical Cost | Motor hp | kwH | Year 1 Runtime | Total kwH | Year 2 Runtime | Total kwH | Year 3 Runtime | Total kwH | Year 4 Runtime | Total kwH | Year 5 Runtime | Total kwH |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| M+E | 66 | 49.5 | 100% | 433,620 | 88% | 381,586 | 78% | 338,224 | 78% | 338,224 | 67% | 290,525 |
| GMP | 140 | 105 | 100% | 919,800 | 93% | 855,414 | 86% | 791,028 | 71% | 653,058 | 64% | 588,672 |
| WWTP | 33 | 24.75 | 100% | 216,810 | 88% | 190,793 | 80% | 173,448 | 72% | 156,103 | 72% | 156,103 |
| Dewatering Pumps | 244 X 2 | 366 | 100% | 3,206,160 | 90% | 2,885,544 | 81% | 2,596,990 | 72% | 2,308,435 | 69% | 2,212,250 |
| | | | Totals | 4,776,390 | | 4,313,336 | | 3,899,689 | | 3,455,820 | | 3,247,551 |
| | | | Cost @ 10cents/kwH | $ 477,639.00 | | $ 431,333.64 | | $ 389,968.92 | | $ 345,582.00 | | $ 324,755.10 |

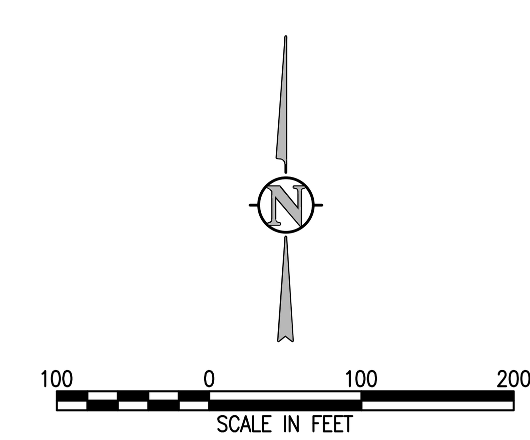
# APPENDIX D – DRAWINGS




PARK BASIN OVERFLOW TO WWTP PROFILE

SCALE: 1"=100' HORIZ
1"=2' VERT




JVA CONSULTING ENGINEERS
JVA, Incorporated 1319 Spruce Street
Boulder, CO 80302 Phone: 303.444.1951
www.jvajva.com E-mail: info@jvajva.com

| NO. | DATE | DES'D | DWN | REVISION DESCRIPTION |
|---|---|---|---|---|

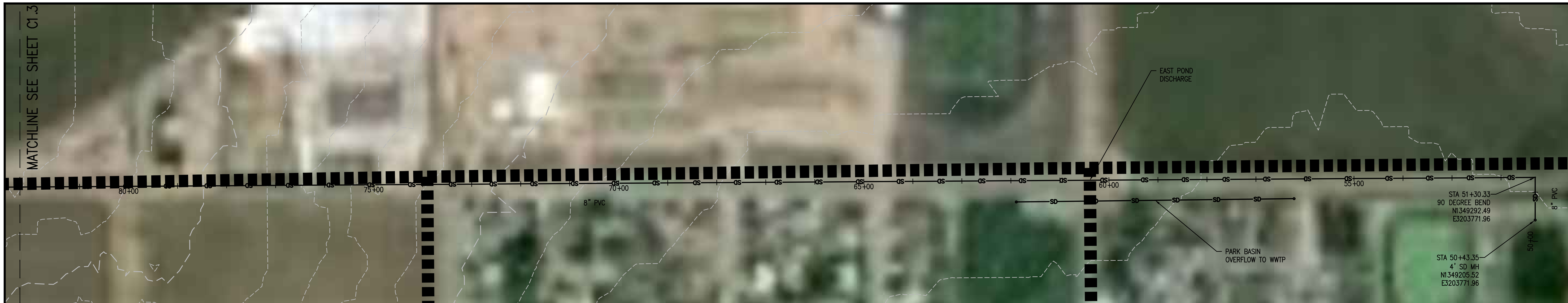
| | |
|---|---|
| DESIGNED BY: | CEC |
| DRAWN BY: | AMF |
| CHECKED BY: | KAT |
| JOB #: | 2278.4c |
| DATE: | 04.13.2016 |



TOWN OF GILCREST
STORMWATER EVALUTAION

PARK BASIN OVERFLOW TO WWTP

SHEET NO.
C1.1





**EAST POND DISCHARGE PROFILE**

SCALE: 1"=100' HORIZ
1"=2' VERT

100 0 100 200
SCALE IN FEET

CONSULTING ENGINEERS
JVA, Incorporated 1319 Spruce Street
Boulder, CO 80302 Phone: 303.444.1951
www.jvajva.com E-mail: info@jvajva.com

| NO. | DATE | DES'D | DWN | REVISION DESCRIPTION |
|---|---|---|---|---|

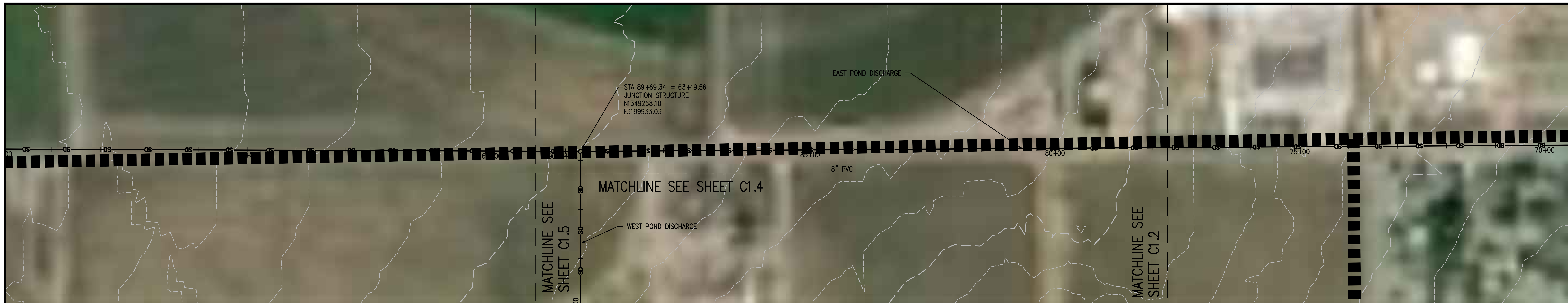
| | |
|---|---|
| DESIGNED BY: | CEC |
| DRAWN BY: | AMF |
| CHECKED BY: | KAT |
| JOB #: | 2278.4c |
| DATE: | 04.13.2016 |

