THE EFFECT OF ANTECEDENT DRY TIME ON WATER QUALITY INFLOW AND OUTFLOW TO WET PONDS AND POLLUTANT REMOVAL EFFICIENCY

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ABSTRACT

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Storm water is a significant source of pollutants in urban areas. Urbanization results in reduced water quality of surface waters due to the loss of vegetation and pervious areas, and increased impervious cover, pollutant loading, and volume and velocity of storm water runoff. The potential for flooding and increased loading of pollutants into surface waters negatively impacts drinking water supplies, outdoor recreation, wildlife, aesthetics, and local economies. To alleviate the negative effects of urbanization, municipalities utilize "best management practices" (BMPs). A constructed wet pond is a type of BMP used to address flooding and pollutant loading. The City of Austin (COA), TX utilizes wet ponds to improve storm water quality before it is discharged into surface water. This study analyzed how antecedent storm event time affects the water quality discharged into and from wet ponds located in Austin, TX. This research study determined that longer antecedent storm event time resulted in a significant increase of influent chemical oxygen demand (COD), nitrate + nitrite (NO23),

volatile suspended solids (VSS), and total suspended solids (TSS), and effluent TSS. Additional research is necessary to determine if the engineering requirements used by the COA, and potentially by areas with similar rainfall patterns, to construct wet pond should be revised.

CHAPTER I

INTRODUCTION

Storm water is a significant source of pollutants, especially in urban areas (US EPA 1983, 1994; Barrett et al. 1998). As urbanization of the landscape continues, pervious areas and pollutant-filtering vegetation is lost. In place of these surfaces, parking lots, buildings, houses, sidewalks, and other impervious structures are constructed (Arnold and Gibbons 1996). These impervious areas are not able to filter contaminants like vegetated areas (Dillaha et al. 1988; US EPA 1994). Instead, the pollutants on these surfaces wash off during storm events (what is known as "storm water runoff") either directly or indirectly into streams, lakes, rivers, and other surface waters.

Urbanization also results in the generation of pollutants that are transported by storm water runoff to surface waters. These pollutants can cause acute and chronic toxicity for aquatic organisms (TCEQ 2003). Cars and trucks contribute pollutants such as oil, grease, and metals (MacKenzie and Hunter 1979; Kim et al. 2007). Particulates released into the atmosphere from industrial processes, and energy and agricultural production include mercury (Hg), lead (Pb), cadmium (Cd), copper (Cu), zinc (Zn), nickel (Ni), and nitrogen (N) which settle on soil and are transported to surface water during storm events (EPA 1983, 2002; USGS 1999; Mielke et al. 2000). Maintenance of manicured lawns causes an increased loading of pesticides, herbicides, and fertilizers. Fertilizers and discharge from wastewater treatment systems (such as wastewater treatment plants and septic systems) contribute phosphorus and nitrogen which can result in increased oxygen demand (US EPA 1994, USGS 1999).

New construction and redevelopment of single and multi-family and commercial structures causes an increased sediment loading. Suspended solids may negatively affect fish by irritating the gills, and reducing egg laying and hatching success (US EPA 1994). Sediment loading also causes turbidity, decreasing the amount of sunlight available for photosynthetic plants (Van Nieuwenhuyse and LaPerriere 1986). Many pollutants bind onto sediments (Vaze and Chiew 2004); therefore, sediment mobilized by land disturbing activities can carry pollutants to surface waters during storm events.

Increase in impervious cover and loss of vegetation also increases the volume and velocity of storm water runoff since there are no exposed soils to absorb the runoff or vegetation to intercept it and slow it down (Chin and Gregory 2001). This increased volume and velocity creates larger flood events that peak quickly (Guillemette et al. 2005). The potential for floods and the increased loading of pollutants into surface waters negatively impacts drinking water supplies (Gaffield et al. 2003; TCEQ 2003; Braden and Johnston 2004), recreation (TCEQ 2003; Novotny et al. 2007), and wildlife (Van Nieuwenhuyse and LaPerriere 1986; US EPA 1994; Paul and Meyer 2001; TCEQ 2003). Reduced water quality and stream or lake aesthetics can lower the value of adjacent land (Braden and Johnston 2004).

Texas is experiencing significant urbanization, leading the country for the number of acres of land developed between 1982 and 2003 (USDA 2000, 2007). The population in the City of Austin (COA) is increasing rapidly, with a current population of 735,088 and a projected population in 2025 of over 1 million (COA 2007). Texas is also especially vulnerable to flash flooding due to urbanization because of the state's rainfall pattern of intense storms (Baker 1977; Slade and Patton 2003).

Managing storm water can positively affect downstream areas that receive the flow, including reducing flooding and pollution treatment, and improving water quality and stream aesthetics (although the specific benefit is difficult to quantify) (Braden and Johnston 2004). The Environmental Protection Agency (EPA) requires many municipalities across the country to manage the construction, industrial, residential, and post-construction storm water which results from urbanization through a permit program entitled "National Pollutant Discharge Elimination System" (US EPA 2005). To fulfill the permit requirements, municipalities utilize, and/or require developers to utilize, various techniques known as "best management practices" (BMPs) (US EPA 1994, 1999, 2005).

Storm water quality controls are typically designed so water quality and quantity conditions predevelopment are maintained (Behera et al. 1999). A common BMP used to address flooding and pollutant loading is a constructed wet pond (US EPA 1999). A wet pond is engineered so storm water runoff enters the pond, and sediments and other pollutants settle out or are taken up by vegetation. Water over a certain volume will be discharged over a spillway and the remaining water will be retained to create a permanent pool (COA 2003; Wang et al. 2004; Weiss et al. 2006).

COA allows the use of wet ponds as a mechanism to address post-construction runoff control (Figure 1). COA's "Environmental Criteria Manual" (ECM) establishes the engineering standards for this type of pollution control as well as many others (Figure 2) (COA 2003). The ECM uses the time between storm events as one of the factors in the calculation of permanent pool size. Since wet ponds are a complex type of control, the perfect pond design has not yet been determined. COA has collected data on several wet ponds within the city limits and plans to evaluate whether the requirements for engineering a wet pond should be revised (COA 2006a).

The objective of this thesis was to use the data collected by COA to evaluate whether the length of antecedent dry time (Han et al. 2005; Kim et al. 2007) affects the pollutant removal efficiency of a wet pond. The study was undertaken with the hypothesis that with an increased time between storm events, pollutant loading to the ponds will be higher, and the effectiveness of the ponds to perform pollutant removal will be lower (Figure 3).

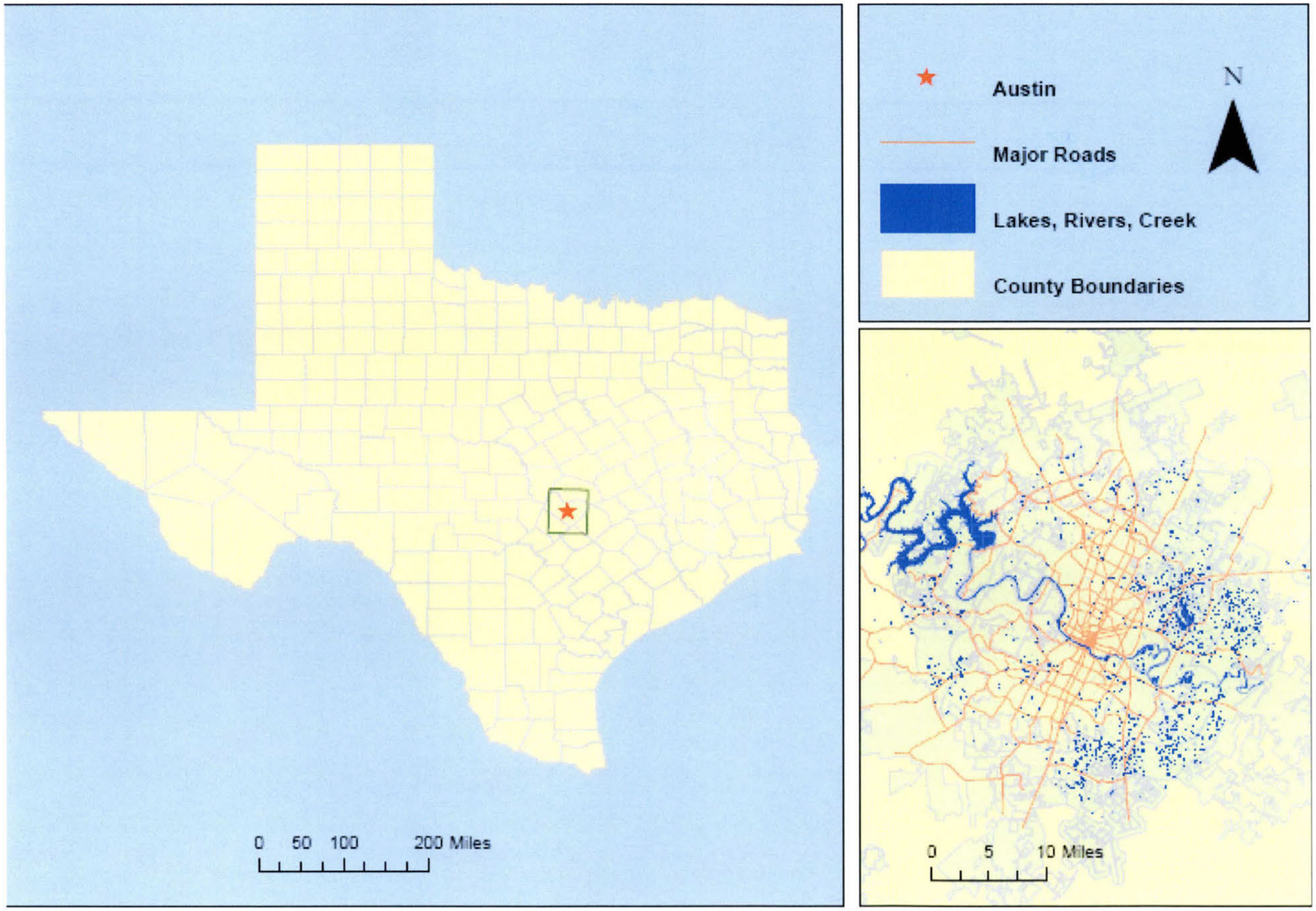


Figure 1. Map of Austin, Texas, the study area.

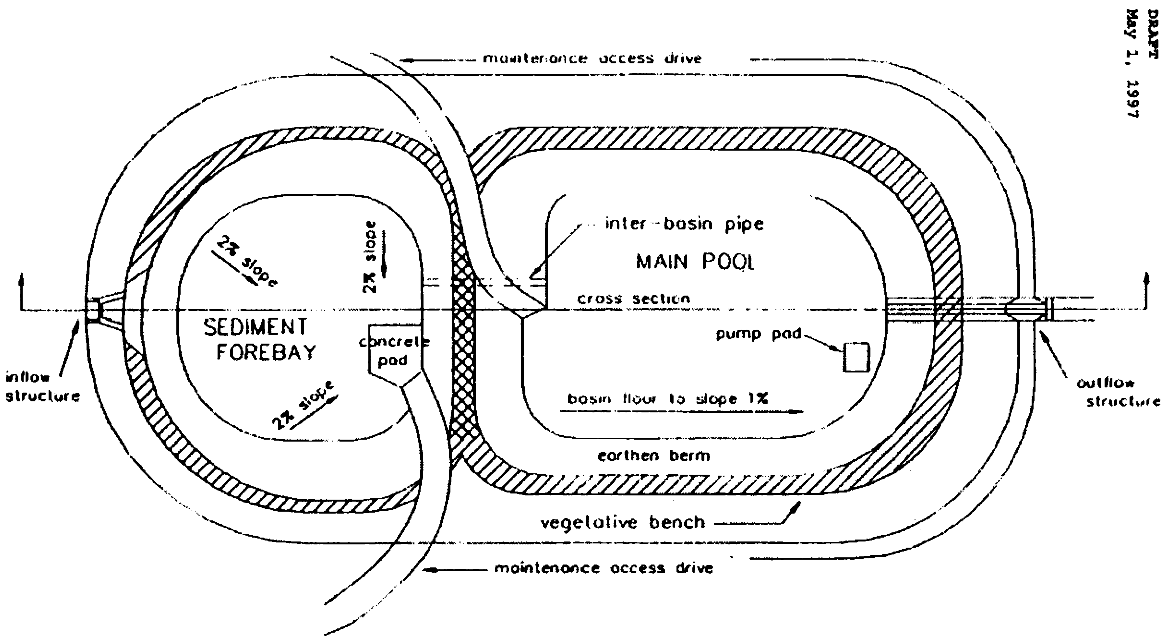


Figure 2. Typical design of an engineered wet pond as included in the City of Austin's Environmental Criteria Manual (COA 2003).

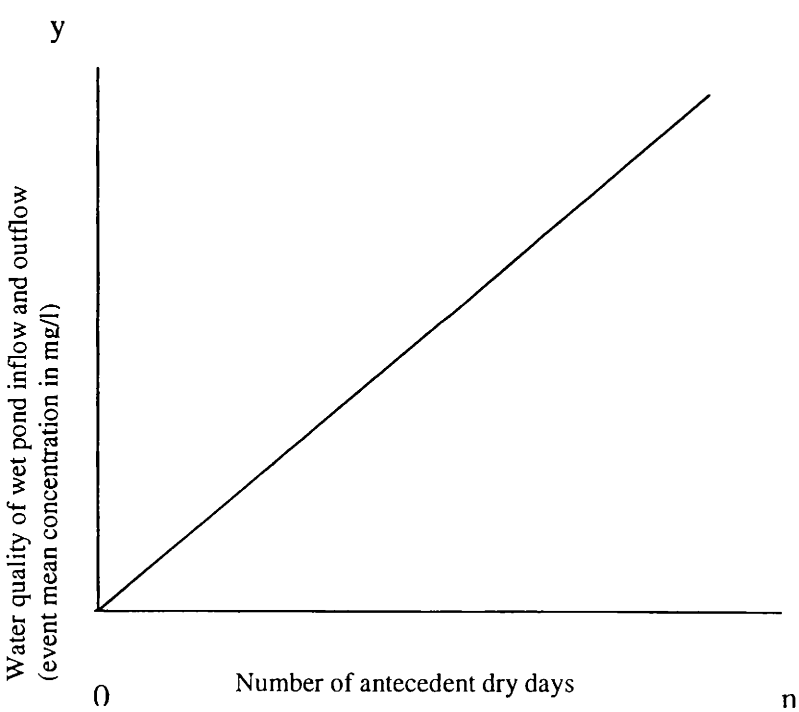


Figure 3. Anticipated results of this study.

This thesis also proposes to determine if the formula used by COA to calculate the size of a wet pond should be revised based on the number of days between storm events. It is recognized that the effects of storm intensity and the "first flush" characteristic of certain pollutants also influences the effectiveness of wet ponds. Unfortunately, accounting for these factors would introduce a level of complexity that is beyond the scope of this research. A study on the effect of antecedent dry time on wet pond effectiveness will complement studies currently in progress by COA. If the number of antecedent dry hours is determined to have a significant effect on the pollutant removal capability of wet ponds, then this finding could impact the pond design criteria established by COA.

CHAPTER II

LITERATURE REVIEW

Storm Water Quality and Quantity

The effect of urban storm water runoff on surface water quality and quantity has been studied extensively. Urbanization has been shown to increase runoff with greater amounts of impervious cover. For example, more frequent flooding in the East Branch of Brandywine Creek in Pennsylvania was attributed to urbanization (Leopold 1968). Increased impervious surfaces in Fountain Hills, a residential community near Phoenix, Arizona also led to greater quantities of storm water runoff (Chin and Gregory 2001).

Characklis and Wiesner (1997) examined runoff from a large watershed in Houston, Texas and concluded that urban runoff increased the concentration of particles, suspended solids, organic carbon, iron, and zinc in receiving waters. Elevated levels of pollutants in runoff were most likely due to human activities (Characklis and Wiesner 1997). In western Georgia, the loss of rural areas to urbanization had led to an increase in pollutants such as fecal coliform and nutrients discharged to local streams. Elevated levels of pollutants in this study were also attributed primarily to manmade causes (Schoonover and Lockaby 2006). Comings et al. (2000) evaluated elevated levels of total, soluble, and bioavailable phosphorus in Lake Sammamish, Washington. From this study, the increased loading of pollutants was attributed to urbanization and the increase in use of detergents, fertilizers, animal waste, and septic tank leachate (Comings et al.

2000). Highways receiving drainage from an urban area in Austin, Texas discharge the highest concentration of pollutants in storm water compared to highways with drainage from more rural and residential areas (Barrett et al. 1998).

The effectiveness of wet ponds has been studied, but these studies have been relatively few and the results have not been consistent (Comings et al. 2000). The small number of wet ponds studied is likely because design requirements for treatment ponds have not been perfected (Weiss et al. 2006). Wet ponds are complicated BMPs to design because flow and pollutant concentration frequently vary across time (Wang et al. 2004). Many factors have been considered significant in the engineering of an effective pond. Pond volume, detention time, and contaminant removing characteristics have been included in wet pond analysis (Weiss et al. 2006). Higher volumes of storm water runoff detained in a pond results in increased removal of pollutant mass and concentration (Sansalone and Cristina 2004). The configuration of wet ponds (i.e., short-circuiting) was also determined to affect the pollutant removal efficiencies (Comings et al. 2000). Wang et al. (2004) consider the stormwater inflow to and outflow from a wet pond an important factor in pond design.

Size of sediment particles and the relationship to pollutant loading is another factor determined to influence wet pond effectiveness. Vaze and Chiew (2004) found that less than 15% of total phosphorus and total nitrogen attached to sediment particles greater than 300 μm. Fine particles require a longer time to settle; as a result, areas with finer particles may require ponds with longer detention periods. Surface to area ratios may be an effective tool to determine pond size based on the desired pollutant removal efficiency. The settling time for particles in wet ponds is important (Wu et al. 1996;

Comings et al. 2000), and a calculation of surface to area ratio has been developed to consider the time particles require to settle out (Wu et al. 1996).

Although urban storm water runoff and wet pond effectiveness and design have been presented in the literature, the antecedent storm event time and the effect on wet pond efficiency is not typically incorporated into the studies' hypotheses. Kim et al. (2007) considered the number of antecedent dry days (ADDs) preceding each storm event in a study on rainfall runoff quality and rate in Korea. The number of ADDs before a storm event along with the rate of rainfall and the drainage area was found to affect the runoff coefficient. The runoff coefficient rate was higher in larger storm events, resulting in higher runoff volume. Antecedent rainfall conditions affected the event mean concentration of pollutants in the runoff. Typically, event mean concentration of pollutants was low with few ADDs. Paradoxically, a long period of dry days in between storms also showed a low event mean concentration after high rainfall rates due to dilution (Kim et al. 2007).

Konrad and Burges (2001) studied the difference between storm water runoff from land converted from forest to residential and commercial developments in Puget lowlands in Washington. Based on the storm pattern in the study area, the authors determined that the best method to evaluate storm water treatment systems was to analyze extended rainfall events that occurred with different numbers of ADDs. The study results showed that small on-site detention systems can be used to address storm water runoff from storm events that are frequent and low in intensity but high magnitude, low frequency storms should be assessed relative to their intensity and frequency (Konrad and Burges 2001).

Another study performed in Washington (Spokane) created a model to predict pollutant removal efficiency of wet ponds. Factors considered in the model were the probability of a storm occurring and antecedent moisture conditions both of which are affected by the time between storm events. This study concluded that storm hyetograph (a graph of water input over time) was significant in pond design along with storm return period, watershed features, and wet pond location (Wang et al. 2004). In a study on storm water runoff quality, Characklis and Wiesner (1997) only sampled storm events with seven ADDs to eliminate water quality variability resulting from the number of ADDs.

In contrast to the previous studies performed which include ADDs as part of the data, this thesis proposes to focus on the time between storm events in particular and determine if there is a significant affect on the water quality flowing into and out of select wet ponds.

COA requires that the permanent pool of a wet pond be sized to retain the amount of rainfall produced in two weeks. This volume is based on the following calculation:

Formula 1. $V = (RT/RI)*WMMS*R_f*Ls*DA*1'/12''$,

where V is the permanent pool volume (acre-feet), RT is the residence time (14 days), RI is the frequency of the mean wettest monthly storm (determined to be 5.45 days), WMMS is the wettest mean monthly rainfall daily event (calculated to be 0.72 inches), R_f is the annual runoff coefficient, Ls is the storage loss coefficient, and DA is the drainage area (acres) (COA 2003). Since RI and WMMS are empirical values, the permanent pool calculation can only be as accurate as what has been determined to be RI and WMMS. The formula is currently based on the determination that a storm event size of 0.72 inches occurs every 5.45 days. An analysis of the relationship of antecedent storm event time and water quality into and out of wet ponds will show if this formula is appropriate or if the values used for RI and WMMS need to be revised.

Water Collection and Analysis Techniques

Automatic sampling and monitoring equipment is widely used to measure flow volume and rates for storm water analysis (Barrett et al. 1998; Wu et al. 1996; Comings et al. 2000; Wang et al. 2004; Kim et al. 2007). In some instances, flow and rain data are obtained from other sources. For example, flow data from US Geological Survey were used by Characklis and Wiesner (1997) and rain data from National Climatic Data Center were utilized by Bartone and Uchrin (1999) and Schoonover and Lockaby (2006). Some rain and flow data are collected manually although it is not as common as automatic collection. Li et al. (2005) and Wang et al. (2004) measured rainfall using tipping bucket rain gauges. Site constraints led Comings et al. (2000) to manual calculation of water volume. From the literature, it appears that automatic samples are typically used to obtain water quality samples (Wu et al. 1996; Barrett et al. 1998; Comings et al. 2000; Kim et al. 2007), although manual collection is still employed (Characklis and Wiesner 1997; Sansalone and Cristina 2004).

Water quality samples used in analysis of storm water were collected using grab or composite samples or a combination of both. Vaze and Chiew (2004) used continuous grab sampling to assess nutrient loading. Grab samples were also used in the analysis of metals and solids in runoff from large watersheds, a comparison of pollutant removal efficiency of two residential wet ponds, and an analysis on the effect of land cover on water quality (Characklis and Wiesner 1997; Bartone and Uchrin 1999; Schoonover and Lockaby 2006). Kim et al. (2007) used grab and composite samples to characterize pollutants discharged from bridges and parking lots during storm events. Li et al. (2005) used both grab and flow-weighted composite samples in their study of particle size distribution in highway runoff and mass-based first flush (Li et al. 2005). Flow weighted composite samples were also commonly collected (MacKenzie and Hunter 1979; Barrett et al. 1998; and Comings et al. 2000).

Event mean concentration (EMC) is a common characteristic calculated for storm water quality to account for high variability in pollutant concentrations during a rain event (Sansalone and Cristina 2004). The EMC ensures that a composite sample represents pollutant concentration through water volume averaging (Comings et al. 2000). There are many ways to describe EMC using words or formulas, although all descriptions appear equivalent based on the literature reviewed. Kim et al. (2007) presented the following formula for EMC:

Formula 2. $$\text{EMC (mg/l)} = \sum_{t=1}^{t=t} C(t)*q_{run}(t) / \sum_{t=1}^{t=t} q_{run}(t),$$

where C(t) is pollutant concentration and $q_{run(t)}$ is runoff flow rate discharged at time t. This formula for EMC adequately accounts for the randomness of runoff quality and quantity (Kim et al. 2007).

Grab samples obtained by Vaze and Chiew (2004) were combined for each storm event to determine the EMC of each storm. EMCs were determined by the mean concentration of a parameter for each runoff event in a study on highway runoff by Barrett et al. (1998). Wu et al. (1996) used both EMC and storm-averaged concentration (SAC) which allows the quality of water flowing into and out of detention ponds to be directly compared. The SAC describes the runoff similar to EMC but it uses the flow-weighted average of the pollutant concentrations for the entire runoff period, whereas EMC is a flow-weighted composite of the pollutant mass over the volume of runoff (Wu et al. 1996). In their first flush study, Sansalone and Cristina (2004) also used EMC in the analysis, using a formula equivalent to Kim et al. (2007).

COA has a protocol for the collection and analysis of flow, rainfall, and water quality data. Similar to research described in this literature review, COA collects flow data using automatic stage recorders and data recorders. Water quality samples are collected as grab and/or composite samples. Rain data are obtained by tipping-bucket rain gauges. EMCs are utilized for analysis and are calculated as the sum of the load divided by the sum of the volume, similar to the formula employed by Kim et al. (2007) (COA 2006a, 2006b). As a result of the data having already been collected by COA using established protocol, the techniques employed in this study will reflect the procedures used by COA.

Statistical Analysis

Many statistical methods are used to describe and compare water quality and quantity data; however most of the literature does not contain details on the type of methods used. Environmental data, including urban storm water runoff quality typically show a logarithm (log) normal distribution (Gilbert 1987; COA 2006a, 2006b). Comings et al. (2000) plotted pollutant concentration data as arithmetic and log-transformed in order to determine the distribution. Based on the results of the distribution test, lognormal distribution was determined to best describe the water quality data (Comings et al. 2000). As a result, when calculating the mean of EMCs, the log-transformed data were used based on the following formula:

Formula 3. $$C_{mean} = e^{\left(\mu + \frac{s^2}{2}\right)}$$,

where exp is the exponentiation on the base of the natural log e, μ is the mean of the natural log of EMCs, and s^2 is the variance of the natural log of EMCs (Comings et al. 2000). COA staff plotted EMC for each pollutant and determined through visual interpretation that the data fit a lognormal rather than a normal distribution (COA 2006a). COA uses several types of means for statistical analysis of water quality data: geometric, "Driscoll", and "Gilbert". The geometric mean is considered a bias estimator of the true mean (Gilbert 1987). This mean is appropriate for analysis of EMCs. The geometric mean is the nth root of the arithmetic mean of the log-transformed data (COA 2006a, 2006b). The Driscoll mean is used to estimate μ and σ^2 for data with log-normal distribution. This mean minimizes bias as the sample size increases. The Driscoll mean is defined as:

Formula 4. $$\mu = e^{\left(y + \frac{s_y^2}{2}\right)}$$, and

Formula 5. $$\sigma^2 = \mu^2\left(e^{s_{\hat{y}}^2} - 1\right)$$,

where μ is the estimate of the mean of data from a log-normal distribution, y is the arithmetic sample mean of the log transformed data, σ^2 is the estimate of the variance of data from a log normal distribution, and $s_{\hat{y}}^2$ is the sample variance of the log transformed data.

COA also uses the Gilbert mean, which more accurately represents the mean for lognormal data, particularly with smaller data sets. The Gilbert mean is defined as:

Formula 6. $$\mu = (e^{\hat{y}})\Psi_n\left(\frac{s_{\hat{y}}^2}{2}\right)$$, and

Formula 7. $$\sigma^2 = e^{2\hat{y}}\left[\Psi_n\left(2s_{\hat{y}}^2\right) - \Psi_n\left(\frac{s_{\hat{y}}^2(n-2)}{n-1}\right)\right]$$,

where μ is the estimate of the mean from lognormal distribution, ŷ is the arithmetic sample mean of the log transformed data, is $s_{\hat{y}}^2$ the sample variance of the log transformed data, σ^2 is the estimate of the variance of data from a log normal distribution, and Ψ_n is an infinite series (Gilbert 1987; COA 2006a).

Median EMC and coefficient of variation were used to compare runoff water quality by Barrett et al. (1998). Coefficient of variation was also calculated for storm number, rainfall volume, rainfall duration, and ADD by Wu et al. (1996). Coefficient of variation is calculated by:

Formula 8. $$C_v = \frac{\sigma}{\mu}$$, where c_v is the coefficient of variation, σ is the standard deviation, and μ is the arithmetic mean. Standard deviation was used by Characklis and Wiesner (1997) to describe pollutants concentrations before and during storm events.

Correlation can be used to determine the relationship between variables such as concentration of pollutants (Characklis and Wiesner 1997). COA analyzed the relationship between mean pollutant concentration in storm water runoff and impervious cover, and mean pollutant concentration and developed/undeveloped watershed using the General Linear Model regression analysis (COA 2006a).

CHAPTER III

METHODS

Site Selection

The longitude and latitude of influent and effluent outfall locations, and rain and water quality data for five wet ponds in Austin were obtained from COA's Environmental Resource Management staff. Water quality data included pollutant load and event mean concentrations for various influent and effluent monitoring events. Rain data included rain event date, total runoff volume, total rain, and dry time between qualifying storm events as established by COA staff using the guidelines detailed in *Stormwater quality and quantity from small watersheds in Austin, Texas* (COA 2006a).

Using an interactive map, the COA GIS Viewer (http://coagis1.ci.austin.tx.us/website/COAViewer_dev/viewer.htm), I identified the site plan or subdivision construction plan review case number associated with the wet pond at the recorded coordinates (Table 1). Using the review case numbers, I retrieved a hard copy of the site plan or subdivision construction plan files from the COA Research Files room (located at 505 Barton Springs Road, Austin, Texas).

I analyzed the plan sheets detailing each of the five wet ponds to assess whether the ponds were constructed according to the criteria established by the COA. Criteria used in this assessment were from the ECM and included factors such as: permanent pool volume, pond liner material and thickness, ratios of plant categories (e.g. tall marsh, short marsh, submergents and floating aquatics), and species of plants (Appendix A). Note that the ECM is updated periodically so the criteria used in this analysis were based on the criteria in effect when the site plan or construction plans for the proposed ponds were submitted to COA staff for review. Two of the five ponds were found to have been built according COA's ECM criteria: Ceylon Tea and Berdoll Farms wet ponds.

The water quality data of Ceylon Tea and Berdoll Farms wet ponds were compared to determine which pond had the greatest number of monitoring events. Berdoll Farms pond was monitored from August 2003 to April 2006, whereas Ceylon Tea was monitored from July 2005 to June 2006. Since more monitoring data were available for Berdoll Farms, Berdoll Farms pond was selected for further analysis.

Data Analysis

Parameters considered in the study were: cadmium (Cd), chemical oxygen demand (COD), copper (Cu), dissolved phosphorus (DP), ammonia (NH3), nitrate+nitrite (NO23), lead (Pb), total Kjedhal nitrogen (TKN), total nitrogen (TN), total organic carbon (TOC), total phosphorus (TP), total suspended solids (TSS), volatile suspended solids (VSS), and zinc (Zn). The water quality data for Berdoll Farms pond were analyzed to determine "paired events" for each parameter. A paired event was considered those monitoring events that experienced influent and effluent discharges associated with a particular rainfall event. The rainfall event for 2/4/2004 had two influent samples analyzed for water quality. For this rain event, the data obtained from the first sample were used for the paired event. Monitoring events that were more than 72 hours apart were not considered paired events.

Table 1. List of wet ponds considered in this study.

Pond Names	COA Subdivision Construction/ Site Plan Number	Latitude	Longitude
Central Market	Unable to locate*		
Effluent		30.303438	-97.73842166
Influent		30.30646078	-97.7405014
Ceylon Tea	C8-00-2083.1B		
Effluent		30.41767311	-97.64016011
Influent East		30.41842901	-97.63955227
Influent North		30.41878308	-97.63980162
Influent West		30.41861576	-97.64070927
Berdoll Farms	C8-00-2113.1B		
Effluent		30.16914107	-97.61124425
Influent		30.17052342	-97.61023838
St. Elmo	SP-91-0072B		
Effluent		30.20701306	-97.75307063
East Influent		30.20762171	-97.75192677
West Influent		30.20759514	-97.75335523
Convention Center	SP-90-0029C		
O/G Chamber Effluent		30.2633698	-97.73736598
3rd Street @ Neches		30.26412677	-97.73934735
Convention Center O/G Wet Pond Effluent		30.2633698	-97.73736598

*Based on discussion with COA staff, this pond was not built per ECM criteria.

The dry time for the influent monitoring was established as the time between the end of the last qualifying rain event and the time of the influent monitoring (which requires runoff entering the pond). The dry time for the effluent monitoring was established as the time between the end of the last rain event and the time of effluent monitoring (which requires a discharge from the pond). This should provide a "snapshot" of the data beginning when runoff first enters the pond and when the runoff (or the mixed runoff) is discharged.

Although the data from COA staff included total runoff volume and EMCs for all monitoring events, these values were recalculated for each parameter considered a paired event to familiarize myself with the technique and confirm accuracy of data provided. EMC data were transformed to natural log for statistical analysis. Since environmental data are widely accepted as exhibiting lognormal distribution, the transformed values were used throughout the statistical analyses.

Using SPSS software, a paired t-test was performed to compare the means of the influent and effluent water quality for each paired event. The null hypothesis was that there is no significant difference between the means ($H_0: \mu_1 = \mu_2$). The alternate hypothesis was that there is a significant different between the means ($H_0: \mu_1 \neq \mu_2$). If there was a significant difference between the means, a correlation analysis of the influent and effluent water quality and dry hours was performed. The null hypothesis was that there is no significant correlation between water quality influent and dry hours and/or water quality effluent and dry hours. The alternate hypothesis was that there is a significant correlation between water quality influent and dry hours and/or water quality effluent and dry hours. If either influent or effluent showed a significant relationship with dry hours, a regression analysis was performed.

CHAPTER IV

RESULTS

Study Site

Berdoll Farms wet pond is the only pond that met ECM criteria out of the five ponds for which water quality and rain data were provided by COA staff. The pond is located in east Austin, southeast of the intersection of State Highway 71 and Farm-to-Market Road 973 (Figure 4). The pond was constructed as part of a single-family subdivision (Figures 5-8). Construction of the pond began in September 2000 and was completed in February 2001 (Pasquarella 2008). According to the subdivision construction plan sheets submitted to COA for review prior to development, the drainage area is 16.59 acres (Appendix B). The storm water discharge flowing into this wet pond is from within this subdivision; there is no off-site drainage into this pond. Based on impervious cover assumptions established by the COA and the information provided by the engineer that submitted the construction plans, the impervious cover within the drainage area for the pond is 10.87 acres. This value includes streets, houses, and driveways.

The volume capacity of Berdoll Farms wet pond is 9.86 acre-feet. The pond is approximately 250 feet by 100 feet, with depths up to 11 feet. The design height of the

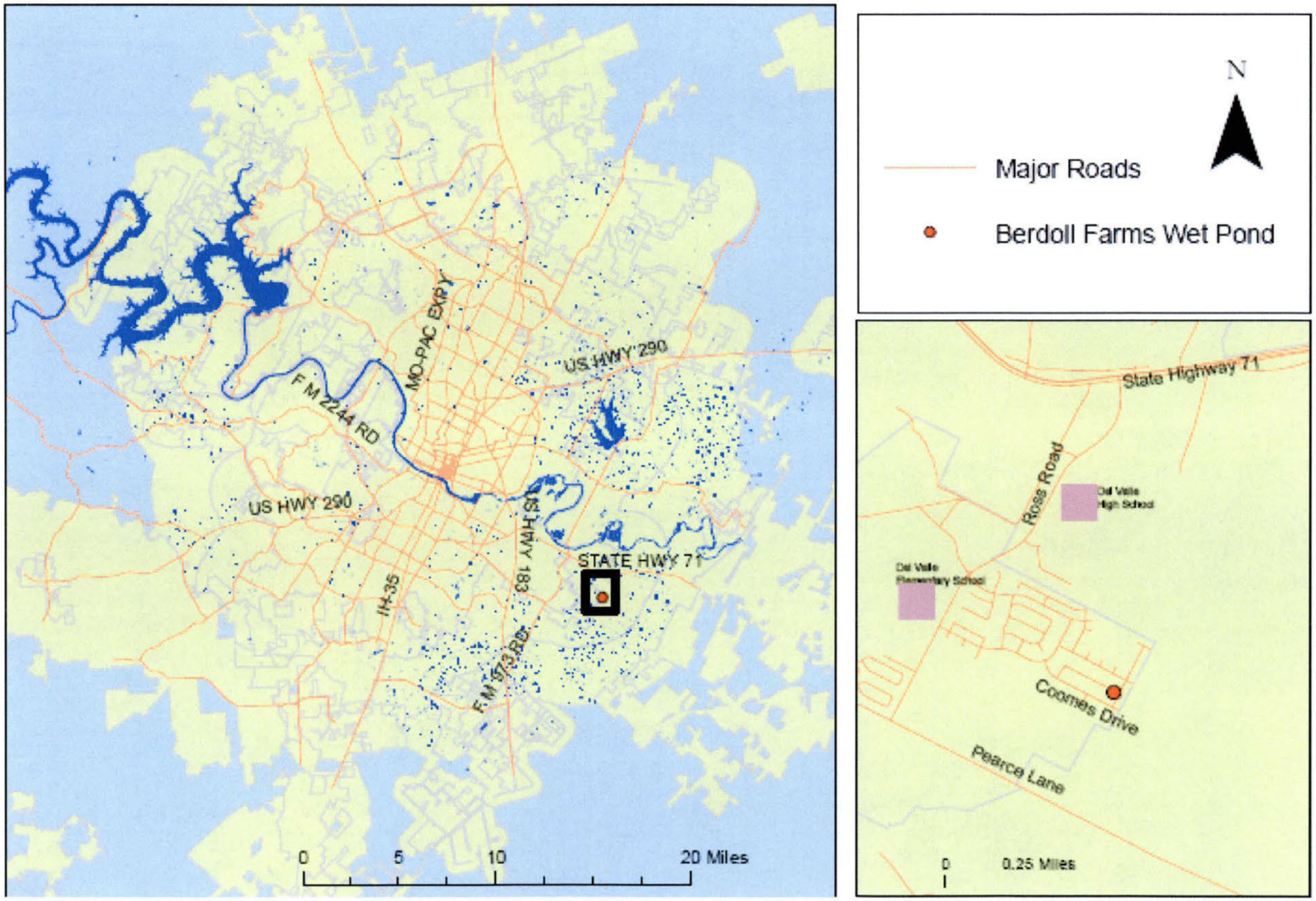


Figure 4. Location map for Berdoll Farms wet pond.

Figure 5. View of Berdoll Farms wet pond, with the sediment bay in the foreground and the main pool in the background.

Figure 6. The inflow structure for Berdoll Farms wet pond.

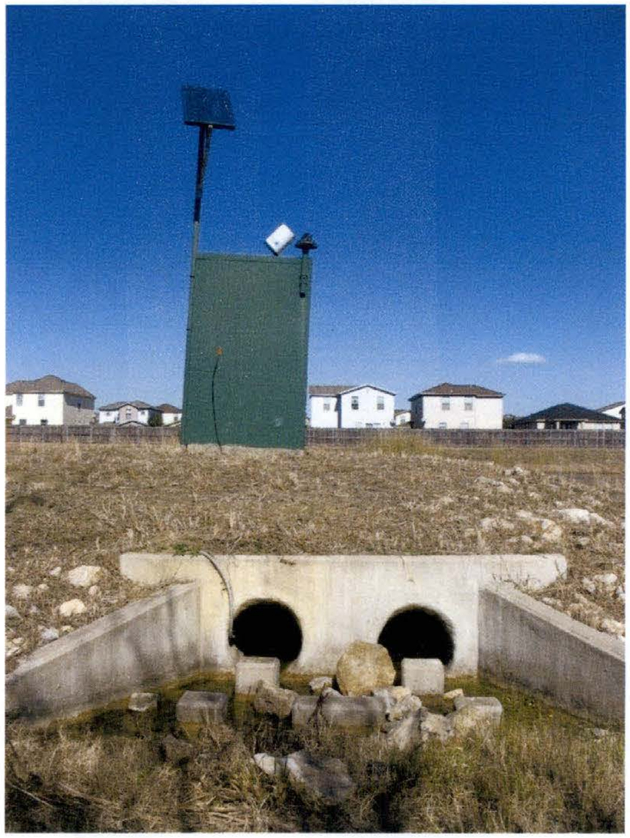


Figure 7. Outfall structure and monitoring equipment for Berdoll Farms wet pond.

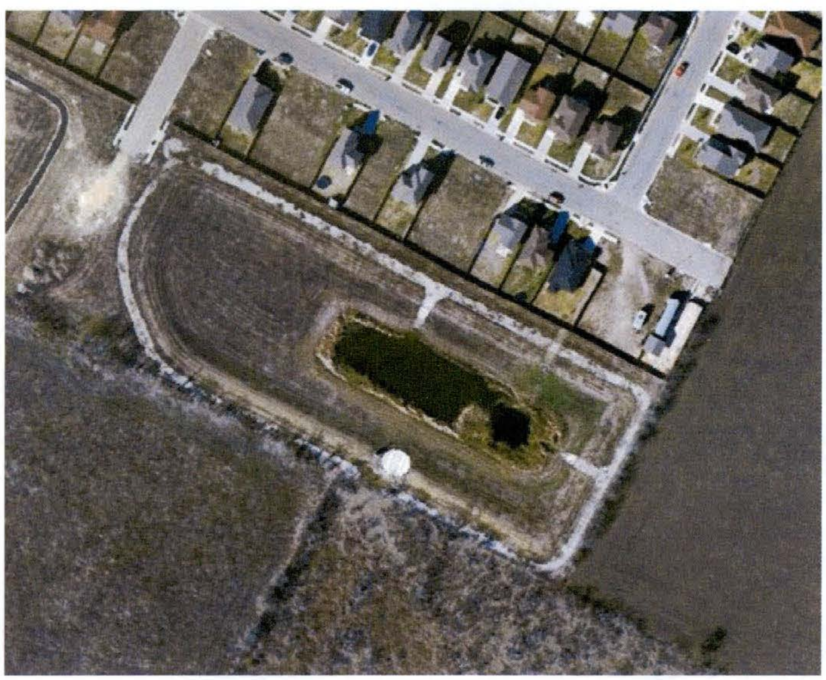


Figure 8. Aerial view of Berdoll Farms wet pond.

permanent pool volume is contour line 464.0 which is eight feet from the floor of the main pool (Appendix C). Vegetation consists of spikerush at or near the pond edge, bulrush at the pond edge and along the area separating the forebay and main pool, and various marsh and aquatic plants throughout the water (Appendix D, Figures 5-7).

Data

The event date, pollutant load, rainfall volume, event mean concentration, and dry hours for each parameter for influent and effluent water studied is included in Appendix E.

Statistical Analysis

Table 2 provides a summary of results of the statistical analysis for each parameter, including sample size, mean (presented as the geometric mean), standard deviation, P value from T-test, Pearson P value from correlation analysis, and R square, constant, and coefficient from regression analysis. The complete results and the output from statistical analyses performed are presented in Appendix F.

Cd influent water quality had a mean of -.5809 µg/l and a standard deviation of .37234. Cd effluent water quality had a mean of -.6931 µg/l and a standard deviation of .0000. All but one influent and all effluent values obtained for Cd were below the minimum detention limit (TCEQ 2003). The T-test for Cd had a P value of .341 which failed to reject the null hypothesis; therefore, the transformed influent and effluent means are not significantly different. No correlation or regression analysis was performed (Table 2).

Table 2. Summary of statistical analysis for all parameters.

	Sample Size (N)	Ln (Mean) (mg/l)*		Standard Deviation		T-test (P values)	Correlation (Pearson P values)		Regression***		
		Influent	Effluent	Influent	Effluent		Influent	Effluent	R Square	Constant	Dry Hours Coefficient
Cd	11	[-.5809]	[-.6931]	.37234	.0000	P=.341	N/A**	N/A	N/A	N/A	N/A
COD	12	3.8685	3.2948	.71556	.38883	P=.007	P=.001	P=.468	.684	3.228	.007
Cu	12	[1.6480]	[1.3131]	.61818	.48092	P=.004	P=.714	P=.207	N/A	N/A	N/A
DP	12	-1.6661	-2.5333	.72923	1.03769	P=.002	P=.479	P=.653	N/A	N/A	N/A
NH3	12	-1.5934	-2.3796	.62538	.84405	P=.005	P=.159	P=.866	N/A	N/A	N/A
NO23	12	-.4170	-2.4256	.67439	1.15430	P<.0005	P=.045	P=.817	.345	-.845	.005
Pb	12	[.6874]	[.5430]	.36037	.34738	P=.026	P=.883	P=.655	N/A	N/A	N/A
TKN	12	-.3119	-.4867	.76777	.63905	P=.360	N/A	N/A	N/A	N/A	N/A
TN	12	.3806	-.2968	.64911	.62944	P=.001	P=.134	P=.588	N/A	N/A	N/A
TOC	12	2.1919	1.9870	.53782	.25500	P=.128	N/A	N/A	N/A	N/A	N/A
TP	12	-1.0070	-1.6980	.65718	.57062	P=.013	P=0.064	P=.831	N/A	N/A	N/A
TSS	12	5.1317	3.4705	.77093	.68515	P<.0005	P=.007	P=.034	.535 / .376	4.522 / 3.908	.007 / -.006
VSS	11	3.0479	2.0166	.69808	.52637	P=.001	P=.006	P=.967	.123	3.415	.002
Zn	12	[3.6169]	[2.2163]	.53174	.80503	P=.001	P=.264	P=.967	N/A	N/A	N/A

*All values within [ ] are in µg/l.

**Not applicable since results of T-test and/or correlation did not necessitate further analysis

***For TSS, the regression is presented for both the influent and effluent water quality. The first half of the cell shows the influent values and the second half of the cell shows the effluent values.

Mean influent water quality for COD was 3.8685 mg/l with a standard deviation of .71556. Mean effluent water quality was 3.2948 mg/l with a standard deviation of .38883. The T-test for COD had a P value of .007 which rejected the null hypothesis. The alternate hypothesis that the means are significantly different can be accepted. The correlation analysis for COD rejected the null hypothesis for influent water quality and dry hours (P=.001) but did not reject the null hypothesis for effluent water quality and dry hours (P=.468). The alternate hypothesis that a significant correlation exists between influent water quality and dry hours was accepted. The liner regression for lognormal influent water quality can be expressed as: ln(influent water quality) = 3.228 + .007(dry hours) (Table 2).

Cu had a mean of 1.6480 μg/l for influent water quality and a standard deviation of.71556. Effluent water quality had a mean of 1.6480 μg/l and a standard deviation of .48092. The T-test for Cu rejected the null hypothesis with a P value of .004. Therefore, the influent and effluent means are significantly different. The correlation analysis for influent and effluent water quality and dry hours showed P values of .714 and .207, respectively, which failed to reject the null hypothesis. Since there is no correlation between influent and effluent water quality and dry hours, no regression analysis was performed (Table 2).

DP influent water quality had a mean of -1.6661 mg/l and a standard deviation of .72923. DP effluent water quality had a mean of -2.5333 mg/l and a standard deviation of 1.03769. The T-test for DP rejected the null hypothesis with a P value of .001. Therefore, the alternate hypothesis that the influent and effluent means are significantly different can be accepted. The influent and effluent water quality and dry hours correlation analysis had P values of .479 and .653, respectively, which failed to reject the null hypothesis. Since there is no correlation between influent and effluent water quality and dry hours, no regression analysis was performed (Table 2).

The mean influent water quality for NH3 was -1.5934 mg/l; the standard deviation was .62538. The mean effluent water quality for NH3 was -2.3792 mg/l; the standard deviation was .84405. The T-test for NH3 showed a P value of .005 which rejected the null hypothesis and accepted the alternate hypothesis. The influent and effluent means are significantly different. The influent and effluent water quality and dry hours correlation analysis had P values of .159 and .866, respectively, which failed to reject the null hypothesis. Since there is no correlation between influent and effluent water quality and dry hours, no regression analysis was performed (Table 2).

Influent water quality for NO23 had a mean of -.4170 mg/l and a standard deviation of .67439. Effluent water quality for NO23 had a mean of -2.4256 mg/l and a standard deviation of 1.15430. The paired sample T-test rejected the null hypothesis for NO23 with a P value of <.0005. The alternate hypothesis that the influent and effluent means are significantly different can be accepted. The influent water quality and dry hours correlation rejected the null hypothesis (P=.045) but the effluent water quality and dry hours analysis did not show a significant correlation (P=.817). Since the alternate hypothesis was accepted for correlation between influent water quality and dry hours, a regression analysis was performed. The linear regression for NO23 entering Berdoll Farms wet pond is: ln(influent water quality = -.845 + .005(dry hours) (Table 2).

Pb influent water quality had a mean of .6874 μg/l and a standard deviation of .36037. Pb effluent water quality had a mean of .5430 μg/l and a standard deviation of

.34738. The T-test for Pb rejected the null hypothesis with a P value of .026. Therefore, the alternate hypothesis that the influent and effluent means are significantly different can be accepted. The influent and effluent water quality and dry hours correlation analysis had P values of .883 and .655, respectively, which failed to reject the null hypothesis. Since there is no correlation between influent and effluent water quality and dry hours, no regression analysis was performed (Table 2).

The mean of TKN influent water quality was -.3119 mg/l; the standard deviation was .76777. The mean of TKN effluent water quality was -.4867 mg/l; the standard deviation was .63905. The paired T-test for TKN influent and effluent water quality mean had a P value of .360; therefore, the null hypothesis was not rejected and the means are not significantly different. As a result, no correlation or regression analyses were performed (Table 2).

TN had a mean influent water quality of .3806 mg/l and a standard deviation of .64911. The effluent water quality mean was -.2968 mg/l with a standard deviation of .62944. TN showed a significant difference between influent and effluent means based on the T-test (P=.001). However, the correlation analyses did not reject the hypothesis for influent water quality and dry hours (P=.0134), or effluent water quality and dry hours (P=.588) (Table 2).

TOC had an influent water quality mean and standard deviation of 2.1919 mg/l and .53782, respectively. The effluent water quality mean and standard deviation were 1.9870 mg/l and .25500, respectively. The T-test for TOC failed to reject the null hypothesis (P=.128). Therefore, the influent and effluent means are not significantly different and no further analysis was performed (Table 2).

TP had an influent water quality mean of -1.0070 mg/l and a standard deviation of .65718. The mean effluent water quality for TP was -1.6980 mg/l with a standard deviation of .57062. The influent and effluent means were determined to be significantly different by the T-test (P=.013). There was no correlation between influent water quality and dry hours (P=.64) and effluent water quality and dry hours (P=.64) so no regression analysis was performed (Table 2).

TSS had a mean influent water quality of 5.1317 mg/l and a standard deviation of .77093. Mean effluent water quality was 3.4705 mg/l with a standard deviation of .52637. The paired T-test rejected the null hypothesis that the two means were equal (P<.0005), so influent and effluent means are significantly different. The correlation analysis of influent water quality and dry hours had a P value of .006, which rejected the null hypothesis. The correlation analysis for effluent water quality and dry hours was significant, also rejecting the null hypothesis that there is no correlation (P=.034). The linear regression for TSS influent water quality is: ln(influent water quality) = 4.522 + .007(dry hours). The linear regression for TSS effluent water quality is: ln(influent water quality) = 3.908 + -.006(dry hours) (Table 2).

VSS had a mean influent water quality of 3.0479 mg/l with a standard deviation of .69808. The mean effluent water quality was 2.0166 mg/l with a standard deviation of .52637. The T-test for VSS rejected the null hypothesis and accepted the alternate hypothesis that the means are significantly different (P=.001). The P value for the correlation between influent water quality and dry hours was .006; therefore, there is a significant correlation. The P value for effluent water quality and dry hours correlation was .967; therefore, the null hypothesis was not rejected and there is no significant correlation. The linear regression for VSS influent water quality entering Berdoll Farms wet pond is: ln(influent water quality) = 3.415 + .002(dry hours) (Table 2).

Zn influent water quality had a mean of 3.6169 μg/l with a standard deviation of .53174. Effluent water quality had a mean of 2.2163 μg/l with a standard deviation of .80503. The T-test for influent and effluent means had a P value of .001 so the null hypothesis is rejected. A correlation analysis on influent water quality and dry hours and effluent water quality and dry hours both failed to reject the null hypothesis with P values of .264 and .967, respectively. Since a significant correlation does not exist for either value, no regression analysis was performed (Table 2).

CHAPTER V

CONCLUSIONS

Based on the results of the paired t-tests, Berdoll Farms wet pond is generally effective in removing pollutants prior to discharge since the mean water quality discharged from the pond was lower than the mean water quality entering the pond for all but three variables (Cd, TKN, TOC). This relationship suggests that a significant amount of pollutants discharged into the wet pond either settle out or are taken up by vegetation and not discharged.

For four variables, COD, NO23, TSS, and VSS, the number of hours between rain events significantly affected the water quality going into or out of the pond. For COD, NO23, TSS, and VSS, it can be concluded that the greater the time between storm events, the poorer the water quality entering the pond. Unfortunately, this relationship was not seen for all parameters studied (i.e. Cd, Cu, DP, NH3, Pb, TKN, TN, TOC, TP, and Zn). This could be a result of small sample size or that certain parameters do not experience correlation with dry time. Based on the statistical results, it can also be concluded that levels of TSS in water flowing out of the pond increases as the length of dry time between storm events increases.

The majority of significant correlation relationships existed between dry hours and influent water quality. COD, TSS, NO23, and VSS had a positive correlation between dry hours and pollutant concentration entering the pond. Approximately 68% of the variability in influent water quality can be accounted for by dry hours between storm events for COD; 53.5% for TSS; 35% for NO23; and 12% for VSS. TSS had a positive correlation between dry hours and pollutant concentration discharged from pond. Thirty-eight percent of TSS effluent water quality variability can be accounted for by the number of dry hours.

The four variables that showed a significant positive relationship between dry hours and influent water quality (COD, NO23, TSS, and VSS) and dry hours and effluent water quality (TSS), were either organic or inorganic; none were metals. This observation suggests that inorganic and organic pollutants respond differently to the number of dry hours compared to metals. If this relationship is present only for influent water quality, this would have no effect on COA's wet pond criteria. Since TSS showed a significant positive relationship between dry hours and effluent water quality in this study, TSS should be evaluated further. These results may suggest an inadequacy in this pond that should be addressed. This inadequacy could be due to errors in the initial construction/design of the pond or a lack of maintenance (i.e. sediment buildup or vegetation that died and was not replaced). Second, these results may be beginning evidence of a trend for other organic and inorganic parameters, and possibly metals as well.

Recommendations as to how to revise COA's wet pond criteria are not appropriate based on the small sample size and lack of significant correlation for most variables. A long-term study with a larger data set would be appropriate to further investigate the relationship between pollutants and the time between storm events.

CHAPTER VI

CHALLENGES

The availability of water quality data was a limiting factor in this study. Initially, this study intended to analyze and compare water quality data from two wet ponds. After determining that Berdoll Farms and Ceylon Tea wet ponds met COA criteria, the water quality data were compared to determine overlapping dates of collection. Water quality data were obtained from 7/3/2003 to 6/21/2006 for Berdoll Farms pond and from 7/7/05 to 6/24/06 for Ceylon Tea pond, which resulted in only one year of overlap. Once paired events were determined, each parameter had approximately five paired events. Since the number of paired events was so low, the study was revised to include all of the data for one pond.

Since the criteria for wet ponds were established in 2000, the longest monitoring timeframe for any data set received was seven years. The data provided for Berdoll Farms pond (and others) initially appeared to provide a ploethera of water quality information. However, after removing non-paired influent and effluent values from the Berdoll Farms data, the usable data were reduced to as few as 11-12 paired events. A follow-up study using the methods employed by this study should be performed when additional data are available for paired events. A larger sample size would likely result in additional correlations between influent (and possibly effluent) water quality and dry hours.

APPENDIX A

City of Austin Wet Pond Criteria

(COA 2003)

1.6.0 DESIGN GUIDELINES FOR WATER QUALITY CONTROLS

1.6.1 Introduction

This document provides guidelines for both the design of stormwater controls to enhance water quality and for the long-term maintenance of these facilities. These guidelines should be followed in order to provide protection for the water resources in the Austin area and to minimize time and effort in obtaining project review and approval. It is recognized that not all sites will permit ponds to be designed strictly according to these guidelines and that innovative designs are possible However, such deviations from these guidelines must be approved by the Director of the Watershed Protection and Development Review Department (WPDR) based upon a showing by the responsible party that site constraints prohibit conformance to the guidelines and that the alternative design will provide at least equivalent water quality benefit. Innovative designs must be based upon sound engineering and scientific principles and must, in the judgment of the Watershed Protection and Development Review Department, show reasonable likelihood of achieving water quality benefit equivalent to ponds designed according to the guidelines.

Stormwater can have significant impact on the water quality of Austin's creeks and the Colorado River To minimize the effect of non-point source pollutants in stormwater, best management practice (BMP) water quality controls are required to serve development. These water quality controls are designed to improve water quality by removing suspended particulate matter and associated constituents such as bacteria, nutrients and metals Sedimentation/filtration basins are the standard water quality control for new development (which is not required to meet a non-degradation standard) and are discussed in detail in Section 1.6.5. Alternative water quality controls which provide a level of water quality equal to or greater than sedimentation/filtration may be acceptable, but must be approved by the Director of the Watershed Protection and Development Review Department.

Applicants are encouraged to contact the WPDR staff prior to submitting plans proposing these alternatives. Minimum design guidelines for several alternatives are outlined in Section 1.6 6

Sites of less than one (1) acre may be subject to different requirements than larger sites Refer to Section 1.9.0 "Stormwater Filtration Criteria" for more information

Figures 1-46 and 1-47 in Appendix V of this manual illustrate water quality design options for suburban and water supply suburban watersheds, and water supply rural watersheds

1.6.2 General Design Guidelines

The following section discusses general design parameters which most BMP water quality controls have in common. These parameters include the volume of run-off which is to be treated, a method to isolate this volume, and liner requirements.

A. **Water Quality Volume.** The primary control strategy for water quality basins is to capture and isolate at least a minimum volume of stormwater runoff for treatment The minimum volume is the first one-half (0 5) inch of runoff plus an additional one-tenth (0 1) inch for each ten (10) percent increase of gross impervious cover over twenty (20) percent within the drainage area to the control. This depth of runoff from the contributing drainage area to the control is and will be referred to as the "Water Quality Volume." The water quality volume must consist of runoff from all impervious surfaces such as roadways, parking areas and roof tops, and all developed pervious areas Water quality treatment is not required for runoff from lands left in their natural state, e g , greenbelts and open spaces Runoff from these areas must be routed around the water quality basin or it must be included in the water quality volume. Off-site contributing drainage should be routed around the water quality basin. If this is not done, off-site contributing areas must be included in the water quality volume or a hydrologic study must be presented which indicates insignificant mixing with the on-site water quality volume. A separate case from the above is a commercial subdivision. Since development on individual lots in commercial subdivisions will incorporate water quality controls, the water quality volume for roadways in commercial subdivisions may be based on only the likely contributing drainage area of the roadway after the lots are developed That is, contributing drainage to roadways from the individual lots does not have to be included in the water quality volume for a commercial subdivision provided that the total drainage area contributing to the roadway pond does not exceed fifty (50) acres Section 1.6.10 includes example calculations for determining water quality volumes.

Because travel time from distant contributing areas reduces the effectiveness of the water quality controls in capturing all of the water quality volume, a maximum contributing drainage area of fifty (50) acres per water quality control basin is recommended.

B **Water Quality Volume Diversion Structures.** Off-line water quality controls are required to have a diversion structure or splitter box which will capture and isolate the water quality volume A typical approach for achieving isolation of the water quality volume is to construct an isolation/diversion weir in the stormwater channel such that the height of the weir equals the elevation of the water quality volume in the pond When runoff in excess of the water quality volume enters the stormwater channel it will spill over the isolation/diversion weir with minimal mixing with the already isolated water quality volume The splitter design must be capable of passing the peak flow rate of a twenty-five (25) year storm into the water quality pond, and pass the peak flow rate of the one-hundred (100) year design storm past the basin without overtopping the pond walls

Figures 1-48 through 1-50 in Appendix V of this manual present examples of these structures

C **Basin Liners.** Impermeable liners are required for water quality basins located over the South Edwards Aquifer Recharge Zone and in areas where there is surface runoff to groundwater conductivity. Impermeable liners may be either clay, concrete or geomembrane If geomembrane is used, suitable geotextile fabric shall be placed on the top and bottom of the membrane for puncture protection Clay liners shall meet the following specifications

TABLE 1-6 CLAY LINER SPECIFICATIONS			
Property	**Test Method**	**Unit**	**Specification**
Permeability	ASTM D-2434	Cm/Sec	1×10^{-6}
Plasticity Index of Clay	ASTM D-423 & D-424	%	Not less than 15
Liquid Limit of Clay	ASTM D-2216	%	Not less than 30
Clay Particles Passing	ASTM D-422	%	Not less than 30
Clay Compaction	ASTM D-2216	%	95% of Standard Proctor Density
Source: City of Austin			

The clay liner shall have a minimum thickness of twelve (12) inches.

If a geomembrane liner is used it shall have a minimum thickness of thirty (30) mils and be ultraviolet resistant.

The geotextile fabric (for protection of geomembrane) shall meet the following specifications:

TABLE 1-7 GEOTEXTILE FABRIC SPECIFICATIONS			
Property	**Test Method**	**Unit**	**Specification**
Material	Nonwoven geotextile fabric		
Unit Weight		Oz /Sq Yd	8 (min)
Filtration Rate		In/Sec	0 08 (min)
Puncture Strength	ASTM D-751 (Modified)	Lb	125 (min)
Mullen Burst Strength	ASTM D-751	Psi	400 (min)
Tensile Strength	ASTM D-1682	Lb.	200 (min)
Equiv. Opening Size	US Standard Sieve	No	80 (min)

Equivalent methods for protection of the geomembrane liner will be considered by the Watershed Protection and Development Review Department on a case by case basis. Equivalency will be judged on the basis of ability to protect the geomembrane from puncture, tearing and abrasion.

Concrete liners may be used for sedimentation chambers and for sedimentation and filtration basins less than one-thousand (1,000) square feet in area. Concrete shall be five (5) inch thick Class A or better as defined in the City of Austin Standard Specifications and shall be reinforced by steel wire mesh. The steel wire mesh shall be six (6) gauge wire or larger and six (6) inch by six (6) inch mesh or smaller An Ordinary Surface Finish (as specified in Item 410 25 of the City of Austin Standard Specifications) is required. When the underlying soil is clay or has an unconfined compressive strength of one-quarter (0.25) ton per square foot or less, the concrete shall have a minimum six (6) inch compacted aggregate base consisting of coarse sand and river stone, crushed stone or equivalent with diameter of three-quarters (0.75) to one (1) inch. Where visible, the concrete shall be inspected annually and all cracks shall be sealed.

When required for sedimentation/filtration basins, the liner shall underlie both the sedimentation and filtration chambers.

1.6.3 Maintenance and Construction Requirements

A. **Maintenance Responsibilities.** Proper maintenance is as important as engineering design and construction in order to ensure that water quality controls will function effectively. Section 25-8-231 of the Land Development Code requires maintenance be performed on water quality controls when necessary as defined by this section

Water quality controls required for commercial and multi-family development shall be maintained by the property owner

Maintenance of full sedimentation/filtration basins for single family or duplex residential development shall be by the City of Austin, unless otherwise approved during the review process.

The City will be responsible for the maintenance of ponds designed to service primarily publicly owned roads and facilities. These ponds must be designed and built according to the full sedimentation/filtration configuration

B **Maintenance Requirements–Design and Construction.** The design of drainage facilities (including but not limited to headwalls, open channels, storm sewers, area inlets, and detention, retention and water quality controls and their appurtenances) shall comply with the requirements of Section 1 2.4.E of the Drainage Criteria Manual. In addition, drainage facilities shall comply with the following construction requirements:

1. Sediment removed from detention, retention, or water quality facilities may be disposed of on-site if properly stabilized according to the practices outlined in the erosion and sedimentation control criteria found in Section 1 4 0 of this manual An off-site disposal site must either be an approved landfill or be issued a permit through the Watershed Protection and Development Review Department

2. The temporary erosion and sedimentation control plan must be configured to permit construction of detention, retention or water quality facilities while maintaining erosion and sedimentation control.

3. No runoff is to enter the sand filtration chamber of the sedimentation/filtration basin prior to completion of site construction and revegetation. Construction runoff may be routed to the sedimentation chamber but outflow from this structure shall bypass the sand filtration basin. It should be noted that good temporary erosion/sedimentation controls are essential to prevent heavy sediment loads caused by home construction from clogging the filtration media.

C. **Major Maintenance Requirements.**

1. Sedimentation and Detention Basins.

   a. Silt should be removed when the accumulation exceeds six (6) inches in sediment basins without sediment traps. In basins with sediment traps, removal of silt shall occur when the accumulation exceeds four (4) inches in the basins, and the sediment traps shall be cleaned when full. In detention basins, silt shall be removed and the basin restored to original lines and grades when standing water conditions occur or the basin storage volume is reduced by more than ten (10) percent.

   b. Accumulated paper, trash and debris should be removed every six (6) months or more often as necessary to maintain proper operation.

   c. Vegetation within the basin shall not exceed eighteen (18) inches in height at any time, except as called for in the design

   d. The basin shall be inspected annually and repairs shall be made if necessary.

   e. Corrective maintenance is required any time a sedimentation basin does not drain the equivalent of the Water Quality Volume within sixty (60) hours of cessation of inflow or a detention basin does not drain completely.

   f. Corrective maintenance is required any time the sediment trap in a sedimentation basin does not drain completely within ninety-six (96) hours of cessation of inflow.

   g. To limit erosion, no unvegetated area shall exceed ten (10) square feet.

   h. Structural integrity of basins shall be maintained at all times.

2. Filtration Basins.

   a. Accumulated paper, trash and debris should be removed every six (6) months or as necessary.

   b. Vegetation within the basin should not be allowed to exceed eighteen (18) inches in height at any time.

   c. Corrective maintenance is required any time draw-down does not occur within thirty-six (36) hours after the sedimentation basin has emptied

   d. The basin should be inspected annually and repairs should be made if necessary.

3. Wet Ponds.

Due to the nature of wet ponds being full of water when in operation, the need for maintenance is not easily visible and the ponds can be difficult to maintain. However, when the ponds are built in stable upland areas, the need for maintenance of these ponds should be infrequent. Accumulation of sediment in the basin is the primary reason the pond will require intensive maintenance. Because of this, very careful attention should be paid to adequate, well-maintained erosion and sedimentation controls in the contributing drainage area during construction This, in combination with the sediment forebay, should prevent the requirement of maintenance of the main pool soon after the pond is put online. The following are guidelines for pond maintenance:

During Site Construction - The sediment load to the sediment forebay shall be closely monitored after every storm event. If heavy sediment loads are detected during an inspection, the source should be corrected. Sediment shall be removed from the sediment forebay when one-third of the forebay volume is lost.

Upon Completion of Site Revegetation - Any sediment build-up (greater than 5% volume loss) shall be removed from the forebay upon completion of site revegetation. The sediment build-up in the main pool shall be checked and if more the ten- percent of the volume is lost, it should be cleaned at that time.

Every Three Months for the First Two Years - During the three month initial inspection cycle, if more than fifteen percent of the volume of the forebay is lost, it shall be cleaned at that time.

Every Three Months - Turf areas around the pond should be mowed. Accumulated paper, trash, and debris shall be removed every three months or as necessary. Cattails, cottonwoods, and willows can quickly colonize shallow water and the edge of the pond. These species, or any areas of plant overgrowth may be thinned at this time or as needed.

Annually - The basin should be inspected annually for side slope erosion and deterioration or damage to the structural elements. Any damage shall be repaired. Large areas, which have dead or missing vegetation, shall be replanted.

Every Three Years - The sediment build-up in the sediment forebay shall be checked. The sediment forebay shall be cleaned if more than one-third of the forebay volume is lost.

Every Six Years - The sediment build-up in the main pool shall be checked. Sediment shall be removed from the main pool when twenty percent of the main pool volume is lost.

1.6.4 Types of Water Quality Controls and Selection Criteria

Sedimentation/filtration is the primary structural water quality control to reduce non-point source pollution in Urban, Suburban, Water Supply Suburban and Water Supply Rural Watersheds. In the Barton Springs Zone, non-degradation water quality controls are required (Please refer to Section 1.6 9 for design criteria for non-degradation controls).. Alternative controls may be acceptable if they are designed to result in a level of water quality equivalent to or better than sedimentation/filtration based upon sound engineering evidence. However, these systems must be approved by the Director of the Watershed Protection and Development Review Department (WPDR) The guidelines for several alternative controls are being developed and the WPDR should be contacted for guidance.

A. **Sedimentation/Filtration Systems.** Sedimentation/filtration systems are the primary water quality control structures. In these systems, the water quality volume is directed to a sedimentation structure followed by a filtration basin; subsequent additional runoff is diverted to a stormwater detention basin as specified in the Drainage Criteria Manual. The sediment basin is required prior to the filtration basin in order to ensure the long-term effectiveness of these systems by protecting the filter media from excessive sediment loading. Two configurations of filtration systems are described in Section 1.6.5.

In full sedimentation/filtration systems, the sedimentation structure is a basin designed to hold the entire water quality volume and to release the water quality volume to the filtration basin over an extended draw-down period.

In partial sedimentation/filtration systems, a sedimentation chamber is located upstream of the filtration basin which is not required to hold the entire water quality volume and will not incorporate an extended draw-down period. This system is designed to remove the heavier sediment and trash litter only and may require more intensive maintenance than the full sedimentation systems. However, partial sedimentation/filtration systems require less depth than the full sedimentation system and may be applicable where topographic constraints exist.

Full sedimentation/filtration systems shall be required where the City is responsible for maintenance unless topographic constraints make this design unfeasible. Unfeasible is considered: assuming (for the purposes of this selection process only) a maximum ponding depth of three (3) feet in the sedimentation basin, if it is not feasible to obtain an outlet for the drainage from the filtration basin within one-hundred (100) feet of the crest of the filtration embankment, then the partial sedimentation/filtration configuration system may be used. If the City is not to be responsible for maintenance of the pond system, either configuration is allowable.

B **Wet Ponds.**

The design of wet ponds for stormwater quality and quantity control may, more than any other control, requires more planning and thoughtful design When properly designed, wet ponds are highly effective in removing stormwater pollutants and can add to the aesthetics of a site or neighborhood. These systems can also be used when the grade of the site is relatively flat. A drawback with these systems can be the long-term maintenance of the facility. Proper measures must be taken to reduce the sediment load, which can be the largest single factor which contributes to the need for maintenance of a wet pond.

The design goal for wet ponds is to have a permanent pool with an average minimum hydraulic residence time of 14 days. This capturing and holding of runoff allows settling of suspended solids and biological uptake of nutrients

Section 1.6.6A outlines the design criteria for wet ponds. When wet ponds are designed to this criteria, they are assumed (based upon local monitoring data) to provide a level of water quality treatment equivalent to sedimentation/filtration. Specific removal efficiency information will be provided when additional monitoring data is available.

A wet pond, when designed and maintained according to the following criteria, will not become a critical environmental feature as defined by the City of Austin

C. **Sedimentation Systems.**

Sedimentation systems are not considered equivalent to sedimentation-filtration controls in terms of water quality treatment. Sedimentation systems may be appropriate when used as part of a series of water quality controls. The use of sedimentation systems will be evaluated on a case-by-case basis to determine if the proposal can be expected to achieve water quality standards. In sedimentation systems, the water quality control strategy is to optimize settling characteristics within the water quality basin in order to remove pollutants by deposition. Water quality enhancement shall be achieved by providing extended draw-down time for the water quality volume.

C **Pollutant Removal Efficiencies for Sand Filtration Systems.** For filtration systems designed in accordance with the guidelines in this section, the following pollutant removal efficiencies are to be assumed:

Removal Efficiency	TSS	TP	TN	COD	BOD	Pb	FC	FS	TOC	Zn
(%)	87	61	31	67	51	80	36	65	61	80

These values are based on a report titled "Removal Efficiencies of Stormwater Control Structures" dated May 1990 by the Environmental Resource Management Division of the WPDR These values will be updated as more data becomes available. For estimating pollutant loading for runoff, the data in Section 1.6.9.3 should be used.

1.6.6 Design Guidelines for Wet Ponds

A. **Wet Ponds.** Wet ponds are designed to use gravitational forces and biological activities to remove urban stormwater pollutants before discharging the treated runoff into a waterway They are typically designed as on-line systems which can also meet the onsite stormwater detention requirement for streambank erosion protection and flood mitigation A literature review of wet ponds (References 111-119 in the Bibliography) was conducted in order to establish design criteria. Figures 1-59B and 1-59C in Appendix V of this manual illustrate a typical system Characterization of the subsurface strata and groundwater, through borings and/or piezometers as per standard geotechnical investigation methods, must be performed with resulting information incorporated into the pond design.

1. Capture Volume Wet ponds in general are designed to have three stages with three corresponding volumes, which are intended to meet the water quality and detention requirements. The first two stages, permanent pool and extended detention, are required for all ponds and function primarily as a water quality control. The second stage may also serve as a streambank erosion prevention measure. The third stage, flood control detention, serves as a flood control measure and is optional to the design of the wet pond. The permanent pool and extended detention volume shall be designed for the entire drainage area contributing to the control for which water quality controls are not already provided. Offsite areas, which are currently undeveloped, may be assumed undeveloped in the design. The primary reason to require extended detention for all of the developed drainage areas, which have not provided detention, is to prevent pond washout caused by high flow-through rates.

    a. Permanent Pool - The permanent pool, the lowest stage of the pond, is designed to hold and treat a volume of runoff between storm events through quiescent settling and biological uptake. The permanent pool should remain nearly full at all times to provide a source of water for wetland plants which are used for biological uptake and to minimize turbulence within the pond during storm events which may result in resuspension of sediment. During storm events, the pond is designed to flush out the treated water and replace it with "new" runoff.

    The removal efficiency of wet ponds is directly related to the time the runoff is held in the pond. The longer the runoff is held in the pond, the more settling and biological uptake that can occur. Based upon national and local monitoring data, a hydraulic residence time of two weeks would provide an equivalent level of water quality treatment as sedimentation/filtration Therefore, the permanent pool volume should be as large as the amount of runoff produced in a two-week period To ensure that the removal efficiency can be achieved during the "rainy" season, the rainfall data used is based upon the statistics for the average wettest month. In addition, the volume should be increased to account for losses associated with 15 years of sediment build-up. When the drainage area to the pond contains only uplands, an increase of volume by five percent is acceptable to account for this loss. If the pond is located where it may receive streambed loads, a more detailed analysis will be required to account for storage losses

The wettest mean monthly storm, which generates runoff in the Austin area, produces 0.72 inches of rainfall and occurs every 5.45 days The amount of runoff from 0.72 inches of rainfall can be estimated by multiplying the annual runoff coefficient found in Table 1-9 of Section 1.6.9 and the rainfall depth. To achieve the fourteen-day minimum residence time an adjustment coefficient is determined by dividing the desired residence time by the storm reoccurrence interval (5.45 days) Then the runoff depth, reoccurrence coefficient, loss factor, and drainage area are multiplied to determine a volume. The permanent pool volume may be calculated using the following equation:

$$V = (RT/RI) * WMMS * R_f * Ls * DA * 1'/12"$$

where "V" is the permanent pool volume (ac-ft), "RT" is the desired hydraulic residence time (14 days), "RI" is the reoccurrence interval for the wettest mean monthly storm (5.45 days), "WMMS" is the wettest mean monthly storm depth (0.72"), "Rf" is the annual runoff coefficient (Table 1-9 of Section 1.6.9), "Ls" is the storage loss coefficient, and "DA" is the drainage area (ac). By replacing the variables with local values and simplifying, the equation for permanent pool volume for ponds receiving upland runoff is:

$$V = 0.162 * R_f * DA$$

b Extended Detention - The extended detention portion of the pond minimizes turbulence in the pond by decreasing the pond flow-through rate and increasing the time in which sedimentation can occur during the storm through dynamic settling The extended detention volume for wet ponds should be designed to detain the one-year, three-hour storm for 24 hours, (Table 1-9A). Through the use of these guidelines, the extended detention volume is considered to meet the streambank erosion requirements. The extended detention volume cannot include the volume provided in the permanent pool because the permanent pool is designed to be full at the start of the rainfall event.

Table 1-9A
City of Austin 1-Year, 3-Hour design storm distribution
Cumulative Values (inches), 5 minute time increment

0 0	0.006	0 012	0.019	0 026
0.034	0.043	0 053	0.064	0 077
0 092	0.110	0.134	0.166	0 212
0 287	0.384	0 542	0 802	1.262
1 462	1.587	1.688	1.746	1 784
1 811	1.832	1 849	1.863	1.875
1 885	1.894	1 902	1.910	1.917
1.924	1.93	1.93	1.93	1 93

c. Flood Control Detention (optional) - The standard detention volume should be designed to meet the city's flood control requirement, in accordance with Section 8 of the Drainage Criteria Manual and it may include the volume contained as extended detention.

2. Drainage Area Limits - The drainage area to the pond must be large enough to allow an adequate supply of runoff. In addition, the need to provide pond depths great enough to minimize water surface fluctuations, an adequate area for vegetation, and enough surface area to allow aeration dictates this minimum drainage area. Due to these factors, a minimum drainage area of twenty acres is needed. Smaller drainage areas will be considered based upon a demonstration that these factors can be met.

   With very large drainage areas, disturbance of waterways can be excessive, high sediment bed loads can be expected, higher turbulence within the pond due to higher flow-through rates may occur, and maintainability may be decreased. Because of these factors, the drainage area may not exceed 320 acres. This upper limit, however, does not allow, recommend, nor encourage construction within the Critical Water Quality Zone established along waterways.

3. Basin Details - The permanent pool volume shall be held in two compartments. The first is called the sediment forebay and the second is called the main pool. These basins shall consist of deep pools and shallow vegetated benches. Other aspects of the pond include maintenance access points, maintenance pads, an outlet structure, and an impermeable liner.

   a. Sediment Forebay - All run-off shall enter the sediment forebay. Energy dissipation is needed at the inflow point(s) to prevent scouring of the basin floor and to quickly reduce the turbulence within the forebay. The forebay shall hold fifteen to twenty-five percent of the permanent pool volume. The sediment forebay and main pool shall be separated using a six inch or thicker reinforced concrete wall as required for structural integrity or earth berm. The separating wall will serve as a barrier for heavy sediments, trapping the majority of the sediment in the forebay, which should extend the maintenance interval for draining the entire pond. The top of the wall should be set at twelve inches below the permanent pool water surface elevation. This will allow the two basins to be hydraulically connected during normal operation. If a submerged earth berm is used, it should have a minimum top width of ten feet and meet the following conditions: 1.) The material used for construction must be stable when saturated and when the maximum hydrostatic force is applied, 2.) the side slope must be stable when saturated, and 3.) the berm must protect against erosive forces on the top of the berm in high flow conditions. When the earth berm is used, it should also be included as part of the vegetated bench.

      The forebay and main pool should be hydraulically connected with a horizontal twelve inch or larger Schedule 40 PVC pipe called an inter-basin pipe to ensure that there will be an adequate supply of water in the forebay in dry conditions. The elevation of the inter-basin pipe should be two feet above the bottom of the forebay and a plug valve included in the line to allow independent draining (by pump) of the sediment forebay after drawing both basins down to the top of the separating wall.

The depth of the sediment forebay shall be four to six feet and shall include vegetatec benches (as discussed in the main pool section below). The bottom of the forebay shoulc have a minimum two percent slope towards a low point. A reinforced concrete pad minimum twelve feet by sixteen feet, shall be provided to form a maintenance pad. Thi: maintenance pad shall be enlarged as needed to cover the portion of the basin whict can not be sloped inward at two percent. The purpose of the maintenance pad is to allov for routine removal of sediment using heavy equipment soon after the basin is drainec without requiring additional time for the basin bottom to dry. An examination of th hydrostatic forces on the maintenance pad when the forebay is empty and the main poo is full should be performed when designing the thickness of the pad. In no case shall th thickness of the pad be less than four inches. A twelve foot wide concrete maintenanc access ramp with a maximum slope of four to one and broom finish should lead from a least twelve inches above the permanent pool elevation to the maintenance pad.

b. Main Pool - The main pool shall contain the remainder of the permanent pool volume The pond shall contain two water depths. The first is called the deep pool and it sha' have a depth from six to eight feet. The bottom of the main pool should slope at on percent toward the maintenance drain or pump pad as discussed below when feasible Unless the pond has a large surface area to enhance aeration, areas deeper than eigh feet may result in the pond becoming anaerobic, possibly resulting in odors. The mai pool should have a length to width ratio greater than two to one (measured from eac inlet to the outlet) to prevent short-circuiting of the pond. Short-circuiting and th presence of dead storage areas in wet ponds are a common problem, exacerbated whe multiple inlets are used to discharge runoff into ponds. In order to prevent problems, th design engineer may be required to perform short-circuiting and dead storage analyses.

A permanently submerged shallow area surrounding the pond of approximately twenty percent of the total pond area should be used as and called a vegetated benct Pinnacles and islands may also be used to achieve the necessary area or to enhanc the aesthetics. This vegetated bench area should be a minimum of ten feet wide, slop inward at five to fifteen percent toward the deep pool, and have a maximum inundation c eighteen inches This vegetated bench should be planted with wetland plants a discussed in Section 5 below. Figure 1-59D in Appendix V of this manual is an exampl of a typical cross-section of the vegetated bench area.

c Pond Liner and Side Slopes - The sediment forebay and main pool shall have a minimu twelve inch (or thicker as required by geotechnical investigations) impermeable clay lin to prevent excessive seepage which may result in ground water contamination or i severe pond drawdown Clay liner specifications can be found in Table 1-6 In genera earthen side slopes of ponds should not exceed three to one, but the slope to be use should be designed carefully to ensure that it will be stable when saturated.

4. Outlet Structures - The design of the outlet pipe is important to enhance the plug flo characteristics of the pond. This section provides guidelines in designing the outlet structur Other designs will be evaluated for their ability to provide plug flow and maintainability. I most cases, the ponds will be designed with two primary outlet structures and a maintenanc drain In all cases, energy dissipation is required to prevent erosion at the outfall locatio Figure 1-59E in Appendix V of this manual is an example of a typical outlet structure

a. Extended Detention - The extended detention outlet structure should be constructed using an inverted PVC pipe with the soffit of the inlet set at an elevation which is two-thirds of the permanent pool depth from the bottom. The flow line of the outlet of the pipe shall be set at the permanent pool elevation. No outlet other than the extended detention outlet will be permitted below the extended detention volume In all cases, the pond will be designed so that the minimum pipe diameter is no less than six inches to minimize clogging potential, the size of the orifice at the end of the pipe may be smaller than six inches in order to achieve the required extended detention. If an orifice plate is used to achieve the required 24 hour drawdown, the orifice must be removable and accessible when the pond is at the extended-detention elevation in order to service blockage. It is recommended that this line discharge into the manhole required for the maintenance drain and discussed in that section.

If an orifice is not used to control the drawdown, the flow in the inverted discharge pipe used for extended detention should be calculated using a method which more accurately accounts for energy losses than the orifice equation. One equation which may be used is:

$$Q = A * ((2*g*h)/(1+k_e+k_b+k_f))^{0.5}$$

where Q is flow (cfs), A is the cross-sectional area of the pipe (sf), g is the acceleration due to gravity (32 2 ft/sec²), ke is the entrance loss coefficient (Table 7-1, DCM), kb is bend losses (Table 5-4, DCM), and kf is the friction loss coefficient. The friction loss coefficient can be found using the equation·

$$k_f = 29*L*n^2/R^{1.33}$$

where L is the pipe length (ft), n is the Manning's roughness coefficient (Table 4-2, DCM), and R is the hydraulic radius (ft).

b. Flood Control Detention - The Drainage Criteria Manual should be referenced for design of the outlet structure to serve for flood control This outlet should be designed for the 10, 25, and 100-year storm or as required in the DCM. When flood control detention is not needed, an overflow spillway capable of passing the 100-year storm is required at or above the elevation at which the extended detention volume is provided. To enhance water quality, a two to one length to width ratio from the inflow to the outflow should be maintained.

c. Maintenance Drain - A drain line, which can completely or partially drain the permanent pool, shall be included where topographic relief exists. The purpose of the drain is to allow for the pond to be drained for long-term maintenance activities. A plug valve shall be installed in the line and the valve should be protected by enclosing it in a manhole set in the pond berm. If the maintenance drain can not completely drain the pond, a six square foot concrete pump pad must be provided at the lowest point in the main pool which will provide a base for temporary installation of a submersible pump.

5. <u>Biological Elements</u> - Biological elements are an important aspect to the function as well as the aesthetics of the wet pond system However, these systems may also attract biological activity that is undesirable in an urban setting. The following criteria should be followed to enhance pollutant removal and minimize undesirable activity

a. Wetland Plantings - Wetland species plants are used in wet ponds to remove dissolvec nutrients and shall be planted on the vegetated benches as specified below

Minimum requirements for Wet Pond landscaping:

1. Minimum wetland plant quantity: Multiply the surface area (in square feet) of the permanent pool by three percent (.03) to determine the minimum quantity of plants to be installed in the vegetative bench.

2. The following chart provides plant category ratios and minimum plant sizes Additional information can be found in the plant list in Table 1-9B.

		MINIMUM SIZE	
PLANT CATEGORY	RATIO	Containers	Bare Root
A Bulrush:	40%	2 gallon	1 bare root
B. Spikerush:	20%	2.5" liners	1 bare root
C. Marsh diversity:	20%	1 gallon	1 bare root
D. Arrowhead	10%	1 gallon	1 bare root
E. Aquatics:	10%	1 gallon	1 bare root

3 Wetland plants provided in bare-root form shall be equal in root ball size to the listed minimum container sizes

4. All wetland plants which fulfill the minimum landscape requirements shall be propagated or harvested from regionally adapted stock (whenever possible) These are plant species or genotypes which are native to a range of within 250 miles of the project site

5 A minimum of 90% of the vegetation shall be alive and viable for one year following installation

Notes:

a Wetland plants must be installed at water depths appropriate to the species Th water depths noted in Table 1-9B show the range of natural zones in which thes plants can be found. Planting depths are usually shallower due to the small size (the plants at the time of installation. If using the minimum-sized plant material, plant shall be installed at the shallow water depth listed.

b Cattails (Typha spp) tend to invade almost all wetlands and aggressively coloniz the shallow water bench. Therefore cattails shall not be specified on the plantin plan

c. The designer is not limited to the species described. Additional species used f(aesthetic reasons, etc. are encouraged. Plants not intended to meet minimu requirements do not need to be native or regionally adapted stock

d. Microbial Initiation - A substantial portion of the pollutant removal in wet ponds is due to biological processes that occur in the sediment. Bacteria in the pond substrate remove nutrients through a process of denitrification. These microbial processes require an organic food source, such as decaying plant litter. Because it is the supply of organic carbon that determines nutrient removal – more than uptake by living plants – denitrification can be expected to continue even during cold-weather plant dormancy. In mature ponds with abundant vegetation, aquatic plants supply the necessary litter layer and aerobic zone for microbial activity. However, since new ponds lack a sufficient source of organic matter, an appropriate amount of carbon (straw, hay, leaf clippings, and other non-woody material) shall be installed during construction. After the pond liner is in place yet prior to allowing the pond to be filled, spread a minimum of one inch of plant litter evenly on the sides of the pond (below the permanent pool level). