U.S. DEPARTMENT OF HOMELAND SECURITY

US Customs and
Border Protection

Well Report
for
Brown Field Border Patrol Station

March 2018
Revised May 2018

1561 E. Orangethorpe Avenue, Suite 240
Fullerton, California 92831
TEL (714) 526-7500 | FAX (714) 526-7004
www.cwecorp.com

Well Report
for
Brown Field Border Patrol Station
Dulzura, California

Prepared for:

US Army Corps of Engineers

And

US Customs and Border Protection

Prepared by:

1561 E. Orangethorpe Avenue, Suite 240
Fullerton, California, 92831

TEL (714) 526-7500 | FAX (714) 526-7004 | www.cwecorp.com

REGISTERED PROFESSIONAL ENGINEER
WILLIAM F. YOU
No. C 35715
Exp. 09/30/19
CIVIL
STATE OF CALIFORNIA

March 2018
Revised May 2018

Table of Contents

Appendices

1. Introduction

The U. S. Customs and Border Protection (CBP) is proposing to construct the Brown Field Border Patrol Station (BPS), near Dulzura, California. The proposed new facility will be constructed on a 122 acre site approximately 15 miles southeast of San Diego and 0.4 miles east of Dulzura, Assessor Parcel Number 600-160-13-00. The site is accessible via State Route 94 (SR94) and is surrounded by rural properties, vineyards, a fire station, and open space. The property, including mineral rights, is owned by the United States of America under the management of the CBP.

The proposed Border Patrol Station will need utility services to properly function and provide adequate facilities to meet health and safety requirements. The site location is in a rural area that has minimal public utilities available. Electrical power and communications are available from existing power poles along Highway 94. A water supply will need to be developed to provide potable water and water for fire protection. Natural gas will need to be supplied by a service and stored within an onsite tank.
Wastewater treatment and disposal will need to consider onsite methods such as septic tank and leach field or package wastewater treatment systems, both systems will infiltrate the treated wastewater effluent. The design parameters for these facilities are discussed in further detail within the following sections.

A municipal water supply source is not available at this site. Potential methods for obtaining water are; buying water from an offsite source and trucking to the site, or drill an on-site water well. An onsite well is the recommended method of supply and it is anticipated that groundwater extracted from an onsite well will provide sufficient water source for the proposed facility. On-site water supply system includes the water well pump, mechanical piping, and electrical power/controls for the pumps. On-site water distribution system includes three (3) storage tanks, pump house with a potable water treatment system, two booster pumps and two fire pumps, piping and appurtenances for water service and fire protection services to the building complex, helipad, fuel station, vehicle maintenance facility, and vehicle wash facility. Two large capacity storage tanks will be installed to meet the storage required for fire flow demand per fire code requirements. A smaller raw water storage tank and potable water equalization tank will supply potable water service to the building complex at a variable flowrate as needed to meet the facility potable water demand. Piping and instrumentation controls will allow use of a single equalization tank with raw water supply directly pumped from the water well to the potable water treatment system during tank maintenance.

Water usage data obtained from the Campo Border Patrol Station (with a total occupancy of 400 capita) was used to estimate the water demand for this facility. The annual total water demand is estimated to be 1.54 million gallons per year for its operation with 400 agents assigned to that facility. Water usage for this facility was estimated assuming water used for spraying hills and fire control during the summer months at the Campo facility is not needed for this facility. Estimated water demand prorated for a possible future expansion to 600 agents is 2.37 million gallons per year for this ultimate operation. The average daily water demand ranges from 3,585 gallon per day in January to 8,245 gallon per day in August for ultimate conditions. The water treatment plant capacity will be adjusted with an average plant output of 9,000 gallon per day as design demands.

2. Well Installation and Evaluation

The Brown Field BPS site is located in an area where existing topography slopes up generally 10% to 15% to the east and southeast until it reaches the base of the hills where slopes increase to 35% to 70%. An initial site layout for the facility was prepared by HNTB-Halff JV and potential locations for the well were reviewed. Ultimately an area close to the well's ultimate location was selected as the best location. Stehly Brothers Drilling Inc., a local licensed well driller on San Diego County's list of approved drillers, was contracted to drill the Brown Field BPS well at the proposed site. The water well construction permit (Permit No. LWELL 001874 dated 11/27/17) was obtained from San Diego County Department of Environmental Health Division (DEH), to drill the water well at the proposed site. A copy of the permit from DEH is included in **Appendix A**. Prior to drilling activities, a site review was conducted with DEH staff to confirm site characteristics and location of the proposed water well with respect to the adjacent property and water courses. Minor adjustments were made to the wells location and approved by DEH. The well drilling began on January 02, 2018 and was completed on January 12, 2018. The well is located at the following coordinates; Latitude 32.64700ºN, Longitude 116.77502ºW.

2.1 Geology and Hydrogeology

The proposed BPS site and water well is located near the community of Dulzura, as shown in **Figure 2-1**, located approximately 22 miles southeast of the City of San Diego, California. This area of San Diego County lies within the Southern Peninsular Ranges Geomorphic Province. The Peninsular Ranges were subject to regional uplift and erosion throughout the Tertiary Period. Continued erosion and headcutting of drainage courses through the Quaternary Period have resulted in the present topography. In general, trends of several of the major drainage courses that have developed appear to be controlled by ancient fractures or major joint systems within the crystalline bedrock.

This area is characterized by mountainous ridges and hills interspersed by intermountain valleys and basins. In the immediate area most of the hills and mountains meander in north- south mountainous ridgelines with elevations that range from about 2,000 to 3,000 foot elevation. The project site is near a north south ridgeline which divides the area into two hydrologic regions, the Otay River and Tijuana River hydrologic regions. The project site is within the Otay River hydrologic region with runoff from the site flowing southwesterly to Dulzura Creek which meanders northwesterly to its confluence with Jamul Creek and then southwesterly to Lower Otay Lake reservoir.

The mountainous terrain in this area has a relatively thin layer of soil over bedrock. In the vicinity of the project soil is 2 to 20 feet with boulders outcroppings frequently exposed on the ground surface. The underlying bedrock consists of weathered granite and fractured granite that is characterized as having fractured rock aquifers underlying approximately three quarters of the area. Fractured rock aquifers are present in this area of San Diego County and precipitation is typically higher than in the lower westerly coastal areas. With the higher rainfall rates and smaller storage capacities, fractured rock recharge rates can be greater in this area. Fractured rock aquifers also have a lower storage capacity (0.001% to 1% of the volume), hence recharge to fractured rock aquifers can cause relatively fast rises to the water table, and similarly fast declines to the water table from groundwater pumping in years without significant recharge. In some areas of the County with fractured rock aquifers, the static groundwater levels have risen or declined in excess of 100 feet in particularly rainy or dry seasons.

Figure 2-1 Aerial Photo of Well Location

The proposed BPS site is located on the west slopes of some local foothills in this area and slopes to Dulzura Creek , which drains to the Lower Otay Reservoir. There are three water wells immediately adjacent to the Brown Field BPS site. The vineyard on the northwest side of the property has a water well located approximately 300 feet west of the property. The Calfire station located south of the property at the southwest corner has a water well that services their facility located approximately 50 feet south of the property. The third adjacent well is a residential well located approximately 100 feet south of the site at the south east corner.

2.2 Well Characteristics and Installation

Stehly Brothers Drilling, based out of Valley Center, California, conducted the drilling and installation of the well. The well was constructed in accordance with the California Well Standards (Bulletin 74-81 and 74-90). Stehly Brothers utilized a rotary drill rig to advance an initial 6-inch diameter pilot borehole to a maximum depth of 1010 feet below ground surface (bgs), as shown in **Figure 2-2**. The mud rotatory drill rig utilizes a mud and water circulation technique and allows soil cuttings to be buoyed up to the surface where it is collected with a strainer. Collected soil cuttings were recorded during the progression of the pilot borehole and listed in the Well Completion Report found in **Appendix A**. Well drilling soil samples were collected and logged at depths in increment sections where changes of material occurred in the pilot borehole.

Figure 2-2 Mud Rotary Drill Rig

The well boring log show the types of soil and bed rock encountered while drilling the well. The top 4 feet is clay soil with decomposed granite mixed in with the soil. From 4 feet to 80 feet the material is generally classified as decomposed granite. At 80 feet granite bedrock was encountered and the well drilling continued through the weathered granite and into the harder fractured granite. When the drilled pilot hole reached 600 feet bgs water was encountered within a granite fr**a**cture. This water was under hydraulic pressure and rose within the bore hole to 223 feet bgs. Drilling continued to 800 feet bgs and preliminary estimates were made for the well production by Stehley Brothers at 40 gpm based upon using compressed air to clear the bore hole. Based upon discussions with Stehly Brothers and a potential to encounter additional water bearing granite fractures, Stehly Brothers was directed by CWE to continue the well to 1,000 feet bgs.

Figure 2-3 Installation of 8 $^5/_8$-inch Casing

Upon completion of the pilot hole, the hole was reamed out and the 8$^5/_8$-inch diameter steel casing was installed (**Figure 2-3).** to a total depth of 101 feet bgs. Well screen intervals were determined based on the borehole boring log. The well screens were set at four 20- foot intervals, 257 to 277, 377 to 397, 477 to 497, 677 to 679 feet bgs and one 50-foot interval 557 to 617 feet bgs, to allow for 130 feet of 6.9-inch PVC (SDR17) well screen. The gravel pack was installed using 1/8-inch to 3/8-inch gravel from 70 to 640 feet bgs. The 1/8-inch to 3/8-inch gravel materials were washed before being placed in the well, as required by the California Water Well Standards (Bulletin 74-81, and 74-90) Chapter II Section 11 disinfection and other sanitary requirements. A concrete sanitary seal was set from 70 feet bgs to surface grade and inspected by a San Diego County Senior Hazardous Materials Specialist in accordance to California Water Well Standards Chapter II (Bulletin 74-81 and 74-90) Section 13 describing strata sealing. The final report and well permit are included in **Appendix A.**

DWR Standards require that if the pump is not installed immediately or if there is a prolonged interruption in construction of the well, a watertight cover shall be installed at the top of the casing. Upon completion of test pumping, a watertight cap was installed onto the well casing and left in a condition to be readily converted into a production well in the future. The finished product is shown in the following **Figure 2-4**. The design plans for the well surface construction features will address the final components of the well and related piping and electrical controls. The finished well will be within a fenced area to maintain a secured facility.

Figure 2-4 Brown Field BPS Well Sealed and Capped

2.3 Well Development

California Water Well Standards (Bulletin 74-81 and 74-90) Chapter II Section 14 describes the necessary procedures for well development where air lifting, overpumping, and backwashing methods were used. Air development, also called air lifting, was used following well construction. High pressure air was injected near the base of the well, forcing the water out of the well. Air lifting removed much of the residual drilling mud that was left in the well from the drilling activities. Following air lifting, the well was swabbed where the swab is slowly lowered to the desired depth then drawn upward, creating a pressure differential below and above the swab which draws the fine particles, from near the well area, inward for removal. This method is effective in breaking up and removing the residual drilling mud left on the borehole sidewalls.

Following the air lifting and swabbing techniques, the well was overpumped, a method where pumping is conducted above the designed discharge rate for a prolonged period of time. Pumping rate during this process reached 60 gallons per minute (gpm) for a two hour period. At the conclusion of well development, the majority of residual drill mud as nearly non-existent and the particles within the groundwater consisted mostly of sand and fine particles, as seen in **Figure 2-5**.

Figure 2-5 Water Discharge during Well Development

Once overpumping was completed, the well was backwashed as a final measure of development. Backwashing, also referred to as surging, is the process of alternately starting and stopping a pump**.** Backwashing reverses water flow and helps in the dilution, agitation and removal of sediment, fine particles, and drilling fluids. This reversal causes the collapsing of bridges in the particles near the well area, removing fine particles, and creating a cleaner discharge. During this process, pumping rate reached 60 gpm followed by shutting off the pump, then pumping rate was brought back to 60 gpm and shut off again. At the conclusion of the three well development methods, approximately 7,500 gallons of drill mud/groundwater were pumped from the well and into the storage/infiltration basin. Prior to discharging onto the site, trenches were dug to allow for an increase in infiltration, and berms were constructed out of soil cuttings to prevent the development water from reaching the local drainage courses.

2.4 Pumping Tests

An evaluation of the well and aquifer capacity was conducted by performing a 72 hour pump test utilizing a temporary pump with a maximum pumping rate of 90 gpm. Our initial analysis of the Brown Field BPS daily demand was 5,000 gpd based upon 400 agents being served, and our initial expectation for the pumping tests were to determine if the well/aquifer can maintain a production rate of 40 gpm. This flow rate would be approximately twice the expected peak pumping rate that may be expected for 4 hours each day. The well pump test was conducted which included a step draw down test, a constant rate test, and an aquifer recovery test. Drawdown measurements from the well were utilized in lieu of observation well drawdown since the well is located in a fractured rock aquifer and adjacent wells may not be connected to the same rock fractures. Water pumped from the well was stored within a constructed settling basin where it was infiltrated into the ground.