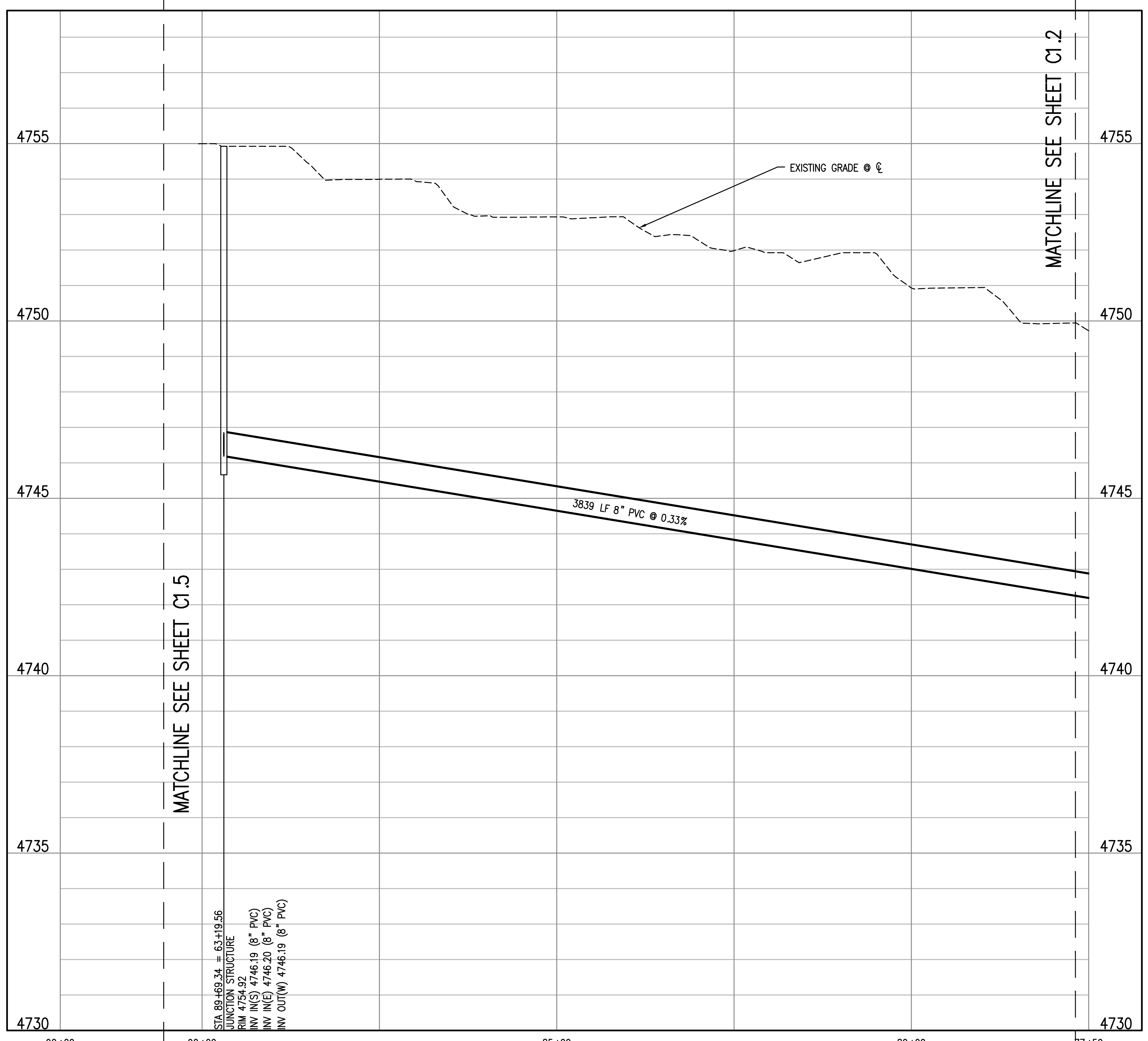


TOWN OF GILCREST
STORMWATER EVALUTAION

EAST POND DISCHARGE PLAN AND PROFILE

SHEET NO.
C1.2




EAST POND DISCHARGE PROFILE

SCALE: 1"=100' HORIZ
1"=2' VERT

100 0 100 200
SCALE IN FEET


CONSULTING ENGINEERS
JVA, Incorporated 1319 Spruce Street
Boulder, CO 80302 Phone: 303.444.1951
www.jvajva.com E-mail: info@jvajva.com


| NO. | DATE | DES'D | DWN | REVISION DESCRIPTION |
|---|---|---|---|---|

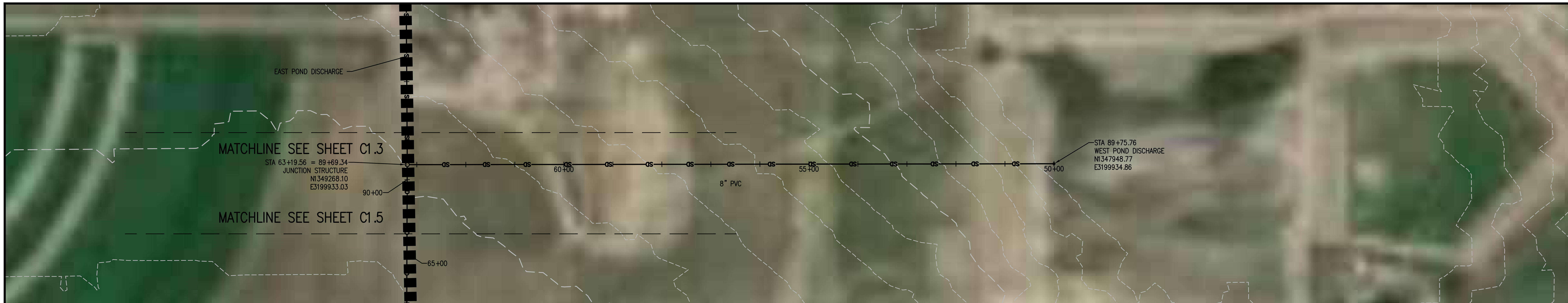
| | |
|---|---|
| DESIGNED BY: | CEC |
| DRAWN BY: | AMF |
| CHECKED BY: | KAT |
| JOB #: | 2278.4c |
| DATE: | 04.13.2016 |

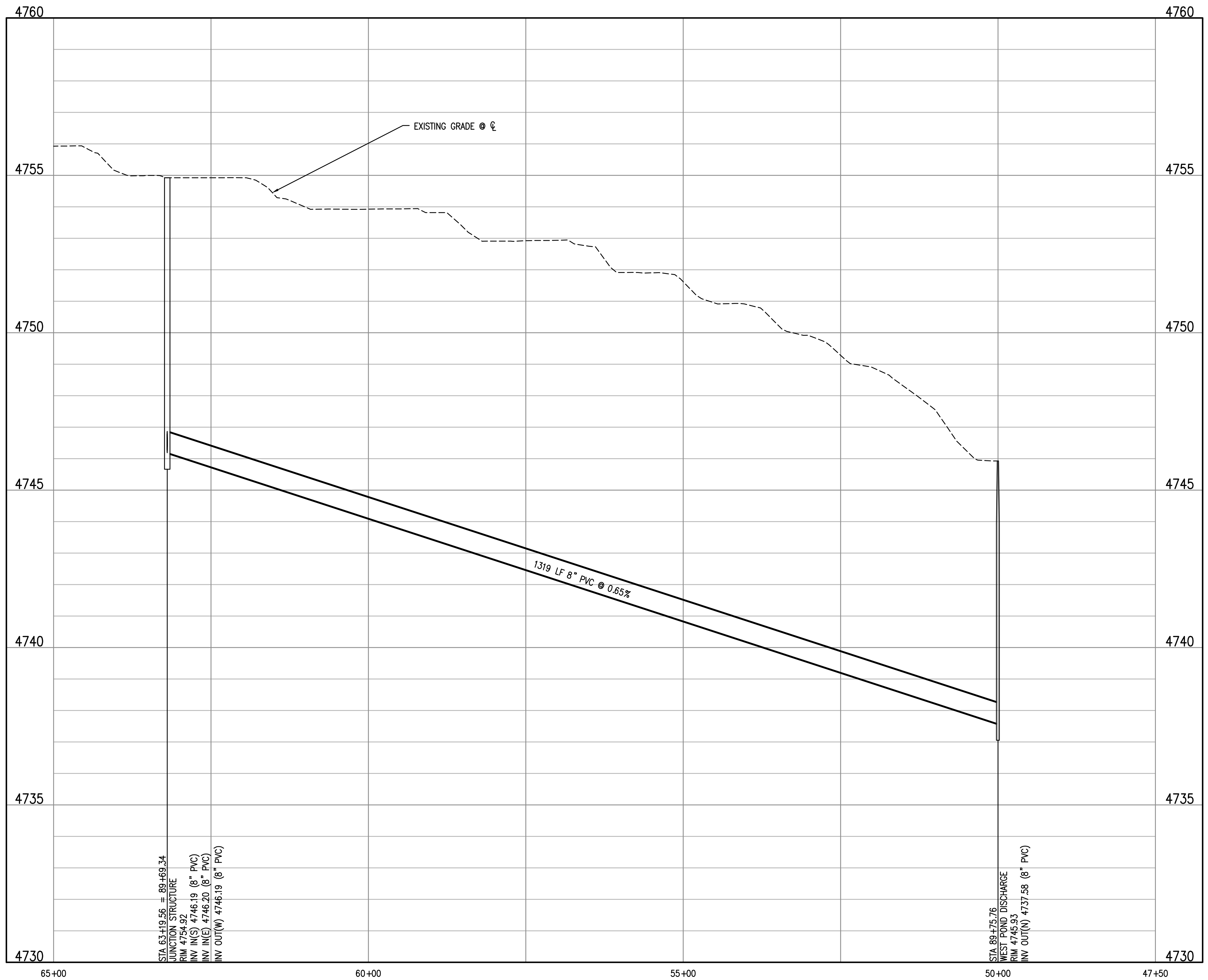


TOWN OF GILCREST
STORMWATER EVALUTAION

EAST POND DISCHARGE PLAN AND PROFILE

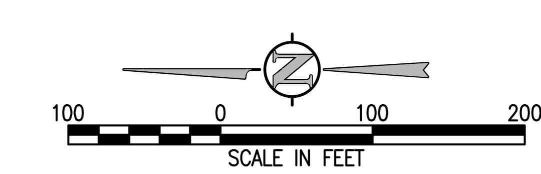
SHEET NO.
C1.3




WEST POND DISCHARGE PROFILE

SCALE: 1"=100' HORIZ
1"=2' VERT




CONSULTING ENGINEERS

JVA, Incorporated
1319 Spruce Street
Boulder, CO 80302
Phone: 303.444.1951
www.jvajva.com
E-mail: info@jvajva.com

| NO. | DATE | DES'D | DWN | REVISION DESCRIPTION |
|---|---|---|---|---|

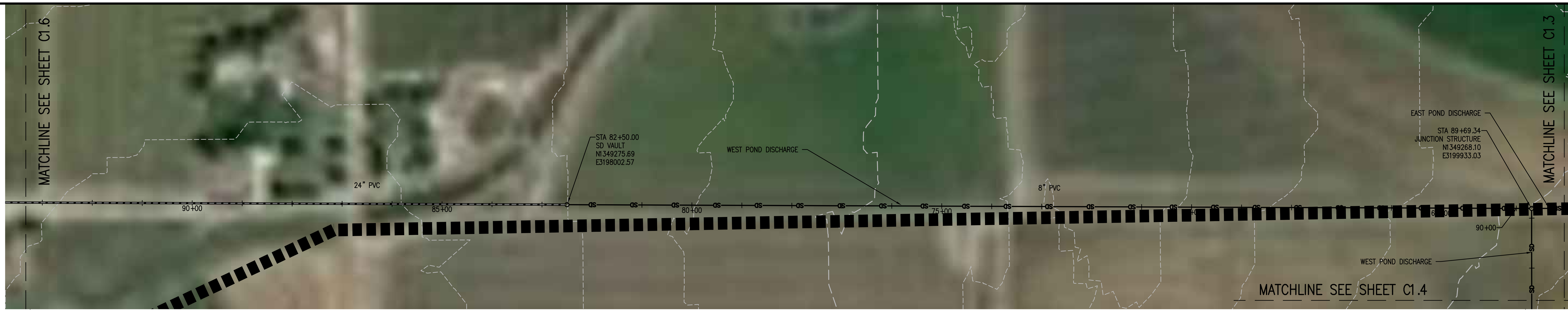
| | |
|---|---|
| DESIGNED BY: | CEC |
| DRAWN BY: | AMF |
| CHECKED BY: | KAT |
| JOB #: | 2278.4c |
| DATE: | 04.13.2016 |

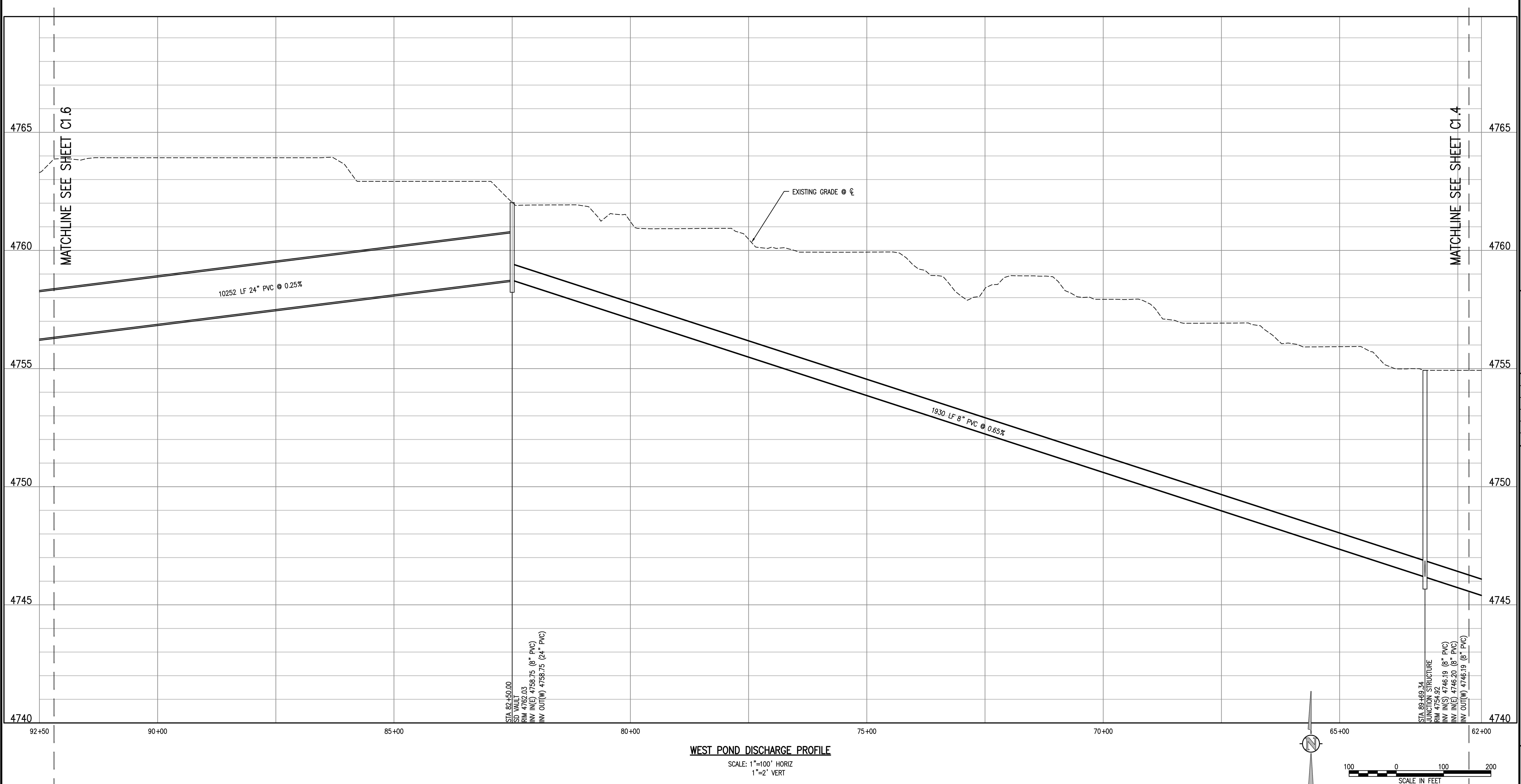


TOWN OF GILCREST
STORMWATER EVALUTAION

WEST POND DISCHARGE PLAN AND PROFILE

SHEET NO.
C1.4




**WEST POND DISCHARGE PROFILE**

SCALE: 1"=100' HORIZ
1"=2' VERT


JVA CONSULTING ENGINEERS

JVA, Incorporated 1319 Spruce Street
Boulder, CO 80302 Phone: 303.444.1951
www.jvajva.com E-mail: info@jvajva.com


| NO. | DATE | DES'D | DWN | REVISION DESCRIPTION |
|---|---|---|---|---|

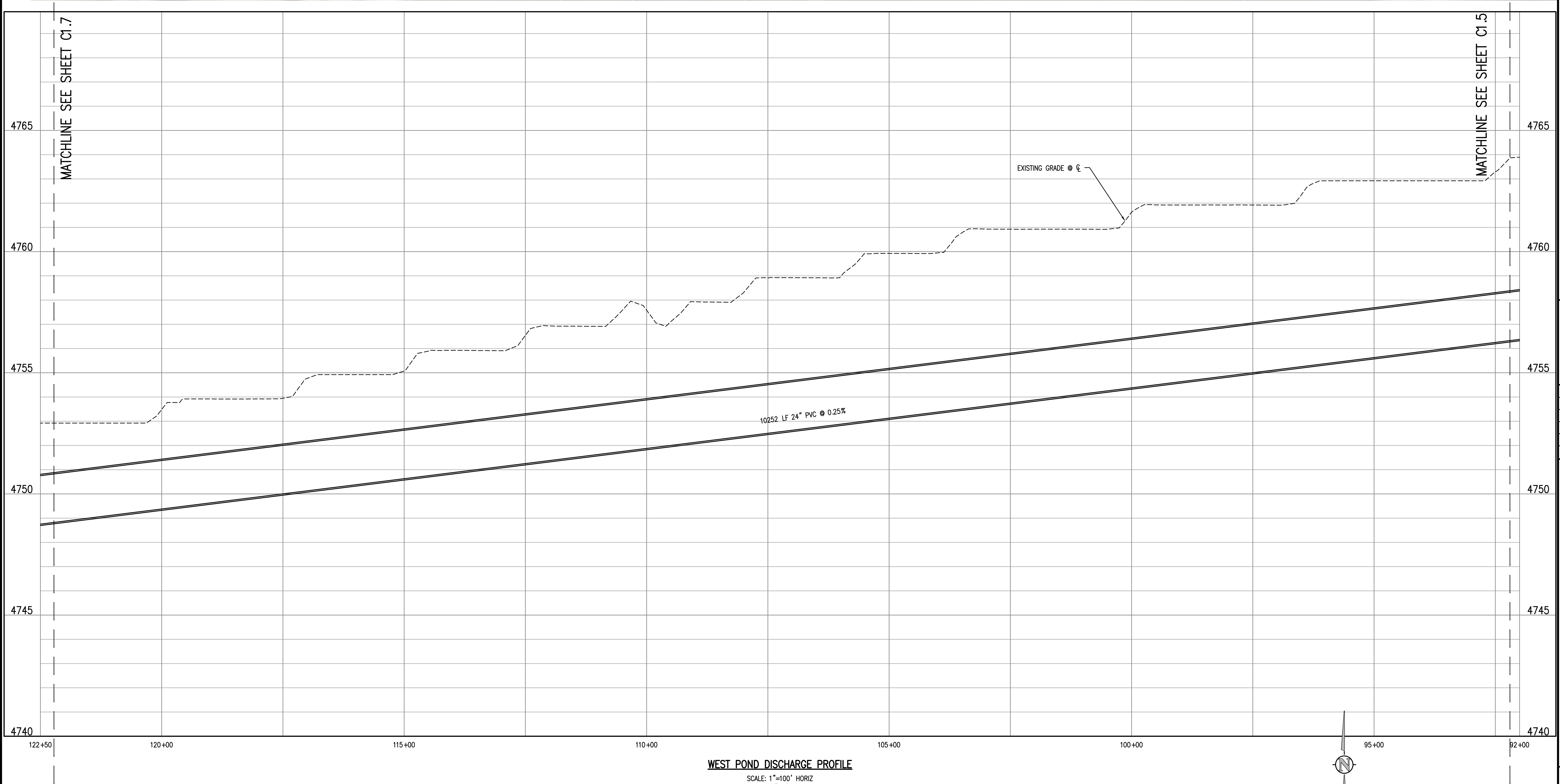
| | |
|---|---|
| DESIGNED BY: | CEC |
| DRAWN BY: | AMF |
| CHECKED BY: | KAT |
| JOB #: | 2278.4c |
| DATE: | 04.13.2016 |



TOWN OF GILCREST
STORMWATER EVALUTAION

WEST POND DISCHARGE PLAN AND PROFILE

SHEET NO.
C1.5


WEST POND DISCHARGE PROFILE

SCALE: 1"=100' HORIZ
1"=2' VERT

JVA CONSULTING ENGINEERS
JVA, Incorporated 1319 Spruce Street
Boulder, CO 80302 Phone: 303.444.1951
www.jvajva.com E-mail: info@jvajva.com

| NO. | DATE | DES'D | DWN | REVISION DESCRIPTION |
|---|---|---|---|---|

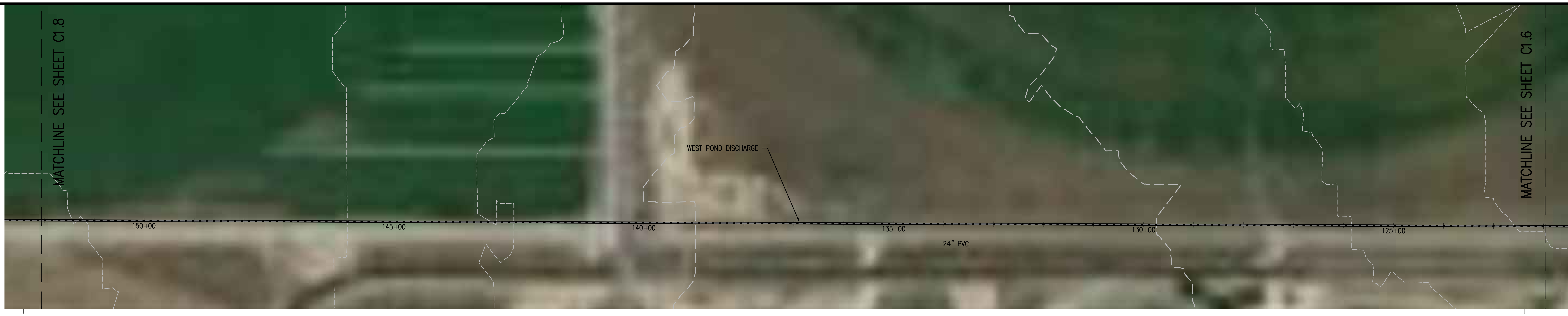
| | |
|---|---|
| DESIGNED BY: | CEC |
| DRAWN BY: | AMF |
| CHECKED BY: | KAT |
| JOB #: | 2278.4c |
| DATE: | 04.13.2016 |

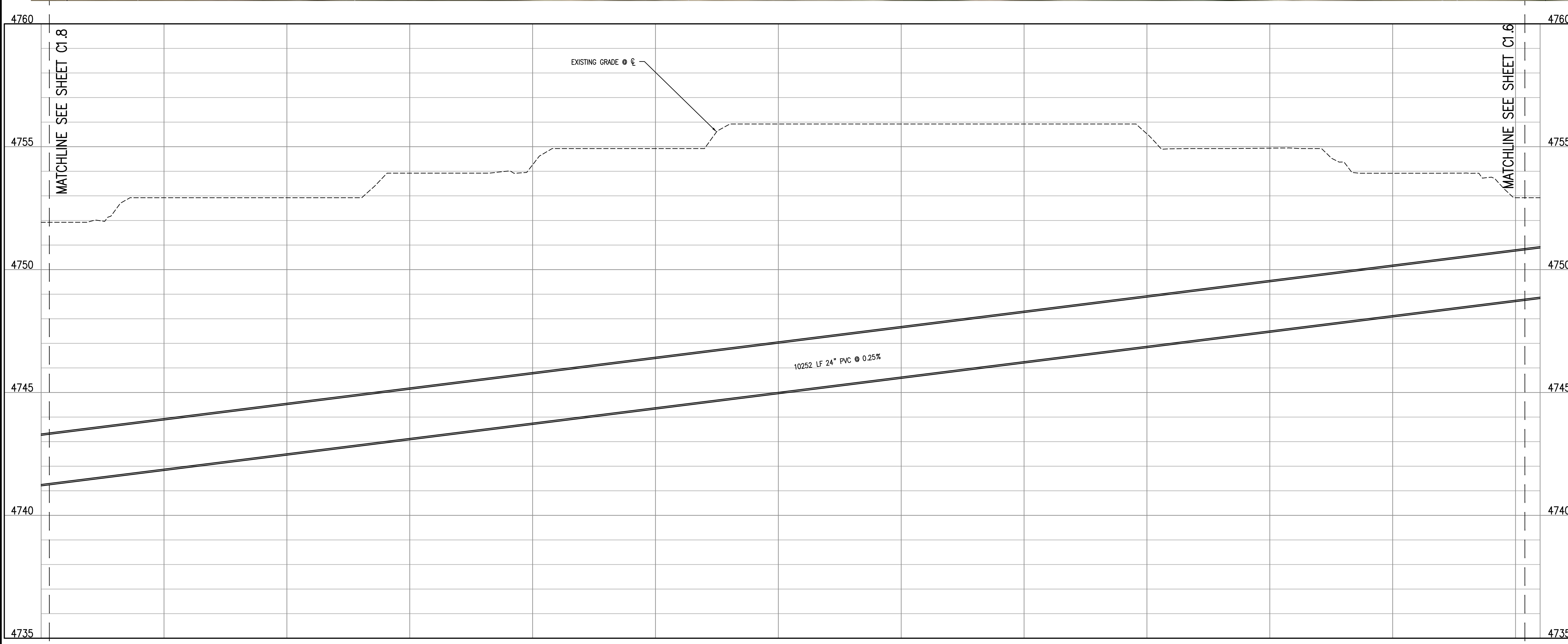


TOWN OF GILCREST
STORMWATER EVALUTAION

WEST POND DISCHARGE PLAN AND PROFILE

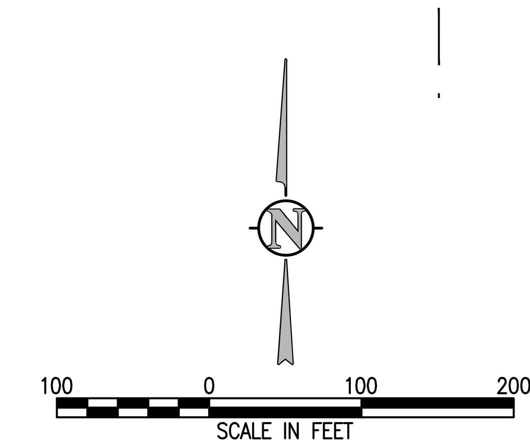
SHEET NO.
C1.6




## WEST POND DISCHARGE PROFILE

SCALE: 1"=100' HORIZ
1"=2' VERT





JVA CONSULTING ENGINEERS
JVA, Incorporated 1319 Spruce Street
Boulder, CO 80302 Phone: 303.444.1951
www.jvajva.com E-mail: info@jvajva.com


| NO. | DATE | DES'D | DWN | REVISION DESCRIPTION |
|---|---|---|---|---|

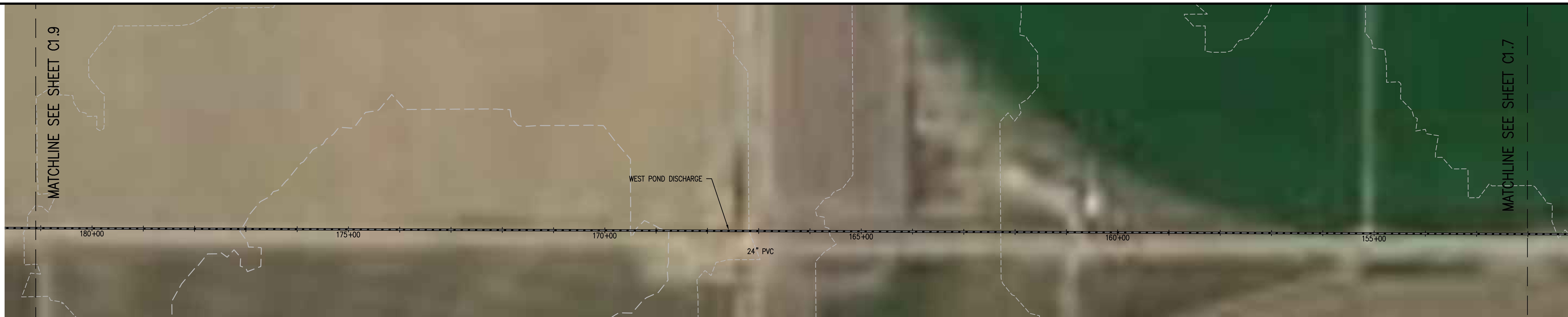
| | |
|---|---|
| DESIGNED BY: | CEC |
| DRAWN BY: | AMF |
| CHECKED BY: | KAT |
| JOB #: | 2278.4c |
| DATE: | 04.13.2016 |

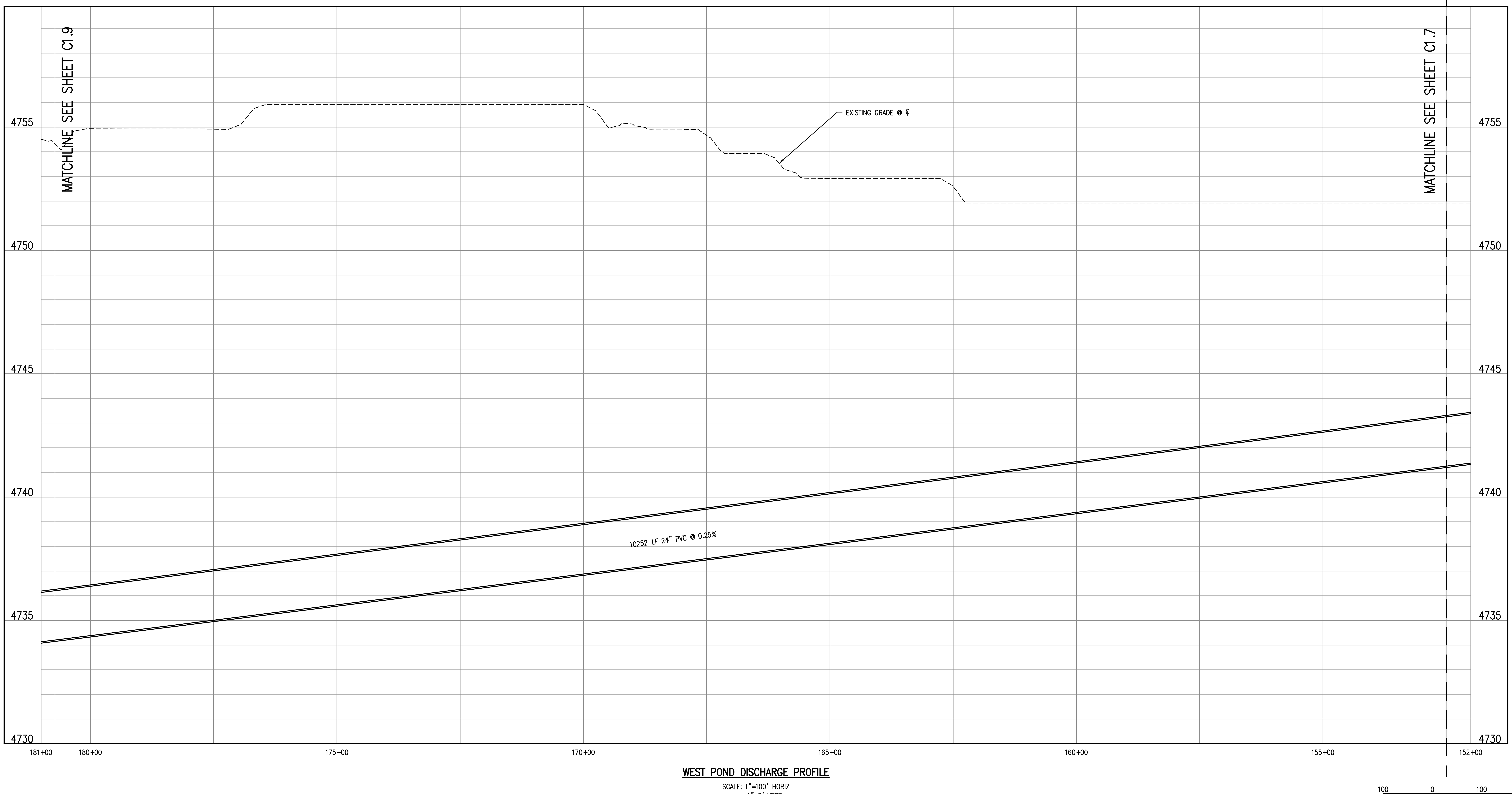


TOWN OF GILCREST
STORMWATER EVALUTAION

WEST POND DISCHARGE PLAN AND PROFILE

SHEET NO.
C1.7




WEST POND DISCHARGE PROFILE

SCALE: 1"=100' HORIZ
1"=2' VERT

100 0 100 200
SCALE IN FEET

JVA CONSULTING ENGINEERS


JVA, Incorporated 1319 Spruce Street
Boulder, CO 80302 Phone: 303.444.1951
www.jvajva.com E-mail: info@jvajva.com


| NO. | DATE | DES'D | DWN | REVISION DESCRIPTION |
|---|---|---|---|---|

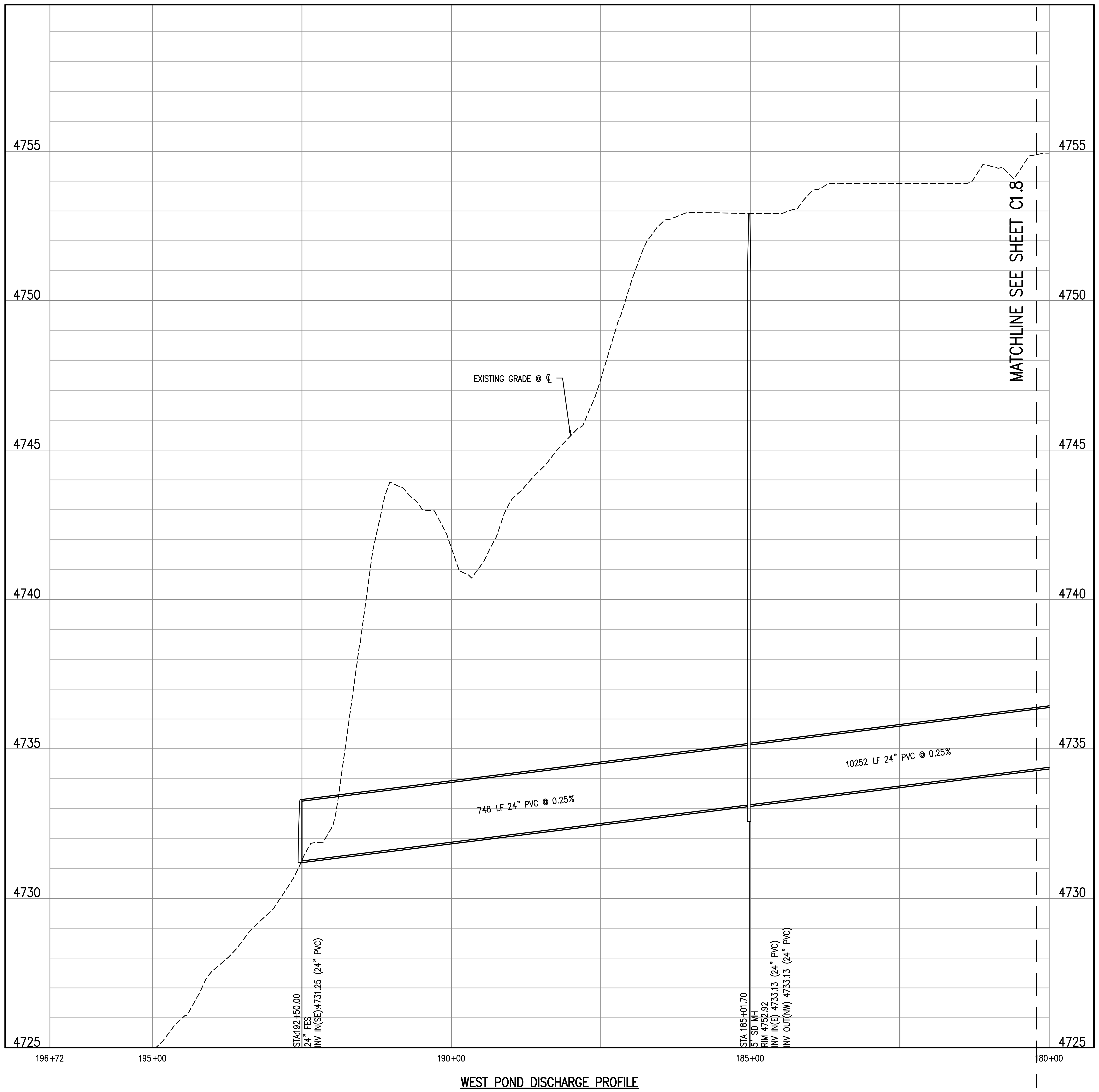
| | |
|---|---|
| DESIGNED BY: | CEC |
| DRAWN BY: | AMF |
| CHECKED BY: | KAT |
| JOB #: | 2278.4c |
| DATE: | 04.13.2016 |



TOWN OF GILCREST
STORMWATER EVALUTAION

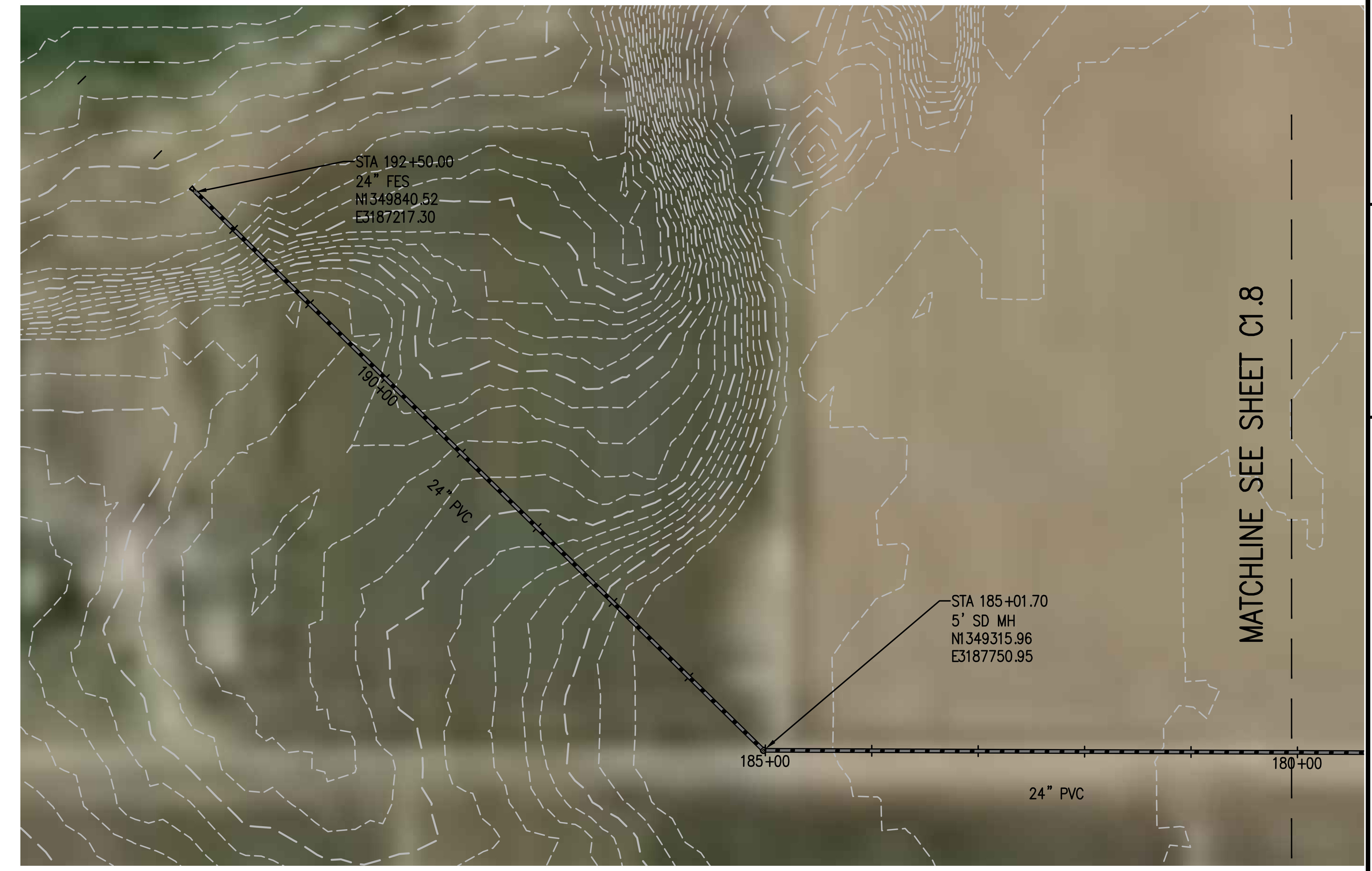
WEST POND DISCHARGE PLAN AND PROFILE

SHEET NO.
C1.8




WEST POND DISCHARGE PROFILE

SCALE: 1"=100' HORIZ
1"=2' VERT




100 0 100 200
SCALE IN FEET

JVA CONSULTING ENGINEERS


JVA, Incorporated 1319 Spruce Street
Boulder, CO 80302 Phone: 303.444.1951
www.jvajva.com E-mail: info@jvajva.com


| NO. | DATE | DES'D | DWN | REVISION DESCRIPTION |
|---|---|---|---|---|

| | |
|---|---|
| DESIGNED BY: | CEC |
| DRAWN BY: | AMF |
| CHECKED BY: | KAT |
| JOB #: | 2278.4c |
| DATE: | 04.13.2016 |



TOWN OF GILCREST
STORMWATER EVALUTAION

WEST POND DISCHARGE PLAN AND PROFILE

SHEET NO.
C1.9