SOUTH PLATTE RIVER DIURNAL FLOW STUDY

DIURNAL FLOW MANAGEMENT OPTIONS IN THE SOUTH PLATTE RIVER

Prepared for:

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EXECUTIVE SUMMARY

INTRODUCTION

The Metro Wastewater Reclamation District ("Metro District") owns and operates the Robert W. Hite Treatment Facility ("RWHTF"), which treats wastewater flows from multiple municipalities and water districts in the Denver Metro area. Effluent from the RWHTF is discharged to the South Platte River ("SPR") just upstream of the confluence of Sand Creek and the SPR. Water users on the SPR both downstream and upstream of the RWHTF point of discharge have expressed concerns that the diurnal fluctuation of the SPR as a result of the operation of the RWHTF causes a reduction in the amount of water delivered to various water rights located downstream of the effluent discharge point of the RWHTF and/or creates difficult conditions for allocating diversions to ditch shareholders due to the fluctuating flow rates experienced within the ditch over the course of a day. The diurnal fluctuation of the SPR downstream of the RWHTF effluent discharge point is primarily due to the hourly variations in the plant's effluent discharge that largely mimics the typical fluctuating water use patterns of the customers it serves.

In April of 2014, the Colorado Water Conservation Board ("CWCB") approved a grant to the Metro District to conduct a study of the diurnal flow conditions on the SPR including potential alternatives to mitigate the impact of the diurnal flow. The scope of the study was cooperatively developed with approximately 15 different parties including municipalities and ditch companies with water rights on the SPR and their consultants through a stakeholder process. The State and Division 1 Engineer's Office were also stakeholders in the study process. In the development of the scope of the study, the stakeholders identified several possible alternatives to address the diurnal flow fluctuation of the SPR, including:

A. Use of upstream storage at Chatfield Reservoir for the purpose of timing releases of stored water to attenuate (i.e. reduce or damped the magnitude of the peaks and troughs) and/or offset the diurnal fluctuation.

B. Use of storage at an existing gravel pit reservoir located in the stretch of the SPR between RWHTF and the headgate of the Western Mutual Ditch (a.k.a. Hewes & Cook Ditch), hereinafter "Western Ditch" or "Western," for the purpose of timing releases of stored water to attenuate and/or offset the diurnal fluctuation.

C. Construction of a new gravel pit storage reservoir downstream of RWHTF and upstream of the Western Ditch headgate for the purpose of timing releases of stored water in order to attenuate and/or offset the diurnal fluctuation.

D. Use of storage at agreed upon locations, including agreed upon timed releases by parties using effluent discharged at RWHTF, for the purpose of timing releases to attenuate and/or offset the diurnal fluctuation.

E. Construction of a storage reservoir near the headgate of the Western Ditch at the Gilcrest Reservoir site or other locations in order to attenuate the impact of the diurnal fluctuation to the Western Ditch, which could benefit other water users that have been historically subject to calls by the Western Ditch.

F. Use of existing or new river check dams (a.k.a. diversion dams) that could be modified in order to regulate the diurnal fluctuation.

G. Use of groundwater diversions for ditches to offset the diurnal fluctuation.

D&A conducted a screening analysis to evaluate the preliminary engineering feasibility of each of the above proposed mitigation alternatives. In addition to the analysis of engineering feasibility, this study identifies potential legal, institutional, and permitting issues associated with each alternative. Additional engineering, legal, and permitting research will be required to determine if any of the alternatives presented herein will be suitable for implementation. D&A's findings contained within this study should be construed only as D&A's objective analysis of each presented option and such findings should not impose any limitation on any of the presented options or any subsequent independent engineering, legal and permitting research related to the feasibility of each such option.

DIURNAL FLUCTUATION

Typical diurnal variations in influent flow experienced by municipal wastewater treatment plants are generally characterized by two peaks resulting from morning and early evening water usage and decreasing flows late at night and early in the morning. The maximum flow of the diurnal flow period is defined as the "peak hourly flow" (U.S. EPA, 1981). Variations in the waveform of the diurnal fluctuation (e.g., amplitude or relationship of peak hourly flow to average) can be seen on a daily basis, with the largest variations occurring on weekends and holidays.

The diurnal fluctuation of the influent to the RWHTF is typical of a wastewater treatment facility with both industrial and nonindustrial (i.e., municipal) contributions. The RWHTF's effluent fluctuation is very similar to the influent fluctuation but slightly delayed due to the treatment processes. The peak effluent discharges from RWHTF occur at approximately 1:00 p.m. and 11:00 p.m., while the low flow discharge generally occurs between the hours of 6:00 a.m. and 7:00 a.m. Analysis of the average hourly effluent discharge at the RWHTF for the years 2001 through 2014 indicates that the average daily discharge from this facility was 200 cfs. However, the discharge from the RWHTF varied on average from a low of 99 cfs at 6:00 a.m. up to a high of 259 cfs at 1:00 p.m.

CURRENT RIVER ADMINISTRATION

The historical dry-up locations within District 2 generally occur below the Burlington Ditch, below the Farmers Independent Ditch, below the Jay Thomas Ditch, and at the Lower Latham Ditch. Recent changes in the operation of the Jay Thomas Ditch water rights (i.e., 100 percent owned by PSCo and used at PSCO's Fort St. Vrain and Cherokee stations) have caused the dry-up point that historically occurred below the Jay Thomas Ditch to now occur below the Western Ditch headgate.

Historical administration affecting District 2 primarily consisted of bypass calls placed by the Jay Thomas Ditch; however, subsequent to PSCo's change of use of the Jay Thomas Ditch rights in 2006, it now primarily consists of bypass calls placed by the Western Ditch.

A bypass call is the partial curtailment of a junior upstream right expressed as a call by the junior right bypassing to a named downstream senior right. For example, an 1885 Burlington direct flow bypass call to the 1871 Western Ditch water right will result in the total curtailment of rights upstream of the Western Ditch junior to 1885, and the Burlington bypassing or curtailing a portion of the 1885 water at its headgate as necessary to satisfy the downstream 1871 right at the Western Ditch. Subsequent to PSCO's change of use of the Jay Thomas Ditch rights in 2006, the call records indicate the water rights most frequently subject to bypass calls from the Western Ditch are the Burlington Ditch's 1885 priority or the Evans No. 2 Ditch 1871 priority. The same call records show an infrequent bypass call of short duration that affects the more junior rights diverted at the Burlington Ditch (i.e., 1908 and 1909), Evans No. 2 Ditch (i.e. 1909 priorities), Fulton Ditch, Brantner Ditch and the Brighton Ditch placed by the Western Ditch necessary to satisfy its August 10, 1871 priority. These bypass calls primarily occur during the months of July, August, and September.

The 1871 priority of the Western Ditch is the primary calling right in this reach of District 2, making the Western Ditch the "swing ditch" (i.e., the ditch that dictates the presence of a call). The Water Commissioner determines the need for a call on the SPR in District 2, upstream of the Saint Vrain Creek confluence, by: 1) discussing the daily water needs of the Western Ditch with a ditch company representative, 2) examining the low flow "trough" of the daily hydrograph at the Henderson gage, 3) examining gaged and known inflows within the reach upstream of the Western Ditch to determine their potential contribution to demand, 4) estimating unmeasured inflows (ungaged surface inflows and groundwater gains) based on weather conditions and experience, and finally, 5) the initial distribution of the water to all in-priority water users, according to their communicated demands, so that the Western's 1871 priority and all intervening water rights are satisfied when the trough of the diurnal flow reaches the Western headgate. If the Water Commissioner determines the Western's demand will not be completely satisfied, the Water Commissioner will place a bypass call within District 2. The bypass call allows the Water Commissioner to work with upstream junior water users so that only a partial curtailment may be required to satisfy the Western Ditch's demand. The Water Commissioner's goal when administering the typical bypass call is to fully satisfy the Western's calling right while avoiding any "spills" (i.e., flow over Western's check dam) to the SPR downstream of the Western Ditch.

IMPACT OF DIURNAL FLUCTUATION

The largest impacts as a result of the diurnal fluctuation are to those structures most frequently subject to a bypass call (i.e., the Burlington Ditch and Evans No. 2 Ditch). Based on current District 2 administrative practices, the ditch system subject to the bypass call is allowed to "chase the peak" of the diurnal fluctuation. This means that the Water Commissioner allows the bypassing structure to gradually increase its diversion rates during the period in which the trough of the diurnal fluctuation has passed the bypassing structure and the rising limb of the diurnal fluctuation allows for increased diversions.

The impact of chasing the peak of the diurnal fluctuation and the resulting variable diversion rates is felt by the ditch company, individual shareholders, and irrigators of the bypassing ditch. That is, the variable flow rates within the ditch over the course of the day create variable water stages and hydraulic heads that make it difficult for the ditch company to allocate and regulate water deliveries, for irrigators to adjust their farm headgates, and also for the setting of siphon tubes. For the purpose of this study, we are calling these impacts to the ditch downstream of the main headgate "down-ditch impacts." D&A understands that not only does the "diurnal fluctuation" within the ditch and the resulting down-ditch impacts require significant effort and resource expenditures by the ditch companies and their shareholders, it reduces the overall efficiency and utilization of the water by the ditch systems.

In addition to affecting the bypassing structures, the diurnal fluctuation can also impact the senior ditch placing the call. The impacts to the senior calling ditch are a result of being shorted water at times over the course of the day. Depending on how accurate the estimates of inflow, outflow, demands, etc. are when the Water Commissioner makes the initial morning allocation of supply, it is possible the calling right may be shorted if actual inflows to the SPR are less than estimated inflows.

ANALYSIS OF MITIGATION ALTERNATICES AND CONCLUSIONS

D&A examined a series of stakeholder developed and proposed mitigation alternatives. A preliminary engineering feasibility analysis was conducted for each of the alternatives as well as the identification of potential legal, institutional, and permitting issues. In addition to the physical mitigation alternatives (e.g., equalization ponds), D&A researched improvements to streamflow gaging and infrastructure that could help with the water supply allocation and administration of the bypass call. Based on the results of the various analyses that were conducted, the following conclusions have been developed:

Upstream Storage at Chatfield Reservoir

The use of Chatfield Reservoir storage and releases as a diurnal flow mitigation measure has some advantages, but also some potentially fatal flaws. This alternative has the advantage that the storage structure is already in place. In this alternative, the daily volume released from Chatfield Reservoir would be the same as what would occur historically, but the release rates would be changed throughout the day to help fill in the trough of the diurnal fluctuation. Based on an analysis of historical water supply, the alternative may not be a complete solution due to limited water supply in dry years and certain months. Due to the 22 mile distance between Chatfield Reservoir and the RWHTF outfall, the determination of timed releases and the effects of intervening river operations complicate this alternative. The changes to the State's and Corps' operation of the Chatfield Reservoir outlet works necessary to make this alternative work involve both legal, institutional, and environmental issues that will likely be difficult to overcome. Additional research beyond the scope of this study is required to determine if such changes in the operation of Chatfield Reservoir would be feasible.

Use of Existing Gravel Pit for Flow Equalization Prior to RWHTF Discharge

Based on the institutional and legal opinions provided by the Metro District, the alternative of using an existing gravel pit for effluent equalization prior to discharge of the effluent from RWHTF to the SPR has potentially insurmountable legal and regulatory flaws that should effectively remove it from further consideration.

Use of Existing Gravel Pit between RWHTF and Western Ditch Headgate

The use of an existing gravel pit downstream of the RWHTF outfall to divert and regulate the diurnal fluctuation is a potentially feasible solution. There are sizable capital costs (e.g., $8.3 million) associated with this alternative related to the potential reimbursement of storage costs, construction of a pump station, and possible improvements to inflow infrastructure (e.g., adding constructed capacity to an existing ditch). In addition to capital costs, the estimated annual energy costs to operate a pump station to deliver temporarily stored water from the gravel pit to the SPR would be approximately $42,000 to $45,000.

Potential legal issues associated with the use of a gravel pit to divert the peak of the diurnal fluctuation off of the SPR for temporary storage and retiming include the need for a water court decree and likely the cooperation of the District 2 water users that would have a portion of their water rights temporarily detained by the upstream equalization pond.

Construction of a New Gravel Pit

The construction and use of a new gravel pit for the purpose of an equalization pond would have similar costs and legal issues associated with that of an existing gravel pit.

Use of Storage and Timed Releases by Parties using RWHTF Effluent

This alternative considers the use of storage and timed releases by parties with gravel pit storage facilities who store and release augmentation and substitute supply water within the Study Reach to help offset the trough of the diurnal fluctuation. This alternative would require participating water users and the State to develop an operating procedure in which the augmenters would agree to make strategically timed releases during the trough of the SPR diurnal fluctuation. This alternative may be feasible but its effectiveness may be limited, at least in the short term due to the limited number of parties that store and release augmentation water and substitute supplies in the Study Reach. If future operations increase the amounts and consistency of augmentation and releases within this reach via gravel pits, this alternative could be an effective mitigation alternative.

Storage Near Ditch Headgates

Constructing and utilizing an equalization pond downstream of ditch companies' river headgates are a proven method to better regulate deliveries to downstream shareholders as evidenced by the Western Mutual Equalization Pond. While these equalization ponds are effective at regulating flows to shareholders, they do not mitigate the fluctuations experienced at the river headgates or reduce the potential of being shorted by upstream water users if over-diversion occurs during a bypass call.

Use of Existing or New River Check Dams

This alternative mitigation proposal would utilize existing river check dams that are modified to regulate the diurnal fluctuation, or constructing a new river check dam. A rise of 6 to 7 feet above normal water surface elevations at the check dams would be necessary to create storage behind the check dams sufficient to regulate the diurnal fluctuation. This alternative presents the potential for significant flooding, riverbank destabilization, and adjacent land issues as a result of a rise and drop of the SPR river stage of this magnitude. Because of these issues, D&A did not fully investigate the costs associated with this alternative.

Utilization of Groundwater Diversions to Offset Diurnal Fluctuation

This alternative considers the use of groundwater diversions (i.e., wells) to offset the diurnal fluctuation experienced by individual ditch systems. In our opinion, the legal issues surrounding the use of headgate wells would likely require the well users to obtain a water court decreed plan for augmentation. Furthermore, based on the magnitude of the troughs experienced by the majority of the ditch companies, this alternative would likely only be feasible for a few ditch systems that have smaller fluctuations. Given the quantity and expense of the wells required to supplement the troughs of the ditch systems and the requirement, per current water law, of an augmentation plan, this alternative doesn't appear as feasible as others presented within this report.

Improved Measurement and Reporting

Based on information gathered from discussions with the Division Engineer and District 2 Water Commissioner, there could be some improvements made to the measurement and reporting of flows in the SPR and its tributaries that would assist in the administration of the diversions in the Study Reach, especially during times of low flow when the diurnal fluctuation is problematic. This would be accomplished with the enhancement of existing streamflow gages and the construction of gaging on currently ungaged tributary inflows. The improvements to existing streamflow gages may include the more frequent calibration and rating of the two USGS gages within this reach (i.e., 64th Avenue and Fort Lupton gages) so that they are more reliable in low flow conditions.

In addition to improving existing gages, adding gaging instrumentation and infrastructure to currently ungaged tributaries such as Little Dry Creek and the Graflin Slough would provide the Water Commissioner more definitive information as to the amount of water supply available for allocation. While this alternative will do nothing to mitigate the physical diurnal fluctuation, the improvements represent fairly low cost options to help reduce the impacts created by the diurnal fluctuation. However, these improvements alone will not be sufficient to mitigate all the negative impacts of the diurnal fluctuation on water diverters within the Study Reach.

The total capital cost of the recommended gage improvements ranges from $40,000 to $90,000. This cost includes constructing a streamflow gage on Little Dry Creek and the Graflin Slough. The total annual operations and maintenance costs associated with these gages is approximately $8,000 to $12,000. The additional cost associated with funding the bi-weekly calibration of the USGS gages at 64th Avenue and Fort Lupton, CO would be approximately $8,000 per year.

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1.0 INTRODUCTION

On April 20, 2011, the City of Aurora, acting by and through its utility enterprise ("Aurora Water" or "Aurora"), the City of Thornton, and the Metro Wastewater Reclamation District ("Metro District") collectively referred to as "Co-Applicants" filed an application with the District Court, Water Division No. 1, in Case No. 11CW74. The Co-Applicants applied for a change of water rights to: 1) obtain approval of the relocation of the treatment and discharge of a portion of Thornton's effluent that is generated in Thornton and currently treated at the Metro District's Robert W. Hite Treatment Facility ("RWHTF") to Metro District's proposed new Northern Treatment Plant ("NTP"), and 2) to obtain approval of a trade of effluent between Thornton and Aurora, referred to as the "Effluent Trade". During the course of negotiations to settle Case No. 11CW74, several Opposers to the application expressed concerns that the diurnal fluctuation of the South Platte River ("SPR") and the corresponding method of administration of water rights in Water District No. 2, caused a reduction in the amount of water delivered to various water rights located downstream of the effluent discharge point of the RWHTF and/or created difficult conditions for allocating diversions to ditch shareholders due to the fluctuating flow rates over the course of a day. The diurnal fluctuation of the SPR downstream of the RWHTF effluent discharge point is primarily due to the hourly variations in the plant's effluent discharge that largely mimics the typical fluctuating water use patterns of the customers it serves.

A stipulated settlement in Case No. 11CW74 was eventually achieved, which led to a Final Decree of the Court. As part of the stipulation between Co-Applicants and various Opposers, the Co-Applicants agreed to fund and oversee a study of the diurnal flows discharged from the RWHTF, including the impacts of such discharged flows and the potential benefits of attenuating (i.e., reducing or dampening the magnitude of the peaks and troughs) of the flows on the SPR downstream of the RWHTF discharge. It was anticipated that funding of the study could be obtained from a grant by the Colorado Water Conservation Board (CWCB). The stipulation between the Opposers and the Co-Applicants specified that the study and the grant application would include:

- Defining the diurnal flow issues
- Identifying the water users likely affected by the diurnal flows and to what degree
- Identifying the potential benefits of mitigating or attenuating the diurnal flows
- Identifying potential administrative or physical actions, including a flow equalization pond that could provide those benefits
- Identifying the costs of providing potential administrative and physical benefits

In April of 2014, the Metro District received notice that the Water Supply Reserve Account (WSRA) grant application submitted to the CWCB for funding of the diurnal flow study was approved. The scope of the study was cooperatively developed with Opposers and their consultants through a stakeholder process. All Opposers in Case No. 11CW74, including the State and Division Engineers, are considered stakeholders in the study process. In the development of the scope of the study, the stakeholders identified several possible alternatives to address the diurnal flow fluctuation of the SPR. These alternatives included:

A. Use of upstream storage at Chatfield Reservoir for the purpose of timing releases of stored water to attenuate and/or offset the diurnal fluctuation.

B. Use of storage at an existing gravel pit reservoir located in the stretch of the SPR between RWHTF and the headgate of the Western Mutual Ditch (a.k.a. Hewes & Cook Ditch), hereinafter "Western Ditch" or "Western," for the purpose of timing releases of stored water to attenuate and/or offset the diurnal fluctuation.

C. Construction of a new gravel pit storage reservoir downstream of RWHTF and upstream of the Western Ditch headgate for the purpose of timing releases of stored water in order to attenuate and/or offset the diurnal fluctuation.

D. Use of storage at agreed upon locations, including agreed upon timed releases by parties using effluent discharged at RWHTF, for the purpose of timing releases to attenuate and/or offset the diurnal fluctuation.

E. Construction of a storage reservoir near the headgate of the Western Ditch at the Gilcrest Reservoir site in order to dampen the impact of the diurnal fluctuation to the Western Ditch, which could benefit other water users that have been historically subject to calls by the Western Ditch. In this study, the investigation of a storage location between the RWHTF and the Western Ditch was not limited to the Gilcrest Reservoir site. D&A also investigated storage locations near other ditch headgates on the SPR between RWHTF and the Western Ditch.

F. Use of existing or new river check dams (a.k.a. diversion dams) that could be modified in order to regulate the diurnal fluctuation.

G. Use of groundwater diversions for ditches to offset the diurnal fluctuation.

D&A conducted a screening analysis to evaluate the preliminary engineering feasibility of each of the above proposed mitigation alternatives. In addition to the analysis of engineering feasibility, this study identifies potential legal, institutional, and permitting issues associated with each alternative. Additional engineering, legal, and permitting research will be required to determine if any of the alternatives presented herein will be suitable for implementation. D&A's findings contained within this study should be construed only as D&A's objective analysis of each presented management option and such findings should not impose any limitation on any of the presented options or any subsequent independent engineering, legal and permitting research related to the feasibility of any such option.

2.0 STUDY REACH

The reach of the SPR that experiences the diurnal fluctuation created by the RWHTF effluent discharge is contained within Water District No. 2 ("District 2") of Water Division No. 1. The mainstem of District 2 is comprised of a 70 mile reach beginning upstream at the Denver gage (PLADENCO) and ending immediately upstream of the Kersey gage (PLAKERCO). Major surface inflows within District 2 include: Sand Creek, Clear Creek, Big Dry Creek, St. Vrain Creek, Big Thompson River, and the Cache La Poudre River. Because the inflows from St. Vrain Creek, the Big Thompson River, and the Cache La Poudre River are significant, the reach of District 2 most affected by the diurnal fluctuation is specifically the reach at and below the headgate of the Burlington Canal and above the confluence of St. Vrain Creek and the SPR. Based on discussions with the Division Engineer, the Water Commissioner for District 2, other stakeholders and our own review of streamflow records, there is very little diurnal fluctuation downstream of the confluence of St. Vrain Creek and the SPR. Therefore, the reach of the SPR that is subject to this study (hereinafter the "Study Reach") is comprised of the mainstem of the SPR beginning at the Burlington Canal headgate and terminating downstream at the Jay Thomas Ditch headgate (see **Figure 1**).

The SPR flow at the upstream end of the Study Reach is largely dependent on discharge from the RWHTF. For a majority of the year, the Study Reach can be considered an effluent-dominated reach, i.e., the flow within the Study Reach is largely comprised of treated effluent from the RWHTF and other municipal wastewater treatment plants located within the Study Reach (e.g., South Adams County Water and Sanitation District's Williams-Monaco Treatment Plant, City of Brighton, Fort Lupton, Platteville, and the new Metro District Northern Treatment Plant set to become operational in 2016). Each of these facilities contributes its own inherent diurnal fluctuation. By examining the RWHTF discharge, the South Platte River at Henderson gage records, and the Fulton and Brantner Ditch diversions for the period of June 2010 through December 2014, D&A estimated the percentage of the SPR flows upstream of the Fulton Ditch headgate that were comprised of effluent from the RWHTF. On a daily basis during this period, the average percentage of the SPR flow comprised of RWHTF treated effluent ranged from a maximum of 87 percent in the month of January to a minimum of 31 percent in the runoff influenced month of June. Individual monthly averages ranged from 99 percent in January 2012, to a low of 16 percent in June 2014 (See **Table 1**). On average, RWHTF effluent comprised 55 percent of the daily flow in the SPR upstream of the Fulton Ditch headgate.

The Sand Creek and Clear Creek confluences with the SPR are located approximately 700 feet and 1.2 miles downstream of the RWHTF outfall, respectively. Surface diversions within the Study Reach are primarily made by the Burlington, Fulton, Brantner, Brighton, Lupton Bottom, Platteville, Meadow Island No.1, Evans No.2, Meadow Island No.2/ Beeman, Farmers Independent, Western, and the Jay Thomas ditches.

The Colorado Division of Water Resources' South Platte River at Henderson streamflow gage (PLAHENCO or "Henderson gage") is located approximately 10.6 miles downstream of the RWHTF outfall and provides a near continuous daily streamflow record dating back to May of 1926. The Henderson gage, as will be discussed in more detail later in this report, provides a useful record of the influence of the RWHTF discharge on flows within the Study Reach.

The adjacent lands on either side of the river within this reach have been heavily mined for sand and gravel. The vast majority of the reclaimed pits have been lined and acquired by local municipalities and water providers for recapture and storage of reusable effluent supplies discharged at the RWHTF, changed water rights, and junior storage rights. The City of Thornton owns a large percentage of the reclaimed pits primarily made up of multi-cell complexes such as their East Gravel Lakes (aka, Tani Lakes Storage Complex), West Gravel Lakes, Cooley East, Hammer facilities, and Rogers Reservoir. Other owners of gravel pit storage in this reach include: South Adams County Water and Sanitation District (SACWSD), Denver Water Board, Aurora Water, and the City of Brighton (see **Figure 2**).

3.0 METRO WASTEWATER RECLAMATION DISTRICT BACKGROUND

3.1 Robert W. Hite Treatment Facility

The Metro Wastewater Reclamation District is the wastewater treatment authority for much of metropolitan Denver and its surrounding suburbs. The Metro District, originally the Metropolitan Denver Sewage Disposal District No.1, is a stand-alone special district formed by the Colorado legislature in 1961. Prior to 1966, wastewater from the Denver Metro area was treated at the Northside Wastewater Treatment Plant ("Northside Plant") located upstream of the Burlington Ditch headgate. The Northside Plant became inadequate due to population growth and higher health standards and therefore construction of the Metro District's first facility, RWHTF, began in 1964 and was completed in 1966. The Northside Plant continued to operate and provide primary treatment for Denver and many small sanitation districts. In 1987, the Northside Plant was decommissioned.

The Metro District currently utilizes only one wastewater treatment facility - the RWHTF, located at 6450 York Street, Denver, Colorado. The Metro District's service area is approximately 715 square miles and serves a population of approximately 1.7 million people. The Metro District customers include 60 local governments consisting of cities and sanitation districts. Large contributors of wastewater to the RWHTF include the cities of Thornton, Aurora, Denver, Lakewood, Westminster, and Arvada.

The RWHTF discharges treated effluent to the SPR approximately 700 feet upstream of the confluence of Sand Creek with the SPR and approximately 2 miles downstream of the Burlington Ditch headgate. The legal description of the RWHTF outfall is as follows: in the Northwest One-Quarter of Section 12, Township 3 South, Range 68 West, 6th P.M., Adams County, Colorado.