2.4.1 72-Hour Pump Test

The 72-hour pump test was started at 2:38 PM on January 16th 2018 with an initial static water surface of 223 feet bgs. During the first hour, the well drawdown, defined as the difference from the initial static water level to the water level measured at the cone of depression, was 53 feet (276 feet bgs). This was a continual drawdown with the pumping rate at 67 gpm. The pumping rate was decreased to 65 gpm at 60 minutes to attempt to stabilize the drawdown. The total drawdown during the second hour of the pump test slowed but continued to fall from 53 feet (276 feet bgs) to 54 feet (277 feet bgs). The pumping rate was decreased to 64 gpm and the pump ran overnight. The well was observed the next day at 11:00 AM when the total drawdown had dropped to 62 feet (285 feet bgs). The drawdown continued to drop and was observed at 64.5 feet (287.5 feet bgs) at 6:00 PM. The pump continued to run overnight and the drawdown was observed to be 72 feet (295 feet bgs) at 12:00 PM. The pumping rate was adjusted to 60 gpm and at 12:30 PM the drawdown had risen 2 feet to 70 feet (293 feet bgs). Within 15 minutes the drawdown lowered to 71 feet (294 feet bgs) the appeared to stabilize for two hours. When observed at 3:00 PM the drawdown had dropped only 0.25 feet to 71.25 feet (294.25 feet bgs). After the overnight continuous pumping the well was observed at 11:00 AM and the drawdown was measured to be 71.75 feet (294.75 bgs). The drawdown was measured again at 3:00 PM and found to be stable at 71.75 feet (294.75 feet bgs). The pump was then turned off and the Recovery Test begun.

The well drawdown stabilized at a pumping rate of 60 gpm and an distance of 71.75 feet (294.75 feet bgs). A rough estimate can be made from this flow rate and available head. Using an equivalent pipe size for 60 gpm an 71.75 feet of head an approximation can be made that at 15 gpm the drawdown would be less than 10 feet. The initial laboratory water samples tested positive for Coliform Bacteria which required a retest of samples for Coliform Bacteria. Since new samples were required and the well needed to be chlorinated and flushed, additional pump tests were ordered for 15 gpm and 30 gpm flow rates. This pump test was started on February 7, 2018 at 2:25 PM with a static water surface of 232 feet bgs. The 15 gpm pumping rate immediately stabilized at a drawdown of 6 feet (238 feet bgs) and ran for 2 hours without any fluctuation in the water surface. At 4:45 PM the pumping rate was changed to 30 gpm and the drawdown began moving down and at the last observation of the day, 6:30 PM, the drawdown was observed to be 7.2 feet (239.2 feet bgs). The pump continued to run overnight, and the drawdown was observed to be 10 feet (242 feet bgs) at 12:30 PM. The drawdown was checked again at 2:30 PM and 3:00 PM and observed to be stable at 10 feet (242 feet BGS).

2.4.2 Recovery Test

At the conclusion of the 72-hour pump test, water level measurement was recorded as part of the well recovery test. The pump was turned off and the rate of recovery was measured to determine how well the aquifer will re-establish equilibrium. The well recovered to within 95% percent of the original static water level of 223 feet bgs within 30 to 40 minutes, this information is shown in **Figure 2-6** below.

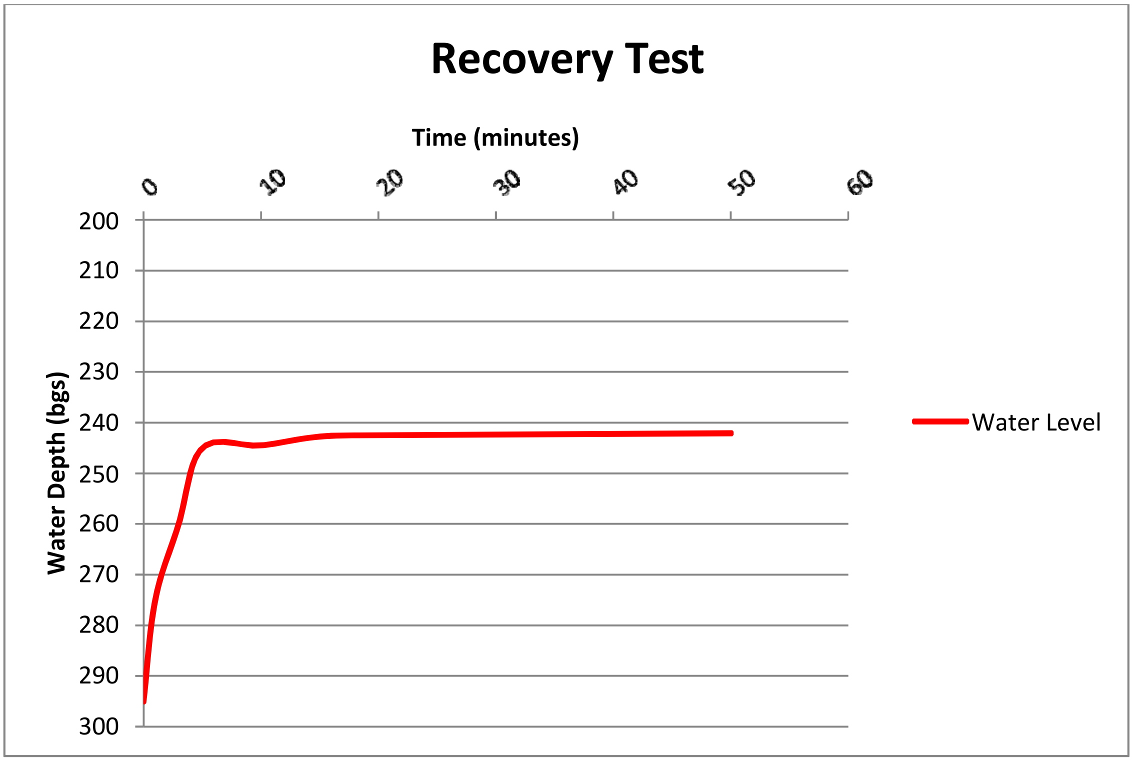


Figure 2-6 Results of Recovery Test

Based on the pump tests and recovery test results, the well is able to sustain a consistent pumping rate of 60 gpm. CWE has prepared preliminary design estimates for the well to pump between 15 gpm and 20 gpm for 4 – 6 hours per day to refill the potable water storage tank. The well appears to have sufficient capacity to pump at these design rates with a drawdown estimated at less than 10 feet.
Additional pump tests were performed on February 7, 2018 utilizing the design pumping rate of 15 gpm. The initial static water surface was 232 feet bgs and when the pump was started the water surface dropped and stabilized at 238 feet bgs for 2 hours. The pumping rate was increased to 30 gpm and the water surface dropped to 239 feet bgs. The pump was run until 3:00 PM the next day. The drawdown was measured at 242 feet bgs and was stable during the 2 ½ hours that was recorded by the operator.

2.5 Aquifer Evaluation

There are three primary elements needed to determine the impacts to the aquifer from well construction and use. To perform this evaluation, we must estimate the volume of the aquifer, the rate of water entering the aquifer (upstream and surface recharge), and the rate of water exiting the aquifer (downstream draw and well draw). For this evaluation we have simplified the numbers to assume the rate of upstream recharge is equal to the rate of downstream draw. The demand for potable water at the site was determined through an estimation of both the population of the border patrol station and the per capita use of water. The Brown Field BPS will be constructed for a population of 400 agents, but with room for expansion to 600 agents. The per capita use of water was determined through the water demands from another border patrol station in the San Ysidro Mountains, Campo BPS. Campo BPS has 400 agents, data from this facility revealed that the average monthly demand was 4,290 gallons per day (gpd). Assuming the same per capita rate, Brown Field BPS will have a demand of approximately

6,500 gpd if the facility expands to accommodate 600 agents. This translates to an annual demand of 2.37 million gallons per year, or 7.28 acre-feet per year (afy).

On-site drilling of a well helped to determine the characteristics of the aquifer beneath the future Brown Field BPS. The geotechnical borings and well drilling logs showed that the aquifer beneath the future Brown Field BPS is confined and composed of fractured bedrock. Well drilling logs show that bedrock was encountered at 80 feet below ground surface (bgs) and water was encountered at 600 feet bgs, but the water in the well rebounded to a depth of about 232 feet bgs. The effective porosity of the fractured bedrock was assumed to be 0.005. The well was drilled to a depth of 1,010 feet bgs with the fractured rock aquifer extending to full depth. The well pump is assumed to be installed at a depth of 1,000 feet bgs.

2.5.1 Five-Year Projection of Drawdown

Calculation of the projected drawdown requires zones to be delineated around the well using the Calculated Fixed Radius method. The default shape of these zones is circular, and the radius of the zones is based on the Time of Travel (TOT) of water from a point in the aquifer to the well. The three zones are defined as:

Zone A	(2 year TOT)
Zone B5	(5 year TOT)
Zone B10	(10 year TOT)

The delineation of the ground water protection zones was determined from the State Water Source forms, attached as **Appendix B**. Calculations performed with data from the pumping rate revealed that the Zone B5 circular area extends to the default minimum 1,500 foot radius from the well if the pump is operated at the 6,500 gpd estimated demand. Using the 1,500-foot radius Zone B5 area as the available capacity, the surface area is calculated to be 162 acres. Using this area and the depth of the available aquifer storage provides an estimation of the available 5-year water bearing rock volume of 162 acre-feet/foot.

The 5-Year predicted drawdown of the well was estimated by calculating the 5-year TOT volume of the aquifer capacity that would be required to provide the annual water demand of 7.28 afy. For fractured bedrock with a porosity of 0.005, the volume of aquifer required to supply this amount of water is 1,456 acre-feet, which, when divided by the 162-acre area of the cone of depression, translates to a height of 8.97 feet. Thus the annual drawdown of the potentiometric water surface with these assumptions for the water demand and the aquifer characteristics, and assuming no additional input of water, can be assumed to be 8.97 feet.

If there was an extended drought at the site and no additional groundwater recharge was available for five years, the potentiometric surface would only drop 44.86 feet, from 232 feet below the surface to 277 feet below the surface. This effect is illustrated in **Figure 2-7.** The total depth of the available aquifer is 768 feet and the 45 foot 5-year drawdown is approximately 5.8% of the aquifer.

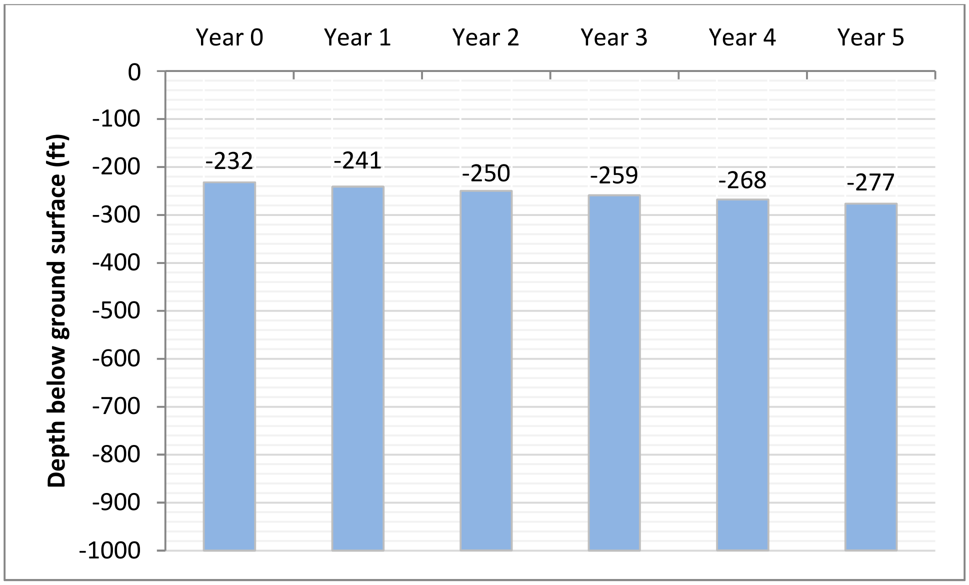


Figure 2-7 Five-Year Projection of Drawdown Under Drought Conditions

The Brown Field BPS will dispose of wastewater utilizing a treatment system that will infiltrate wastewater effluent into the soil. Under the assumption that 80% of the potable water becomes wastewater and this wastewater will infiltrate through the soil to the groundwater aquifer even during drought years, the potentiometric surface of the aquifer after five years of drought would drop by a rate of only 1.79 feet per year, or around 9 feet after five years. The total depth of the available aquifer is 768 feet and the 9 foot 5-year drawdown is approximately 1.2% of the aquifer.

Given these assumptions, there would be little long-term impact on the aquifer if the water well at Brown Field BPS were to be constructed and operated as planned.

2.5.2 Well Interference

The 5-Year projection of drawdown calculations estimate the total drawdown to be 45 feet (5.8% of the aquifer depth) if no recharge is estimated for the Brown Field BPS wastewater leach field infiltration and 9 feet (1.2% of the aquifer depth) if the leach field infiltration is included. The County of San Diego Guidelines for Determining Significance and Report Format and Content Requirements document addresses Well Interference in fractured rock within Section 4.3.1. Within this section the documents states that “as an initial screening tool, well interference will be considered significant if it results in a decrease of 20 feet or more in the offsite wells”. The 5-year projection of drawdown estimates that the drawdown at the Brown Field BPS will be 45 feet for the worst case scenario, drought conditions and no recharge from wastewater effluent. Under these conditions the drawdown at the Brown Field BPS well is 5.8% of the aquifer depth and it can be estimated that the drawdown would be less at the nearest well located approximately 500 feet southwest of this well. Under these conditions it can be assumed that the adjacent well static water surface may not be decreased more than 20 feet. When considering the groundwater recharge that will occur due to the Brown Field BPS wastewater treatment system effluent infiltration, the project will have no impact on the adjacent wells' static water surface.

2.5.3 50% Reduction of Groundwater in Storage (Water Balance Analysis)

The extent of the groundwater aquifer that will be influenced by the new well can be assumed to take the shape of a cylinder with a height equal to the thickness of the aquifer and a radius defined by the Radius of Influence (ROI). The inflows and outflows to this cylinder are depicted in **Figure 2-8**

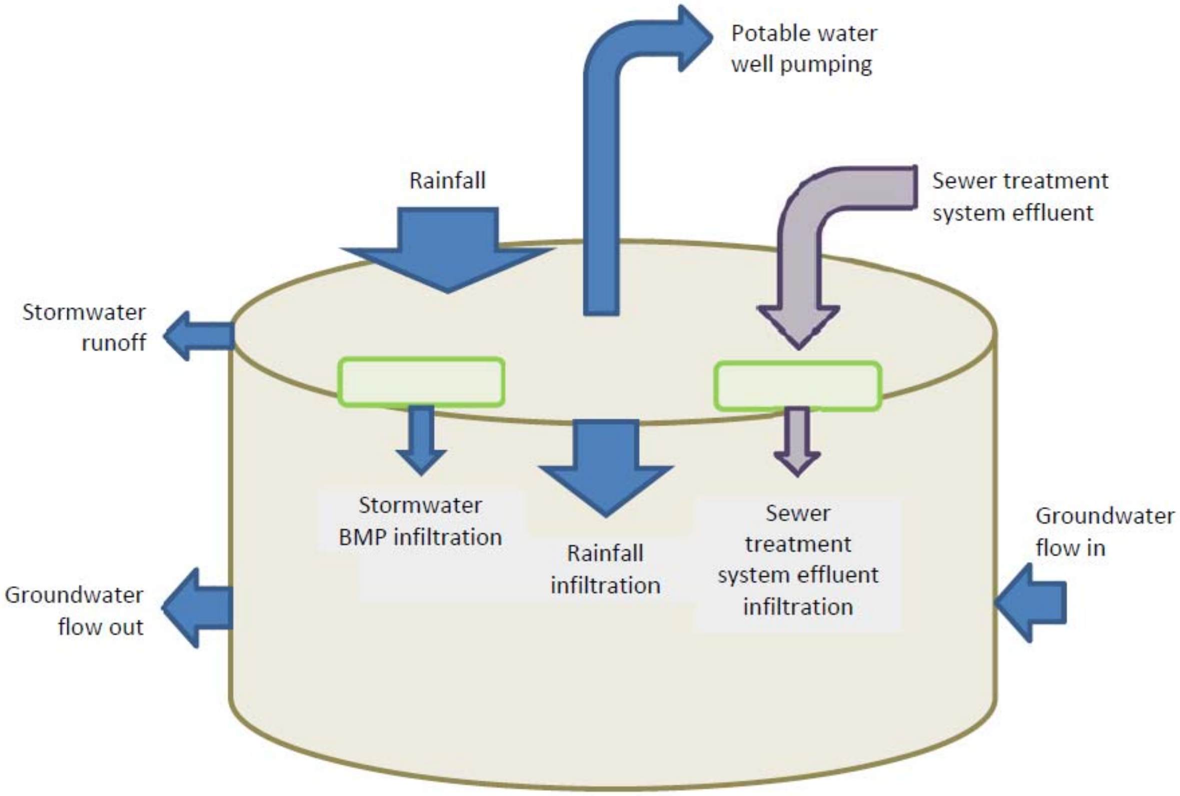


Figure 2-8 Groundwater Inflows and Outflows

The site will include some infiltration from stormwater runoff treatment and sewage treatment system effluent. The inflows to the proposed groundwater aquifer are rainfall infiltration, sewage treatment system effluent infiltration, and subsurface upstream groundwater inflow. The outflows are well pumping and groundwater outflow to the downstream aquifer.

Under existing conditions, this scenario does not exist, and therefore the sewage treatment system effluent infiltration become zero, as does the well pumping.

2.6.3.1 Radius of Influence

There are several equations to estimate the ROI given the characteristics of an aquifer and the pumping rate. On-site drilling of a well helped to determine the characteristics of the aquifer beneath the future Brown Field Station. A pumping test was performed at the site to determine the transmissivity of the aquifer, which was found to be 495 ft^2 per day. The storage coefficient was estimated to be 0.0006. The ROI was determined through the Cooper-Jacob equation for drawdown rearranged to solve for ROI. The ROI was calculated to be 1,362 feet. This gives an area of influence of 133.79 acres.

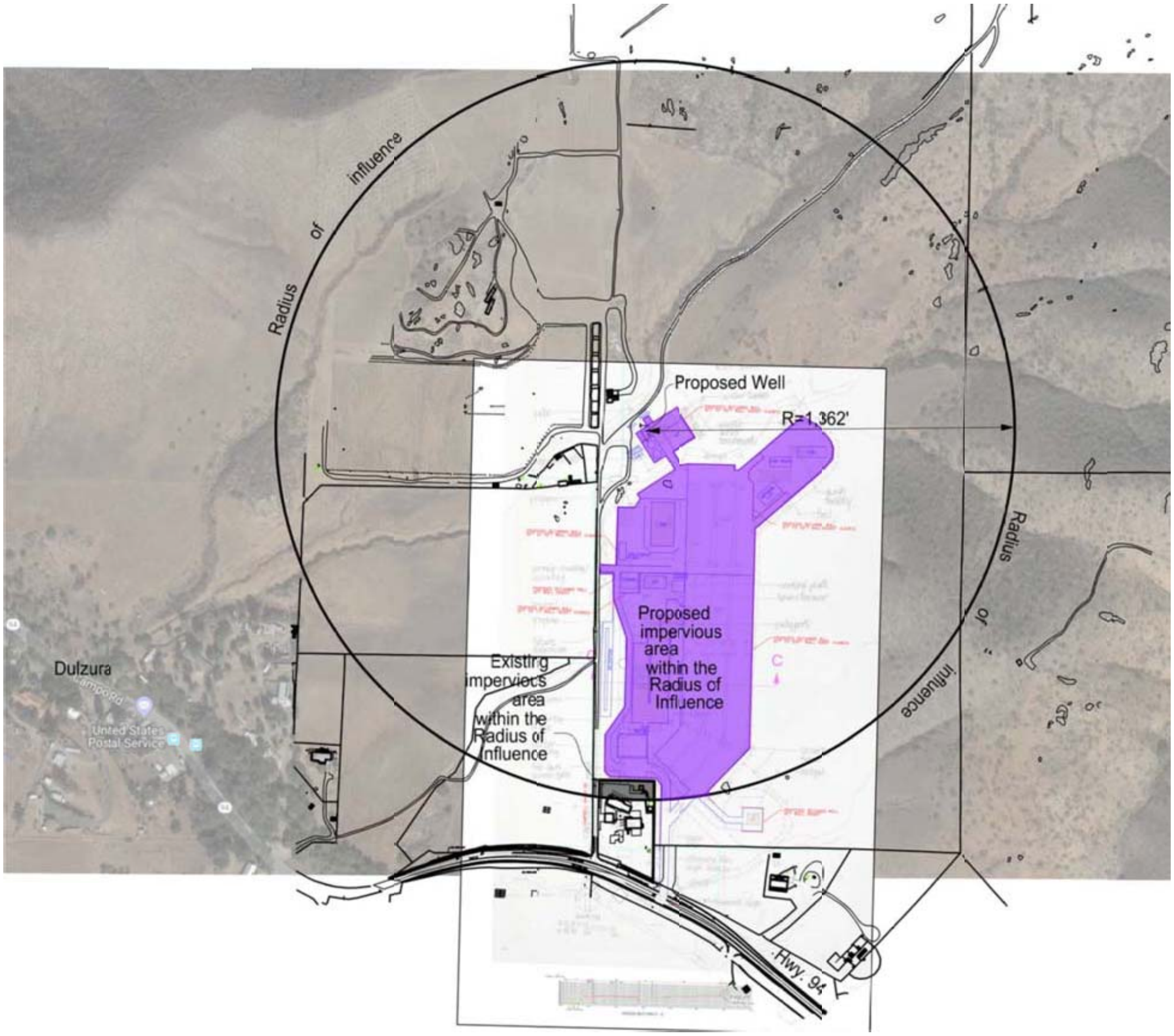


Figure 2-9 Radius of Influence

The existing impervious area within the ROI is composed of exposed rock outcrops within the ROI and the northernmost sliver of the CalFire property at the corner of Highway 94 and Campbell Ranch Road, as shown in **Figure 2-9**. The existing impervious area is 3.00 acres. The proposed impervious area within the ROI includes the exposed rock outcrops and most of the proposed site. The proposed impervious area is approximately 17.85 acres total, of which approximately 15.16 acres is Brown Field Station.

2.6.3.2 Rainfall and Infiltration

Rainfall data was acquired from the Dulzura Summit rain gage (Station no. 48) maintained by the San Diego County Flood Control District. The Dulzura Summit rain gage is only about three miles southeast of the site. Due to its proximity to the site, the Dulzura Summit rain gage was selected for this analysis. The dataset included daily rain gage data from January 1, 1987 to December 31, 2017, 31 complete years.

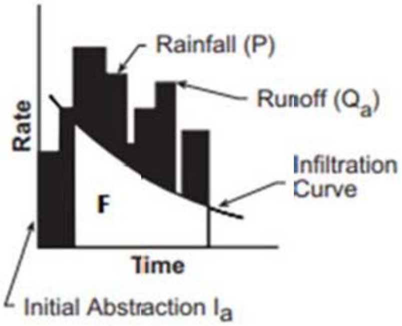


Figure 2-10 Partition of Precipitation in the NRCS Method

The partitioning of rainfall into runoff, infiltration, and evapotranspiration was governed by the Natural Resources Conservation Service (NRCS) Hydrologic Method described in Section 4 of the San Diego County Hydrology Manual (the Manual). **Figure 2-10**, adapted from Figure 4-3 in the Manual, shows a diagram of the components of the NRCS curve number method. According to the theory behind the NRCS method, precipitation that falls to earth is separated into three components: runoff (Q), initial abstraction (I_a) and retention (F). The initial abstraction includes the portion of the rainfall that is not available for either infiltration or runoff and includes the portion of the rainfall that is used to wet surfaces rior to reaching the ground. Generally, the I_a component is assumed to leave the drainage area through evapotranspiration. If the amount of rainfall is less than the I_a component, then neither infiltration nor runoff occurs.

The retention F is the portion of the rainfall that is assumed to infiltrate to groundwater, while the runoff Q is the portion of the rainfall that leaves the site through surface channels as stormwater runoff. Both Q and F are related to the potential maximum retention S, which is determined through curve number (CN) values that are dependent on the surface and soil properties. The maximum CN value, for a completely smooth i pervious surface, is 100, and this value would result in a potential maximum retention S of zero. Pervious areas at the site were assumed to have a CN_2 of 89 (for D soil), while the impervious areas were assumed to have a CN_2 of 98. I_a is also dependent on the CN of a watershed; in hydrology, I_a is usually set to equal 0.2 times S. Impervious surfaces lack the plants, soil, and varied topography that tends to intercept rainfall without producing runoff. Therefore, the higher the CN value is in a watershed, the lower the I_a value will be.

The CN varied with time based on the Precipitation Zone Number (PZN), as described in the Manual. A PZN of 1 was assigned to each day where there was less than a tenth of an inch of precipitation in the preceding five days, a PZN of 2 was assigned to each day where there was between a tenth of an inch and one inch of precipitation in the preceding five days, and a PZN of 3 was assigned to each day where there was over an inch of rain in the preceding five days. For impervious surfaces, CN_1 was 94 and CN_3 was 99, and for pervious surfaces, CN_1 was 76 and CN_3 was 96. These values are given from Table 4-10 in the Manual.

2.6.3.3 Aquifer Extraction and Replenishment

The demand for potable water at the site was determined through an estimation of both the population of the border patrol station and the per capita use of water. Brown Field Station will be constructed for a population of 400 agents, but with room for expansion to 600 agents. The per capita use of water was determined through the water demands from another border patrol station in the San Ysidro Mountains, Campo Station. Campo Station has approximately 400 agents. Data from Campo Station revealed that the average monthly demand was 4,290 gallons per day (gpd). Assuming the same per capita rate, Brown Field Station will have a demand of approximately 6,500 gpd if the facility expands to accommodate 600 agents. This translates to an annual demand of 2.37 million gallons per year, or 7.29 acre-feet per year (afy).

The site will contain a treatment unit to treat sanitary sewer flows onsite. Tertiary effluent will then be pumped to a new retention pond at the southwest side of the site for infiltration. It is assumed for this analysis that the retention pond will be able to infiltrate 100% of the sewer treatment system effluent, and that the effluent will be 80% of the water demand.

The site will require post-construction stormwater management in the form of a structural Best Management Practice (BMP). The BMP will be designed to capture the flow from an 85th percentile storm, which for the Dulzura site is a storm that drops about 0.85 inches of rain over 24 hours, according to the maps published in the San Diego County BMP Design Manual.

Though soils onsite support high infiltration rates, it is likely that an infiltration BMP will not be feasible due to the location of bedrock at a depth less than 15 feet below the site. A biofiltration basin will likely be necessary to treat stormwater runoff generated onsite. According to the San Diego County BMP Design Manual, biofiltration basins can be either lined or un-lined to provide incidental infiltration. However, for the purposes of this analysis, the infiltration component is assumed to be zero, the worst case scenario.

The amount of groundwater entering and exiting the theoretical cylinder of soil beneath the ROI is an unknown quantity. An assumption was made that the potentiometric surface of the aquifer is unchanging in time under existing conditions on average. For this assumption to be true, the long term average of groundwater leaving the cylinder must equal the long term average of groundwater entering the cylinder plus the long term average of infiltration within the ROI from rainfall. Therefore it was assumed that the cylinder experienced a constant loss equal to the average infiltration under existing conditions.

2.6.3.4 Aquifer Calculations

Calculations were performed on a daily basis for the 31-year period covered by the Dulzura Summit rain gage data between 1987 and 2017. Two design scenarios were analyzed covering the same geographic space defined by a circle centered on the proposed well with a radius equal to the ROI calculated in **Section 2.6.3.1**. Both the existing and the proposed scenarios include a rainfall-runoff analysis involving two land use types to model 1) the pervious areas within the ROI and 2) the impervious areas within the ROI. The proposed site is modeled as 15.16 acres of additional impervious area, and 15.16 acres of reduced pervious area, within the ROI.

Under the proposed scenario, well pumping and sewer infiltration are also taken into account in the groundwater analysis. In both the existing and proposed scenarios, a constant rate of groundwater flux is assumed to flow out of the aquifer underlying the ROI.

The average annual rainfall over the 31-year period studied between 1987 and 2017 was 12.53 inches, according to the Dulzura Summit rain gage. The annual rainfall in each calendar year is shown in **Figure 2-11.**

Over 31 years, a total of 4,330 acre-feet of rain fell on the 133.79-acre circular area defined by the ROI of the proposed well. The average annual rainfall was 139.66 acre-feet.

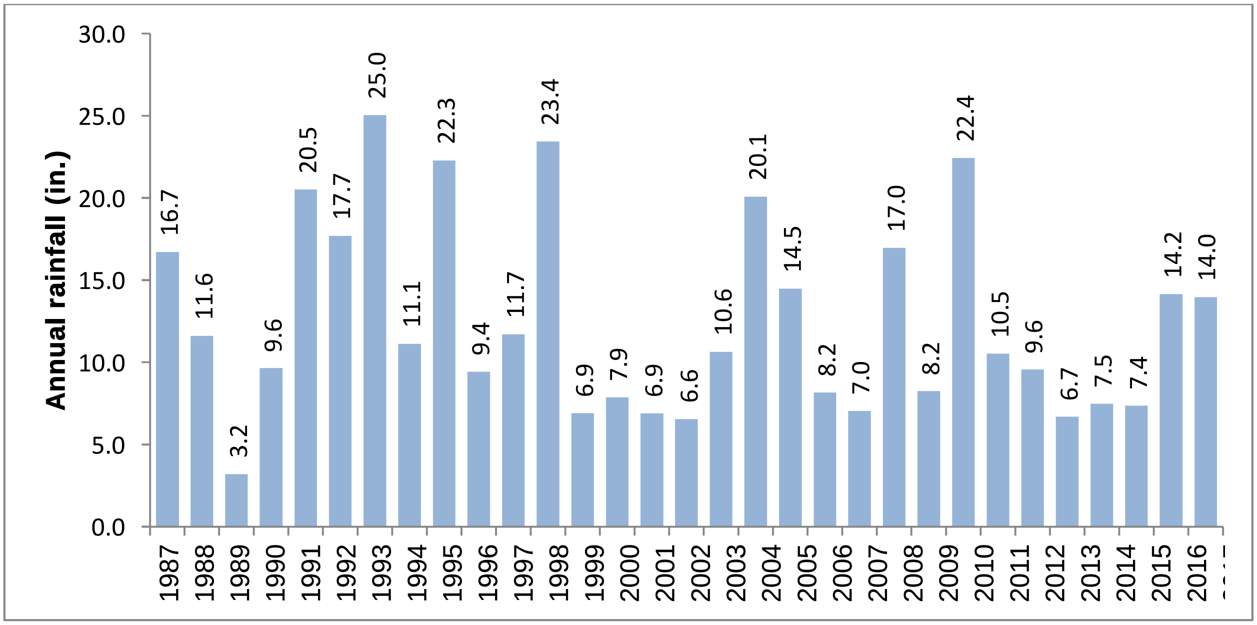


Figure 2.11 Annual Precipitation at Dulzura Summit Rain Gage

The rainfall was partitioned three ways into runoff, initial abstraction, and infiltration, according to the NRCS Hydrologic Method described in **Section 2.6.3.2**. **Figure 2-12** below and **Figure 2-13** on the following page depict the partitioning of the 139.66 acre-feet of annual precipitation

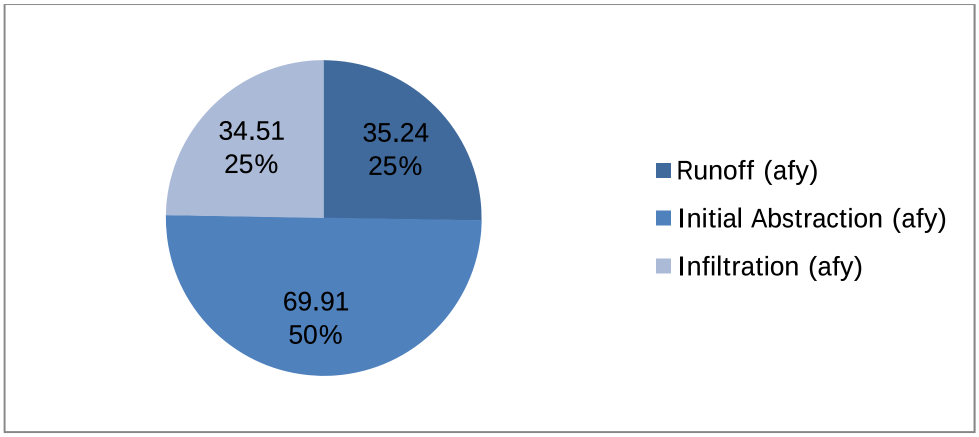


Figure 2-12 Average Annual Partition of Precipitation, Existing Conditions

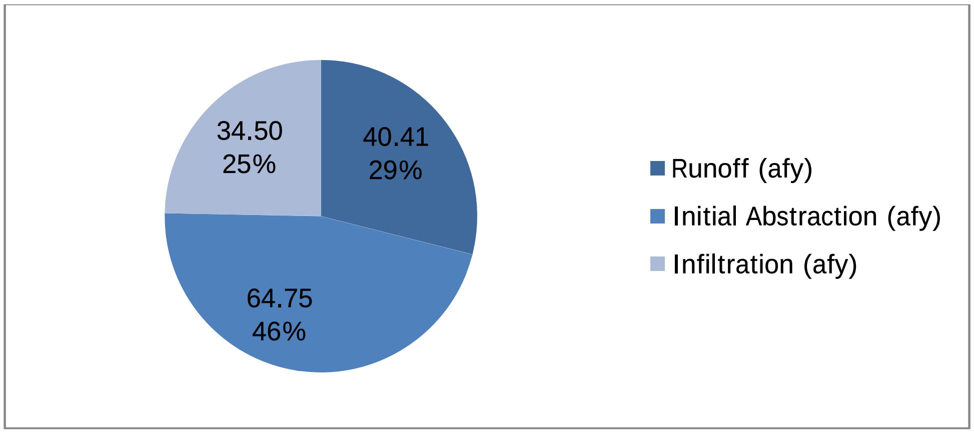


Figure 2-13 Average Annual Partition of Precipitation, Proposed Conditions

In both existing and proposed conditions, the NRCS model shows about half the rainfall became initial abstraction and left the site through evapotranspiration. About a quarter of the rainfall became runoff, while the final quarter infiltrated to groundwater.

More of the area bounded by the ROI became impervious surface in the proposed conditions, which decreased the initial abstraction quantity, while runoff increased as expected. The infiltration quantity was mostly unchanged (34.51 acre-feet under existing conditions, 34.50 acre-feet under proposed conditions) because the lower acreage of pervious soils was offset by the increased amount of rainfall made available as initial abstraction was reduced.

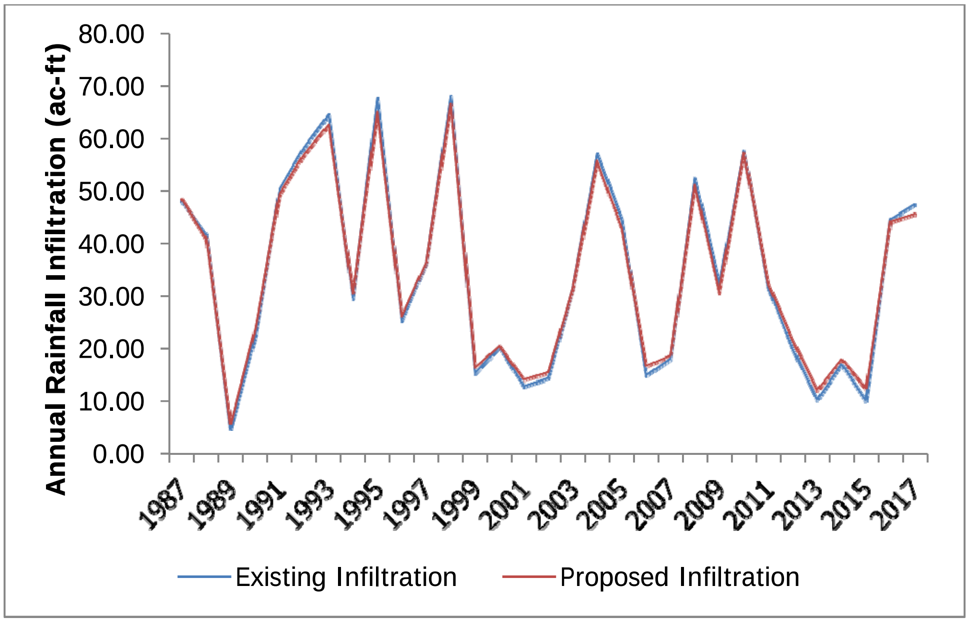


Figure 2-14 Annual Infiltration from Rainfall

Figure 2-14 shows the annual infiltration in each year within the circle bounded by the ROI under existing and proposed conditions.

The well was assumed to operate under the proposed condition scenario. The well was assumed to operate at a constant rate of 7.28 acre-feet per year. Effluent from the sewer treatment system onsite was assumed to infiltrate at a constant rate equal to 80% of the well pumping rate, or 5.83 acre-feet per year.

2.6.3.5 31-Year Water Balance

The full water balance within the ROI over the 31-year span of time analyzed in this report for the existing condition scenario is shown in **Table 2-1**.

Table 2-1 Water Balance - Existing Conditions

Year	In			Out		Net Ground-water Recharge (ac-ft)	Balance Over Time (ac-ft)
	Rainfall Infiltration (ac-ft)	BMP Infiltration (ac-ft)	Sewer Infiltration (ac-ft)	Well Pumping (ac-ft)	Net Ground-water Flux (ac-ft)		
1987	48.14	0.00	0.00	0.00	34.51	+14	+14
1988	41.67	0.00	0.00	0.00	34.51	+7	+21
1989	4.62	0.00	0.00	0.00	34.51	-30	-9
1990	22.20	0.00	0.00	0.00	34.51	-12	-21
1991	50.42	0.00	0.00	0.00	34.51	+16	-5
1992	58.41	0.00	0.00	0.00	34.51	+24	+18
1993	64.65	0.00	0.00	0.00	34.51	+30	+49
1994	29.60	0.00	0.00	0.00	34.51	-5	+44
1995	67.89	0.00	0.00	0.00	34.51	+33	+77
1996	25.38	0.00	0.00	0.00	34.51	-9	+68
1997	36.31	0.00	0.00	0.00	34.51	+2	+70
1998	68.18	0.00	0.00	0.00	34.51	+34	+103
1999	15.23	0.00	0.00	0.00	34.51	-19	+84
2000	20.34	0.00	0.00	0.00	34.51	-14	+70
2001	12.78	0.00	0.00	0.00	34.51	-22	+48
2002	14.33	0.00	0.00	0.00	34.51	-20	+28
2003	31.55	0.00	0.00	0.00	34.51	-3	+25
2004	57.21	0.00	0.00	0.00	34.51	+23	+48
2005	44.73	0.00	0.00	0.00	34.51	+10	+58
2006	14.85	0.00	0.00	0.00	34.51	-20	+38
2007	18.03	0.00	0.00	0.00	34.51	-16	+22
2008	52.48	0.00	0.00	0.00	34.51	+18	+40
2009	32.19	0.00	0.00	0.00	34.51	-2	+38
2010	57.73	0.00	0.00	0.00	34.51	+23	+61
2011	31.63	0.00	0.00	0.00	34.51	-3	+58
2012	19.84	0.00	0.00	0.00	34.51	-15	+43
2013	10.19	0.00	0.00	0.00	34.51	-24	+19
2014	16.97	0.00	0.00	0.00	34.51	-18	+1
2015	10.02	0.00	0.00	0.00	34.51	-24	-23
2016	44.69	0.00	0.00	0.00	34.51	+10	-13
2017	47.50	0.00	0.00	0.00	34.51	+13	+0

The full water balance within the ROI over the 31-years analyzed in this report for the proposed condition scenario is shown in **Table 1-2**.

Table 1-2 Water Balance - Proposed Conditions

Year	In			Out		Net Ground-water Recharge (ac-ft)	Balance Over Time
	Rainfall Infiltration (ac-ft)	BMP Infiltration (ac-ft)	Sewer Infiltration (ac-ft)	Well Pumping (ac-ft)	Net Ground-water Flux (ac-ft)		
1987	48.45	0.00	5.83	7.29	34.51	+12	+12
1988	40.95	0.00	5.83	7.29	34.51	+5	+17
1989	5.57	0.00	5.83	7.29	34.51	-30	-13
1990	23.58	0.00	5.83	7.29	34.51	-12	-25
1991	49.49	0.00	5.83	7.29	34.51	+14	-12
1992	57.03	0.00	5.83	7.29	34.51	+21	+9
1993	62.71	0.00	5.83	7.29	34.51	+27	+36
1994	30.51	0.00	5.83	7.29	34.51	-5	+31
1995	65.09	0.00	5.83	7.29	34.51	+29	+60
1996	26.09	0.00	5.83	7.29	34.51	-10	+50
1997	36.45	0.00	5.83	7.29	34.51	+0	+50
1998	66.66	0.00	5.83	7.29	34.51	+31	+81
1999	16.31	0.00	5.83	7.29	34.51	-20	+61
2000	20.47	0.00	5.83	7.29	34.51	-15	+46
2001	14.01	0.00	5.83	7.29	34.51	-22	+24
2002	15.44	0.00	5.83	7.29	34.51	-21	+3
2003	31.43	0.00	5.83	7.29	34.51	-5	-1
2004	55.79	0.00	5.83	7.29	34.51	+20	+19
2005	42.99	0.00	5.83	7.29	34.51	+7	+26
2006	16.67	0.00	5.83	7.29	34.51	-19	+6
2007	18.62	0.00	5.83	7.29	34.51	-17	-11
2008	51.16	0.00	5.83	7.29	34.51	+15	+4
2009	30.73	0.00	5.83	7.29	34.51	-5	-1
2010	57.31	0.00	5.83	7.29	34.51	+21	+20
2011	32.34	0.00	5.83	7.29	34.51	-4	+17
2012	21.47	0.00	5.83	7.29	34.51	-14	+2
2013	12.07	0.00	5.83	7.29	34.51	-24	-22
2014	17.92	0.00	5.83	7.29	34.51	-18	-40
2015	12.37	0.00	5.83	7.29	34.51	-24	-63
2016	44.22	0.00	5.83	7.29	34.51	+8	-55
2017	45.70	0.00	5.83	7.29	34.51	+10	-45

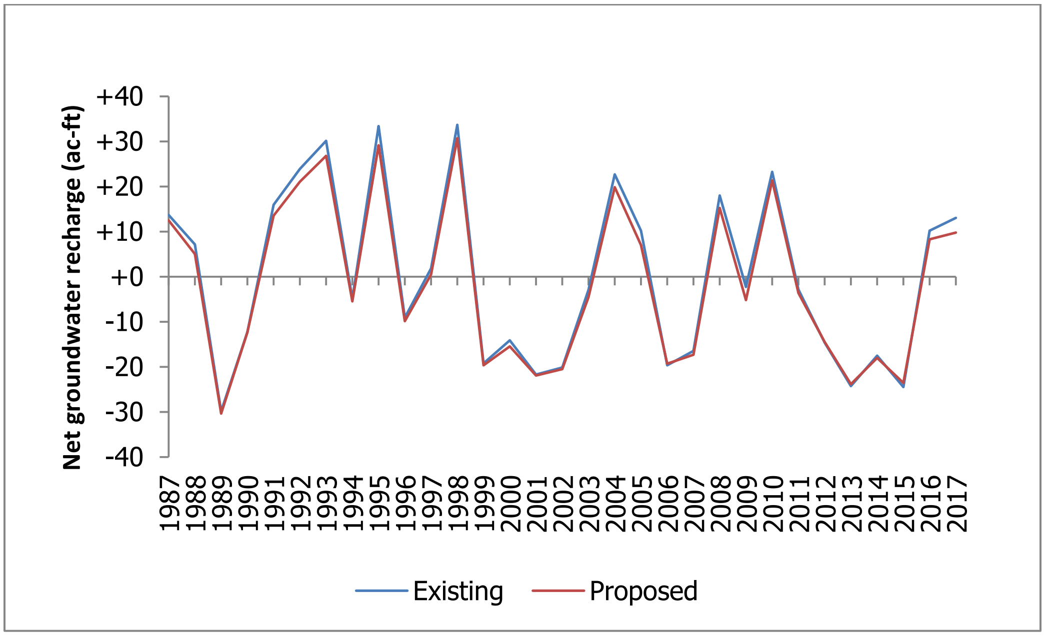


Figure 2-15 Net Groundwater Recharge

Figure 2-15 shows the net groundwater balance for the site. Less water is shown infiltrating into the subsurface due to the proposed conditions than under existing conditions.

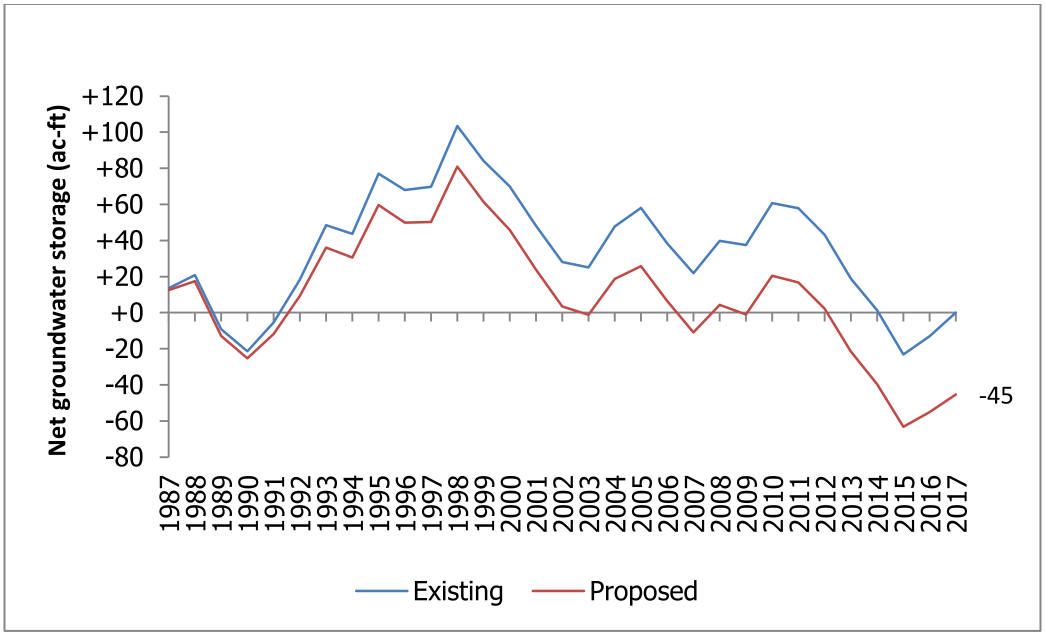


Figure 2-16 Groundwater Balance

Figure 2-16 shows the groundwater balance within the ROI assuming a starting volume of zero acre-feet in 1986. Over the 31-year span of time analyzed as part of this report, construction of the site would have yielded a reduction of 45 acre-feet of groundwater over the baseline existing conditions.

3. Well Water Evaluation and Analysis

The water quality results from the Brown Field BPS well were sampled and tested to determine if treatment will be necessary to permit the potable water supply at this facility. The proposed facility will have two wells upon completion of the facility construction; however, only one well has been drilled at this time to evaluate t e pumping rate, water quality, and aquifer capacity. The water system facilities will include a pump building that will house water treatment equipment, assumed to be chlorine injection at a minimum, and booster pumps for the potable water system and additional booster pumps for the fire suppression system. The sections below provide additional details on the water quality analysis and pre-treatment options.

3.1 Water Sample Collection and Analysis

The groundwater well is intended to provide potable water and fire water for the Brown Field BPS. The samples were collected from well during the final hours of the pump test. CWE staff trained in the methods for sampling took care in collecting the samples to reduce the possibility of sample contamination. A valve located near the well head was utilized to fill the sample bottles as depicted in **Figure 3-1**.

Figure 3-1 Pump Valve for Water Quality Sample Collection

3.2 Analytical Results

During the water sample collection, field measurement and observations were collected by CWE. Water samples were collected during the final hours of the pump test. Sample bottles were packed in ice and transported to Analytical Chemical Labs, Inc. in San Diego for analysis. Analytical results for laboratory wet chemistry are summarized in **Table 3-1.** The results of the water analysis show high levels of Fluoride, Manganese, and detectable levels of Coliform Bacteria. Based upon the presence of Coliform

Bacteria detected in the initial analysis and the need to test for the Most Probable Number (MPN) for Coliform Bacteria if present, additional test for Coliform Bacteria was taken after thoroughly chlorinating and flushing the well. Since Analytical Chemical Labs does not perform the MPN, D-TEK Analytical Laboratories in Carlsbad was requested to perform the Coliform Bacteria analysis and if detected provide the MPN. The analysis of the water sample did not detect the presence of Coliform Bacteria and hence further analysis for the MPN was not required. Chain of Custody forms and lab reports are included in **Appendix C** at the end of this report.

Table 3-1 Water Quality Results for Sample 1

Comparison of Drinking Water Sample Analysis to Title 22 Maximum Contaminant Levels (MCL)

Sample Collection Date: 01/18/2018

Paramater	**Results**	**Maximum Contaminant Levels (MCL)**	**Table 6449-A Secondary MCL**	**Table 6449-B Secondary MCL**	**Units**	**Detection Limit for Reporting**	**Method**	**Notes**
Fluoride (F)	**5.57**	2.000	-----	-----	mg/L	0.05	SM 4500-F C	
Nitrate as Nitrogen	2.02	10.000	-----	-----	mg/L	0.02	SM 4500-NO3-D	
Nitrite as Nitrogen	0.006	1.000	-----	-----	mg/L	0.003	HACH 8507	
Total Hardness	126.6[b]	-----	-----	-----	mg/L	0.01	EPA 200.7	MCL not specified in Title 22
Total Dissolved Solids (TDS)	351	-----	-----	500[a]	mg/L	1	SM 2540 E-1997	
Carbonate Alkalinity	138.6[b]	-----	-----	-----	mg/L	0.5	SM 2320 B	MCL not specified in Title 22
Bicarbonate Alkalinity	169.1[b]	-----	-----	-----	mg/L	0.5	SM 2320 B	MCL not specified in Title 22
Hydroxide Alkalinity	382[b]	-----	-----	-----	mg/L	0.5	SM 2320 B	MCL not specified in Title 22
pH	7.49[b]	-----	-----	-----	pH	0.01	SM4500-H+R	MCL not specified in Title 22
Specific Conductance	701[b]	-----	-----	900[a]	µS/cm	0.1	EPA 120.1	
Total Dissolved Solids	496	-----	-----	500[a]	mg/L	1	SM 2540 C	
Calcium	43.37[b]	-----	-----	-----	mg/L	0.005	EPA 200.7	MCL not specified in Title 22
Magnesium	14.45[b]	-----	-----	-----	mg/L	0.001	EPA 200.7	MCL not specified in Title 22
Sodium	58.27[b]	-----	-----	-----	mg/L	0.005	EPA 200.7	MCL not specified in Title 22
Iron	0.2278[b]	-----	0.3[a]	-----	mg/L	0.002	EPA 200.7	
Manganese	**0.1902[b]**	-----	0.05[a]	-----	mg/L	0.0001	EPA 200.7	
Colifom Bacteria	**+**[c]	—	-----	-----	Presence	1/100ml	Colisure SM 9223B	
E. *coli*	**—**	—	-----	-----	Presence	1/100ml	Colisure SM 9223B	

[a] Recommended values- Title 22 California Code of Regulations California Regulations Related to Drinking Water Table 6449-A and 6449-B

[b] Transient-noncommunity water systems shall monitor their sources or distribution system entry points representative of the effluent of source treatment for bicarbonate, carbonate, and hydroxide alkalinity, calcium, iron, magnesium, manganese, pH, specific conductance, sodium, and total hardness at least once

[c] Well-derived water to undergo treatment

4. Potential Treatment Measures

Potential treatment measures for removal of excess Fluoride and Manganese that will be considered for use at the Brown Field BPS are presented in the sections below. Treatment mechanisms and information regarding each measure is provided for each treatment method as well as advantages, disadvantages, and cost.

4.1 Reverse Osmosis

Reverse osmosis (RO) is a water purification technology that uses a semipermeable membrane to remove ions, molecules, and larger particles from water. Osmosis is the natural process in which solvents move from an area of low solvent concentration to high concentration through a membrane. This is due to high water potential in the low concentration water versus low water potential in the high concentration water. RO is the opposite of this process and does not occur naturally, requiring an applied pressure to overcome osmotic pressure and move water with high solvent concentration through a membrane to low solvent concentration. The result is clean water on one side of the membrane and an accumulation of particles on the other side of the membrane which is flushed out of the system and in some cases cycled through the system again. RO systems are designed such that constituents at the molecular level are unable to pass through the semipermeable membrane, allowing hydrogen and oxygen molecules to pass through. The semipermeable membrane can be made of different materials but the commonality is its pores are small enough that only water molecules can pass through, leaving behind dissolved solids. RO is highly effective in removing dissolved constituents such as lead, iron, chromium, arsenic, and aluminum. Pretreatment is required before utilizing a RO system to prevent fouling of the membrane by fine particles or biological growth. Usable water is reduced due to the production of waste water when using an RO treatment and therefore must be considered when designing production flow rate. The proposed dimensions of an RO system to treat 15 gpm is approximately 4 feet long, 2 feet wide, and 4 feet high, requiring a footprint of about 8 square feet. This includes area required for a high pressure pump, storage tanks, pipes, and system framework. Pretreatment methods will increase the footprint depending on which system or systems are chosen. Well water from the test well suggests using an ion exchange pretreatment system to reduce hardness followed by a RO system to remove TDS and metals.

4.1.1 Advantages

RO systems provide the purest form of water if used in combination with pretreatment methods. Dissolved solids that are difficult to remove by other treatment systems are removed by RO systems by up to 97%.

4.1.2 Disadvantages

Not all water entering the system is recoverable in RO systems. Typically between 75% to 80% of feed water is recovered in industrial/municipal RO systems, leaving 25% to 20% as waste water. To prevent scaling of the system, it is recommended that water is pretreated by activated carbon, ion exchange, sand filters, or a combination of these. High pollutant loading causes increased frequency of system maintenance. As a high pressure system, RO systems require more energy input to the system. Water treated by RO and intended for potable water use will need further treatment or blending to add minerals back into the water. Training is necessary to operate and monitor the programmable system.

4.1.3 Cost

RO systems are dependent on designed flow rate and pollutant loading. RO system costs can range between $3,500 for 5 gpm flow rate system to $10,000 for 25 gpm flow rate system. Installation, operation, and maintenance is not included in this price and will be additional. Operation and maintenance costs will vary with usage and pollutant loading. High pollutant loading will require frequent replacement of filters.

4.2 Activated Alumina

Activated Alumina is manufactured from aluminium hydroxide by dehydroxylating it in a way that produces a highly porous material; this material can have a surface area significantly over 200 m^2/g.. Adsorption is both the physical and chemical process of accumulating a substance at the interface between liquid and solids phases. Activated Alumina utilizes its high surface area to adsorb dissolved solids in liquids or gas, creating a surface film of adsorbates, removing these contaminants from water. Activated Alumina is effective in removing organic contaminants, chlorine, constituents producing foul taste and odor, and pharmaceutical byproducts s. Although Activated Alumina removes pollutants from water it does not destroy or degrade them, it simply stores the molecules on its surface. Typically, Activated Alumina is packed into cylindrical columns, tanks, or pipes, allowing contaminated water to flow through the Activated Alumina. A benefit to Activated Alumina treatment systems comes from the ability to reactivate and reuse the Activated Alumina once it is saturated with pollutants. Using a two adsorber tank system requires a land area of about 10 square feet. This configuration is desirable as it permits carbon exchange or backwash operations to be performed on one adsorber tank without interrupting the second adsorber tank.

4.2.1 Advantages

Activated Alumina is effective in removing organics, such as unwanted taste and odors, chlorine, fluorine, pharmaceuticals, and volatile organic compounds. Activated Alumina is harder and longer lasting than powdered activated carbon, clean to handle, purify large volumes of gas or liquids of a consistent quality, and can be reactivated and reused many times. The period between reactivation varies significantly but can average 18 months. The frequency of reactivation is dependent on pollutant loading onto the Activated Alumina. As Activated Alumina is an adsorptive process, no chemicals are added to the water for treatment.

4.2.2 Disadvantages

Activated Alumina is not effective in removing dissolved minerals, heavy metals, bacteria, or other microorganisms. When pollutant loading is high, reactivation of Activated Alumina will be required more frequently, increasing maintenance costs.

4.2.3 Costs

An Activated Alumina system able to sustain a 15 gpm flow composed of a two tank system can be purchased for $10,000. Additional Activated Alumina can also be purchased for $2.50/pound plus applicable tax and freight. Generally, price per pound is less expensive when buying in higher quantities. Installation, operation, and maintenance costs should also be considered but will vary with pollutant loading. The Activated Alumina system will require monitoring of the pressure system and Activated Alumina saturation.

4.3 Ion Exchange

Ion exchange is an exchange of ions between two electrolytes or between an electrolyte solution and a complex. This type of process is a form of surface phenomenon called sorption. In the case of ion exchange, sorption describes the surface interaction of a liquid and solid where positively or negatively charged particles of the ion exchange resins attract anions or cations which bind to the surface. Ion exchange resins can be designed to remove only cations, anions, or can be mixed to remove both. The positively or negatively charged particles of ion exchangers enables these systems to efficiently remove single and doubly charged ions such as sodium, potassium, chlorine, or calcium and magnesium effectively reducing hardness. Large scale ion exchange systems operate on a cyclic basis where water flows through the resin container until the resin is considered exhausted when water leaving the exchanger contains more than the desired maximum concentration of the ions being removed. Typically, water flows from a tank designed to remove either anions or cations then flow to a second tank designed to remove whichever charged particles not removed by the first. Resin is then regenerated by sequentially backwashing the resin bed to remove accumulated solids, flushing removed ions from the resin with a concentrated solution of replacement ions, and rising the flushing solution from the resin. Acids or bases are added to the ion exchange resins for regeneration, effectively washing off cations or anions binded to the ion exchange surface. Ion exchange systems require a storage tank for the ion exchange resin, a bulk brine storage tank, a brine waste storage tank, and land area for the network of pipes and pumps.

4.3.1 Advantages

Ion exchange water treatments is widely used for water softening, efficient for single-charged or double charged monatomic ions, polyatomic inorganic ions, organic acids and bases, and biomolecules that can be ionized. Ion exchange systems are also effective for removal of metals.

4.3.2 Disadvantages

Ion exchange treatment systems require disposal of brine waste and does not remove bacteria or other microorganisms. High pollutant loading will increase the frequency of ion exchange regeneration. Ion exchange does not work on particles with zero net charge.

4.3.3 Costs

Designed based on contaminant loading, can be purchased for $20,000 per ion exchange bed unit suitable for a 15 gpm flow rate. Operation and maintenance costs vary with pollutant loading.

5. Conclusion

CWE oversaw the drilling and installation of the Brown Field BPS well in Dulzura, California for the potable water and fire water supply at this facility. Pumping tests were conducted on the well and it was determined that the aquifer is sufficient to sustain a flow rate of 60 gpm for at least four hours of continuous pumping, resulting in a water surface drawdown in the well of 72 feet. At the completion of the 72 hour pump test the immediately rose 20 feet when the pump was shut off and an additional 30 feet within the next 5 minutes.

Additional pump tests were run when the well had to be chlorinated and flushed for additional Coliform Bacteria samples to be taken to the laboratory for testing. These additional pump tests physically confirmed the well drawdown of 6 feet at our proposed pumping rate of 15 gpm. This pumping rate will only occur for approximately 4-6 hours per day to refill the potable water storage tank. This additional pump test is a confirmation that the well that was drilled at the Brown Field BPS is adequate to serve this facility for the 400 agent completed capacity and the 600 agent future expanded capacity.

Water quality samples were collected from the well to determine treatment required prior to supplying the Brown Field BPS with a potable water supply. Although the well water constituents were generally found below MCLs with the exception of Fluoride and Manganese. Typical treatment of well water for potable use includes disinfection, usually chlorine injection. This water supply source will need additional treatment to remove excess Fluoride and Manganese. Potential treatment options include:

- Reverse osmosis;
- Activated Alumina; and
- Ion exchange

These three treatment options were reviewed for Advantages, Disadvantages, and Cost. The RO treatment system is recommended for this installation due to the thorough removal of constituents and the water treatment costs. Approval of the treatment system and potable water supply from this well will require permitting through the State of California Water Resource Control Board, Division of Drinking Water.

6. References

County of San Diego Department of Planning and Land Use. 2010. *General Plan Update Groundwater Study.* https://www.sandiegocounty.gov/content/dam/sdc/pds/gpupdate/docs/BOS_Aug2011/EIR/Appn_D_GW.pdf

County of San Diego Department of Public Works Flood Control Section. 2003. *San Diego County Hydrology Manual* https://www.sandiegocounty.gov/dpw/floodcontrol/floodcontrolpdf/hydro-hydrologymanual.pdf

California Department of Water Resources. 2003. *California's Groundwater Bulletin 118*. http://www.water.ca.gov/groundwater/bulletin118/docs/Bulletin_118_Update_2003.pdf. 2016.

California Department of Water Resources. 1980. *Ground Water Basins in California, A Report to the Legislature in Response to Water Code Section 12924*. January 1980

California Department of Water Resources. *California Well Standards, Combined (Bulletin 74-81 and 74-90)* http://wdl.water.ca.gov/groundwater/wells/california_well_standards/well_standards_content.cfm

California State Water Resource Control Board. 2016. *Comparison of MCLs and PHGs for Regulated Contaminants in Drinking Water*. http://www.waterboards.ca.gov/drinking_water/certlic/drinkingwater/MCLsandPHGs.shtml. November 2016

United States Environmental Protection Agency. 2010. *40 CFR Part 131 Water Quality Standards; Establishment of Numeric Criteria for Priority Toxic Pollutants for the State of California; Rule*. 2016.

USGS Water Resources: *http://nwis.waterdata.usgs.gov/ca/nwis/uv?cb_all_=on&cb_00010=on&cb_00010=on&cb_00010=on&cb_00055=on&cb_00060=on&cb_00065=on&cb_00095=on&cb_00095=on&cb_00300=on&cb_00301=on&cb_00400=on&cb_32295=on&cb_32315=on&cb_63680=on&cb_63680=on&cb_63680=on&cb_72137=on&cb_99133=on&cb_99409=on&format=gif_default&site_no=11447650&period=&begin_date=2012-11-01&end_date=2016-11-29*

Appendix A
Well Completion Report

STEHLY BROTHERS DRILLING, INC.

License: C-57 #709686
13268 McNally Road
Valley Center, California 92082
760-742-3668 / 760-742-4564 Fax

1/16/18

U.S. Customs and Border Protection
c/o CWE, 1561 E. Orangethorpe Ave., Suite 240
Fullerton, CA 92831 Attn: William Young
714-526-7500 x103 wyoung@cwecorp.com

CWE Project No. 17251:08s
Well Site: APN: 600-160-13-00
NE Corner Campo/Campbell Ranch
Dulzura, CA 91917
Permit LWELL-001874 11/27/17

Community Water Well drilled for U.S. Customs and Border Protection, Brown Field Border Patrol Station at the NE Corner of Campo Road and Campbell Ranch Road in Dulzura, CA 91917. Started 1/2/18 and Finished Well 1/12/18. APN: 600-160-13-00 Permit #LWELL 001874 11/27/17

0-4	Clay and D.G.
4-75	D.G. and a little Black & White (B&W) Granite
75-80	Slightly Fractured Brown D.G.
80-90	Brown D.G. and B&W Granite
90-300	B&W Granite
300-310	Slightly Fractured B&W Granite
310-598	B&W Granite
598-603	Fractured (Bigger Rock) B&W Granite Water: 50 GPM
603-1010	B&W Granite

SET AND PULLED 20' OF 8 5/8" TEMPORARY STEEL CASING
SET 101' OF 8 5/8" .250 STEEL CASING.
REAMED HOLE FROM 6" TO 8" FROM 101-717' WATER: 90 GPM TOTAL

Comments:

Total Depth Drilled:	1010'
Total Well Depth:	717'
Hole Diameter:	8" hole from 101-717' 6" hole from 717-1010'
Casing:	101' of 8 5/8" .250 Wall Steel Casing
Surface Seal:	Cement
Liner:	140' of 6.9" PVC SDR Screen 577' of 6.9" PVC SDR Solid
Water:	90 GPM

0-257'	Solid
257-277'	Screen
277-377'	Solid
377-397'	Screen
397-477'	Solid
477-497'	Screen
497-557'	Solid
557-617'	Screen
617-677'	Solid
677-697'	Screen
697-717'	Solid

*The free Adobe Reader may be used to view and complete this form. However, software must be purchased to complete, save, and reuse a saved form.

File Original with DWR

State of California

Well Completion Report

Refer to Instruction Pamphlet

No. e0362969

Page 1 of 2

Owner's Well Number Dulzura Brown Field Border

Date Work Began 01/02/2018 Date Work Ended 1/12/2018

Local Permit Agency County of San Diego

Permit Number LWELL-001874 Permit Date 11/27/17

DWR Use Only – Do Not Fill In

State Well Number/Site Number

Latitude N Longitude W

APN/TRS/Other

Geologic Log

Orientation ● Vertical ○ Horizontal ○ Angle Specify

Drilling Method Air Rotary Drilling Fluid Air

Depth from Surface Feet	to Feet	Description Describe material, grain size, color, etc
0	4	Clay and D.G.
4	75	D.G. and a little Black & White (B&W) Granite
75	80	Slightly Fractured Brown D.G.
80	90	Brown D.G. and B&W Granite
90	300	B&W Granite
300	310	Slightly Fractured B&W Granite
310	598	B&W Granite
598	603	Fractured (Bigger Rock) B&W Granite
		Water: 90 GPM
603	1,010	B&W Granite

Total Depth of Boring 1010 Feet

Total Depth of Completed Well 717 Feet

Well Owner

Name U.S. Customs and Border Protection, c/o CWE

Mailing Address 1561 E. Orangethorpe Ave., Suite 240

City Fullerton State CA Zip 92831

Well Location

Address NE Corner of Campo Rd. & Campbell Ranch Rd.

City Dulzura County San Diego

Latitude Deg. Min. Sec. N Longitude Deg. Min. Sec. W

Datum Dec. Lat. 32.64700 N Dec. Long. 116.77502 W

APN Book 600 Page 160 Parcel 13

Township 17S Range 2E Section 33

Location Sketch

(Sketch must be drawn by hand after form is printed.)

North

See Attached

West East

South

Illustrate or describe distance of well from roads, buildings, fences, rivers, etc. and attach a map. Use additional paper if necessary. **Please be accurate and complete.**

Activity

● New Well

○ Modification/Repair

○ Deepen

○ Other

○ Destroy

Describe procedures and materials under "GEOLOGIC LOG"

Planned Uses

● Water Supply

☐ Domestic ☑ Public

☐ Irrigation ☐ Industrial

○ Cathodic Protection

○ Dewatering

○ Heat Exchange

○ Injection

○ Monitoring

○ Remediation

○ Sparging

○ Test Well

○ Vapor Extraction

○ Other

Water Level and Yield of Completed Well

Depth to first water 598 (Feet below surface)

Depth to Static Water Level 222 (Feet) Date Measured 02/22/2018

Estimated Yield * 90 (GPM) Test Type Air Lift

Test Length (Hours) Total Drawdown (Feet)

*May not be representative of a well's long term yield.

Casings

Depth from Surface Feet	to Feet	Borehole Diameter (Inches)	Type	Material	Wall Thickness (Inches)	Outside Diameter (Inches)	Screen Type	Slot Size if Any (Inches)
0	101	15	Blank	Low Carbon Steel	.250	8 5/8"		
257	277	8	Screen	SDR17	.406	6.9"		
377	397	8	Screen	SDR17	.406	6.9"		
477	497	8	Screen	SDR17	.406	6.9"		
557	617	8	Screen	SDR17	.406	6.9"		
677	697	8	Screen	SDR17	.406	6.9"		

Annular Material

Depth from Surface Feet	to Feet	Fill	Description
0	98	Cement	
98	101	Bentonite	

Attachments

☐ Geologic Log

☐ Well Construction Diagram

☐ Geophysical Log(s)

☐ Soil/Water Chemical Analyses

☑ Other Location Sketch

Attach additional information, if it exists.

Certification Statement

I, the undersigned, certify that this report is complete and accurate to the best of my knowledge and belief

Name Stehly Brothers Drilling, Inc.
Person, Firm or Corporation

13268 McNally Road Valley Center CA 92082
Address City State Zip

Signed Paul Stehly 2/23/18 709686
C-57 Licensed Water Well Contractor Date Signed C-57 License Number

DWR 188 REV. 1/2006

IF ADDITIONAL SPACE IS NEEDED, USE NEXT CONSECUTIVELY NUMBERED FORM

COUNTY OF SAN DIEGO
DEPARTMENT OF ENVIRONMENTAL HEALTH

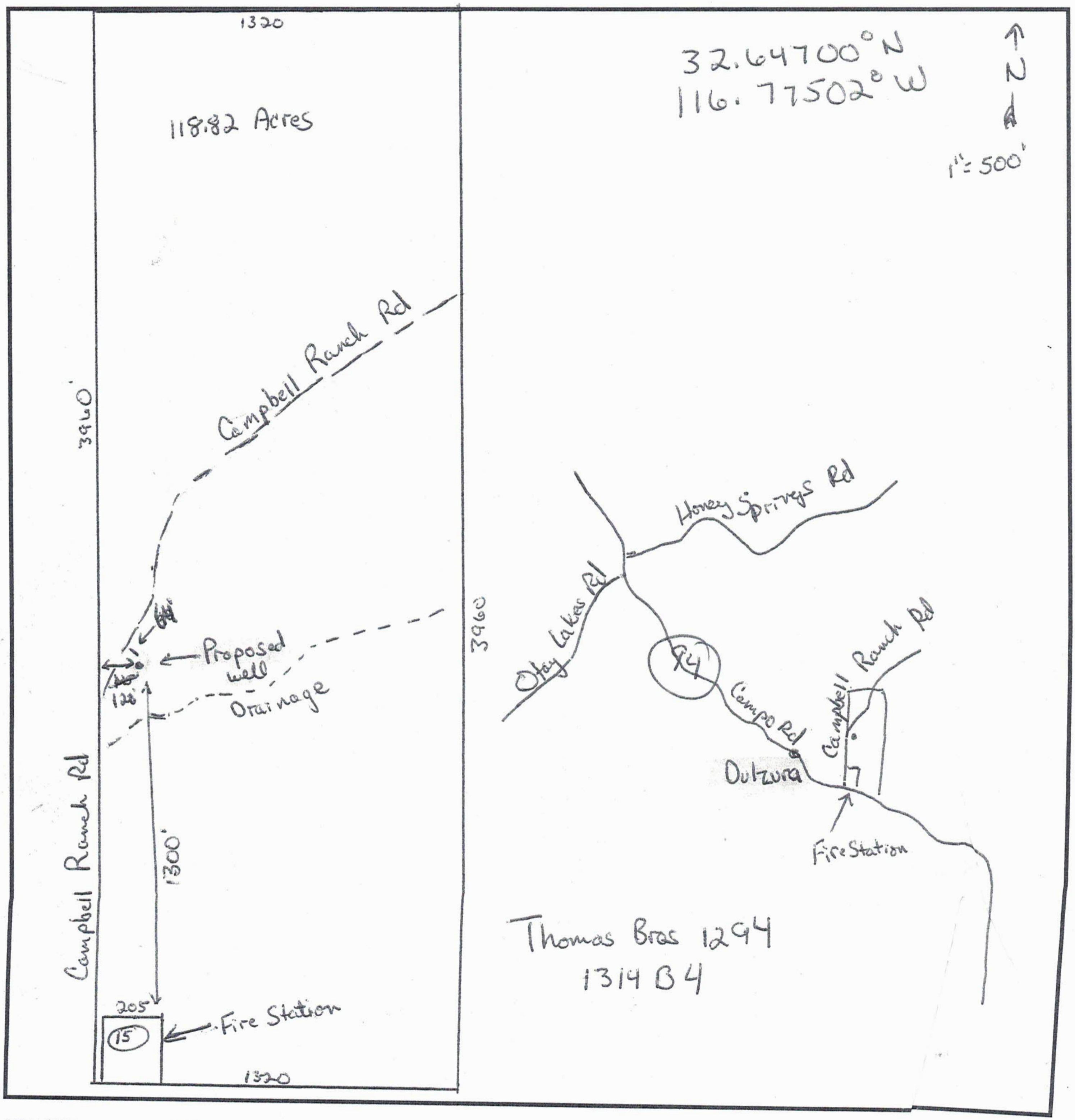
Control #: ____________________

Assessor's Parcel Number: 600-160-13

~~32.647049°N~~
~~116.775076°W~~

LOCATION

Indicate below the vicinity and exact location of well with respect to the following items: Property lines, water bodies or water courses, drainage pattern, easements, roads, existing wells, sewers and private sewage disposal systems and other potential contamination sources, including dimensions.



32.64700°N
116.77502°W

Thomas Bros 1294
1314 B4

DEH:LU 731b (Rev. 7/2002) NCR

STEHLY BROTHERS DRILLING, INC.

Date: **1/16/2018 to 1/20/18**

Customer: Delzura Border Patrol Station Phone:

Location: Delzura

Test Pump: **65L20** **AMPS:** 27A

Generator Hrs: **Start** **5539**

Finish **5611**

Static Water Level: 223' **Total Well Depth: 1010'** **Total Pump Depth: 567'**

TIME	GPM	Pumping Levels	AMPS	PSI	NOTES
2:38PM	67	223			**DAY 1** 1/16/18
2:40	67	258			88% or 52.8hz
2:45	65	259			
2:50	67	273			
2:55	67	274			
3:00	67	274			
3:15	67	275			
3:30	65	276			
3:45	65	272		>100	
4:00	64	277			
4:30	64	277			
5:00	64	277.5			
5:30	64	277.5			
11:00	65	285		>100	**DAY 2** 1/17/18
3:00	64	286.5			
4:00	64	287			
5:00	64	287			
6:00	64	287.5			

Comments:

Continued…

Page 2:

TIME	GPM	Pumping Levels	AMPS	PSI	NOTES
12:00	64	295		>100	**DAY 3** 1/18/18
12:30	60	293	Adjusted Flow Rate 85% or 51hz		
12:45	60	294			
1:00	60	294			
1:15	60	294			
1:30	60	294			
2:00	60	294			
2:30	60	294			
3:00	60	294.25			
3:30	60	294.25			
4:00	60	294.25			
5:00	60	294.25			
11:00	60	294.75			**DAY 4** 1/19/18
12:00	60	294.75			
1:00	60	294.75			
2:00	60	294.75			
3:00	60	294.75			
					RECHARGE LOG
3:00	SHUT DOWN	275			Quickly recharging in first 5 min. (Approx 20'+)
3:05		245			
3:10		244.5			Slowed down after 5-7 min.
3:15		242.75			
3:20		242.5			
3:25		242.25			
3:30		242.25			
3:40		242			
3:50		242			
4:00		242			
4:15		242			
4:30		241.75			
5:00		241.5			
5:30		241.25			
6:00		241			
3:00		237'			1/20/18 Saturday, after 24 hr recovery

Comments:

Continued…

	TIME	GPM	Pumping Levels	AMPS	PSI	NOTES
2/7/18	2:25	Static Water Level: 232'				
	2:30	15	238'			Set Flow rate @ 15GPM at 30hz
	2:45	15	238'			
	3:00	15	238'			
	3:15	15	238'			
	3:30	15	238'			
	4:30	15	238'			
						Changed Flow rate to 30GPM
	4:45	30	238'			
	5:00	30	238.5'			
	5:15	30	238.75'			
	5:30	30	239'			
	5:45	30	239'			
	6:00	30	239'			
	6:30	30	239.2'			
2/8/18	12:30	30	242'			
	2:30	30	242'			
	3:00	30	242'			Shut Down

Comments:

Appendix B
State Water Source (SWS) forms

GROUND WATER ASSESSMENT FORMS

Delineation of Ground Water Protection Zones

Procedures

Three zones are delineated around a well (see specific guidance for springs and horizontal wells), using the Calculated Fixed Radius method. The default shape of these zones is circular and the radius of the zones is based on the Time of Travel (TOT) of water from a point in the aquifer to the well. The three zones are defined as:

Zone A	(2 year TOT)
Zone B5	(5 year TOT)
Zone B10	(10 year TOT)

For porous media aquifers (consisting primarily of rocks, sands, gravels and clays), the radius also considers the pumping rate of the well (Q in gallons per minute), the screened interval of the well (H in feet), and the effective porosity of the aquifer (η - assumed to be 0.2). For fractured rock aquifers, the procedures are the same, but the radius of the zones is increased by 50%.

There are more complicated methods for determining the size, shape and location of zones. Water systems interested in these methods should consult with a hydrogeologist or other knowledgeable professional.

The following table has been developed to assist water systems and regulators in determining the procedures to use in delineating protection zones.

TABLE 1

Aquifer Media	Type of System	Pumping Rate (Q gpm)	Radius Zone A (R_2 feet)	Radius Zone B5 (R_5 feet)	Radius Zone B10 (R_{10} feet)
Porous Media			600’min.	1,000’min.	1,500’min.
	Transient Noncommunity	Any	600’	-----	-----
	Non-Transient Noncommunity	0 to 100 gpm	Calculate or Refer to Table 2		
	Non-Transient Noncommunity	> 100 gpm	Calculate		
	Community	0 to 100 gpm	Calculate or Refer to Table 2		
	Community	> 100 gpm	Calculate		
Fractured Rock			900’min.	1,500’min.	2,250 min.
X	Transient Noncommunity	Any	900’	-----	-----
	Non-Transient Noncommunity	0 to 100 gpm	Calculate or Refer to Table 3		
	Non-Transient Noncommunity	> 100 gpm	Calculate		
	Community	0 to 100 gpm	Calculate or Refer to Table 3		
	Community	> 100 gpm	Contact DHS*		

Appendix C
Laboratory Reports

Analytical
Chemical
Labs, Inc.
EPA Reg. # CA01419 ELAP Cert. # 2505

Client:	US Customs and Border Protection C/O CWE 1561 E. Orangethorpe Ave. Fullerton, CA	**Report Date:**	January 25, 2018
		Received Date:	January 18, 2018 15:30
Attn.:	Bill Young	**Phone:** 714-526-7500	**Fax:** 714-526-7004
Project Name:	Braun Field Border Patrol Station Dulzura, CA	**Project #:**	Item #
Purchase Order #:	Verbal	Regular TAT	

Certificate of Analysis

Lab No: 180030-1	**Sample ID:** # 1		**Matrix:** Aquatic
Sampled By: Allen Xie	**Date:** 01/18/2018	**Time:** 14:30	**Source:** Well pump test

Parameter	Results	Units	DLR	Method	Analyzed
Fluoride F	**5.57**	mg/l	0.05	SM 4500-F C	2018/01/19
Nitrate as N	**2.02**	mg/l	0.02	SM4500-NO3-D	2018/01/19
Nitrite as N	**0.006**	mg/l	0.003	HACH 8507	2018/01/19
Hardness, Total	**126.6**	mg/l	0.01	EPA 200.7	2018/01/19
Dissolved Solids, Total (TDS)	**351**	mg/l	1	SM 2540 E-1997	2018/01/19
Carbonate Alkalinity as $CaCO_3$	**138.6**	mg/l	0.5	SM 2320 B	2018/01/19
Bicarbonate Alkalinity as $Ca(HCO_3)_2$	**169.1**	mg/l	0.5	SM 2320 B	2018/01/19
Hydroxide Alkalinity as OH	**382**	mg/l	0.5	SM 2320 B	2018/01/19
pH	**7.49**	pH	0.01	SM4500-H+R	2018/01/18
Specific Conductance	**701**	µS/cm	0.1	EPA 120.1	2018/01/18
Dissolved Solids, Total (TDS)	**496**	mg/l	1	SM 2540 C	2018/01/22
Calcium (Ca)	**43.37**	mg/l	0.005	EPA 200.7	2018/01/19
Magnesium (Mg)	**14.45**	mg/l	0.001	EPA 200.7	2018/01/19
Sodium (Na)	**58.27**	mg/l	0.005	EPA 200.7	2018/01/19
Iron Fe	**0.2278**	mg/l	0.002	EPA 200.7	2018/01/19
Manganese Mn	**0.1902**	mg/l	0.0001	EPA 200.7	2018/01/19

Parameter	Results	Units	MDL	Method	Analyzed
Coliform Bacteria P/A	Positive (1)	presence	1 / 100 ml	Colisure SM 9223B	2018/01/19; 16:00
E. Coli P/A	**Negative (0)**	presence	1 / 100 ml	Colisure SM 9223B	2018/01/19; 16:00

Approved: **Andrew Moroz, MSChE**
Laboratory Director

ND = Not Detected
DLR = Detection Limit for Reporting
MDL = Method Detection Limit

Any remanding sample(s) for testing will be disposed of two weeks from the final report date unless other arrangements are made in advance.

Analytical Chemical Labs Inc.
8222 Vickers St., Suite 110
San Diego, CA 92111-2118
Phone: (619) 276-1558
FAX: (619) 276-1581
e-mail: acl@chemist.com

CHAIN OF CUSTODY RECORD

ACL Work Order 180030

Page 1 Of 1

CLIENT NAME: US Customs and Border Protection c/o CWE

ADDRESS: 1561 E. Orangethorpe Ave.
Fullerton, CA

PROJECT: Brown Field Border Patrol, Station, Dulzura CA

PHONE #: 714-526-7500 x103
FAX #: 714-526-7004
e-mail: wYoung@CWEcorp.com

PROJECT MANAGER: Bill Young
SAMPLER: Allen Xie
P.O. #

ANALYSIS REQUESTED

LAB I.D. #	DATE SAMPLED	TIME SAMPLED	SAMPLE TYPE ☆	SAMPLE DESCRIPTION	# OF CONT.	Fluoride	Nitrate, Nitrite, NO_3^- F	Bicarbonate Alkalinity, Carbonate Alkalinity	Hydroxide Alkalinity	Calcium, pH, specific conductance, Sodium	Total Hardness, Metals, Iron, Magnesium	Manganese, BacT 〃	REMARKS
DULZURA well	01/18/18	1430	DW	Well pump test	4	✓	X	X	X	X	X	X	

SPECIAL HANDLING

○ 1-2 Hours Rush TAT (plus 250%)
○ 3-4 Hours Rush TAT (plus200%)
○ Same Day Rush TAT (plus 150%)
○ 24 to 36 Hours TAT (plus 100%)
○ 48 - 72 Hours TAT (plus 50%)
● [✓ Regular TAT (5 business Days)]

RELINQUISHED BY:	DATE/TIME	RECEIVED BY:
Allen Xie	JAN 18 2018 15:30	
RELINQUISHED BY:	DATE/TIME	RECEIVED BY:
RELINQUISHED BY:	DATE/TIME	RECEIVED BY:

O = AQ - Aqueous; NA - Nonaqueous; SL - Sludge; DW - Drinking Water; WW - Wastewater; GW - Groundwater; SO - Soil; SW - Solid Waste; AF - Air Filters; WP - Wipes; PT - Paint Chips; OF - Oil or Fuel; OT - Other

SAMPLE CONDITION:

Precooled [✓ Y]/N
Preserved [✓ Y]/N
Evidence seals intact [✓ Y]/N
Preserved at Lab Y/[✓ N]

DISPOSITION OF SAMPLES:

Pick Up by client Y/[✓ N]
Return Y/[✓ N]
Dispose by Lab ** [✓ Y]/N
** Extra charge added

SPECIAL REQUIREMENTS OR INSTRUCTIONS:

D-TEK ANALYTICAL LABORATORIES, INC.
2722 Loker Ave. West, Suite B
Carlsbad, CA 92010
(760) 930-2555 FAX (760) 930-2510

Stehly Brothers Drilling
13268 McNally Rd.
Valley Center, CA 92082

Attn: Doug Hansen

Date Reported: 02/09/18
Date Sampled: 02/08/18
Date Received: 02/08/18
Sample Type: DR-WATER

Project ID: CWE #17251

Log Number: **18-593**
Sample ID: **US Border Dulzura**

ANALYTICAL RESULTS

Analysis	**Results**	**Units**	**Method**	**Analyst/Date**
T. Coliform	< 1	MPN/100ml	SM9223B	CK 02/08/19
E. coli	< 1	MPN/100ml	SM9223B	CK 02/08/19

D-TEK Analytical Laboratories, Inc.
2722 Loker Ave. West, Suite B
Carlsbad, CA 92010
(760) 930-2555, FAX (760) 930-2510

CHAIN OF CUSTODY

COC No. 1842833

PAGE 1 OF 1

CUSTOMER INFORMATION

COMPANY: STEHLY BROTHERS DRILLING
CONTACT PERSON:
PROJECT MANAGER: DOUG HANSEN
ADDRESS: 13268 McNALLY RD.
CITY: VALLEY CENTER STATE: CA ZIP: 92082
PHONE: 760-742-3668 FAX:

PROJECT INFORMATION

PROJECT NAME/NUMBER: CWE # 17251

BILLING INFORMATION

BILL TO:
ADDRESS:
PHONE:
P.O.#:

D-TEK LOG # LAB USE ONLY	SAMPLE # / SAMPLE IDENTIFICATION	SAMPLE DATE	SAMPLE TIME	SAMPLE MATRIX	CONTAINER TYPE	# OF CONTAINERS	ANALYSIS REQUEST: T-COLIFORM (MPN)
18-0543	VS BORDER DULZURA	2/8/18	12:30 PM	H2O	PLASTIC		

SAMPLES COLLECTED BY: CLIENT
EMPLOYER:

California DPH EDT required? Y N
PWS Number:

SAMPLE INTEGRITY

1. RELINQUISHED BY
PRINTED NAME: MATT FOX
SIGNATURE:
DATE AND TIME: 2/8/18 1715

1. RECEIVED BY
PRINTED NAME: CRIS K. KROEGER
SIGNATURE:
DATE AND TIME: 2/8/18 1718

2. RELINQUISHED BY
PRINTED NAME:
SIGNATURE:
DATE AND TIME:

2. RECEIVED BY
PRINTED NAME:
SIGNATURE:
DATE AND TIME:

3. RELINQUISHED BY
PRINTED NAME:
SIGNATURE:
DATE AND TIME:

3. RECEIVED BY
PRINTED NAME:
SIGNATURE:
DATE AND TIME:

SAMPLE RECEIPT
RECEIVED IN ICE [✓ Y] N NA
TAPE SEAL INTACT Y N [✓ NA]
PRESERVATIVE [✓ YES] NO NA
pH VERIFIED: YES NO [✓ NA]
Recorded temp 6°C

SPECIAL INSTRUCTIONS

TAT

-- ALL SAMPLES ARE SUBJECT TO TERMS AND CONDITIONS ON REVERSE SIDE --