According to the Metro District, the RWHTF currently collects an average of approximately 140 million gallons (430 acre-feet) of wastewater per day. Effluent records examined for the years 2010 through 2014 indicate the average daily effluent discharge to the SPR for the same time period is 133 mgd, or 205 cfs (**Table 2**). Annual volumetric discharges for 2010 through 2014 averaged approximately 147,000 acre-feet (**Table 2**).

3.1.1 Projected Effluent Amounts at RWHTF

The Metro District currently estimates that projected flows at the RWHTF following the construction of the new NTP located near Brighton, CO, will not be materially different than the effluent discharge amounts experienced today. Additional population growth within the RWHTF service is expected to replace the volume of Thornton's wastewater currently treated at RWHTF that will be moved to the NTP following its opening in 2016.

3.2 The Northern Treatment Plant

The Metro District began construction of the NTP in late 2012. It is located within the western half of Section 31, Township 1 North, Range 66 West, of the 6th P.M. The outfall of the NTP to the SPR is located near the intersection of Weld County Road 2 (168th Avenue) and State Highway 85.

The outfall is located approximately 6.8 miles downstream of the Henderson gage or 2.75 miles downstream of the Brighton Ditch headgate.

The completion of the construction activities and the commissioning of the NTP are estimated to occur in 2016. The NTP is scheduled to be built in phases with an initial capacity (average daily flow) of 28.8 mgd, expandable to 60 mgd at build-out. Upon startup in 2016, the new facility will be capable of serving approximately 300,000 people. Ultimately, the facility could be expanded to be capable of serving approximately 750,000 people.

Previous studies conducted by D&A in support of Case No. 11CW74 concluded that the introduction of the NTP's effluent discharge, including flows currently treated at RWHTF, to the SPR at the NTP outfall will not cause the low flow trough of the existing diurnal fluctuation within the SPR to decrease and therefore will not trigger increased calls from water users downstream of the NTP outfall (D&A, 2012).

4.0 PREVIOUS STUDIES

As far back as 1997, the Metro District has evaluated the concept of flow equalization for treated wastewater discharging to the SPR. The concept was not considered for the purpose of mitigating the impacts of the diurnal flow fluctuation on downstream water users, but rather to determine the potential effects that the daily flow variations had on aquatic life downstream of the RWHTF discharge. The Metro District provided D&A with two documents that addressed the flow equalization concept related to the Metro District's studies conducted in 2004 pursuant to the1997 Memorandum of Understanding (MOU) among the District, the Water Quality Control Division, the EPA, and the Colorado Division of Wildlife, now Colorado Parks and Wildlife. As part of the 1997 MOU, the District agreed to implement projects to provide a margin of safety for the dissolved oxygen standard adopted by the Water Quality Control Commission. The proposed projects included the construction of instream re-aeration structures, flow equalization facilities to stabilize the depth of flow and extent of the wetted river channel, and fish exclusion facilities to reduce fish loss (i.e., entrainment) in irrigation ditches. The following are brief summaries of the two documents related to the District's 2004 evaluation of flow equalization.

4.1 William M. Lewis, Jr. March 24, 2004 Letter

On March 24, 2004, William M. Lewis Jr., Ph.D., a professor and director of the Center of Limnology at the University of Colorado at Boulder, wrote a letter to Ms. Barbara Biggs, then the Governmental Affairs Officer for Metro, summarizing his opinion regarding the effectiveness of flow equalization of the RWHTF discharge on the environmental health of the SPR below the RWHTF discharge, as compared with other kinds of restoration activities requiring similar investment. Dr. Lewis's involvement was related to the previously mentioned MOU with the regulatory authorities. Dr. Lewis evaluated the effectiveness of the two identified advantages flow equalization may have on the environmental health of the reach of the SPR below the RWHTF discharge: 1) raising the mean and minimum dissolved oxygen concentrations below the discharge, and 2) increasing the abundance of aquatic life by removal of the oscillating water levels within the river.

Dr. Lewis's letter ultimately concluded that the oscillating flow (i.e., the diurnal flow fluctuation) in the SPR "has a very small effect on oxygen concentrations, and that oxygen concentrations would be essentially as they are now if flow were equalized". Dr. Lewis also offered his opinion regarding the effect that the alternating wetting and drying of the SPR banks has on the aquatic life. Dr. Lewis concluded: "*Given that the oscillations induced by Metro cause alternating wetting and drying of a relatively small portion of sandy sediment, and do not inundate or cut off true backwaters, I cannot find a good rationale for giving flow equalization a high priority*." Dr. Lewis also believed that the oscillations in flow within this reach of the river "*induced by other types of water management and by thunderstorms are quite substantial and are inevitably part of the environmental picture*". Dr. Lewis closed the letter by stating "*I do not believe "that flow equalization is an effective strategy for accomplishing [environmental] restoration*".

4.2 Flow Equalization Summary Report

The Metro District provided D&A with a "Summary Report" titled: "Flow Equalization" dated July 12, 2004. The Summary Report documents the study and pilot demonstration level evaluations of flow equalization facilities at the RWHTF conducted by the Metro District pursuant to the 1997 MOU. As previously discussed, the flow equalization studies conducted pursuant to the MOU focused on the diurnal flow's effects on aquatic life and did not analyze its effects on the abilities of downstream water users to divert their water rights.

The 1997 MOU assumed that an 18 million gallon (mg), 55 acre-feet, facility would be required to provide flow equalization to meet the criteria of the MOU (i.e., achieve a daily variance of less than 10 percent of the average flow under normal discharge conditions). The MOU established the 10 percent variance goal for times when the RWHTF's discharge represents a large component of the SPR flows. Under high flows, flow equalization would not be needed as the RWHTF discharge has a lesser impact on stream depths under this condition. The MOU defines high SPR flows as those exceeding 1,000 cfs at the Henderson gage.

The Metro District's analysis concluded there would be numerous operational complexities associated with flow equalization, especially when trying to comply with the MOU variance criteria. The Summary Report concluded that "*[t]he diurnal flow patterns also include a rapid transition from deficit flow conditions to storage conditions*," and that on many days, "*this transition takes less than one hour*." The Summary Report also found that in addition to the rapid transitions, it would be hard to predict the actual daily flow variability and therefore the actual daily storage requirement "*will not be known until after the entire daily storage and discharge cycle has been completed*." The Metro District found that in order to comply with the MOU variance standard 95 percent of the time, a storage volume in excess of 25 mg (77 acre-feet) was needed. This represented an approximate 35 percent increase in storage volume than originally conceived in 1997. At the time the study was completed, the Metro District estimated that an off-site 25 mg equalization facility would have a capital cost of approximately $19.5 million.

In addition to the operational complexities and capital costs associated with the flow equalization, the Metro District concluded that the facility's ability to reduce ammonia levels was costly and unreliable. Based on these findings, the Metro District concluded that "*flow equalization is less viable than originally considered and has fewer treatment benefits than originally thought*." The major findings of the Summary Report included:

- The storage volume required to achieve flow equalization would be in excess of 77 acre-feet, which is approximately 35 percent greater than what was anticipated in 1997 (55 acre-feet) when the concept was first considered. The larger volume results from more detailed evaluations of flow patterns and operational strategies that would be necessary to meet the requirements of the MOU.

- The site area required for the flow equalization facility would be much larger than was originally expected and would require the use of land already planned for future tertiary treatment facilities.

- Because flow equalization would be operationally complex, uncertainty would exist regarding the ability to equalize flows to meet the criteria set by the MOU.

- Little practical reduction in treatment process sizing at the RWHTF would be realized with flow equalization prior to secondary treatment.

- The ability to achieve additional treatment, e.g., ammonia removal, in the significantly enlarged equalization facility would be costly and unreliable.

Based on the Metro District's studies, the construction of flow equalization or fish entrainment mitigation facilities would not provide the environmental benefits originally anticipated. The Metro District concluded that both would be significantly more expensive to construct and operate than originally anticipated, and the flow equalization facility would be extremely difficult to operate in a manner that would achieve the purpose of its construction. Given these conclusions, the MOU was amended in 2004 (referred to as the "2004 Amendment"). In the 2004 Amendment, the District agreed to complete a comprehensive assessment to develop new recommendations on the best ways to protect and improve aquatic habitat in the SPR downstream of the RWHTF outfall.

5.0 DIURNAL FLOW FLUCTUATION

Typical diurnal variations in influent flow experienced by municipal wastewater treatment plants are generally characterized by two peaks resulting from morning and early evening water usage and decreasing flows late at night and early in the morning. The maximum flow of the diurnal flow period is defined as the "peak hourly flow" (U.S. EPA, 1981). Variations in the waveform of the diurnal fluctuation (e.g., amplitude or relationship of peak hourly flow to average) can be seen on a daily basis, with the largest variations occurring on weekends and holidays.

5.1 RWHTF Diurnal Fluctuation

The influent diurnal fluctuation of the RWHTF is typical of a wastewater treatment facility with both industrial and nonindustrial (i.e., municipal) contributions. The RWHTF's effluent fluctuation is very similar to the influent fluctuation but slightly delayed due to the treatment processes. **Figure 3** illustrates the relationship between RWHTF's average hourly influent and effluent for the 2011 water year. By comparing the two lines, the troughs of the influent and effluent are very similar in magnitude, with an approximate two-hour delay between the trough entering the plant and exiting the plant. The peak of the effluent fluctuation is slightly reduced from the peak of the influent fluctuation, and again delayed by approximately two hours.

The peak effluent discharges from RWHTF occur at approximately 1:00 p.m. and 11:00 p.m., while the low flow discharge generally occurs between the hours of 6:00 a.m. and 7:00 a.m. Analysis of the average hourly effluent discharge at the RWHTF for the years 2001 through 2014 indicates that the average daily discharge from this facility was 200 cfs. However, the discharge from the RWHTF varied on average from a low of 99 cfs at 6:00 a.m. up to a high of 259 cfs at 1:00 p.m. (see **Figure 4**).

5.1.1 Average Hourly Effluent Discharge

Metro's records of effluent discharge for the years of 2001 through 2014 indicate that the average hourly effluent discharge was 200 cfs (see **Figure 4**).

5.1.2 Average Daily Peak Hourly Flow

The average daily peak hourly flow was fairly consistent throughout the year, with a slight increase in the months of April through August (see **Table 3**). The month of May appears to have the highest average daily peak hourly flow of 280 cfs, whereas January had the lowest average daily peak hourly flow of 257 cfs.

5.1.3 Average Daily Minimum Hourly Flow

The average daily minimum hourly flow (i.e., lowest hourly discharge over the course of a day) followed a similar pattern of that of the peak flow (i.e., higher minimum flows within the spring and summer months) (see **Table 4**). For the period of 2001 through 2014, the month of March had the lowest average minimum hourly flow of 83 cfs.

The difference between the peak hourly flow and the minimum hourly flow over the course of 24-hours represents the "peak-to-trough" amplitude of the daily diurnal flow waveform. At times when the RWHTF discharge dominates the flow of the SPR downstream of the outfall, the peak-to-trough amplitude, or the difference between the peak hourly flow and the minimum hourly flow, represents the approximate magnitude of the varying flows downstream irrigators may experience at their river headgates when diverting from the SPR. As shown in **Table 5**, the average daily peak-to-trough amplitude for the period of 2001 through 2014 ranges from 181 cfs in April to 167 cfs in July.

The peaking factor of a plant's influent, a metric often used in the design of wastewater treatment facilities, represents the relationship of the peak hourly influent flow compared to the average daily flow. Rather than calculating the peaking factor for influent, D&A calculated the peaking factor of the RWHTF's *effluent* discharge for the period of 2001 through 2014. In the context of this study, the peaking factor of the effluent discharge is useful for the conceptual sizing and design of flow equalization facilities, especially when projecting maximum potential inflow rates based on future effluent discharges. As shown in **Table 6**, the daily peaking factor of the RWHTF's discharge is very stable throughout the year and averages approximately 1.34. In other words, the peak hourly flow is, on average, 134 percent of the average hourly flow.

The "trough-to-average" factor, or the relationship of the minimum hourly flow compared to the average daily flow was also determined. This factor is useful when projecting the expected magnitude of the minimum hourly flow, or trough, based on projected average effluent discharges. For the period of 2001 through 2014, the trough-to-average factor was calculated as 0.47, or the minimum hourly flow was 47 percent of the average daily flow. As shown in **Table 7**, this factor remains mostly constant through the course of a year, with a slight increase in trough-to-average factor in the months of May, June, July, and August.

5.1.4 Volume of Fluctuation Above and Below the Average Daily Effluent Discharge

The volume of water above the average daily effluent discharge represents the volume that an equalization basin would be required to temporarily detain in order to maintain a constant discharge rate equal to the average daily flow rate. The average daily volume above the average hourly effluent discharge for the years of 2001 through 2014 is approximately 46 acre-feet (see **Figure 5**). However, while this is the average volume, there are days that the effluent discharge pattern varies from the average. Therefore, D&A calculated the volume of discharge in excess of the average daily flow rate for the years of 2001 through 2014 and performed an exceedance probability on the daily volumes. As shown in **Figure 6**, a volume in excess of 60 acre-feet is only exceeded 10 percent of the days. Similarly, a volume in excess of 63 acre-feet is exceeded 5 percent of the days. Therefore, an equalization basin of approximately 63 acre-feet would be capable of storing the daily volume of flow above the average daily flow rate 95 percent of the time.

The volumes above the average also represent the volume of water it would take to increase the trough up to the average. Therefore, a release of 60 to 63 acre-feet would be required on a daily basis to fill in the trough (i.e., bring the flow rates during the trough up to the average rate) of the RWHTF discharge.

As discussed in Section 4.2 of this report, the Metro District's previous flow equalization study concluded after extensive modeling that equalization ponds need to be sized conservatively high to account for the variations in the diurnal fluctuations that are not always known ahead of time. For example, the actual average daily flow rate is not known until after the day is completed. To account for these operational difficulties and inefficiencies, the Metro District study determined that 77 acre-feet was required to minimize large variances in discharges. The Metro District's study was based on the RWHTF's influent variations, whereas ours is based on the effluent diurnal fluctuation. However, both analyses show fairly consistent results and conclude that equalization of the RWHTF's diurnal fluctuation would require a 60 to 80 acre-foot equalization pond depending on the desired ability to equalize most or all diurnal variances.

5.2 South Platte River's Response to RWHTF Diurnal Fluctuation

When a wastewater effluent flow rate is large compared to a receiving stream's flow rate, the flow rate and pattern of the downstream reach of the receiving stream largely mimic that of the effluent discharge. This is the case with the SPR downstream of the RWHTF during the months not highly influenced by snowmelt runoff. As shown in **Table 1**, during the period of 2010 through 2014, the average daily percentage of the SPR flow comprised of RWHTF treated effluent, calculated by month, ranged from a maximum of 87 percent in January to a minimum of 31 percent in the runoff influenced month of June. Individual monthly averages ranged from 99 percent in January 2012, to a low of 16 percent in June 2014. On an annual basis, Metro District effluent comprised 55 percent of the flow in the SPR upstream of the Fulton Ditch headgate. Because the RWHTF discharge constitutes such a large percentage of the SPR flow within the Study Reach, the hydrograph and flow pattern (i.e., hourly fluctuations) often mimic that of the RWHTF outfall.

5.2.1 Diurnal Fluctuation at the Henderson Gage

The Henderson gage is located approximately 10.6 miles downstream of the RWHTF outfall and provides a near continuous daily streamflow record which began in May of 1926. If the flow rate in the SPR is not influenced by a recent storm or spring runoff, the flow rate and hourly flow rate variation at the Henderson gage are largely influenced by the effluent discharge at the RWHTF. As an example, a two-day period during September 5th and 6th of 2011 is shown on **Figure 7**, which illustrates that the flow at this gage during this period fluctuated between approximately 170 cfs to as high as 330 cfs, and averaged approximately 250 cfs. By comparing the Henderson gage hydrograph to the flow discharge at the RWHTF on this day, it is apparent that the diurnal fluctuation at the Henderson gage is strongly influenced by the diurnal fluctuation of the RWHTF effluent discharge.

The diurnal fluctuation of the RWHTF discharge and the diurnal fluctuation present at the Henderson gage are similar; however, a slight attenuation of the fluctuation occurs along that 10.6 mile stretch of the SPR. This occurs largely due to the effluent discharge combining with the flow of the SPR above the RWHTF outfall, the influences of the Sand Creek and Clear Creek inflows, the irrigation diversions and return flows occurring within the 10.6 mile reach, and also as a natural channel, the SPR's inherent reduction of the peak of a hydrograph as the peak moves downstream due to the channel's resistance and storage characteristics.

As will be discussed in more detail later in Section 6.0 of this report, the Evans No. 2 1871 direct flow right is frequently subject to a bypass call. For comparison purposes, D&A examined the average daily hydrograph of the Henderson gage on the days when the Evans No. 2 Ditch was subject to a bypass call. The Henderson gage hydrograph on these days is representative of the conditions on the SPR at the Henderson gage when the combination of the diurnal fluctuation and the bypass call was problematic for the District 2 irrigators. Average hourly discharge records for the Henderson gage for the period of June 2010 through December 2014 were obtained from the Colorado Division of Water Resources website: *Colorado's Surface Water Conditions*[1]. **Figure 8** represents the average hourly flow and hydrograph of the Henderson gage on days when the Evans No. 2 priority was subject to a bypass call. Comparing this hydrograph to the average daily RWHTF discharge (**Figure 4**), one can see the attenuation that occurs in the reach between the RWHTF and the Henderson gage. For example, the amplitude of the RWHTF effluent trough is 101 cfs (**Figure 4**), as compared to the average amplitude of the trough at the Henderson gage of approximately 77 cfs (**Figure 8**). **Table 8** summarizes the average-to-trough, the average-to-peak, and the peak-to-trough amplitudes of the RWHTF outfall, the Henderson gage, the Fort Lupton gage, and the Kersey gage. As shown in **Table 8**, the magnitude of the diurnal fluctuation is attenuated as it moves downstream.

Many of the potential mitigation alternatives presented herein propose to store the peak of the diurnal fluctuation after the RWHTF effluent has been discharged to the SPR. Therefore, similar to the analysis conducted to determine the storage required to regulate the RWHTF effluent prior to discharge to the SPR, D&A determined what the storage requirements would be for diverting and storing the peak of the SPR diurnal fluctuation at the Henderson gage. This storage requirement would consist of a gravel pit or an on-channel reservoir created by an on-channel check dam for the purpose of regulating the diurnal fluctuation.

By examining the Henderson gage hydrograph on days when the Evans No. 2 Ditch was subject to a bypass call, the average daily storage requirement was approximately 34 acre-feet. However, there are fairly significant variations from the average, and therefore D&A conducted an exceedance analysis to determine the range of storage requirements. The exceedance analysis (see **Figure 9**) indicated that to store 95 percent of the variations in the volume above the average daily flow, 70 acre-feet of storage is required. This storage amount is very similar to the storage requirement determined for the RWHTF effluent prior to discharge as well as the storage requirement calculated as part of the Metro District's previous studies. The attenuation of the RWHTF hydrograph that takes place between the RWHTF outfall and the Henderson gage appears to reduce the magnitude of the peaks and troughs but the volume above the average appears to stay fairly consistent. For the purposes of sizing, cost estimates, and evaluating mitigation alternatives that rely on equalization storage, 70 acre-feet will be used herein for the storage requirement.

5.2.2 Diurnal Fluctuation at the Fort Lupton Gage

The U.S. Geological Survey's South Platte River at Fort Lupton, CO streamflow gage (PLALUPCO or "Fort Lupton gage") is located approximately 28 miles downstream of the RWHTF outfall and 17 miles downstream of the Henderson gage. The Fort Lupton gage provides intermittent seasonal and monthly records beginning as early as 1906. However, a consistent daily record only exists

[1] Much of the SPR gage data relied on for this study, including streamflow data and ditch diversion data, was downloaded from the following Colorado Division of Water Resources website: http://www.dwr.state.co.us/SurfaceWater/data/division.aspx?div=1

from October 2003 to present. The Fort Lupton gage is operated by the U.S. Geological Survey (USGS) in cooperation with the Metro District. Because it is located approximately one mile upstream of the Evans No. 2 river diversion, the flow data for this gage provides a record of the hourly fluctuations of the SPR present within the lower portions of the Study Reach and above one of the ditches most impacted by the diurnal fluctuation.

Similar to its Henderson gage analysis, D&A examined the hourly flow data at the Fort Lupton gage on days the Evans No. 2 Ditch is subject to a bypass call for the period of June 2010 through December 2014. As shown in **Figure 10**, the additional 17 miles along with the inflows and outflows occurring within the reach downstream of the Henderson gage results in further attenuation of the diurnal fluctuation. For example, the amplitude of the trough compared to the average daily flow rate at the Fort Lupton gage is approximately 39 cfs, compared to 77 cfs at the Henderson gage and 101 cfs at the RWHTF discharge (**Table 8**).

In addition, the attenuation reduces the volume of flow above the daily average, or the volume that would be needed to temporarily detain in an equalization basin. An exceedance analysis performed on the daily volumes above the average at the Fort Lupton gage on days when the Evans No. 2 Ditch was subject to a bypass call indicated that to store 95 percent of the variations in the volume above the average daily flow, 50 acre-feet of storage is required (**Figure 11**). This is a reduction in required equalization volume of 20 acre-feet when compared to that which is needed at the Henderson gage. However, D&A understands that the Fort Lupton gage is not as accurate as the Henderson gage at low flow conditions due to the reduced frequency in which it is calibrated by the USGS. In addition, because the Henderson gage is located higher within the study reach, D&A chose to use the 70 acre-feet calculated as the storage requirement at the Henderson gage location for the equalization storage requirement for the alternative analyses involving equalization storage.

5.2.3 Diurnal Fluctuation at the Kersey Gage

The Colorado Division of Water Resources' South Plate River Near Kersey streamflow gage (PLAKERCO or the "Kersey gage") is located downstream of the confluences of the St. Vrain Creek, the Big Thompson River, and the Cache La Poudre River with the SPR and represents the downstream terminus of Water District 2 as shown on **Figure 1**. Because of the contributions of these inflows, the previously mentioned natural river attenuation, and the diversion of the majority of the SPR during the irrigation season by the ditches located at or above the Western Ditch, the Kersey gage does not experience a diurnal fluctuation of a similar magnitude to that recorded by the gages above the Western Ditch. For comparison purposes, D&A examined the Kersey gage records on the days in which the Evans No. 2 Ditch was subject to the bypass call for the period of June 2010 through December 2014 and determined that the peak-to-trough amplitude for those days averaged only 8 cfs (**Table 8**). Therefore, this analysis confirmed our previous understanding that the majority of the diurnal fluctuation is diverted and removed from the SPR at or above the Western Ditch headgate.

6.0 CURRENT RIVER ADMINSTRATION

The historical dry-up locations within District 2 generally occur below the Burlington Ditch, below the Farmers Independent Ditch, below the Jay Thomas Ditch, and at the Lower Latham Ditch (see **Figure 1** for dry-up locations). Recent changes in the operation of the Jay Thomas Ditch water rights (i.e., 100 percent owned by PSCo and used at PSCo's Fort St. Vrain and Cherokee stations) have caused the dry-up point that historically occurred below the Jay Thomas Ditch to now occur below the Western Ditch headgate. Historical administration affecting District 2 primarily consisted of bypass calls placed by the Jay Thomas Ditch; however, subsequent to PSCo's change of use of the Jay Thomas Ditch rights in 2006, it now primarily consists of bypass calls placed by the Western Ditch.

A bypass call is the partial curtailment of a junior upstream right expressed as a call by the junior right bypassing to a named downstream senior right. For example, an 1885 Burlington direct flow bypass call to the 1871 Western Ditch water right will result in the total curtailment of rights upstream of the Western Ditch junior to 1885, and the Burlington bypassing or curtailing a portion of the 1885 water at its headgate as necessary to satisfy the downstream 1871 right at the Western Ditch. As previously mentioned, subsequent to PSCo's change of use of the Jay Thomas Ditch rights in 2006, the call records indicate the water rights most frequently subject to bypass calls from the Western Ditch are the Burlington Ditch's 1885 priority or the Evans No. 2 Ditch 1871 priority. The same call records show an infrequent bypass call of short duration that affects the more junior rights diverted at the Burlington Ditch (i.e., 1908 and 1909), Evans No. 2 Ditch (i.e. 1909 priorities), Fulton Ditch, Brantner Ditch and the Brighton Ditch placed by the Western Ditch necessary to satisfy its August 10, 1871 priority.

Through discussions with David Nettles, Division 1 Engineer, and William Schneider, District 2 Water Commissioner, D&A understands that the 1871 priority of the Western Ditch is the primary calling right of this reach of District 2 making the Western Ditch the "swing ditch" (i.e., the ditch that dictates the presence of a call). Furthermore, D&A understands that the Water Commissioner determines the need for a call on the SPR in District 2, upstream of the Saint Vrain Creek confluence, by: 1) discussing the daily water needs of the Western Ditch with a ditch company representative, 2) examining the low flow "trough" of the daily hydrograph at the Henderson gage, 3) examining gaged and known inflows within the reach upstream of the Western Ditch to determine their potential contribution to demand, 4) estimating unmeasured inflows (ungaged surface inflows and groundwater gains) based on weather conditions and experience, and finally, 5) the initial distribution of the water to all in-priority water users, according to their communicated demands, so that the Western's 1871 priority and all intervening water rights are satisfied when the trough of the diurnal flow reaches the Western headgate. If the Water Commissioner determines the Western's demand will not be completely satisfied, the Water Commissioner will place a bypass call within District 2. The bypass call allows the Water Commissioner to work with upstream junior users so that only a partial curtailment may be required to satisfy the Western Ditch's demands. D&A understands the Water Commissioner's goal when administering the typical bypass call is to fully satisfy the Western's calling right while avoiding any "spills" (i.e., flow over Western's check dam) to the SPR downstream of the Western Ditch.

6.1 Water District 2 Call Record Analysis

D&A completed an analysis of call records of District 2 for the period of 1992 through 2012. The analysis includes a tabulation of the various water rights that have placed a call during the study period, as well as the relative frequency of calls, including bypass calls, that affect each particular structure. The call records indicate which particular water users within District 2 were potentially affected by the diurnal flow administration.

6.1.1 Calling Structures

The following table displays the frequency in which a call was placed by a structure within the Study Reach. The table lists the number of calls for the entire period of 1992 through 2012, as well as those calls placed within the last eleven years of the study period (2002 through 2012).

	1992 – 2012[1]			2002 - 2012		
Calling Structure	**Total Days w/ Call**	**Average Days Per Year**	**% of Total Calls**	**Total Days w/ Call**	**Average Days Per Year**	**% of Total Calls**
Burlington[2,3]	1,752	83.4	52.3%	798	72.5	44.2%
Fulton	57	2.7	1.7%	-	-	-
Brantner	58	2.8	1.7%	-	-	-
Brighton	-	-	-	-	-	-
Lupton Bottom	6	0.3	0.2%	3	0.3	0.2%
Platteville	31	1.5	0.9%	-	-	-
Meadow Island No. 1	15	0.7	0.4%	-	-	-
Evans No. 2	138	6.6	4.1%	13	1.2	0.7%
Meadow Island No. 2	2	0.1	0.1%	1	0.1	0.1%
Farmers Independent	120	5.7	3.6%	82	7.5	4.5%
Western	674	32.1	20.1%	616	56.0	34.1%
Jay Thomas	494	23.5	14.8%	293	26.6	16.2%
TOTAL	**3,347**	**159.4**	**100%**	**1,806**	**164.2**	**100%**

1 D&A understands that a large number of the calls recorded prior to the early 2000s were recorded using the name of the bypassing structure when in actuality the calling structure was either the Jay Thomas or the Western Ditch headgate. Therefore, the total calls tabulated above for the 1992 – 2012 period likely overestimate the calls placed by the Fulton, Brantner, Platteville, Meadow Island No. 1, and the Evans No. 2 structures. Likewise, the total calls placed by the Western Ditch and the Jay Thomas Ditch for the period of 1992 – 2012 are likely underestimated.

2 Includes calls by all rights and structures diverted at the Burlington Ditch head-works (e.g., Henrylyn Irrigation District, Horse Creek Reservoir, Prospect Reservoir, and Barr Lake).

3 D&A understands that bypass calls from the Burlington Ditch to the Jay Thomas Ditch or the Western Ditch were recorded correctly for 1992 – 2012. Therefore, the number of calls placed by the Burlington Ditch in this table should reflect actual calls placed by the Burlington Ditch and not bypass calls to either the Jay Thomas Ditch or Western Ditch.

As indicated by the call tabulation, the primary calling structures since 1992 have been the Burlington Ditch, the Jay Thomas Ditch and the Western Ditch. Because the Burlington Ditch is the most upstream structure within the Study Reach, its calls do not affect any of the other ditches within the Study Reach. Therefore, the purpose of this table is to illustrate that calls placed from the Jay Thomas Ditch and the Western Ditch are the predominant call within the Study Reach that, when placed, affect the other ditches within the Study Reach. The calls placed from the Jay Thomas Ditch and the Western Ditch were historically administered as bypass calls. The Western Ditch represents approximately 61 percent of the calls placed on the SPR above the St. Vrain Creek confluence and below the Burlington Ditch since 2002. More recently and as previously mentioned, PSCo's change of the Jay Thomas water in 2006 has increased the Western's share of the call as the Jay Thomas Ditch has not placed a call since 2006.

6.1.2 Structures Subject to a Bypass Call

As previously mentioned, a bypass call is the partial curtailment of a junior upstream right expressed as a call by the junior right bypassing to a named downstream senior right. The bypass call allows the Water Commissioner to work with upstream junior users so that only a partial curtailment may be required to satisfy the downstream senior right's demands. Recent administration of the District 2 call is primarily done utilizing a bypass call. For example, of the 674 total calls the Western Ditch placed between 1992 and 2012, 547 of them, or 81 percent, were done by bypass call. Recent call records indicate the most frequent bypass calls are from the Burlington Ditch's 1885 priority or the Evans No. 2 Ditch 1871 priority to the Western Ditch headgate. The following is a table of how often each ditch system within the Study Reach is subject to a bypass call for the periods of 1992 through 2012 and 2002 through 2012.

	1992 – 2012		2002 - 2012	
Structure Subject of a Bypass Call	Total Bypass Days	Average Days Per Year	Total Bypass Days	Average Days Per Year
Burlington	386	18.4	265	24.1
Fulton	67	3.2	53	4.8
Brantner	51	2.4	48	4.4
Brighton	9	0.4	3	0.3
Lupton Bottom	26	1.2	14	1.3
Platteville	52	2.5	32	2.9
Meadow Island No. 1	41	2.0	40	3.6
Evans No. 2 (incl. Milton Reservoir)	500	23.8	447	40.6
Meadow Island No. 2	48	2.3	48	4.4
Farmers Independent	13	0.6	13	1.2
Western	43	2.0	43	3.9
Jay Thomas	-	-	-	-

The call record indicates that the Burlington Ditch and Evans No. 2 Ditch systems are most frequently impacted by a bypass call. Of the total of 500 bypass calls affecting the water rights diverted at the Evans No. 2 Ditch, 410 were bypass calls to either the Jay Thomas Ditch or to the Western Ditch. These bypass calls primarily occur during the months of July, August, and September.

7.0 IMPACT OF DIURNAL FLUCTUATION

Based on discussions with the Division Engineer, the Water Commissioner for District 2, and other stakeholders, D&A understands that the diurnal fluctuation impacts the water users of District 2 in different ways. Impacts to District 2 include impacts to individual ditch systems, especially to those subject to a bypass call, and to the District as a whole in the form of inefficient water use and water lost to the lower reaches of the SPR downstream of the Western Ditch. D&A understands that the largest impacts as a result of the diurnal fluctuation are to those structures most frequently subject to a bypass call (i.e., the Burlington Ditch and Evans No. 2 Ditch).

Based on current District 2 administrative practices, the ditch system subject to the bypass call is allowed to “chase the peak” of the diurnal fluctuation. This means that the Water Commissioner allows the bypassing structure to gradually increase its diversion rates during the period in which the trough of the diurnal fluctuation has passed the bypassing structure and the rising limb of the diurnal fluctuation allows for increased diversions. Because the Water Commissioner has set the initial bypass amount based on the previously described “initial allocation” method (i.e., the distribution of daily supply based on the communicated demand and the minimum flow within the reach as defined by the trough present at Henderson gage and corrected for known/assumed inflows and outflows), the Water Commissioner is assured that the calling ditch system will still be completely satisfied when the trough of the diurnal fluctuation is present at the calling ditch’s headgate if the bypassing structure’s fluctuating diversions (i.e., chasing of the peak) do not reduce the flow in the river beyond the original amount used as the basis for the morning bypass. D&A understands that to accomplish this, the Water Commissioner and a representative of the bypassing ditch company remain in contact throughout the day and fairly late into the evening. The near-constant communication allows for the Water Commissioner to relay his approval to increase diversions (i.e., reduce curtailment) or conversely, direct a decrease of diversions (i.e., increase curtailment) depending on any mid-day adjustments necessary.

The impact of chasing the peak of the diurnal fluctuation or the resulting variable diversion rates is felt by the ditch company, individual shareholders, and irrigators of the bypassing ditch. That is, the variable flow rates within the ditch over the course of the day create variable water stages and hydraulic heads that make it difficult for the ditch company to allocate and regulate water deliveries, for irrigators to adjust their farm headgates, and also for the setting of siphon tubes. For the purpose of this study, we are calling these impacts to the ditch downstream of the main headgate “down-ditch impacts.” D&A understands that not only does the “diurnal fluctuation” within the ditch and the resulting down-ditch impacts require significant effort and resource expenditures by the ditch companies and their shareholders, it reduces the overall efficiency and utilization of the water by the ditch systems.

7.1 Daily Flow Fluctuations at Bypassing Structures

D&A examined the maximum daily fluctuation[2] in hourly diversion rates for the ditch systems that are subject to a bypass call for the period of June 2010 through December 2014. The fluctuations experienced on bypass call days are a result of the previously described administration that results in the ditch companies varying their diversion rates over the course of a day. The following table

[2] Maximum daily flow fluctuation = Hourly Maximum Flow Rate – Hourly Minimum Flow Rate

summarizes the average and maximum daily fluctuations experienced by the bypassing structure for the period of 2010 through 2014.

Daily Maximum Flow Fluctuation on Bypass Call Days (2010 – 2014)

Ditch System Subject to Bypass Call	No. of Bypass Days	Flow Fluctuation on Bypass Days, cfs		
		Average	Maximum	Ratio of Max. to Avg.
Burlington Ditch	113	118	522	4.4 : 1
Fulton Ditch	17	16	61	3.8 : 1
Brantner Ditch	18	18	99	5.5 : 1
Lupton Bottom Ditch	3	24	61	2.5 : 1
Platteville Ditch	21	15	123	8.2 : 1
Meadow Island No. 1 Ditch	9	19	86	4.5 : 1
Evans No. 2 Ditch	275	49	202[3]	4.1 : 1
Meadow Island No. 2 Ditch	35	18	134	7.4 : 1
Farmers Independent Ditch	15	17	113	6.6 : 1

The table illustrates that the Burlington Ditch and Evans No. 2 Ditch systems generally experience the largest flow fluctuations when subject to a bypass call. As previously mentioned, the large flow fluctuations create difficult ditch delivery and allocation situations that result in decreased ditch system and on-farm efficiencies.

7.2 Impact to Evans No.2 Ditch

Based on the District 2 call record analysis described in Section 6.0 of this report and the flow fluctuation analysis presented above, it appears that the Evans No. 2 Ditch system is currently the system most frequently impacted by the diurnal fluctuation. This finding is consistent with the information D&A obtained during our discussions with the State and the stakeholders of this study.

As previously mentioned, the Evans No. 2 Ditch is frequently subject to a bypass call placed in the months of July, August, and September. Typically, a portion of the 177 cfs, October 5, 1871 water right of the Evans No. 2 is bypassed to the Western's August 10, 1871 water right for 71 cfs. In general, the Evans No. 2 Ditch, aided by a SCADA controlled headgate, must start decreasing its diversions in 10 to 15 cfs increments each hour starting at about 3:00 p.m. to accommodate the Western's call for water while the flow of the SPR is decreasing due to the diurnal fluctuation. Typically, by 6:00 p.m., the trough of the diurnal fluctuation reaches the Evans No. 2 Ditch. By 10:00 p.m., the diversions will level off and they will then gradually open their headgate to chase the rising limb of the diurnal flow. According to Evans No. 2 Ditch representatives, the diurnal flow swing at their headgate is usually on the order of 60 to 80 cfs; however, their diversions can vary from 60 to 177 cfs during the course of a single day. D&A's analysis for the 2010 through 2014 study period indicated the average fluctuation on bypass call days was 49 cfs; however, the diversion records did show individual daily fluctuations consistent with the statements provided by the Evans No. 2 Ditch representatives.

[3] Maximum fluctuation includes water diverted under a 1909 right at the Evans No. 2 headgate. The Evans No. 2 1871 direct flow right is limited to 177 cfs.

The result of chasing the diurnal fluctuation during the periods the ditch is subject to a bypass call requires significant effort by the ditch company, by the irrigators under the ditch, and causes inefficiencies for the 110 headgates along the 25 miles of the Evans No. 2 Ditch. Some individual shareholders have constructed their own equalization ponds downstream of their farm headgates to regulate their deliveries, while others believe the regulation of deliveries is the responsibility of the ditch company. D&A understands that the Evans No. 2 Ditch has conceptually designed an on-ditch equalization pond and identified its potential location. However, it has not finalized financing and continues its coordination with the Farmers Reservoir and Irrigation Company (FRICO) that shares conveyance capacity within the Evans No. 2 Ditch.

7.3 Impact to Burlington Ditch

The 1885 Burlington Ditch direct flow right is subject to a frequent bypass call placed by the Western Ditch. A 1908 and 1909 Burlington Ditch direct flow right is also, on occasion, subject to a similar bypass call. The impacts of the diurnal fluctuation on the Burlington Ditch are interesting in that the Burlington Ditch's headgate is above the RWHTF discharge. However, because the calling structure (i.e., Western Ditch) is downstream of the RWHTF discharge, and the administration of a call originating within the Study Reach and below the RWHTF is largely based on the trough of the diurnal fluctuation, the Burlington Ditch is impacted by this fluctuation. For example, if the Burlington Ditch's 11-20-1885 right is bypassing to the Western Ditch's 8-10-1871 right, the bypass rate is determined the same way as for ditch structures located below the RWHTF discharge. That is, the bypass or curtailment amount at the Burlington Ditch is initially determined based on the magnitude of the trough of the diurnal fluctuation and whether its flow rate is adequate to satisfy the Western Ditch. D&A understands that until recently, the Burlington Ditch did not chase the peak or vary its diversions over the course of a day if it was subject to the bypass call. Essentially, the curtailment amount determined by the initial morning allocation was the curtailment amount for the remainder of the day.

Additional coordination between the Ditch Company and the Water Commissioner more recently has led to Burlington varying its diversion rates over the course of a day similar to the practices of the Evans No. 2 Ditch. However, because of the significant river travel time (i.e., approximately 24 hours) between the Burlington Ditch headgate and the Western Ditch headgate and uncertainty in unmeasured gains and losses between those same structures, the modifications to the Burlington Ditch diversion amounts are made less frequently. Because of the relatively close distance between the Evans No. 2 Ditch headgate and the Western Ditch headgate, the travel and response time between the two structures is only about 6 hours. Therefore, the effects of the varying diversions of the Evans No. 2 Ditch on the available flow at the Western Ditch headgate are more quickly realized. If the curtailment amounts are either overestimated or underestimated, the effects of this can be corrected relatively quickly compared to if these estimation inaccuracies are made to the curtailment estimates at the Burlington Ditch. Because of this, D&A understands the Water Commissioner is more conservative with the allowed headgate adjustments at the Burlington Ditch. This results in the Burlington Ditch usually only making two gate changes over the course of 24 hours as follows: one time at roughly midnight to reduce its diversions and then again at approximately 5 am to increase its diversions. The practice of making two near instantaneous gate changes results in, at times, significant and abrupt changes in the flow rate and stage within the ditch. These changes, as previously described, can make deliveries and down-ditch operations difficult. The down-ditch impacts of the two gate changes on the Burlington Ditch may be more severe than those experienced by the Evans No. 2 Ditch as the adjustments made by the Evans No. 2 Ditch are more gradual due to their proximity to the calling ditch.

7.4 Impacts to Calling Structures

D&A understands that in addition to affecting the bypassing structures, the diurnal fluctuation can also impact the senior ditch placing the call based on information from Western Ditch representatives. The impacts to the senior calling ditch are a result of being shorted water at times over the course of the day. As previously mentioned, the Water Commissioner's goal when administering the bypass call is to fully satisfy the senior calling right, often the Western Ditch's 8-10-1871 right for 71.12 cfs, while only requiring a partial curtailment of the upstream junior users. Depending on how accurate the estimates of inflow, outflow, demands, etc. are when the Water Commissioner makes the initial morning allocation of supply, it is possible the calling right may be shorted if actual inflows to the SPR are less than estimated inflows. In these cases, due to inadequate flow data, the Water Commissioner may unknowingly allow the junior right to divert more water and bypass less to the SPR than would be actually required to fully satisfy the senior calling right. D&A understands that once the Water Commissioner is made aware of any shortages, the Water Commissioner requires an increased curtailment of the junior right in order to satisfy the calling senior right.

7.5 Impacts to Water District 2 Water Users

In addition to the impacts experienced by the individual ditch systems placing a call or subject to a bypass call, D&A understands that the diurnal fluctuation does, at times, result in water flowing over the Western Ditch check dam. As discussed above, the Water Commissioner's goal when administering the bypass call is to fully satisfy the Western's calling right while avoiding any spills (i.e., flow over Western's check dam) to the downstream reach. Depending on how accurate the estimates of inflow, outflow, demands, etc. are when the Water Commissioner makes the initial morning allocation of supply, D&A understands the flow over the Western check dam can be upwards of 10 cfs, plus or minus. The water flowing over the Western check dam would be water that would otherwise be available for diversion by the District 2 irrigators above the Western Ditch headgate. The amount of flow over the Western check dam is also dependent on the bypassing ditch's ability to efficiently and responsively chase the diurnal fluctuation.

No record is kept as to the amount of water flowing over the Western check dam. Generally, the presence of flow over the check dam is known only by visual inspection. According to multiple stakeholder statements, the District 2 water users believe the State's, and specifically District 2 Water Commissioner Bill Schneider's, methodology of estimating and administering for the diurnal fluctuation results in only limited spills. Mr. Schneider did indicate in the same stakeholder meetings that improvements to stream gages, as well as additional gages on tributary inflows could aid and improve his allocation of supplies and further limit spills from this reach. More discussion regarding potential improvements to administration, including additional stream flow measurements, is found in Section 9.0 of this report.

8.0 STRUCTURAL MITIGATION ALTERNATIVES ANALYSIS

Several possible alternatives have been proposed to address the diurnal flow fluctuation and its impacts on District 2 water users. These alternatives were offered by various stakeholders during meetings held for the Effluent Trade case and early stakeholder meetings held in preparation of this study. The possible solutions proposed by the stakeholders include:

A. Use of upstream storage at Chatfield Reservoir for the purpose of timing releases of stored water to attenuate and/or offset the diurnal fluctuation.

B. Use of storage at an existing gravel pit reservoir located in the stretch of the SPR between RWHTF and the headgate of the Western Ditch for the purpose of timing releases of stored water to attenuate and/or offset the diurnal fluctuation.

C. Construction of a new gravel pit storage reservoir downstream of RWHTF and upstream of the Western Ditch headgate for the purpose of timing releases of stored water in order to attenuate and/or offset the diurnal fluctuation.

D. Use of storage at agreed upon locations, including agreed upon timed releases by parties using effluent discharged at RWHTF, for the purpose of timing releases to attenuate and/or offset the diurnal fluctuation.

E. Construction of a storage reservoir near the headgate of the Western Ditch at the Gilcrest Reservoir site in order to attenuate the impact of the diurnal fluctuation to the Western Ditch, which could benefit other water users that have been historically subject to calls by the Western Ditch. In this study, the investigation of a storage location between the RWHTF and the Western Ditch was not limited to the Gilcrest Reservoir site. D&A also investigated storage locations near other ditch headgates on the SPR between RWHTF and the Western Ditch.

F. Use of existing or new river check dams that could be modified in order to regulate the diurnal fluctuation.

G. Use of groundwater diversions for ditches to offset the diurnal fluctuation.

8.1 Preliminary Screening Analysis

D&A conducted a screening analysis to evaluate the preliminary engineering feasibility of each of the above proposed mitigation alternatives. In addition to the analysis of engineering feasibility, this study identifies the more significant potential legal, institutional, and permitting issues associated with each alternative. Not included in this screening is any discussion or analysis regarding potential water quality impacts. Some of the mitigation alternatives could also present significant water quality and/or aquatic life habitat issues. For example, the Metro District has indicated that some of the mitigation alternatives could adversely impact dissolved oxygen (DO) in the SPR and/or other water quality standards. Additional engineering, legal, and permitting research will be required to determine which, if any of the alternatives presented herein, or which combinations of presented alternatives may be suitable for implementation.

8.1.1 Upstream Storage at Chatfield Reservoir

A mitigation alternative presented by the stakeholders involved the use of storage and releases from Chatfield Reservoir as a source of supplementing the flow in the SPR during the trough of the diurnal fluctuation. The Chatfield Reservoir is a 350,000 acre-foot on-channel reservoir located southwest of Denver, at the confluence of the SPR and Plum Creek. Chatfield Reservoir is primarily a flood control structure operated by the US Army Corps of Engineers (Corps); however, Denver Water does have a contract to store approximately 27,000 acre-feet of water within the conservation pool of the reservoir. There is currently a project underway (Chatfield Reservoir Reallocation Project) to reallocate approximately 20,600 acre-feet of the existing flood pool such that it can be utilized by municipal and agricultural water providers of the Front Range area. The Chatfield Reallocation Project is a partnership among nine water providers in the Denver metropolitan area. Each organization will receive a varying amount of storage space at Chatfield once the reallocation is complete. These organizations include: Colorado Water Conservation Board, Colorado Parks and Wildlife, Castle Pines Metropolitan District, Castle Pines North Metropolitan District, Town of Castle Rock, Centennial Water & Sanitation District, Center of Colorado Water Conservancy District, Central Colorado Water Conservancy District, and the Mount Carbon Metro District.

In concept, the Chatfield Reservoir mitigation alternative would use the timed release of natural streamflow flowing into Chatfield Reservoir as a source of water to mitigate the diurnal fluctuation present downstream of the RWHTF. Operations would include the timed release of the daily inflow to Chatfield Reservoir that was subject to release for downstream purposes, such that when the storage release reaches the RWHTF outfall approximately 22 miles downstream, it would fill in or supplement the trough of the diurnal fluctuation. Depending on the volume of water available and set for release to downstream users, the release could potentially supplement more than just the trough-below-the-average. That is, if the volume of water is sufficient, the release could be timed such that the release would supplement the trough-below-the-peak of the RWHTF hydrograph. This would result in a steady rate (i.e., flat top) hydrograph below the RWHTF. The patterned release would likely be pre-determined based on recent RWHTF discharge information, including the approximate timing and magnitude of the peaks and troughs and an estimate of the daily volume of natural streamflow present in Chatfield Reservoir set for release to downstream users. It is important to note that the volume of releases on a daily basis would be the same as they would be under current administration. This alternative would only vary the release rates in an effort to help mitigate the diurnal fluctuation downstream.

In terms of the potential storage space, Chatfield Reservoir would certainly have adequate daily storage capacity to store and retime normal daily inflows during the irrigation season for the purpose of potentially mitigating the trough of the diurnal fluctuation. From a water availability standpoint, D&A was unsure of the reliability of natural stream inflow into Chatfield Reservoir that could be stored and retimed for this mitigation purpose. Therefore, D&A examined the “Chatfield Checksheet” and the historical daily flows present at the South Platte River Below Chatfield Reservoir streamflow gage (PLACHACO). The Chatfield Checksheet is a jointly developed and maintained accounting form, mainly by Denver Water and the State, which accounts for, among other ancillary structures and inflows, the daily operations of the Strontia Springs and Chatfield reservoirs. One of the main metrics tracked by the Chatfield Checksheet is the daily required outlet release of Chatfield Reservoir, which is the amount of inflow into Chatfield Reservoir not allocated for direct storage or storage by exchange within the reservoir, outflow exchanges, or deliveries out of the manifold such as the deliveries to the fish hatchery, Last Chance Ditch, Nevada Ditch, City Ditch, etc. The calculated required outlet release is the daily amount of natural streamflow that should be available to meet downstream water requirements and is therefore used as the basis of determining the Chatfield Reservoir release rate. If the release requirement changes significantly from day to day, the State Engineer's Office (SEO) along with the Corps will make an adjustment to the gate such that the actual release is closer to the calculated release. Therefore, the daily flow record at the streamflow gage located immediately below the Chatfield Reservoir outlet, PLACHACO, provides a good approximate record of the calculated required release from the Chatfield Reservoir on a daily basis and thereby the volume of water that could be temporarily detained and retimed for the purpose of mitigating the downstream diurnal fluctuation. As previously mentioned, the volume of water to be released on a daily basis would be the same amount of water that would have been released daily prior to implementing this practice. Therefore, D&A does not envision this alternative requiring any long-term storage of natural inflows in Chatfield Reservoir.

D&A analyzed the daily flow record for the PLACHACO gage for the period of July 1986 through December of 2013 and found that the average volume of flow released from Chatfield Reservoir on a daily basis would be adequate, in most irrigation months and years, to offset the average 70 acre-foot trough-below-the-average of the RWHTF discharge hydrograph. As shown in **Table 9**, in all but the driest years (e.g., 2002 and 2012), the daily release volumes would be adequate (i.e., in excess of 70 acre-feet) in the irrigation months of April through August. The months of March, September and October are not as reliable, but still average no less than 53 acre-feet of releases. Therefore, from examination of the streamflow releases from Chatfield Reservoir, it would appear that the availability of excess or unallocated natural streamflow present at Chatfield Reservoir for release would have been historically adequate to fill-in the average trough-below-the-average of the diurnal fluctuation present at the RWHTF outfall. It is possible that an increase in the exercise of existing exchanges that store water in upstream storage reservoirs, including Cheesman Reservoir, Strontia Springs Reservoir, and Chatfield Reservoir, would reduce the availably of unallocated natural streamflow at Chatfield Reservoir and potentially render this alternative infeasible due to a lack of water supply.

As previously mentioned, depending on the amount of water set for release out of Chatfield, it is possible the volume of water released could more than just fill in the trough-below-the average RWHTF hydrograph. If adequate natural streamflow is available, it could be released from Chatfield Reservoir such that it would fill in the trough-below-the-peak of the RWHTF discharge. The resulting hydrograph downstream of the RWHTF would be a near steady-state hydrograph with little to no peaks and troughs. Similar to the methodology previously described to determine the 60 to 70 acre-foot volume below trough-below-the-average of the RWHTF hydrograph, D&A determined the daily volume below the *peak* of the RWHTF discharge hydrograph for the years of 2001 through 2014. The exceedance analysis performed on the daily volumes below the peak indicated that 95 percent of the daily volumes were equal to or less than approximately 197 acre-feet (see **Figure 12**). This means that on a daily basis, a release from Chatfield of 197 acre-feet would be required to fill in the trough-below-the-peak of the RWHTF discharge effluent and create a near steady-rate hydrograph below the discharge. Lesser releases could still help mitigate the trough-below-the-peak but would not completely eliminate the variance created by the RWHTF discharge.

The daily streamflow record at the PLACHACO gage was again examined to determine the adequacy of the historical releases to fill in the trough-below-the-peak. As shown in **Table 10**, the historical average daily release volumes are generally inadequate (i.e., less than 197 acre-feet) to completely supplement the daily volume below the peak in the months of March, September, and October. During the months of April, July and August, the volumes released from Chatfield Reservoir would have been inadequate in a number of years. During the months of May and June, outside of the extremely dry years (e.g., 2002 and 2012), the historical release volumes would have been generally capable of supplementing the trough-below-the-peak but not as consistently as they would be able to supplement the trough-below-the-average. As previously mentioned, future changes in the operations of exchanges through this reach may reduce water availability so that very little water would be available during the irrigation season to mitigate the diurnal fluctuation.

While the historical water availability on a daily basis may have been generally adequate to mitigate the trough-below-the-average, and at times, the trough-below-the-peak, the necessity to vary the release rates out of Chatfield Reservoir on an hourly basis presents a significant departure from the historical operations of the Chatfield outlet works. D&A's research on this topic indicated that the operation of the Chatfield Reservoir outlet works is strictly managed by the Corps with input from the SEO. When the stage of Chatfield Reservoir is within the normal "conservation pool," the SEO determines using the previously mentioned Chatfield Checksheet, what daily river release rates are necessary to meet the downstream water requirements and will issue the necessary regulation release orders to the Corps. D&A understands that while these release rates can and are occasionally varied on a daily basis, the normal operation is to make gate adjustments less frequently. To use Chatfield Reservoir storage to offset the trough of the diurnal fluctuation, gate changes would be required on an hourly basis. By examination of multiple months and years of the Chatfield Checksheet, it appears gate changes are only made once a day, if at all. D&A is not aware of how the Chatfield Reservoir Reallocation Project will alter the normal operations of outlet works. However, given the current level of communication and coordination required for gate changes, hourly changes of releases from the Chatfield Reservoir outlet works would require a significant change in the water control plan currently instituted by the Corps.

D&A did not request a legal opinion on whether or not a decree would be necessary to temporarily store and retime the natural streamflow that flows into Chatfield Reservoir. Based on our understanding of how the reservoir outlets are operated, there is likely already some amount of temporary impounding taking place in between changes in release rates. A decree may be necessary and useful to provide terms and conditions for ushering these releases downstream to the Study Reach. These releases would have to flow past the Burlington Ditch headgate which is a historical dry-up location on the SPR. Also, there may be river operations, including diverters, between Chatfield Reservoir and the RWHTF that would be impacted by a change in the flow patterns resulting from this alternative. For example, the fairly large flow fluctuations that would be created by the varied releases would essentially create a diurnal fluctuation within the reach of the SPR downstream of Chatfield. This diurnal fluctuation would likely create diversion difficulties for the Burlington Ditch similar to those experienced by the diverters downstream of RWHTF.

In addition to operational and legal issues, D&A understands that as part of the Corps' approval process of the Chatfield Reallocation Project, the mitigation plan included an Environmental Pool to be utilized in part to maintain flows in the SPR reach below Chatfield Reservoir. The low flows present downstream of Chatfield during certain times of the year are a significant environmental concern. As previously described, the retiming of releases from Chatfield will result in higher than normal flows at times and lower than normal flows at others. It is likely that this release pattern will exacerbate the low flow concerns during certain times of the day. Additional research would be needed to determine the impacts this alternative would have on low flows.

From an infrastructure standpoint, this alternative has the advantage that the structure needed for potentially mitigating the diurnal fluctuation is already constructed and in place. However, it is D&A's opinion that the changes to the State's and Corps' operations of the Chatfield Reservoir outlet works necessary to make this alternative work involve legal, institutional, and environmental issues that may prove difficult and would require additional research beyond the scope of this study to determine if such changes would be feasible.

8.1.2 Use of Existing Gravel Pit for Flow Equalization Prior to RWHTF Discharge

As previously mentioned, the adjacent lands on either side of the river within the Study Reach, especially upstream of Fort Lupton, CO, have been heavily mined for sand and gravel. The reclaimed pits that have been lined have been acquired mainly by local municipalities and water providers for recapture and storage of reusable effluent supplies discharged at the RWHTF. **Figure 2** shows the location of the lined gravel pits and is color-coded by owner. As shown in these figures, the City of Thornton owns a large percentage of the reclaimed pits primarily made up of multi-cell complexes such as their East Gravel Lakes (aka, Tani Lakes Storage Complex), West Gravel Lakes, Cooley East, Hammer facilities, and Rogers Reservoir. Other owners of gravel pit storage in this reach include: SACWSD, Denver Water Board, Aurora Water, and the City of Brighton.

During stakeholder meetings, discussions occurred regarding the potential use of an existing gravel pit that could be utilized to capture and equalize RWHTF effluent prior to its discharge to the river. An inherent requirement of this alternative would be the use of a gravel pit within close proximity to RWHTF to avoid lengthy and cost prohibitive pipelines and/or pump stations. By examining the location of existing gravel pits, the most likely candidates for use as an equalization structure would be Denver Water's Welby (a.k.a. Cat Lake) and/or Bambei-Walker (a.k.a. Miller Lake) pits based solely on their proximity to the existing RWHTF outfall. While these two pits are within 1,500 to 2,000 feet of the existing RWHTF outfall, a pipeline constructed to convey effluent to these pits would require a costly crossing of the SPR and/or Interstate 270. D&A discussed the feasibility of this alternative from an institutional and legal standpoint with the Metro District and its legal counsel. The Metro District stated that the RWHTF effluent discharge permit issued by the Colorado Department of Public Health and Environment (CDPHE) is for specific locations on the SPR. If the discharge from an equalization pond is different from the existing RWHTF discharge location, the Metro District stated that the permit amendment process of moving the discharge location would be lengthy and would present significant regulatory and legal hurdles. These hurdles would involve studying potential water quality impacts caused by the location of the equalization basin and outfall relocation and determining compliance with current permit and stream standards that are based on the location of the existing outfall. The Metro District indicated that water quality parameters potentially impacted by an equalization pond and/or outfall relocation include, but are not limited to, dissolved oxygen (DO) levels, reduced natural nutrient processing, selenium and cadmium levels, and increased water temperatures in the SPR reach above the new outfall.

In terms of legal hurdles, the Metro District believes that moving the discharge point will create very similar legal issues as those seen in Case No. 11CW74. That is, there are numerous Water Court decrees that identify the Metro District's current outfall location for exchanges, augmentation, and the point of municipal return flows. The movement of the outfall from its current location would create issues for the entities with such decrees. Therefore, the Metro District believes that a relocation of the existing outfall would ultimately involve significant litigation with numerous parties. Moreover, the Metro District does not believe that storing effluent to mitigate the diurnal flow is consistent with its statutory purpose of intercepting, receiving, transporting, treating, and disposal of the outfalls of member sewer systems (C.R.S. 32-4-506). The Metro District is also unsure of whether or not changing the timing of its discharge by storing and releasing the effluent at a more constant rate presents any legal issues as it does not own or exercise dominion or control over the water they treat. In general, the Metro District treats influent as it comes in and releases treated effluent in a similar pattern. Based on the institutional and legal opinions provided by the Metro District, it is D&A's opinion that the alternative of using an existing gravel pit for effluent equalization prior to discharge of the effluent to the SPR has potentially insurmountable legal and regulatory flaws that should effectively remove it from further consideration.

8.1.3 Use of Existing Gravel Pit between RWHTF and Western Ditch Headgate

An alternative concept developed by the stakeholders involves the use of an existing gravel pit storage reservoir located downstream of the RWHTF discharge for the purpose of mitigating the diurnal fluctuation. Conceptually, this alternative would use an existing gravel pit storage reservoir, and associated filling and discharging infrastructure, to divert the peak of the diurnal fluctuation off the SPR and release it during the following trough. This alternative would avoid the previously discussed legal and institutional issues regarding the retiming and relocating of the existing RWHTF discharge to the SPR as the effluent discharge would occur the same as it does today. The storing and retiming of the peak would be made by a diversion off of the SPR subsequent to the discharge of the RWHTF effluent to the SPR.

There are numerous lined gravel pit storage reservoirs and complexes, primarily owned and operated by municipal water providers, along the adjacent lands of the SPR within the Study Reach (see **Figure 2**). The gravel pits are filled via diversions of reusable effluent supplies discharged at the RWHTF outfall, changed water rights, and junior storage rights. The water supplies are diverted either by direct river diversions constructed and controlled by the water provider or at the headgates of District 2 irrigation ditches pursuant to contractual carriage agreements. These carriage agreements usually specify the amount of the ditch capacity the municipal water provider can use for the purpose of conveying its supplies to the point of storage. The available capacity can be either the ditch's excess capacity (i.e., ditch's capacity in excess of that needed for ditch company's water rights) and/or additional constructed capacity whereby the ditch and infrastructure upstream of the gravel pit storage are improved to meet the simultaneous needs of both the ditch company and the water provider.

Based on the average peak of the diurnal fluctuation at the Henderson gage compared to its average daily flow, a gravel pit would need to have an inflow capacity of approximately 40 cfs to completely divert the peak flow rate. These diversions would occur over approximately 14 hours and would utilize the gravel pit's existing filling structure and capacity or additional constructed capacity added for this purpose.

Due to the average depth of gravel deposits within this reach of the SPR (i.e., 30 to 50 feet below surface), a majority of the gravel pits require the use of a pump station to make releases to the SPR. Assuming that the release of the previously stored peak of the diurnal fluctuation during the time of the trough would require pumping, the pump station would be required to pump approximately 80 cfs to be capable of fully supplementing the minimum point of the trough.

The costs of this alternative could vary depending on what infrastructure upgrades would be required for the existing gravel pit storage reservoir in terms of inflow capacity and outflow capacity. The current owner of the storage reservoir would likely require payment for the use of the storage, and the purchase and development of lined storage ranges from $7,000 to $10,000 per acre-foot. If 70 acre-feet of equalization storage is required, the cost for storage would be approximately $490,000 to $700,000. If the construction of a pump station is required, an 80 cfs pump station including the pump(s), vault, inflow and outflow piping, controls, etc. could cost an estimated $8.3 million (see **Table 11**) or more depending on specific site conditions and required infrastructure.

In addition to the capital costs, the annual energy costs associated with operating the pump station are substantial. D&A estimated annual energy costs associated with operating the pump station to range from $42,000 to $45,000 (see **Table 12**) depending on the equalization pond's frequency and duration of use. That is, depending on the hydrologic and administrative conditions of the SPR (i.e., average, wet, dry), the reliance on the use of an equalization pond may vary. Annual energy costs were developed for an average and dry hydrologic year. D&A understands that the diurnal fluctuation of the RWHTF discharge is not an issue for irrigators unless the SPR's flows get below a certain flow threshold (e.g., approximately 500 cfs). Below this flow rate, a bypass call is usually placed and the diurnal fluctuation creates the previously described down-ditch issues for the structure subject to the bypass call. Therefore, to estimate the duration of use of the equalization pond and pump station on an annual basis, D&A summarized the District 2 bypass calls for the years of 2000 through 2012. The following table summarizes the average annual number of bypass calls by month and the maximum for the study period which occurred in 2012.

Number of Bypass Calls Per Month

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec	Total
Average	0	0	1	7	9	9	19	22	13	1	0	0	81
2012	0	0	9	23	23	23	21	25	24	0	0	0	148

The range of estimated annual energy costs were developed based on pumping for approximately 81 days a year on average and for a maximum of 148 days.

There will likely be legal issues associated with the temporary diversion and storage of the peak flows of the SPR into an off-channel reservoir. The State generally allows for direct flow water rights to be temporarily detained for up to 72 hours in order to allow more efficient or effective beneficial use of the water. However, this alternative is essentially an off-channel gravel pit used to divert and temporarily store portions of the SPR flow made up of multiple entities' water rights, reusable supplies, and native flow. Downstream water users may be reluctant to have a portion of their water rights temporarily detained for this purpose without a court-approved decree. Also, it is possible that the augmentation of evaporation and conveyance losses associated with the diversion and temporary storage of the SPR water would be required. Accordingly, it is likely a decree would be required to operate the off-channel equalization pond and the cooperation of the District 2 water users. Additional legal research and opinion would be required to identify all legal issues surrounding this alternative.

8.1.4 Construction of a New Gravel Pit

If storage within an existing gravel pit storage facility was not available or was thought to be problematic from a shared operating standpoint, a new gravel pit storage reservoir could be constructed for the purpose of flow equalization. Due to the relatively small amount of storage required for flow equalization (i.e., 70 acre-feet), it is likely the gravel pit constructed will be significantly larger based on the size of the majority of the existing gravel pits within the Study Reach. In this case, it may be again beneficial to share the storage capacity of the gravel pit with an entity with larger storage requirements. From a pumping cost standpoint, it is much more efficient to pump the flow equalization amounts from the "top" of the gravel pit rather than having to lift it from greater depths if the gravel pit is not kept near full.

This alternative may have added expenses over the use of an existing gravel pit and its associated infrastructure. If the new gravel pit did not have the ability to divert water from a nearby irrigation ditch or from an existing river diversion, a new river diversion would be required to divert the peak of the diurnal fluctuation. Given the current regulations and permitting requirements necessary to construct a new river diversion (e.g., Section 404 permitting), constructing a new river diversion would be cumbersome and expensive. Storage and pump station costs would be similar to those discussed above.

This alternative would have very similar legal issues to those discussed for the existing gravel pit reservoir. Because the gravel pit would require diverting and temporarily storing multiple entities' water rights, a water court decree and the cooperation of the District 2 water users will likely be required to do so.

8.1.5 Use of Storage and Timed Releases by Parties Using RWHTF Effluent

Beyond what has been previously discussed regarding the use of gravel pit storage and timed releases of effluent to mitigate the diurnal fluctuation, an additional mitigation alternative was proposed that would involve a cooperative operating principle to be developed by the State and willing participants within the Study Reach, whereby the participants would agree to make strategically timed releases such that the augmentation and/or substitute supply releases would supplement the trough of the diurnal fluctuation when it is present at the outlet works of the storage structure. The participants of the operating principle or agreement would likely be those entities already relying on releases from gravel pit storage to augment depletions within the Study Reach or provide a substitute supply for an exchange originating within the Study Reach.

For example, rather than making an augmentation release of 20 acre-feet on an average basis over the course of a day (e.g., 10 cfs release), the augmenter would make a larger release over a shorter period of time. The timing of the release would be determined based on the timing of the trough within that reach. The increased rate or varied rate would be dependent on factors such as the capacities and limitations of the existing infrastructure (e.g., pump station, outlet pipe, etc.).

The effectiveness of this alternative to mitigate the diurnal fluctuation is dependent on the level of participation of the entities augmenting and/or exchanging within the reach, the participants' ability to increase their release rates during the time of the trough, and most importantly, the amount of consistent augmentation and/or substitute supply being released from gravel pit storage within the Study Reach. The presence of the bypass call would limit the exchange reach to between the calling structure and the bypassing structure or an exchange reach located completely above the bypassing structure. To the extent these exchanges exist and occur, the release of the substitute supply could be timed such that it helps mitigate the trough. D&A was not able to quantify the existing or projected amounts of augmentation or exchange planned for the Study Reach, but understands that the majority of the augmentation currently taking place within the Study Reach is done using augmentation supplies that are left in the SPR after discharge from RWHTF. For example, Aurora Water's current augmentation of the Prairie Waters Project (PWP) well depletions is done almost exclusively using its reusable effluent supplies discharged at RWHTF. Therefore, Aurora does not currently make augmentation releases from its Walker Reservoir and/or Everist Reservoir on a consistent basis. However, as PWP depletions increase with the project's increased capacity and Aurora's increased reliance on it to meet an increasing municipal demand, Aurora expects to rely more regularly upon releases from its gravel pit storage for the purpose of augmenting PWP depletions.

D&A understands that Denver Water and Thornton largely use their gravel pit storage along this reach for exchange purposes and in Thornton's case, storage of its decreed water rights from Clear Creek water, Burlington-Wellington shares, and SPR raw water sources. Therefore, it is understood that neither Denver Water nor Thornton currently has any significant or consistent augmentation occurring within the Study Reach. However, both entities utilize exchanges from their gravel pits to upstream termini. When the bypassing structure is located downstream of the source of substitute supply, a timed release of the substitute supply during the trough could help to mitigate the impacts of the diurnal fluctuation.

This alternative may be feasible but its effectiveness may be limited, at least in the short term. As consistent augmentation via gravel pit releases increases within the reach, this alternative could help mitigate the diurnal fluctuation if augmenters are willing to participate and modify their operations with the goal of mitigating the diurnal fluctuation. This alternative could potentially help fill in the trough of the diurnal fluctuation but would not help reduce the peak flow. This alternative would likely require frequent communication between the Water Commissioner and the participants and likely increase the operating and maintenance expenses of the participants due to operating pumps and infrastructure at higher and possibly varying flow rates.

8.1.6 Storage Near Ditch Headgates

A majority of the alternatives presented by stakeholders and previously discussed herein are "regional" in nature, meaning that they are mitigation alternatives that would attempt to mitigate the diurnal fluctuation for the entire Study Reach or a large portion of it. However, this mitigation alternative looks to utilize individually owned and operated equalization ponds near river headgates in order to regulate the fluctuations delivered to shareholders. This alternative would not mitigate the diurnal fluctuation the SPR experiences and thus would not solve the river allocation problem (i.e., ensuring the proper amount of water goes to the bypassing ditch and the proper amount to the calling ditch). Rather, this alterative is limited to eliminating the large fluctuations and down-ditch problems on a ditch-by-ditch basis. This mitigation alternative also acknowledges that not all ditch systems are impacted equally by the diurnal fluctuation.

The Western Ditch Company constructed the "Western Mutual Equalization Pond" in 2010 to help regulate flows and provide a more stable supply to its shareholders (White Sands Water Engineers, 2014). The clay lined, on-ditch equalization pond, has a capacity of 67 acre-feet and utilizes an Obermeyer Hydro, Inc. spillway gate with an inflatable air bladder to control and regulate the amount of water released to shareholders. D&A understands the equalization pond has been effective in regulating deliveries to Western's shareholders located downstream of the equalization pond. However, because it is located downstream of the river headgate, it has done nothing to regulate the fluctuating flows experienced at the river diversion.

As previously mentioned, the Evans No. 2 Ditch company is discussing and evaluating constructing its own equalization pond to dampen the variations in diversions it experiences over the course of a day, especially during the bypass call. Similar to the analyses D&A performed for the RWHTF and Henderson gage hydrographs to determine the approximate storage requirement for storing the peak of the diurnal fluctuation, D&A examined hourly diversion data for the Evans No. 2 Ditch during days it was subject to a bypass call. The analysis indicated that the average volume of water in excess of the average daily flow, or conversely the volume of water required to fill in the trough of the fluctuation, is approximately10 acre-feet. While this is the average volume required, D&A performed an exceedance probability analysis to determine what the equalization volume would be required to store and regulate (i.e., release at constant flow rate) 95 percent of the daily fluctuations experienced by the Evans No. 2 Ditch. The results of the exceedance analysis indicate that a volume of approximately 35 acre-feet would be required to store and regulate 95 percent of the fluctuations.

This alternative is feasible, as evidenced by the Western Mutual Equalization Pond, and is effective in providing a more regular and stable supply to the Western Ditch shareholders. For other ditch systems, this alternative would require adequate land area along the ditch for construction of the pond and ideally a reach of the ditch with adequate fall necessary to create the required storage to dampen irregular diversion rates. D&A does not foresee any significant legal hurdles as the State generally allows for direct flow water rights to be temporarily detained for up to 72 hours in order to allow more efficient or effective beneficial use of the water.

8.1.7 Use of Existing or New River Check Dams

A mitigation alternative presented and discussed at the stakeholder meetings was utilizing an existing river check dam (or diversion dam) and modifying it to regulate the diurnal fluctuation. As D&A understands it, the concept would be to modify an existing river check dam, or construct a new one, so that the usually static (i.e., non-adjustable) check dam spill elevation could be varied by the use of an Obermeyer Hydro, Inc. inflatable dam (or equivalent). When the beginning of the peak of the diurnal fluctuation is present at the check dam, the inflatable dam would be raised so that it would only allow a predetermined average flow rate over the dam, and "store" the peak (i.e., the flow above the average daily flow rate) behind the dam. Therefore, the river and its banks become a temporary on-channel reservoir. As the diurnal fluctuation recedes such that "inflows" are lower than the average, the inflatable dam would lower and release the on-channel storage in order to maintain the flows within the SPR downstream of the check dam at a more constant average daily flow rate.

D&A analyzed the amount of storage potential available within the channel of the SPR at three different locations utilizing the U.S. Army Corps of Engineers (USACE) HEC-RAS model developed for the 2005 Adams County Flood Hazard Area Delineation for the South Platte River (CDM, 2005). The three locations selected for our analysis were 1) the Fulton Ditch diversion dam, 2) a location 8,750 feet downstream of the Brantner Ditch diversion dam, and 3) a location 4,800 feet upstream of the Brighton Ditch diversion dam (see **Figure 13**). The Fulton Ditch diversion dam was selected based on it being an existing check dam located near the top of the Study Reach and the second and third locations were selected similar to how one would select a suitable location for an on-channel reservoir to maximize potential storage (i.e., a broad and flat reach). Due to limitations of the survey used to create cross sections at other existing check dams, D&A did not evaluate any other existing structures beyond the Fulton Ditch check dam. However, the three locations that were evaluated are representative of the channel geometry and the potential storage available within the Study Reach.

HEC-RAS is capable of calculating the volume of water between river cross-sections. Therefore, for each of the three scenarios, D&A incrementally increased the water surface elevation within the model at the cross-section representing the identified location. The rise in the water surface elevation simulates the stage of the river at the crest of the inflatable check dam. By comparing the volume upstream of the cross-section before and after the artificial water surface rise, we determined the amount of available storage upstream of the check dam created by the rise. The results of this analysis are summarized in the following table.

	Storage Behind Incremental Check Dam Rise, Acre-Feet		
Rise In Stage (ft)	Fulton Ditch Diversion Dam	8,750 ft d/s of Brantner Ditch Diversion Dam	4,800 ft u/s of Brighton Ditch Diversion Dam
1	0.13	1.59	0.39
2	0.90	7.36	7.97
3	4.21	13.64	17.98
4	12.08	21.44	30.97
5	25.39	31.19	47.45
6	46.43	47.63	87.64
7	105.22	73.19	

As previously described, the volume necessary to store the peak of the diurnal fluctuation, or the amount of flow above the average daily discharge, is approximately 70 acre-feet. As shown in the table, a fairly significant rise in the water level of the SPR is required to gain enough volume to substantially store the peak of the diurnal fluctuation. For each of the three scenarios, to be able to fully store the peak volume of 70 acre-feet at one location, a six to seven foot check is required.

This alternative presents the potential for significant flooding, riverbank destabilization, and adjacent land issues as a result of a rise and drop of the SPR river stage of this magnitude. The rapid rise and the decline of the river stage on daily basis would likely increase slumping and bank stabilization issues. Furthermore, the increased stage, even temporarily, could create high groundwater issues for neighboring properties. Using HEC-RAS, the flowrate corresponding to a rise of this magnitude ranges from approximately 3,400 to 4,200 cfs or approximately one-third the flow rate of a 10-year frequency storm.

A version of this alternative that would use more than one inflatable check dam could prove more feasible from a physical and engineering standpoint. The concept would be to use multiple inflatable check dams and therefore reduce the storage requirement and corresponding water rise requirement. However, using multiple dams may complicate operations and water rights issues. In addition, based on the three cross sections we analyzed, it would require two to three checks with a water level increase of over 4 feet at each check dam to create the required 70 acre-feet of storage behind the check dams.

Because of the magnitude of the rise required to gain storage volumes necessary to fully or partially mitigate the diurnal fluctuation and the resulting flood and bank stability issues, this alternative is not likely as feasible as other alternatives presented in this report and therefore we did not fully investigate the costs associated with this alternative. However, based on D&A's experience, there are significant costs associated with inflatable check dams ranging in length from 150 to 250 feet, as well as the construction costs associated with retrofitting the inflatable check dam into one or more existing irrigation river diversions. In addition to these costs, significant bank stabilization work would be required on both sides of the SPR for approximately 5,000 to 6,000 feet upstream of the check dam.

There may also be legal issues associated with the temporary impoundment of water behind the check dams. As previously mentioned, the State generally allows for direct flow water rights to be temporarily detained for up to 72 hours in order to allow more efficient or effective beneficial use of the water. However, a check dam on the SPR would create a regional on-channel reservoir that would temporarily detain multiple entities' water rights, reusable supplies, and native flow. D&A is unsure if a decree would be required to operate the variable check dam or if downstream water users would be reluctant to having a portion of their water supplies temporarily detained. Additional legal research would be required to identify all legal issues surrounding this alternative and to assess the feasibility of this alternative.

8.1.8 Utilization of Groundwater Diversions to Offset Diurnal Fluctuation

An alternative was presented by the group of stakeholders that involves the use of wells to supplement ditch diversions during the time of the trough. The conceptual idea involves pumping wells located near the ditch headgates and close to the SPR and discharging them into the ditch to help regulate the flows within the ditch when diversions decrease below a desired flow rate as a result of the trough. The depletions caused by the pumping of the well would be augmented by the ensuing peak of the diurnal fluctuation.

Alluvial wells that have been constructed along the river in this reach of the SPR generally yield between 1 and 2 cfs. Therefore, depending on the individual ditch system and their fluctuations, this alternative may require a significant number of wells. For example, on average the Evans No. 2 Ditch experiences a 24 cfs fluctuation below the average daily flow during the month of July. That means that during the trough of the diurnal fluctuations, their diversion rate is 24 cfs less than the average daily diversion rate. Therefore, to fully supplement the diversions during the trough, it would require approximately 12 wells assuming the upper range of the expected yields (i.e., 2 cfs). Not only are the capital and operational costs associated with this number of wells financially burdensome, the amount of real estate along the river necessary to construct a well field of this magnitude is sizeable. For example, Aurora's current PWP well field consists of 23 wells that stretch over approximately 2 miles of the western bank of the SPR. That being said, strictly from an engineering and physical feasibility standpoint, this alternative may be better suited for the ditch systems with smaller diversion rates and smaller negative departures from their desired daily average flow rates.

There are likely fairly significant legal issues associated with this alternative related to the operation of the wells. D&A understands that the original concept for this alternative was that the supplemental wells would operate as "headgate wells". That is, the wells would be located within close proximity of the SPR (e.g., less than 100 feet) and that their depletions to the river would be assumed to be instantaneous as if the pumping was an immediate diversion from the river. Therefore the pumping depletions would be replaced on the same day with the ditch company's direct flow water rights available within the SPR during the peak of the diurnal fluctuation. However, we understand that the State's position on the approval and administration of headgate wells has changed within the last 5 to 10 years. The State now requires a detailed groundwater modeling analysis that indicates that the wells have depletions within the same day as pumping. If the modeling doesn't support the same-day depletions, the State will require the wells' stream depletions be augmented pursuant to a decreed plan for augmentation. Current modeling methodologies and expert opinions regarding modeling input parameters results in very few wells being classified and administered as headgate wells. Therefore, this alternative would likely require the ditch company to obtain a decreed augmentation plan in order to operate the wells. The augmentation plan would require daily accounting, an augmentation station at the river headgate, and other administrative requirements that would complicate the operation of the supplemental wells.

Given the quantity and expense of the wells required to supplement the troughs of the ditch systems and the requirement, per current water law, of an augmentation plan, this alternative does not appear to be as feasible as others presented within this report.

9.0 IMPROVED MEASUREMENT AND REPORTING

As previously discussed, the current District 2 Water Commissioner's administrative practice for determining the need for a call or bypass call upstream of the Saint Vrain Creek confluence is to: 1) discuss the daily water needs of the Western Ditch with a ditch company representative, since the Western Ditch is most often the swing ditch; 2) examine the low flow trough of the daily hydrograph at the Henderson gage; 3) examine the gaged and known inflows within the reach upstream of the Western Ditch to determine their potential contribution to demand; 4) estimate unmeasured inflows (ungaged surface inflows and groundwater gains) based on weather conditions and experience; and finally 5) initially distribute the water to all in-priority water users, according to their communicated demands. If the Water Commissioner determines the Western's demand will not be completely satisfied, the Water Commissioner will place a bypass call within District 2. The bypass call allows the Water Commissioner to work with upstream junior users so that only a partial curtailment may be required to satisfy the Western Ditch's demands.

As previously mentioned, the Water Commissioner's goal when administering the bypass call is to fully satisfy the Western's calling right while avoiding any spills (i.e., flow over Western's check dam) to the downstream reach. Depending on the accuracy of the estimates of inflow, outflow, demands, etc. when the Water Commissioner makes the initial morning allocation of supply, D&A understands the flow over the Western check dam can be upwards of 10 cfs, plus or minus. The flow over the Western check dam is also dependent on the bypassing ditch's ability to efficiently and responsively chase the diurnal fluctuation.

No record is kept as to the amount of water flowing over the Western check dam. Generally, the presence of flow over the check dam is known only by visual inspection. According to multiple stakeholder statements, the District 2 water users believe the State's, and specifically District 2 Water Commissioner Bill Schneider's, methodology of estimating and administering for the diurnal fluctuation results in only limited spills. Bill Schneider did indicate during stakeholder meetings, as well as during the September 11, 2014 stakeholder field trip that improvements to stream gages as well as additional gages on tributary inflows could aid and improve his allocation of water supplies and further limit spills from this reach.

The following sections discuss the existing streamflow gages, methodology relied upon for administration of this reach, recommended improvements to existing infrastructure, and the potential benefits of gaging additional tributary inflows. It should be noted that the following recommendations, including improving gage measurement and additional real-time reporting, will help maximize the allocation of available flow, but these things by themselves cannot fully mitigate the impacts of the diurnal fluctuation.

9.1 Current Mainstem Stream Flow Gages

The SPR mainstem gages that aid in the administration of the Study Reach include the South Platte River at 654th Avenue, Commerce City, CO gage (PLASIXCO), the previously mentioned South Platte River at Henderson, CO gage (PLAHENCO), the previously mentioned South Platte River near Fort Lupton, Co gage (PLALUPCO), and to a lesser extent, the previously mentioned South Platte River near Kersey, CO gage (PLAKERCO). The 64th Avenue gage and the Fort Lupton gage are owned and maintained by the USGS whereas the Henderson and Kersey gages are owned and maintained by the Colorado Division of Water Resources.

Through discussions with David Nettles, Division 1 Engineer, and Bill Schneider, District 2 Water Commissioner, D&A understands that the State's ability to rely on USGS gages for allocation of flow, especially when the river is low, is difficult. Bill Schneider indicated that the USGS gages are rated and calibrated on 6 to 8 week intervals whereas the State hydrographers rate the Henderson gage at least every 2 weeks and more frequently if a large storm event shifts the fairly sandy bed in the vicinity of the gage. Because the USGS gages are rated less frequently, Bill believes they are at times unreliable. For example, Bill's experience is that at low flows, the 64th Avenue gage can read high by up to 30 cfs, whereas the Fort Lupton gage can underestimate the actual flow of the SPR by 40 to 50 cfs. These discrepancies make it difficult for the State to have sufficient data to fully evaluate the flow conditions at different locations within the Study Reach and allocate water to the District 2 ditches. Both Bill Schneider and David Nettles agreed that more frequent calibration of the USGS gages within this reach would aid the administration of the SPR within the Study Reach, especially in dry months and years when the diurnal fluctuation is problematic for District 2 irrigators.

9.1.1 Improving Existing Mainstem Gages

D&A contacted Mr. Greg Smith with the USGS to discuss the potential of increasing the frequency in which the USGS currently rates and calibrates the 64th Avenue gage and the Fort Lupton gage. Mr. Smith said that those two gages, which are already entirely funded by the Metro District, are currently rated every six to eight weeks. Mr. Smith mentioned that due to the relatively infrequent rating of these gages, he was not surprised the State did not find them to be accurate at low flow rates. Mr. Smith stated that to increase the rating frequency to a bi-weekly event, similar to the State's frequency, the annual cost increase would be approximately $4,000 per gage site.

9.2 Tributary Inflows

D&A understand that the tributary inflows that enter the SPR downstream of the Henderson gage are of particular interest to the Water Commissioner Bill Schneider when making the initial allocation of supply during periods of low flow. Within the Study Reach, these tributary inflows include Big Dry Creek, Little Dry Creek, the Graflin Slough, the Lorentz Slough, and other minor drainages. Big Dry Creek is the only one of these tributary inflows that is gaged (BIGDAFCO). Therefore, most of tributary inflows are not measured, yet at times contribute measurable and meaningful flow rates to the SPR. Little Dry Creek and the Graflin Slough are downstream of the Evans No. 2 Ditch river headgate and therefore available for diversion at the Western Ditch headgate. Without gages, estimates of the inflows are made. If the estimate of these inflows is significantly different from actual flows, or the inflows change significantly during the course of the day, this can result in either over-curtailment of the ditch subject to the bypass call and water being wasted over the Western check dam, or result in the Western Ditch being shorted.

Therefore, to aid with administration, Bill Schneider mentioned it would be useful to gage and instrument some of these larger tributary inflows. It was Mr. Schneider's opinion that a gage on Little Dry Creek and the Graflin Slough would be especially useful in allocating the flow between the Evans No. 2 Ditch and the Western Ditch. He also mentioned that the gage on Big Dry Creek needed some improvements. D&A has since learned the State has moved and replaced the Big Dry Creek gage.

D&A corresponded with Mr. Russell Stroud, Division 1 Lead Hydrographer, regarding the costs associated with construction of stream flow gages. Mr. Stroud provided us with very recent construction cost data for gages destroyed and replaced following the 2013 flood event. D&A reviewed the various projects and cost estimates and concluded that construction of a stream flow gage on a tributary drainage similar to Little Dry Creek would cost between $20,000 and $45,000 depending on the selected grade control structure. The $20,000 to $45,000 total includes construction costs and material costs such as the shelter, stilling well, data logger, and the instrumentation and electronics required for the State's telemetry. It should be noted that these costs are based on State employees designing, bidding, and overseeing the construction of the gages. Costs would likely be higher if designed and constructed by a non-State entity. Information provided by Mr. Stroud indicated that the State's annual maintenance, calibration, and upkeep costs would range from $4,000 to $6,000 per gage site.

9.3 Stage Recorder at Western Ditch River Check Dam

As previously mentioned, the amount of water flowing over the Western's check dam provides the Water Commissioner a visual idea of how closely the initial allocation of water supply is to meeting the irrigation demands based on actual river conditions. That is, if there is a consistent amount of water going over the Western check dam during the time there is a bypass call to the Western, the amount of curtailment occurring at the junior structure (e.g., Evans No. 2) could possibly be relaxed. Conversely, if there is no water flowing over the Western check dam and the Western is still not satisfied, the bypassing structure may need to further curtail its diversions. D&A understands that the State's ability to monitor the Western check dam and the amount of water flowing over it is by visual inspection. Therefore, it has been proposed that a stage recorder be installed at the Western check dam that continuously measures and reports the stage of the SPR at the check dam. Based on the known elevation of the Western check dam, the Water Commissioner could instantaneously monitor the depth of the water above or below the check dam. The State could develop a rating curve of approximate flow rates at given overtopping depths or simply develop operating rules based on the depth of flow over the dam.

Because there are approximately 9.5 miles between the Evans No. 2 and Western river headgates, there is travel time or a lag time between the two locations. D&A's estimate of the flow velocity of the SPR during the low flow periods suggests that the lag time between the two river headgates is approximately 5 to 6 hours. Because of this lag time, the Water Commissioner would not be able to make instantaneous decisions based on the stage recorder but rather take into account the duration of the excess flow over the dam or lack thereof before modifying the call or the bypass rates. Nevertheless, the stage recorder and the State's ability to monitor the amount of water leaving the Study Reach would be a useful tool to aid administration. We understand that the State is in the process of adding a stage recorder and telemetry to the Western Ditch diversion dam.

10.0 CONCLUSIONS

The diurnal fluctuation of the SPR downstream of RWHTF discharge is primarily due to the hourly variations in the plant's effluent discharge that largely mimics the typical fluctuating water use patterns of the municipalities it serves. Because the Study Reach can be considered an effluent-dominated reach for large parts of the year, the resulting diurnal fluctuation of the SPR impacts the water users of District 2. These impacts include down-ditch fluctuations as a result of chasing the peak flow of the SPR, inefficient water use by shareholders, water lost to the lower reaches, and overall shortage of supply. D&A understands that a large majority of the impacts are experienced by the ditch systems subject to a bypass call. However, the administration of the bypass call, including the allocation of water supply, can also lead to the senior calling structure being shorted over the course of the day.

The ditch systems that experience the largest impacts have either constructed an individual on-ditch equalization pond or have begun to discuss the need for one. The District 2 water users that were parties to Case No. 11CW74 expressed a desire for a study to be conducted to determine, among other things, the feasibility of a more regional mitigation alternative capable of attenuating the fluctuations caused by the RWHTF effluent discharge. D&A examined a series of stakeholder developed and proposed mitigation alternatives. A preliminary engineering feasibility analysis was conducted for each of the alternatives as well as the identification of potential legal, institutional, and permitting issues. In addition to the physical mitigation alternatives (e.g., equalization ponds), D&A researched improvements to streamflow gaging and infrastructure that could help with the water supply allocation and administration of the bypass call.

Based on the results of the various analyses and research conducted and described herein, the following conclusions have been developed:

10.1 Analysis of Diurnal Fluctuation Hydrology

1. The RWHTF effluent discharge constitutes a large percentage of the SPR flow downstream of the outfall. On an average daily basis from 2010 through 2014, the effluent makes up approximately 55 percent of the flow at the Henderson gage. The average monthly percentages vary from a maximum of 87 percent in January, to a low of 31 percent in the runoff month of June (**Table 1**).

2. The Metro District's NTP will treat a portion of the wastewater currently treated at the RWHTF. However, growth is expected to occur within the RWHTF service area so that projected effluent amounts at the RWHTF will not be materially different than current amounts. The introduction of the NTP's effluent discharge to the Study Reach, approximately 6.8 miles downstream of the Henderson gage, will not cause the magnitude of low flow trough of the existing diurnal fluctuation to increase and therefore will not trigger increased calls from water users downstream of the NTP outfall.

3. The Metro District's previous studies related to influent flow equalization concluded that flow equalization was not an effective strategy for accomplishing environmental restoration downstream of the outfall.

4. The Metro District's previous studies related to influent flow equalization concluded that approximately 77 acre-feet of storage space was required to regulate influent flow to within a daily variance of less than 10 percent of the average flow and that it did not possess adequate land area on their property to construct a flow equalization pond of this size.

5. The influent diurnal fluctuation experienced by the RWHTF is typical of a wastewater treatment facility with both industrial and nonindustrial (i.e., municipal) contributions.

6. For the years of 2001 through 2014, the average hourly effluent discharge from RWHTF was 200 cfs.

7. The average daily peaking factor of the RWHTF discharge is approximately 1.34. In other words, the peak hourly flow is on average 134 percent of the average hourly flow. This factor remains mostly constant throughout the year (**Table 6**).

8. The average daily trough-to-average factor of the RWHTF discharge is approximately 0.47, or the minimum hourly flow is 47 percent of the average daily flow. This factor remains mostly constant throughout the year (**Table 7**).

9. The average daily volume of RWHTF effluent discharge above the average daily flow is approximately 46 acre-feet. However, due to variations in water use patterns, a volume of approximately 63 acre-feet would be required to store 95 percent of the peaks of the diurnal fluctuations experienced at the RWHTF (**Figure 6**).

10. When not influenced by snowmelt runoff or a storm event, the flow pattern (i.e., hydrograph) of the SPR downstream of the RWHTF is largely influenced by the RWHTF discharge (**Figure 7**).

11. D&A's exceedance analyses indicated that a storage volume of approximately 70 acre-feet is required to store the peak of the diurnal fluctuation present at the Henderson gage during the times the diurnal fluctuation is problematic for water users.

12. Due to the natural river attenuation and the contributions of flow from St. Vrain Creek, the Big Thompson River, and the Cache La Poudre River, there is little to no diurnal fluctuation at the downstream terminus of District 2 as evidenced by the streamflow record provided by the Kersey gage (PLAKERCO).

10.2 Impact of Diurnal Fluctuation on Water Users

1. Until 2006, the historical administration affecting District 2 primarily consisted of bypass calls placed by the Jay Thomas Ditch. Subsequent to PSCo's change of use of the Jay Thomas Ditch rights in 2006, the Western Ditch is now considered the swing ditch.

2. A bypass call is the partial curtailment of a junior upstream right expressed as a call by the junior right bypassing to a named downstream senior right.

3. Recent call records indicate the most frequent bypass call is from the Burlington Ditch's 1885 right or the Evans No. 2 Ditch 1871 right to the Western Ditch headgate. The same call records show an infrequent bypass call of short duration that affects the more junior rights diverted at the Burlington Ditch (i.e., 1908 and 1909), Evans No. 2 Ditch (i.e. 1909 priorities), Fulton Ditch, Brantner Ditch and the Brighton Ditch placed by the Western Ditch necessary to satisfy its August 10, 1871 priority.

4. The Water Commissioner determines the need for a call in District 2, upstream of the Saint Vrain Creek confluence, by: 1) discussing the daily water needs of the Western Ditch with a ditch company representative, 2) examining the low flow "trough" of the daily hydrograph at the Henderson gage, 3) examining gaged and known inflows within the reach upstream of the Western Ditch to determine their potential contribution to demand, 4) estimating the unmeasured inflows (ungaged surface inflows and groundwater gains) based on weather conditions and experience, and finally 5) distributing the water to all in-priority water users, according to their communicated demands, so that the Western's 1871 priority and all intervening water rights are satisfied when the trough of the diurnal flow reaches the Western headgate. If the Water Commissioner determines the Western's demand will not be completely satisfied, the Water Commissioner will place a bypass call within District 2.

5. The Water Commissioner's goal when administering this typical bypass call is to fully satisfy the Western's calling right while avoiding any "spills" (i.e., flow over Western's check dam) to the SPR downstream of the Western Ditch.

6. The call records for the period of 1992 and 2012 indicate that the primary calling structures since 1992 have been the Burlington Ditch, the Jay Thomas Ditch and the Western Ditch. The Western Ditch represents approximately 61 percent of the calls placed above the St. Vrain Creek confluence and downstream of the Burlington Ditch headgate since 2002.

7. The call records indicate that the Burlington Ditch and the Evans No. 2 Ditch are the ditches most frequently impacted by a bypass call. Of the total of 500 bypass calls during the years of 1992 through 2012 affecting the Evans No. 2 rights, 410 were to either the Jay Thomas Ditch or to the Western Ditch. These bypass calls primarily occur during the months of July, August, and September.

8. The bypassing structure is allowed to "chase the peak" of the diurnal fluctuation, or increase its diversions during the rising limb of the diurnal fluctuation. The impact of chasing the peak of the diurnal fluctuation is felt by the ditch company, individual shareholders, and irrigators of the bypassing ditch. That is, the variable flow rates within the ditch over the course of the day create variable water stages and hydraulic heads that make it difficult for the ditch company to allocate water, for irrigators to adjust their farm headgates, and also for the setting of siphon tubes.

9. The "diurnal fluctuation" within the ditch and the resulting down-ditch impacts require additional effort and resource expenditures by the affected ditch company and its shareholders, and reduce the overall efficiency and utilization of the water by the ditch system.

10. The Burlington Ditch and the Evans No. 2 Ditch systems generally experience the largest flow fluctuations (i.e., the difference between the maximum hourly diversion and the minimum hourly diversion on a given day) when subject to a bypass call.

11. The diurnal fluctuation impacts ditch systems other than just the structure subject to the bypass call. For example, depending on the accuracy of the estimates of the inflows, outflows, demands, etc. used in the Water Commissioner's initial morning allocation, the diurnal fluctuation and the administration of the bypass call may result in the senior calling right being shorted over the course of a day.

12. Depending on the accuracy of the initial allocation, the administration of the bypass call may result in water flowing over the Western river check dam. This inefficiency leads to water being lost from the reach that would have otherwise been available and diverted by the District 2 irrigators located above the Western Ditch headgate.

10.3 Preliminary Feasibility of Mitigation Alternatives

10.3.1 Upstream Storage at Chatfield Reservoir

1. The use of Chatfield Reservoir storage and releases as a diurnal flow mitigation measure has some advantages but also some potentially fatal flaws. This alternative has the advantage that the storage structure is already in place. In this alternative, the daily volume released from Chatfield Reservoir would be the same as what would occur historically, but the release rates would be changed throughout the day to help fill in the trough of the diurnal fluctuation. Based on an analysis of historical water supply, the alternative may not be a complete solution due to limited water supply in dry years and certain months. Due to the 22 mile distance between Chatfield Reservoir and the RWHTF outfall, the determination of timed releases and the effects of intervening river operations complicate this alternative. The changes to the State's and Corps' operation of the Chatfield Reservoir outlet works necessary to make this alternative work involve both legal, institutional, and environmental issues that will likely be difficult to overcome. Additional research beyond the scope of this study is required to determine if such changes in the operation of Chatfield Reservoir would be feasible.

10.3.2 Use of Existing Gravel Pit for Flow Equalization Prior to RWHTF Discharge

1. Based on the institutional and legal opinions provided by the Metro District, the alternative of using an existing gravel pit for effluent equalization prior to discharge of the effluent from RWHTF to the SPR has potentially insurmountable legal and regulatory flaws that should effectively remove it from further consideration.

10.3.3 Use of Existing Gravel Pit between RWHTF and Western Ditch Headgate

1. The use of an existing gravel pit downstream of the RWHTF outfall to divert and regulate the diurnal fluctuation is a potentially feasible solution. There are sizable capital costs (e.g., $8.3 million) associated with this alternative related to the potential reimbursement of storage costs, construction of a pump station, and possible improvements to inflow infrastructure (e.g., adding constructed capacity to an existing ditch). In addition to capital costs, the estimated annual energy costs to operate a pump station to deliver temporarily stored water from the gravel pit to the SPR would be approximately $42,000 to $45,000.

2. Potential legal issues associated with the use of a gravel pit to divert the peak of the diurnal fluctuation off of the SPR for temporary storage and retiming include the need for a water court decree and likely the cooperation of the District 2 water users that would have a portion of their water rights temporarily detained by the upstream equalization pond.

10.3.4 Construction of a New Gravel Pit

1. The construction and use of a new gravel pit for the purpose of an equalization pond would have similar costs and legal issues associated with that of an existing gravel pit.

10.3.5 Use of Storage and Timed Releases by Parties using RWHTF Effluent

1. This alternative considers the use of storage and timed releases by parties with gravel pit storage facilities who store and release augmentation and substitute supply water within the Study Reach to help offset the trough of the diurnal fluctuation. This alternative would require the participating water users and the State to develop an operating procedure in which the water users would agree to make strategically timed releases of substitute supplies during the trough of the SPR diurnal fluctuation. This alternative may be feasible but its effectiveness may be limited, at least in the short term due to the limited number of parties that store and release augmentation water and substitute supplies in the Study Reach. If future operations increase the amounts and consistency of augmentation and releases within this reach via gravel pits , this alternative could be an effective mitigation alternative.

10.3.6 Storage Near Ditch Headgates

1. Use of an equalization pond downstream of ditch companies' river headgates are a proven method to better regulate deliveries to downstream shareholders as evidenced by the Western Mutual Equalization Pond. While these equalization ponds are effective at regulating flows to shareholders, they do not mitigate the fluctuations experienced at the river headgates or reduce the potential of being shorted by upstream water users if over-diversion occurs during a bypass call.

10.3.7 Use of Existing or New River Check Dams

1. This alternative mitigation proposal would utilize existing river check dam(s) that are modified to regulate the diurnal fluctuation, or constructing a new river check dam. An increase in the water surface elevation of the SPR of 6 to 7 feet above historical check dam

water levels would be necessary to create storage behind the check dams sufficient to regulate the diurnal fluctuation. This alternative presents the potential for significant flooding, riverbank destabilization, and adjacent land issues as a result of a rise and drop of the SPR river stage of this magnitude. Because of these issues, D&A did not fully investigate the costs associated with this alternative.

10.3.8 Utilization of Groundwater Diversions to Offset Diurnal Fluctuation

1. This alternative considers the use of groundwater diversions (i.e., wells) to offset the diurnal fluctuation experienced by individual ditch systems. In our opinion, the legal issues surrounding the use of headgate wells would likely require the well users to obtain a water court decreed plan for augmentation. Furthermore, based on the magnitude of the troughs experienced by the majority of the ditch companies, this alternative would likely only be feasible for a few ditch systems that have smaller fluctuations. Given the quantity and expense of the wells required to supplement the troughs of the ditch systems and the requirement, per current water law, of an augmentation plan, this alternative does not appear as feasible as others presented within this report.

10.3.9 Improved Measurement and Reporting

1. Based on information gathered from discussions with the Division Engineer and District 2 Water Commissioner, there could be some improvements made to the measurement and reporting of flows in the SPR and its tributaries that would assist in the administration of the diversions in the Study Reach, especially during times of low flow when the diurnal fluctuation is problematic. This would be accomplished with the enhancement of existing streamflow gages and the construction of gaging on currently ungaged tributary inflows. The improvements to existing streamflow gages may include the more frequent calibration and rating of the two USGS gages within this reach (i.e., 64th Avenue and Fort Lupton gages) so that they are more reliable in low flow conditions.

   In addition to improving existing gages, adding gaging instrumentation and infrastructure to currently ungaged tributaries such as Little Dry Creek and the Graflin Slough would provide the Water Commissioner more definitive information as to the amount of water supply available for allocation. While this alternative will do nothing to mitigate the physical diurnal fluctuation, the improvements represent fairly low cost options to help reduce the impacts created by the diurnal fluctuation. However, these improvements alone will not be sufficient to mitigate all the negative impacts of the diurnal fluctuation on water diverters within the Study Reach.

   The total capital cost of the recommended gage improvements ranges from $40,000 to $90,000. This cost includes constructing streamflow gages on Little Dry Creek and the Graflin Slough. The total annual operations and maintenance costs associated with these gages is approximately $8,000 to $12,000. The additional cost associated with funding the bi-weekly calibration of the USGS gages at 64th Avenue and Fort Lupton, CO would be approximately $8,000 per year.

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TABLES

TABLE 1
RWHTF Discharge as Percentage of South Platte River Flows
South Platte River immediately above Fulton Ditch Headgate

(values in CFS)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2010	ND	ND	ND	ND	ND	22%	37%	36%	63%	75%	55%	83%
2011	97%	97%	92%	69%	58%	31%	17%	41%	64%	66%	51%	88%
2012	99%	77%	72%	55%	51%	55%	58%	64%	60%	77%	77%	91%
2013	82%	80%	74%	75%	48%	30%	38%	46%	25%	42%	48%	66%
2014	72%	76%	67%	57%	33%	16%	25%	41%	52%	21%	31%	31%
Average	**87%**	**83%**	**76%**	**64%**	**48%**	**31%**	**35%**	**46%**	**53%**	**56%**	**52%**	**72%**
Min	**72%**	**76%**	**67%**	**55%**	**33%**	**16%**	**17%**	**36%**	**25%**	**21%**	**31%**	**31%**
Max	**99%**	**97%**	**92%**	**75%**	**58%**	**55%**	**58%**	**64%**	**64%**	**77%**	**77%**	**91%**

ND = No data used. Study period began 6/1/2010

TABLE 2

RWHTF Effluent Discharge (METSEWCO)

Monthly Average of Daily Flow

(values in CFS)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Average
2010	ND	ND	ND	ND	ND	238	227	216	204	207	208	198	214
2011	207	209	198	193	224	222	233	207	201	202	204	202	209
2012	206	211	200	193	198	195	195	190	189	203	212	209	200
2013	196	195	200	205	213	199	195	200	230	208	200	198	203
2014	198	202	204	201	222	215	211	216	175	203	203	194	204
Average	**202**	**204**	**200**	**198**	**214**	**214**	**212**	**206**	**200**	**205**	**205**	**200**	**205**
Min	**196**	**195**	**198**	**193**	**198**	**195**	**195**	**190**	**175**	**202**	**200**	**194**	**200**
Max	**207**	**211**	**204**	**205**	**224**	**238**	**233**	**216**	**230**	**208**	**212**	**209**	**214**

ND = No data used. Study period began 6/1/2010

(values in Acre-Feet[1])

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
2010	ND	ND	ND	ND	ND	14,200	13,900	13,300	12,100	12,700	12,400	12,200	ND
2011	12,700	11,600	12,200	11,500	13,800	13,200	14,300	12,800	12,000	12,400	12,100	12,400	151,000
2012	12,700	11,700	12,300	11,500	12,200	11,600	12,000	11,700	11,200	12,500	12,600	12,900	144,900
2013	12,100	10,800	12,300	12,200	13,100	11,800	12,000	12,300	13,700	12,800	11,900	12,200	147,200
2014	12,200	11,200	12,500	12,000	13,700	12,800	13,000	13,300	10,400	12,500	12,100	12,000	147,700
Average	**12,400**	**11,300**	**12,300**	**11,800**	**13,200**	**12,700**	**13,000**	**12,700**	**11,900**	**12,600**	**12,200**	**12,300**	**147,700**
Min	**12,100**	**10,800**	**12,200**	**11,500**	**12,200**	**11,600**	**12,000**	**11,700**	**10,400**	**12,400**	**11,900**	**12,000**	**144,900**
Max	**12,700**	**11,700**	**12,500**	**12,200**	**13,800**	**14,200**	**14,300**	**13,300**	**13,700**	**12,800**	**12,600**	**12,900**	**151,000**

[1] Rounded to the nearest 100 acre-foot)

TABLE 3

Monthly Average of Daily Maximum RWHTF Discharge

(cfs)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Average
2001	189	204	185	182	201	190	196	191	191	182	231	237	198
2002	244	206	200	282	289	290	280	282	283	277	281	275	266
2003	267	270	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
2004	ND	ND	259	260	266	260	267	277	268	270	271	212	261
2005	212	257	234	269	274	281	266	279	266	271	279	274	264
2006	220	ND	267	266	267	260	273	270	273	266	275	276	265
2007	285	292	287	301	314	291	282	288	290	280	276	273	288
2008	272	274	274	271	281	270	260	ND	ND	283	286	285	276
2009	278	278	273	303	305	313	298	294	288	289	303	286	292
2010	283	282	295	311	318	303	295	285	280	279	278	269	290
2011	277	277	268	264	295	286	307	270	272	271	274	274	278
2012	279	283	268	263	266	257	257	256	260	269	286	287	269
2013	266	267	270	273	281	262	260	266	304	276	272	272	272
2014	263	268	270	267	291	276	274	281	243	271	275	265	270
Average	**257**	**263**	**258**	**270**	**280**	**272**	**270**	**270**	**268**	**268**	**276**	**268**	**268**

ND indicates either no data or incomplete data available.

Table values derived from Metro Sewer Effluent at Denver (METSEWCO) records.

TABLE 4

Monthly Average of Daily Minimum RWHTF Discharge

(cfs)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Average
2001	55	55	56	62	81	81	89	86	74	68	86	84	73
2002	81	74	66	84	104	104	102	97	93	90	90	89	89
2003	87	87	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
2004	ND	ND	81	67	73	91	107	112	95	96	89	60	87
2005	59	83	69	88	97	108	100	106	87	97	90	94	90
2006	67	ND	84	82	87	86	101	96	88	88	84	88	86
2007	89	98	97	102	128	117	108	108	99	90	91	89	101
2008	91	87	86	86	95	97	86	ND	ND	100	95	100	92
2009	95	90	86	107	119	140	125	110	104	105	116	107	109
2010	104	102	106	128	135	132	120	109	96	96	98	95	110
2011	105	112	85	80	114	119	126	102	97	96	96	98	102
2012	98	100	89	85	92	90	93	87	82	86	97	98	92
2013	85	85	85	93	103	88	88	91	122	99	91	93	93
2014	90	93	93	95	115	111	107	111	80	95	99	94	99
Average	**85**	**89**	**83**	**89**	**103**	**105**	**104**	**101**	**93**	**93**	**94**	**92**	**94**

ND indicates either no data or incomplete data available.

Table values derived from Metro Sewer Effluent at Denver (METSEWCO) records.

TABLE 5

Monthly Average of Daily Difference between Maximum and Minimum Hourly RWHTF Discharges

(cfs)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Average
2001	135	149	129	120	119	109	107	105	116	114	146	152	**125**
2002	163	133	134	198	185	186	178	185	190	187	191	186	**176**
2003	180	184	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	**182**
2004	ND	ND	178	193	193	170	160	165	173	174	182	152	**174**
2005	153	174	165	180	177	174	167	173	179	174	189	180	**174**
2006	153	ND	184	184	180	174	172	174	185	179	191	188	**179**
2007	195	193	190	200	186	174	175	180	192	190	184	183	**187**
2008	181	187	187	186	186	173	174	ND	ND	183	192	184	**183**
2009	182	188	187	196	186	173	174	184	184	184	188	179	**184**
2010	179	180	189	183	183	171	175	176	184	183	180	175	**180**
2011	172	165	182	185	181	167	181	168	175	174	179	177	**176**
2012	180	182	179	178	174	167	165	169	179	183	189	188	**178**
2013	181	182	185	180	178	174	172	175	182	177	181	179	**179**
2014	173	175	178	172	175	165	167	171	163	176	176	171	**172**
Average	**171**	**174**	**174**	**181**	**177**	**168**	**167**	**169**	**175**	**175**	**182**	**177**	**174**
Min	**135**	**133**	**129**	**120**	**119**	**109**	**107**	**105**	**116**	**114**	**146**	**152**	**124**
Max	**195**	**193**	**190**	**200**	**193**	**186**	**181**	**185**	**192**	**190**	**192**	**188**	**190**

ND indicates either no data or incomplete data available.
Table values derived from Metro Sewer Effluent at Denver (METSEWCO) records.

TABLE 6

Monthly Average of Daily RWHTF Effluent Peaking Factors

(Peak Discharge / Average Daily Discharge)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Average
2001	1.43	1.44	1.36	1.36	1.30	1.27	1.26	1.27	1.33	1.32	1.40	1.44	**1.35**
2002	1.40	1.37	1.39	1.37	1.34	1.34	1.34	1.36	1.39	1.37	1.41	1.42	**1.37**
2003	1.39	1.41	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	**1.40**
2004	ND	ND	1.41	1.38	1.39	1.32	1.31	1.33	1.37	1.36	1.40	1.16	**1.34**
2005	1.15	1.40	1.31	1.36	1.36	1.32	1.33	1.33	1.38	1.35	1.41	1.41	**1.34**
2006	1.17	ND	1.35	1.36	1.36	1.33	1.32	1.31	1.38	1.36	1.40	1.39	**1.34**
2007	1.37	1.34	1.35	1.34	1.32	1.28	1.29	1.28	1.37	1.38	1.40	1.41	**1.34**
2008	1.36	1.39	1.39	1.40	1.38	1.33	1.34	ND	ND	1.35	1.41	1.37	**1.37**
2009	1.35	1.39	1.38	1.35	1.33	1.26	1.29	1.34	1.37	1.34	1.35	1.34	**1.34**
2010	1.34	1.34	1.32	1.31	1.30	1.27	1.30	1.32	1.37	1.35	1.34	1.36	**1.33**
2011	1.34	1.32	1.35	1.37	1.32	1.29	1.32	1.30	1.35	1.34	1.35	1.36	**1.33**
2012	1.35	1.34	1.34	1.36	1.34	1.32	1.32	1.35	1.38	1.31	1.35	1.37	**1.34**
2013	1.36	1.37	1.35	1.33	1.32	1.32	1.33	1.33	1.33	1.33	1.36	1.37	**1.34**
2014	1.33	1.33	1.33	1.32	1.31	1.28	1.30	1.30	1.20	1.33	1.35	1.36	**1.31**
Average	**1.33**	**1.37**	**1.36**	**1.35**	**1.34**	**1.30**	**1.31**	**1.32**	**1.35**	**1.35**	**1.38**	**1.37**	**1.34**
Min	**1.15**	**1.32**	**1.31**	**1.31**	**1.30**	**1.26**	**1.26**	**1.27**	**1.20**	**1.31**	**1.34**	**1.16**	**1.27**
Max	**1.43**	**1.44**	**1.41**	**1.40**	**1.39**	**1.34**	**1.34**	**1.36**	**1.39**	**1.38**	**1.41**	**1.44**	**1.39**

ND indicates either no data or incomplete data available.

Table values derived from Metro Sewer Effluent at Denver (METSEWCO) records.

TABLE 7

Monthly Average of Daily RWHTF Effluent "Trough-to-Average" Factors

(Minimum Discharge / Average Daily Discharge)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Average
2001	0.42	0.39	0.41	0.46	0.53	0.54	0.57	0.58	0.52	0.49	0.52	0.51	**0.49**
2002	0.46	0.49	0.46	0.41	0.48	0.48	0.49	0.47	0.46	0.45	0.45	0.46	**0.46**
2003	0.45	0.45	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	**0.45**
2004	ND	ND	0.44	0.35	0.37	0.46	0.52	0.54	0.49	0.48	0.46	0.32	**0.44**
2005	0.32	0.45	0.38	0.44	0.48	0.51	0.50	0.50	0.45	0.48	0.46	0.48	**0.46**
2006	0.35	ND	0.43	0.42	0.44	0.44	0.48	0.47	0.44	0.45	0.43	0.44	**0.44**
2007	0.43	0.45	0.46	0.45	0.54	0.52	0.49	0.48	0.47	0.44	0.46	0.46	**0.47**
2008	0.46	0.44	0.44	0.44	0.47	0.48	0.45	ND	ND	0.48	0.47	0.48	**0.46**
2009	0.46	0.45	0.43	0.48	0.52	0.56	0.54	0.50	0.49	0.49	0.52	0.50	**0.49**
2010	0.49	0.48	0.47	0.54	0.55	0.55	0.53	0.50	0.47	0.47	0.47	0.48	**0.50**
2011	0.51	0.54	0.43	0.41	0.50	0.53	0.54	0.49	0.48	0.48	0.47	0.48	**0.49**
2012	0.48	0.48	0.45	0.44	0.46	0.46	0.48	0.46	0.43	0.42	0.46	0.47	**0.46**
2013	0.43	0.44	0.42	0.45	0.48	0.44	0.45	0.45	0.51	0.47	0.46	0.47	**0.46**
2014	0.46	0.46	0.45	0.47	0.52	0.51	0.51	0.51	0.39	0.47	0.49	0.48	**0.48**
Average	**0.44**	**0.46**	**0.44**	**0.44**	**0.49**	**0.50**	**0.50**	**0.50**	**0.47**	**0.47**	**0.47**	**0.47**	**0.47**
Min	**0.32**	**0.39**	**0.38**	**0.35**	**0.37**	**0.44**	**0.45**	**0.45**	**0.39**	**0.42**	**0.43**	**0.32**	**0.39**
Max	**0.51**	**0.54**	**0.47**	**0.54**	**0.55**	**0.56**	**0.57**	**0.58**	**0.52**	**0.49**	**0.52**	**0.51**	**0.53**

ND indicates either no data or incomplete data available.

Table values derived from Metro Sewer Effluent at Denver (METSEWCO) records.

TABLE 8

Hydrograph Characteristics on Days when Evans No. 2 Ditch Subject to a Bypass Call

Hydrograph	Amplitude, cfs		
	Average-to-Trough	Average-to-Peak	Peak-to-Trough
RWHTF Outfall	101	59	160
SPR @ Henderson Gage	77	36	113
SPR @ Fort Lupton Gage	39	22	61
SPR @ Kersey	4	4	8

TABLE 9

Monthly Average of Daily Volumes at South Platte River
Below Chatfield Reservoir Gage (PLACHACO) Available to Fill In the RWHTF Trough-Below-Average
(acre-feet)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1986	ND	ND	ND	ND	ND	ND	104	483	204	15	179	0
1987	28	7	135	680	3,903	2,055	580	438	120	197	83	3
1988	13	130	194	413	540	570	794	474	72	45	138	12
1989	7	31	128	252	290	477	760	450	76	38	9	137
1990	9	0	11	101	365	340	445	313	86	72	203	15
1991	3	1	33	194	231	280	244	389	79	1	69	6
1992	80	95	184	336	297	305	194	150	1	1	134	2
1993	70	101	98	201	270	328	192	139	31	22	116	32
1994	12	41	83	304	462	376	96	99	42	10	74	15
1995	38	38	67	116	1,183	3,506	4,264	777	165	19	163	140
1996	6	33	27	202	281	401	439	184	83	4	88	17
1997	2	70	80	106	355	1,009	512	695	95	61	104	109
1998	121	122	197	650	1,668	540	734	750	144	189	12	1
1999	22	12	87	187	1,535	2,713	1,138	964	82	37	81	80
2000	103	86	103	237	438	439	273	135	28	28	25	55
2001	77	119	94	90	297	283	333	160	61	0	2	0
2002	4	43	93	17	33	95	5	1	1	6	2	11
2003	8	4	90	463	412	326	200	151	191	7	4	6
2004	26	64	31	193	196	166	488	299	64	45	68	19
2005	0	10	22	435	1,004	619	149	259	61	48	107	13
2006	2	28	43	60	262	261	611	444	183	265	67	2
2007	2	83	703	977	3,432	1,732	839	668	325	98	123	88
2008	32	113	230	341	435	605	686	318	107	21	2	7
2009	74	26	46	219	660	1,683	614	170	60	156	0	87
2010	72	69	101	661	840	676	213	580	20	16	69	59
2011	82	88	68	71	41	232	932	337	43	4	0	0
2012	74	75	74	53	35	30	58	38	17	18	1	0
2013	5	21	30	56	173	129	106	88	57	5	23	80
Average	**35**	**57**	**125**	**280**	**601**	**526**	**403**	**270**	**92**	**53**	**36**	**29**
Min	**0**	**4**	**22**	**17**	**33**	**30**	**5**	**1**	**1**	**0**	**0**	**0**
Max	**82**	**119**	**703**	**977**	**3,432**	**1,732**	**932**	**668**	**325**	**265**	**123**	**88**

ND indicates either no data or incomplete data available.

\## Months between March and Octoboer with average daily volumes less than 70 acre-feet needed to fill in trough of diurnal fluctuation.

TABLE 10

Monthly Average of Daily Volumes at South Platte River Below Chatfield Reservoir Gage (PLACHACO) Available to Fill In the RWHTF Trough-Below-Peak

(acre-feet)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1986	ND	ND	ND	ND	ND	ND	104	483	204	15	179	0
1987	28	7	135	680	3,903	2,055	580	438	120	197	83	3
1988	13	130	194	413	540	570	794	474	72	45	138	12
1989	7	31	128	252	290	477	760	450	76	38	9	137
1990	9	0	11	101	365	340	445	313	86	72	203	15
1991	3	1	33	194	231	280	244	389	79	1	69	6
1992	80	95	184	336	297	305	194	150	1	1	134	2
1993	70	101	98	201	270	328	192	139	31	22	116	32
1994	12	41	83	304	462	376	96	99	42	10	74	15
1995	38	38	67	116	1,183	3,506	4,264	777	165	19	163	140
1996	6	33	27	202	281	401	439	184	83	4	88	17
1997	2	70	80	106	355	1,009	512	695	95	61	104	109
1998	121	122	197	650	1,668	540	734	750	144	189	12	1
1999	22	12	87	187	1,535	2,713	1,138	964	82	37	81	80
2000	103	86	103	237	438	439	273	135	28	28	25	55
2001	77	119	94	90	297	283	333	160	61	0	2	0
2002	4	43	93	17	33	95	5	1	1	6	2	11
2003	8	4	90	463	412	326	200	151	191	7	4	6
2004	26	64	31	193	196	166	488	299	64	45	68	19
2005	0	10	22	435	1,004	619	149	259	61	48	107	13
2006	2	28	43	60	262	261	611	444	183	265	67	2
2007	2	83	703	977	3,432	1,732	839	668	325	98	123	88
2008	32	113	230	341	435	605	686	318	107	21	2	7
2009	74	26	46	219	660	1,683	614	170	60	156	0	87
2010	72	69	101	661	840	676	213	580	20	16	69	59
2011	82	88	68	71	41	232	932	337	43	4	0	0
2012	74	75	74	53	35	30	58	38	17	18	1	0
2013	5	21	30	56	173	129	106	88	57	5	23	80
Average	**35**	**57**	**125**	**280**	**601**	**526**	**403**	**270**	**92**	**53**	**36**	**29**
Min	**0**	**4**	**22**	**17**	**33**	**30**	**5**	**1**	**1**	**0**	**0**	**0**
Max	**82**	**119**	**703**	**977**	**3,432**	**1,732**	**932**	**668**	**325**	**265**	**123**	**88**

ND indicates either no data or incomplete data available.

\## Months between March and Octoboer with average daily volumes less than 197 acre-feet needed to fill in trough of diurnal fluctuation.

TABLE 11

Gravel Pit Pump Station Capital Costs

Item	Quantities	Unit Cost	Capital Cost
48" Pipe Length (lf)	500	$10/in/lf	$240,000
Installed Pump Horsepower Requirement (hp)	539 [1]	$15,000/ hp[2]	$8,100,000
Total Cost			**$8,300,000**

[1] The installation horsepower requirement is calculated using the maximum flow rate, (80 cfs) and the maximum total dynamic head of approximately 42-ft (includes maximum static height of 30-ft and friction losses.)

[2] Price per horsepower includes costs associated with pumps, pump vault, inflow piping, instrumentation and controls.

TABLE 12

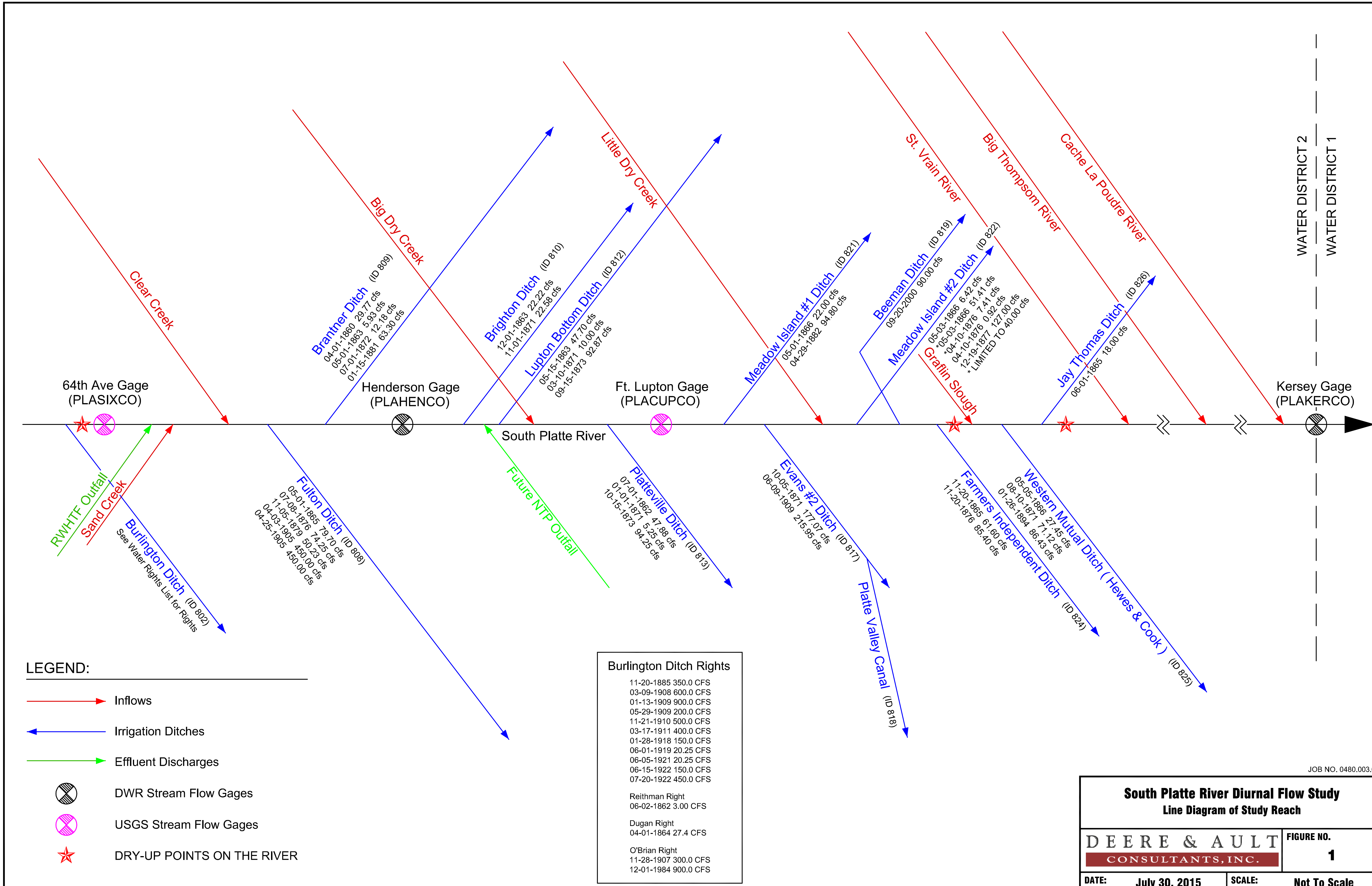
Pump Station Annual Energy Costs

	Average Year Annual Energy Cost	Maximum Year Annual Energy Cost
Days of Operation	81	148
Pumping Flow Rate (cfs)	52	52
Pipe Diameter (in)	48	48
Pipe Length (ft)	500	500
Elevation Change (ft)	20	20
Average Horsepower Requirement (hp)[1]	254	254
Energy Demand Cost[2]	$34,000	$34,000
Energy Usage Cost	$4,000	$7,000
Energy Service Charge	$4,000	$4,000
Total Cost	$42,000	$45,000

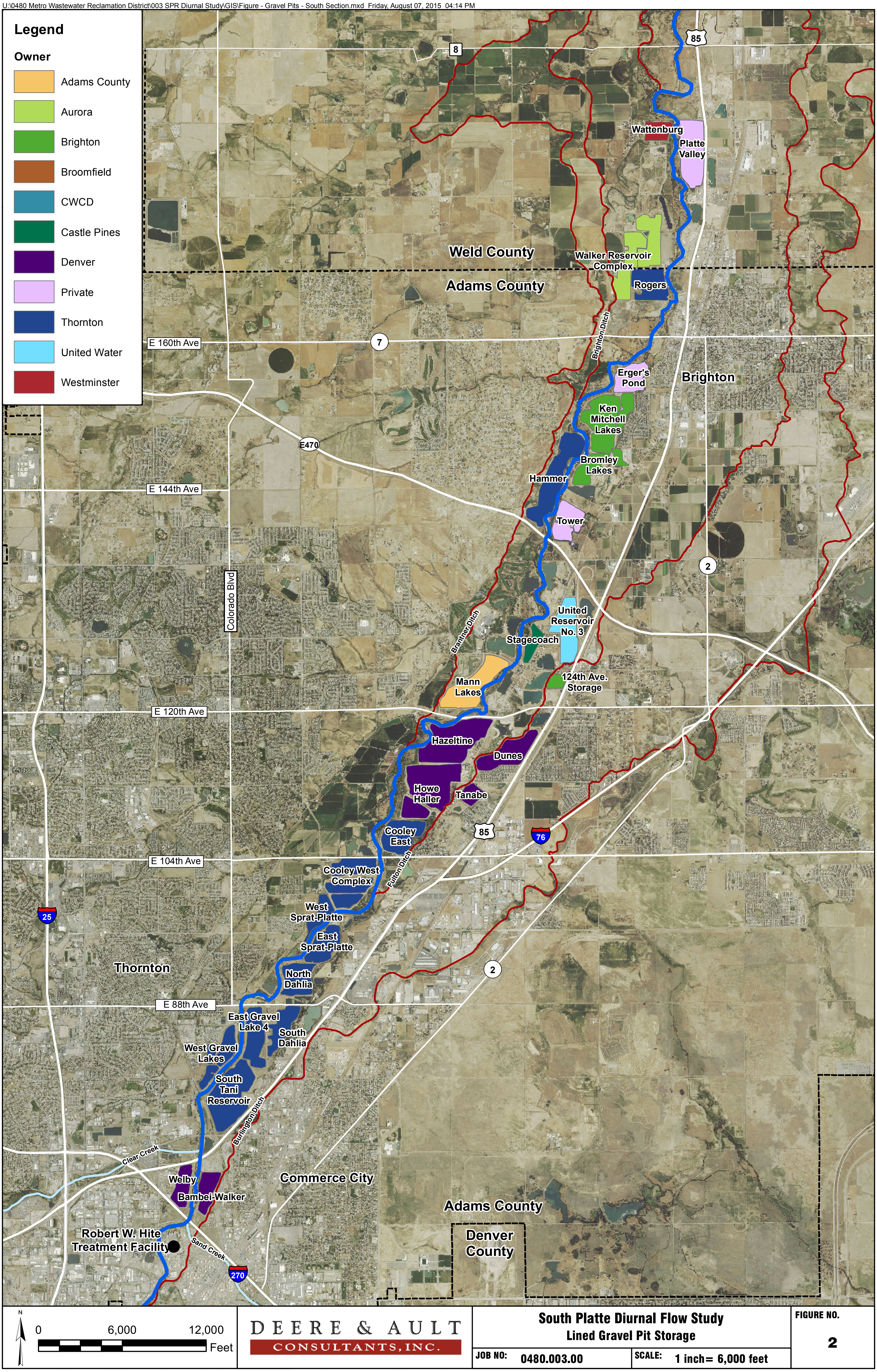
[1] The average annual energy cost determined using the horsepower requirement of pumping the average flow rate of approx. 52 cfs at a total dynamic head of approx. 30-ft.

[2] The annual energy demand cost was assumed to be the same for both scenarios (i.e., pumping occurring at some point in the months of April through September).

FIGURES



South Platte River
64th Ave Gage
(PLASIXCO)
Henderson Gage
(PLAHENCO)
Ft. Lupton Gage
(PLACUPCO)
Kersey Gage
(PLAKERCO)
WATER DISTRICT 2
WATER DISTRICT 1
Clear Creek
Big Dry Creek
Little Dry Creek
St. Vrain River
Big Thompson River
Cache La Poudre River
Graflin Slough
Sand Creek
RWHTF Outfall
Future NTP Outfall
Brantner Ditch (ID 809)
04-01-1860 29.77 cfs
05-01-1863 5.93 cfs
07-01-1872 12.18 cfs
01-15-1881 63.30 cfs
Brighton Ditch (ID 810)
12-01-1863 22.22 cfs
11-01-1871 22.58 cfs
Lupton Bottom Ditch (ID 812)
05-15-1863 47.70 cfs
03-10-1871 10.00 cfs
09-15-1873 92.87 cfs
Meadow Island #1 Ditch (ID 821)
05-01-1866 22.00 cfs
04-29-1882 94.80 cfs
Beeman Ditch (ID 819)
09-20-2000 90.00 cfs
Meadow Island #2 Ditch (ID 822)
05-03-1866 6.42 cfs
*05-03-1866 51.41 cfs
*04-10-1876 7.41 cfs
04-10-1876 0.92 cfs
12-19-1877 127.00 cfs
* LIMITED TO 40.00 cfs
Jay Thomas Ditch (ID 826)
06-01-1865 18.00 cfs
Burlington Ditch (ID 802)
See Water Rights List for Rights
Fulton Ditch (ID 808)
05-01-1865 79.70 cfs
07-08-1876 74.25 cfs
11-05-1879 50.23 cfs
04-03-1905 450.00 cfs
04-25-1905 450.00 cfs
Platteville Ditch (ID 813)
07-01-1862 47.88 cfs
01-01-1871 5.25 cfs
10-15-1873 94.25 cfs
Evans #2 Ditch (ID 817)
10-05-1871 177.07 cfs
06-09-1909 215.95 cfs
Platte Valley Canal (ID 818)
Farmers Independent Ditch (ID 824)
11-20-1865 61.60 cfs
11-20-1876 85.40 cfs
Western Mutual Ditch (Hewes & Cook) (ID 825)
05-05-1866 27.45 cfs
08-10-1871 71.12 cfs
01-26-1894 86.43 cfs
LEGEND:
Inflows
Irrigation Ditches
Effluent Discharges
DWR Stream Flow Gages
USGS Stream Flow Gages
DRY-UP POINTS ON THE RIVER
Burlington Ditch Rights
11-20-1885 350.0 CFS
03-09-1908 600.0 CFS
01-13-1909 900.0 CFS
05-29-1909 200.0 CFS
11-21-1910 500.0 CFS
03-17-1911 400.0 CFS
01-28-1918 150.0 CFS
06-01-1919 20.25 CFS
06-05-1921 20.25 CFS
06-15-1922 150.0 CFS
07-20-1922 450.0 CFS
Reithman Right
06-02-1862 3.00 CFS
Dugan Right
04-01-1864 27.4 CFS
O'Brian Right
11-28-1907 300.0 CFS
12-01-1984 900.0 CFS
JOB NO. 0480.003.00
South Platte River Diurnal Flow Study
Line Diagram of Study Reach
DEERE & AULT
CONSULTANTS, INC.
FIGURE NO.
1
DATE:
July 30, 2015
SCALE:
Not To Scale



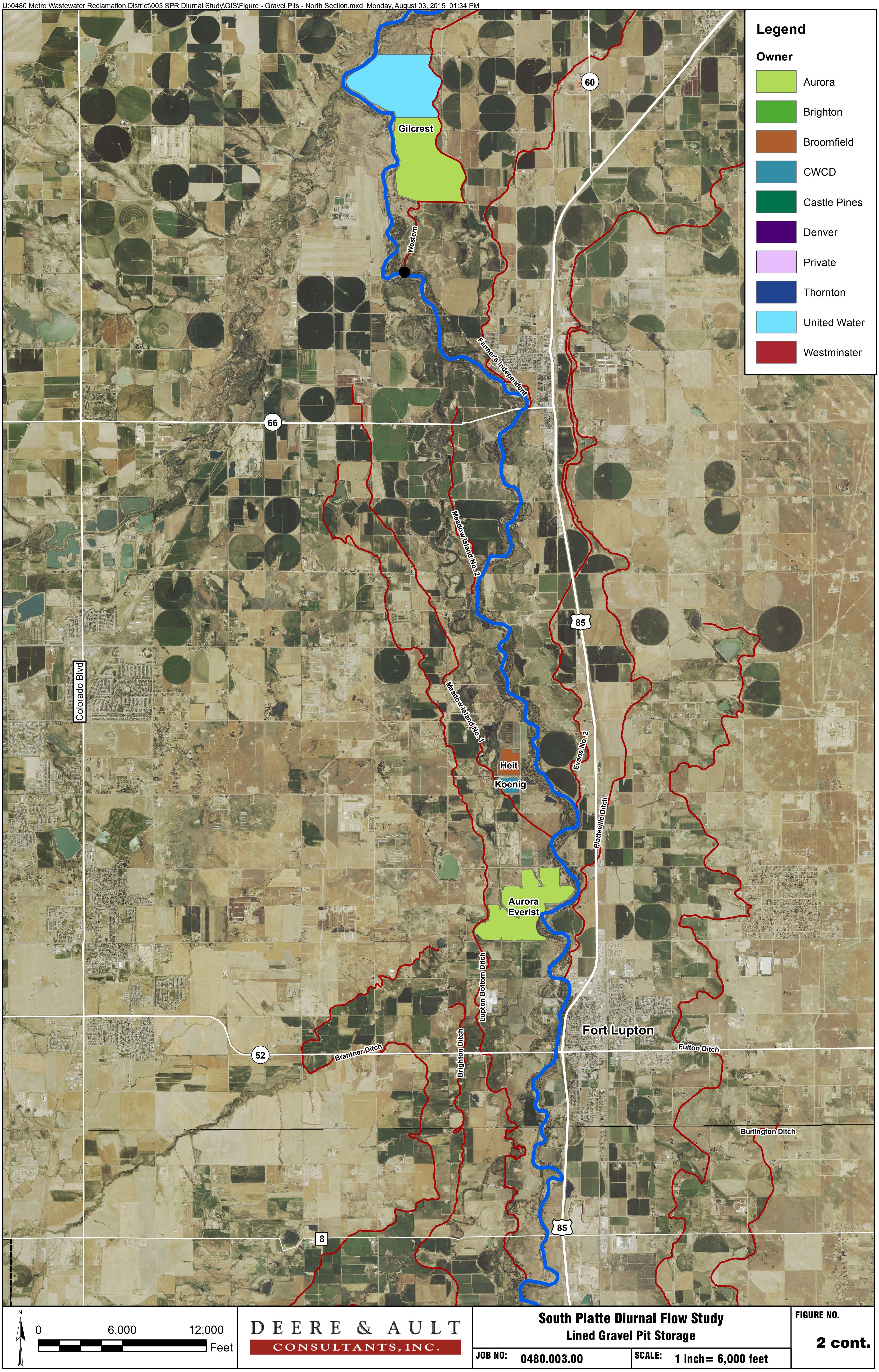
DEERE & AULT
CONSULTANTS, INC.

South Platte Diurnal Flow Study
Lined Gravel Pit Storage

JOB NO:	0480.003.00	SCALE:	1 inch= 6,000 feet

FIGURE NO. 2

U:\0480 Metro Wastewater Reclamation District\003 SPR Diurnal Study\GIS\Figure - Gravel Pits - North Section.mxd Monday, August 03, 2015 01:34 PM



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CONSULTANTS, INC.

South Platte Diurnal Flow Study
Lined Gravel Pit Storage

JOB NO:	0480.003.00	SCALE:	1 inch= 6,000 feet

FIGURE NO.
2 cont.

FIGURE 3

Robert W. Hite Treatment Facility

Average Hourly Influent vs Effluent Discharges for 2011

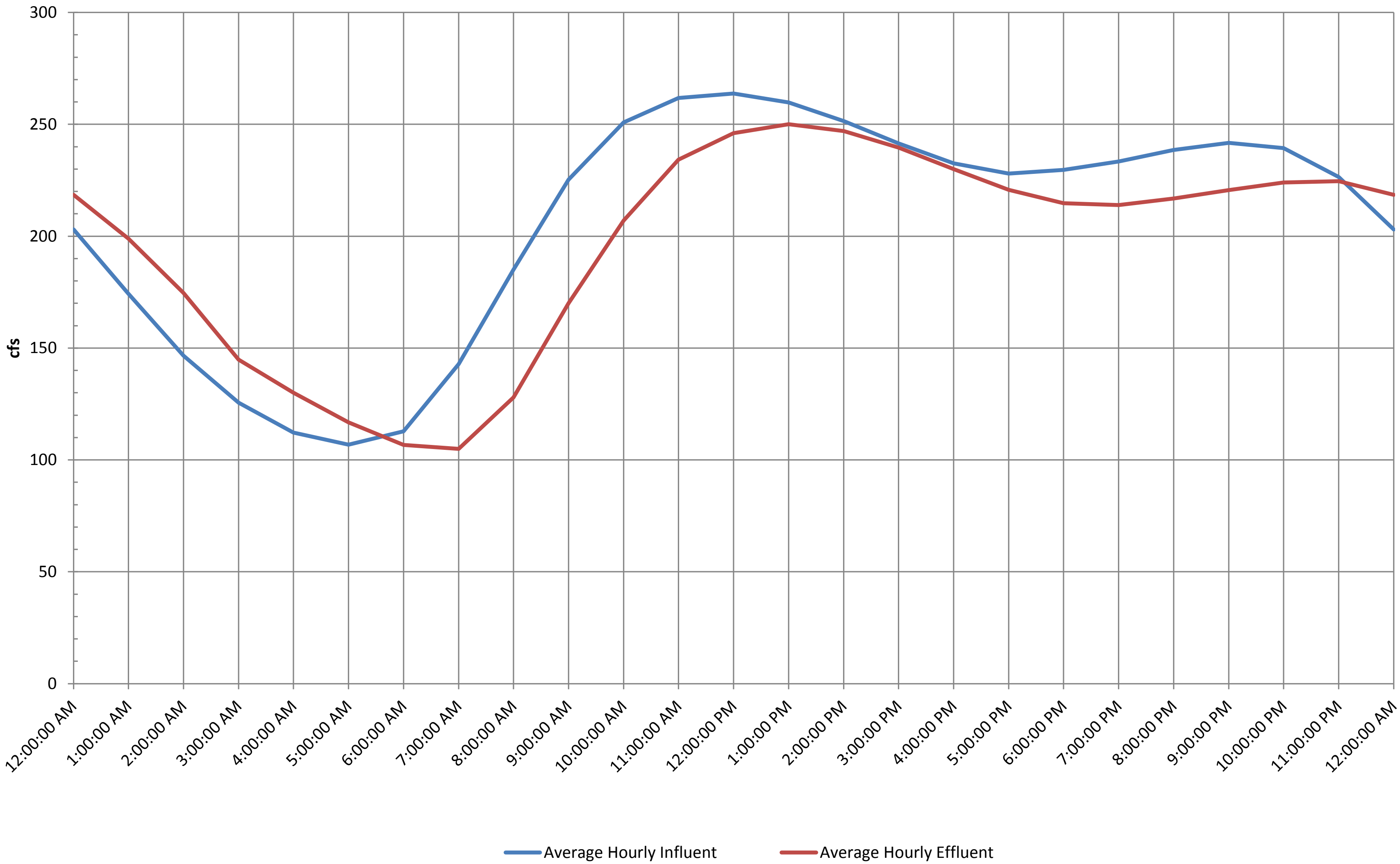
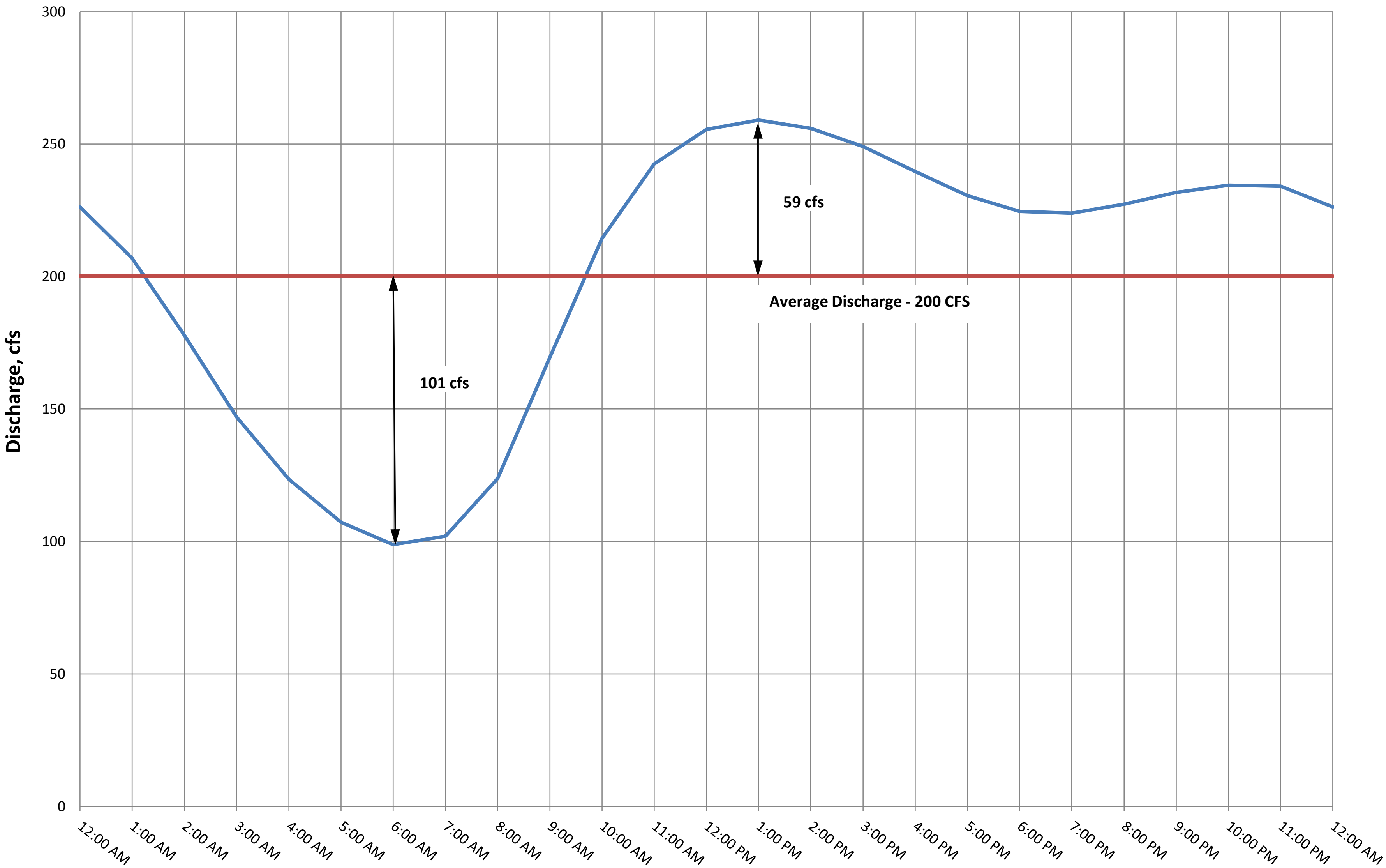


FIGURE 4
Average Hourly RWHTF Discharge[1]
(2001 - 2014)



[1] Hourly discharge value is average of discharges of the preceeding 60 minutes.

FIGURE 5
Average Daily Volumes Above & Below Average Daily RWHTF Discharge
(2001 - 2014)

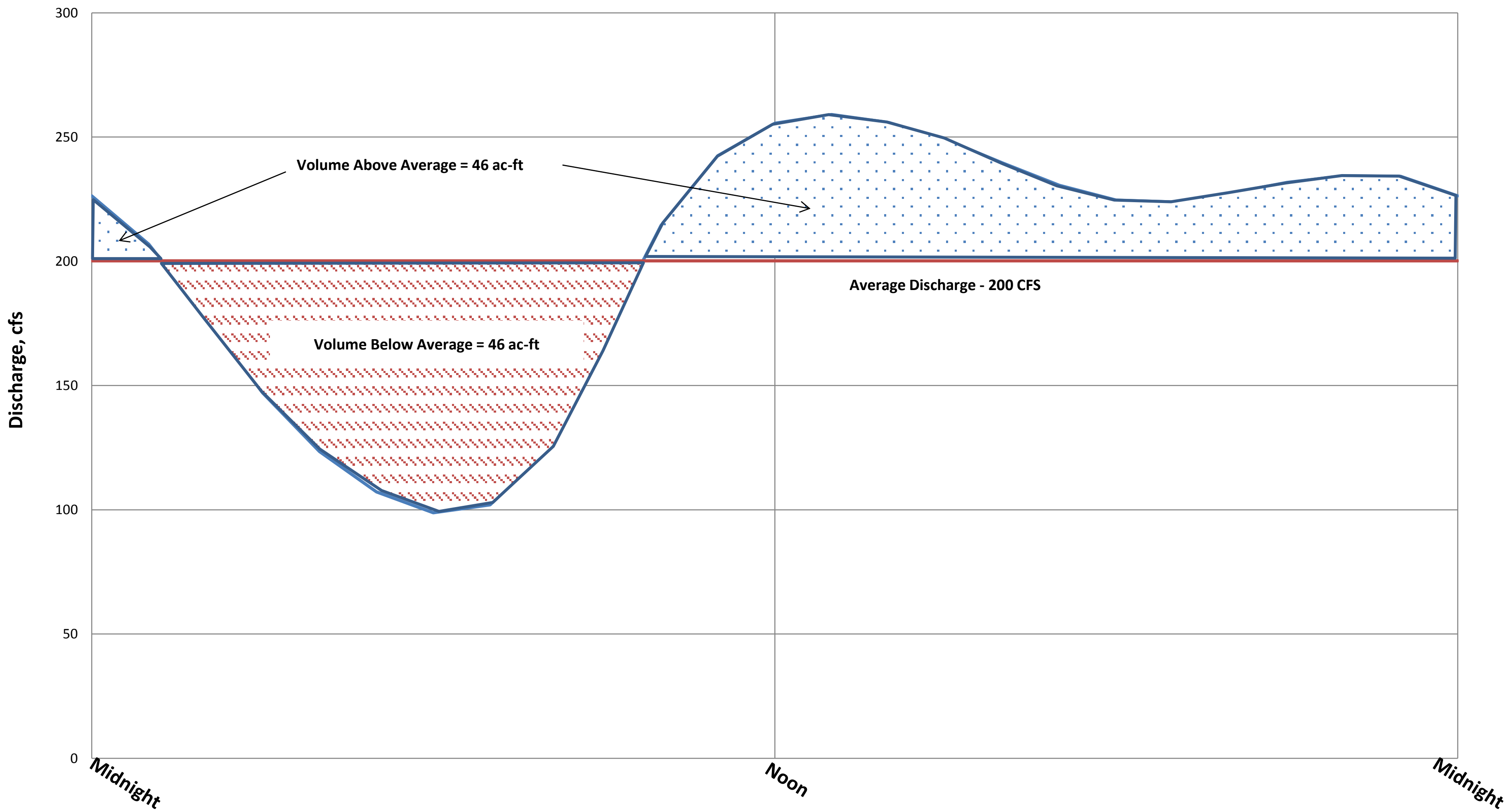


Figure 6
Exceedance Analysis - Volume of Flow Above Average Daily RWHTF Discharge

(2001 - 2014)

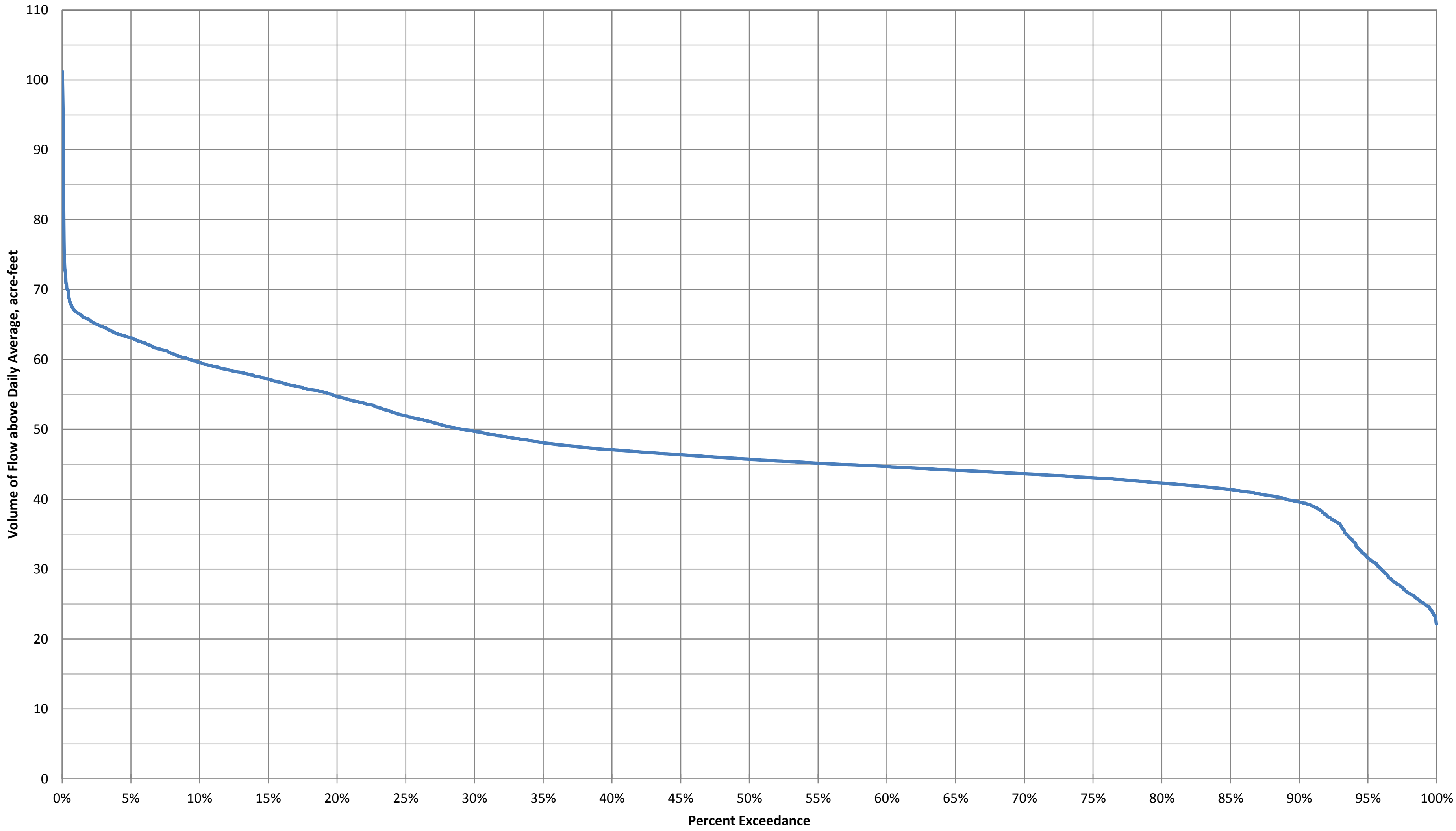


FIGURE 7

48 HOUR DISCHARGE: ***Denver Gage, RWHTF Discharge and Henderson Gage Flows***

(Sept. 5, 2011 through Sept. 6, 2011)

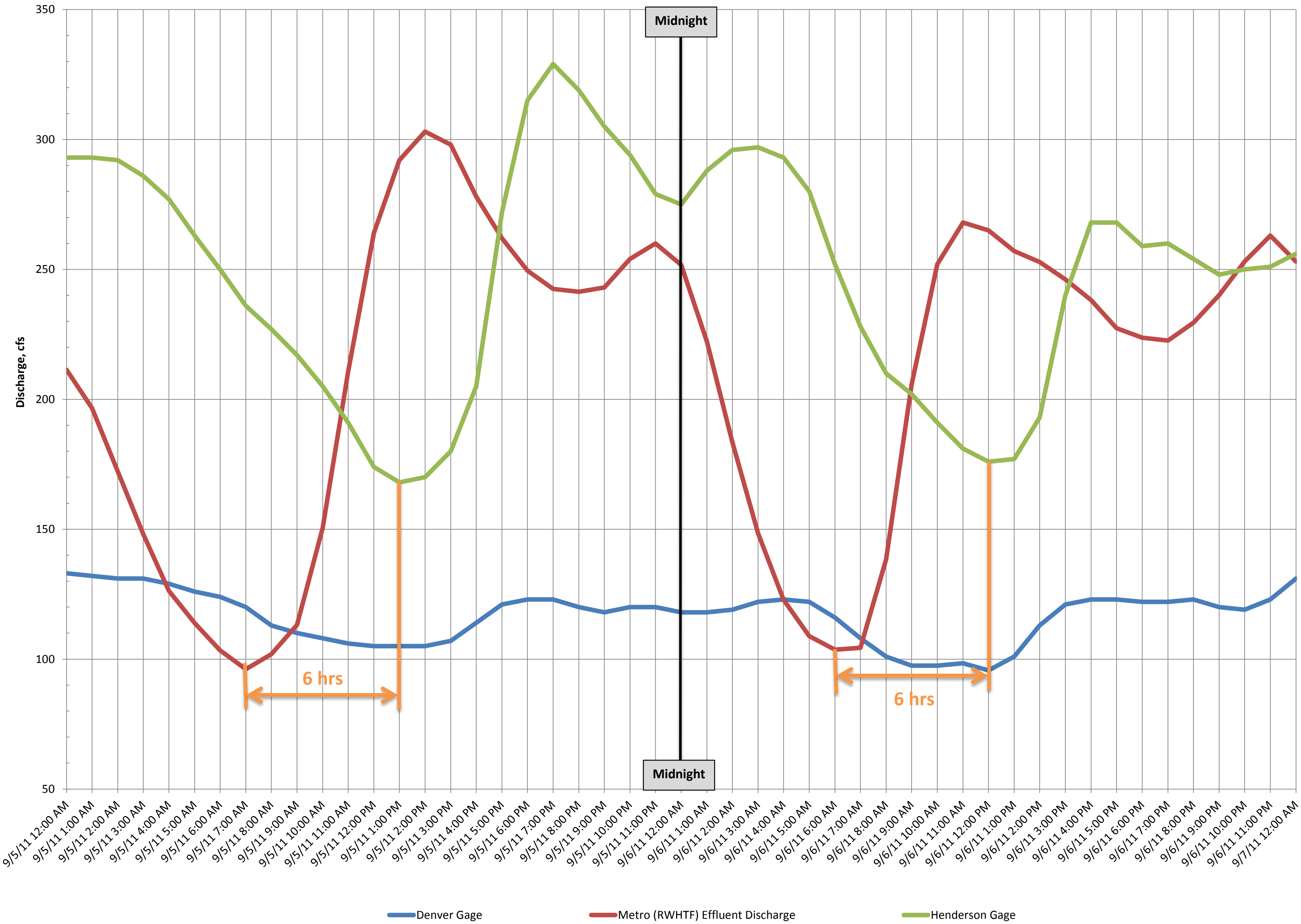
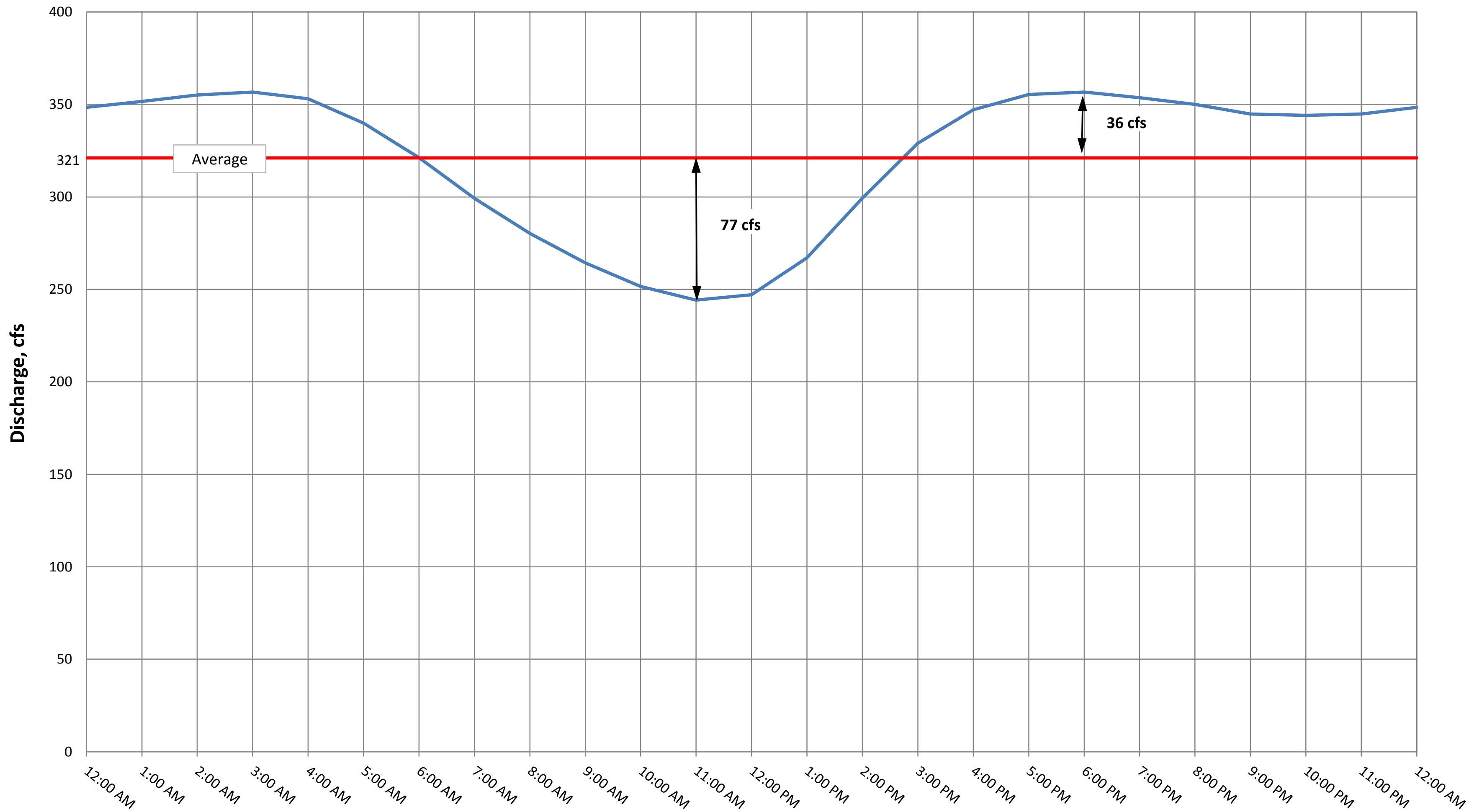


Figure 8

Average Hourly Henderson Gage Discharge[1] on Days with a Bypass Call Affecting Evans No. 2

(2000 - 2012)



[1] Hourly discharge value is average of discharges of the preceeding 60 minutes.

Figure 9

Exceedance Analysis - Volume of Flow Above Average Daily Henderson Gage Flow when Evans No. 2 Ditch Subject to Bypass Call

(2000 - 2012)

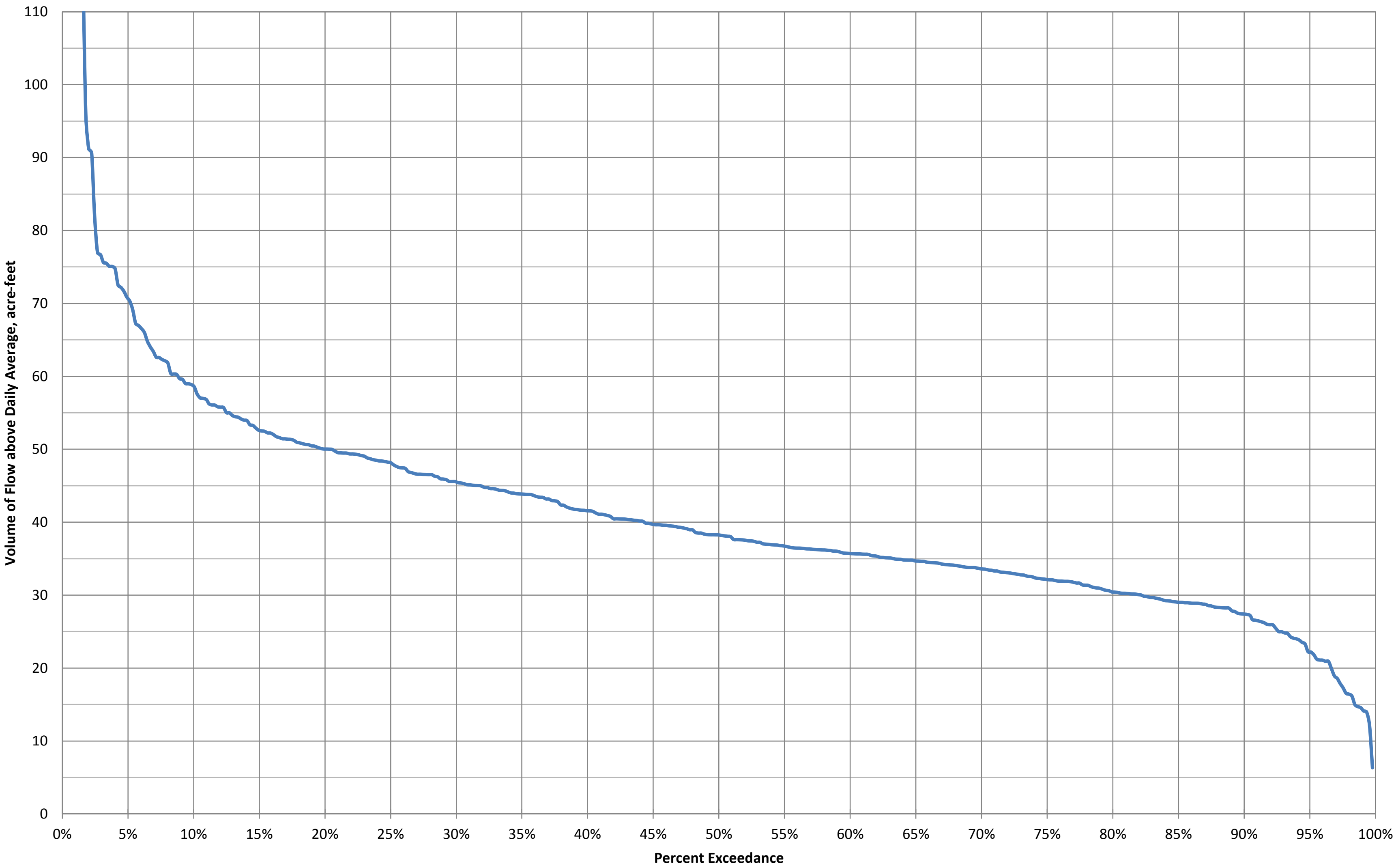
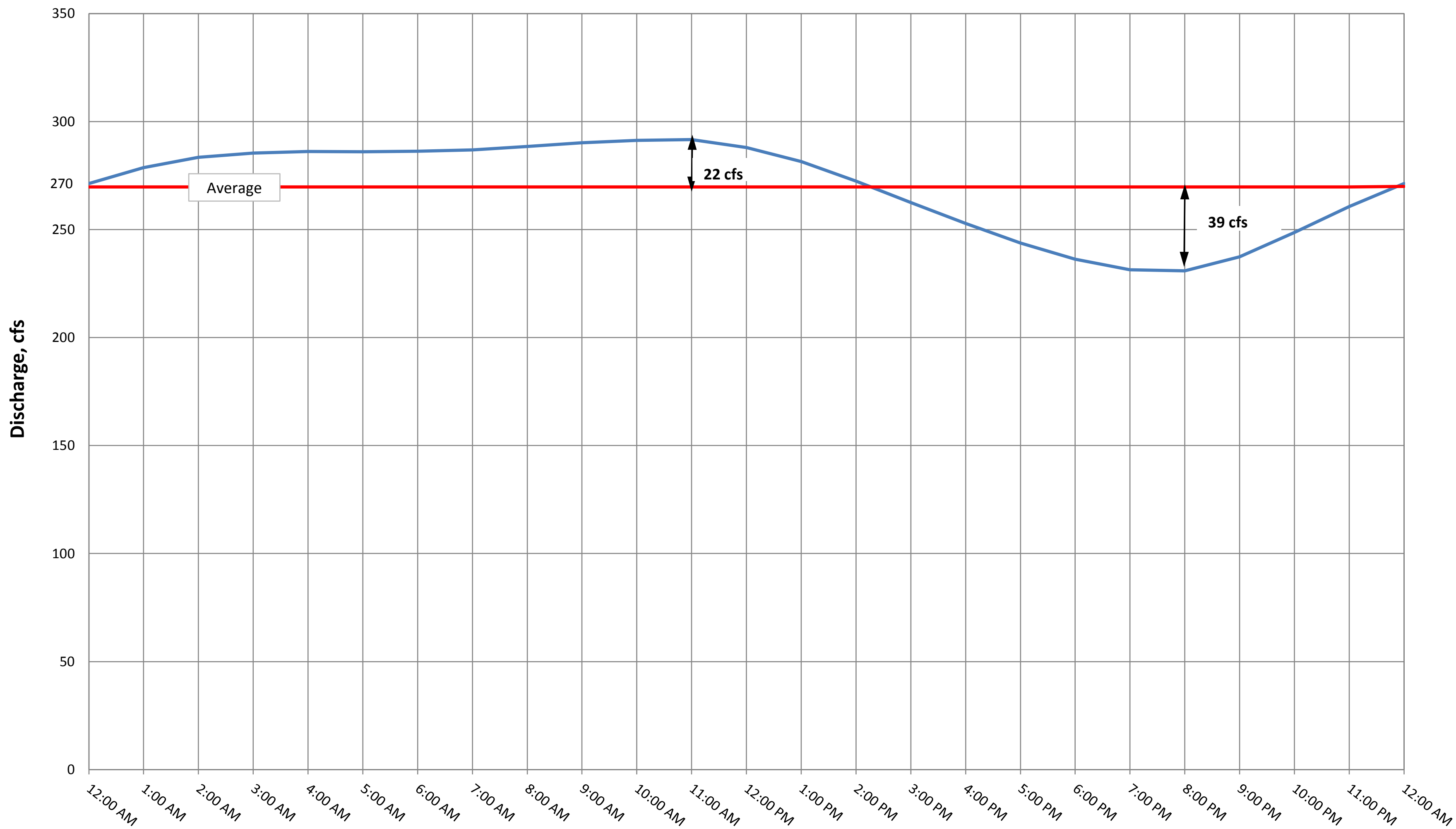


Figure 10

Average Hourly Fort Lupton Gage Discharge[1] on Days with a Bypass Call Affecting Evans No. 2

(2003 - 2012)



[1] Hourly discharge value is average of discharges of the preceeding 60 minutes.

Figure 11

Exceedance Analysis - Volume of Flow Above Average Daily Fort Lupton Gage Flow when Evans No. 2 Subject to Bypass Call

(2003 - 2012)

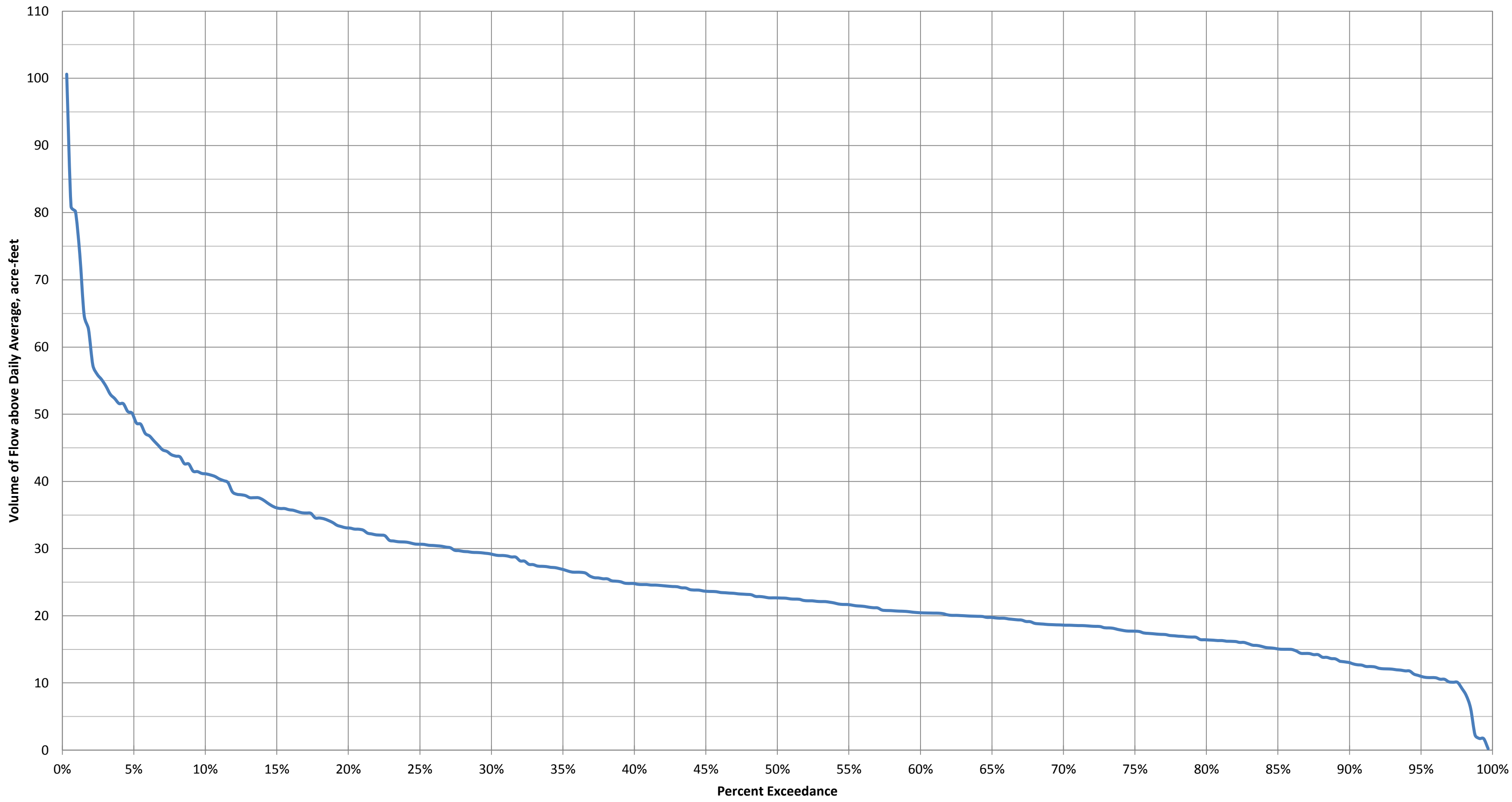
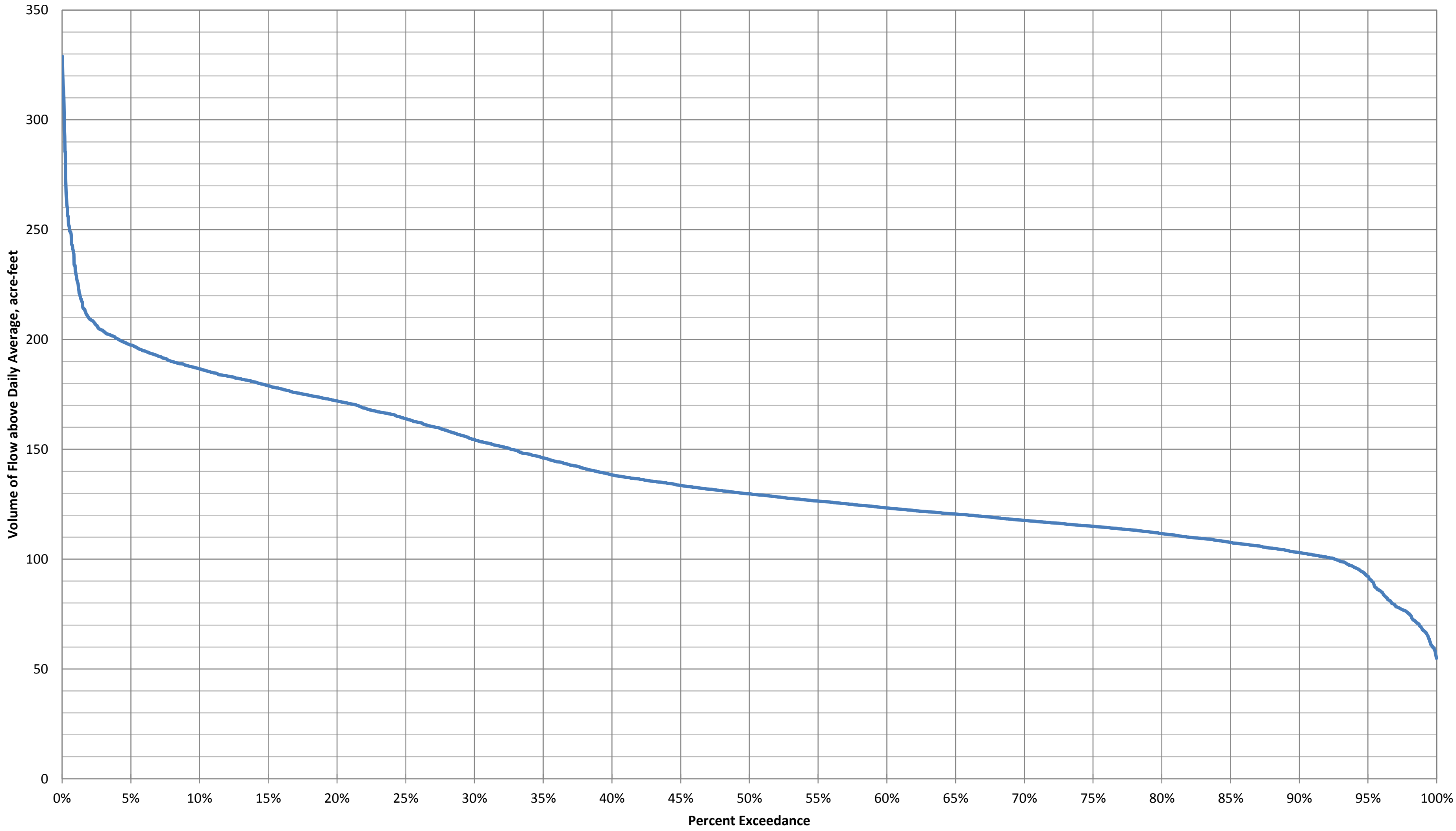
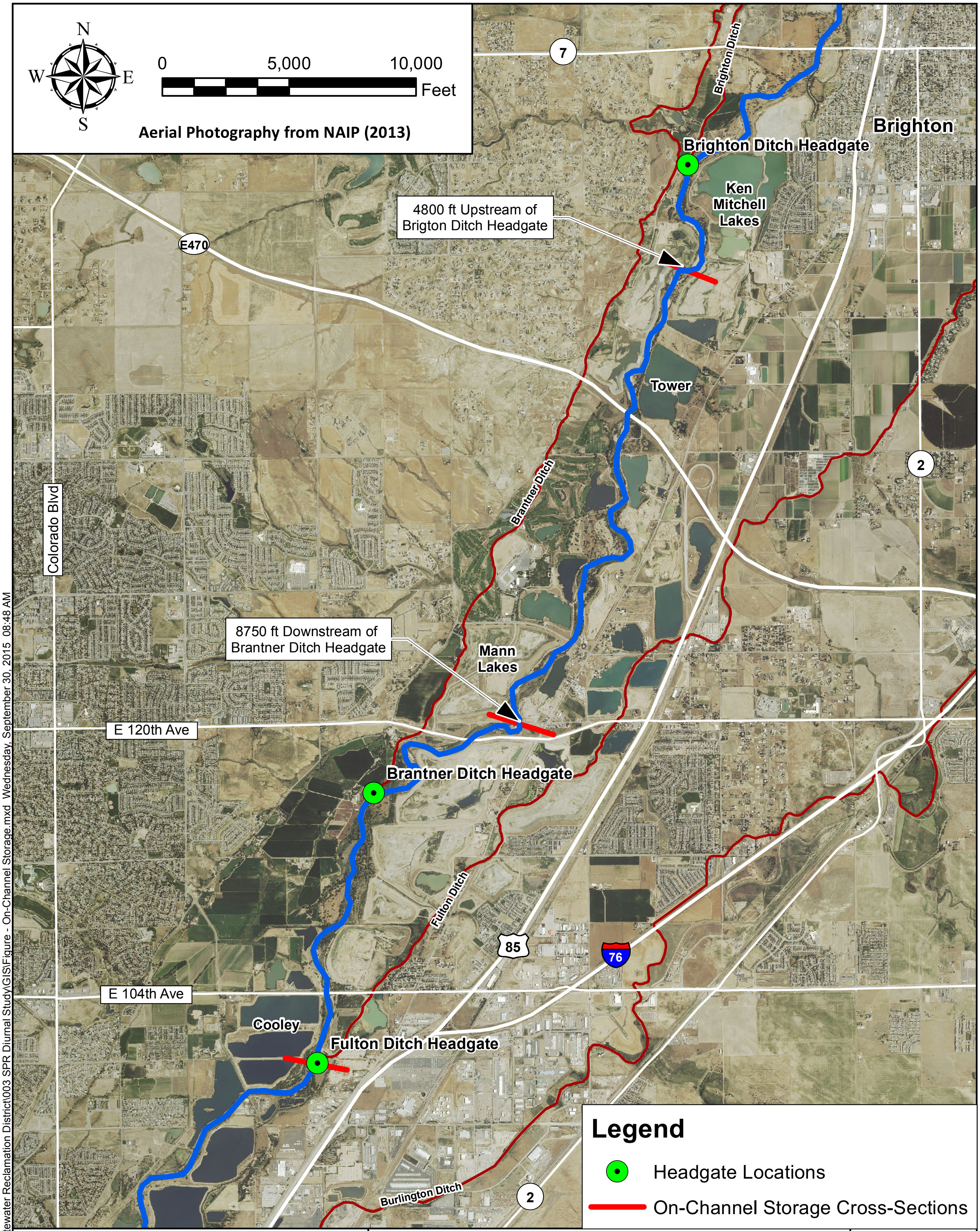


Figure 12
Exceedance Analysis - Volume of Flow Below Daily Peak Discharge at RWHTF

(2001 - 2014)





DEERE & AULT CONSULTANTS, INC.

South Platte River Diurnal Flow Study
Potential On-Channel Storage Locations

FIGURE NO.	13
JOB NO:	0480.003.00
SCALE:	1 inch=5,000 feet

U:\0480 Metro Wastewater Reclamation District\003 SPR Diurnal Study\GIS\Figure - On-Channel Storage.mxd Wednesday, September 30, 2015 08:48 AM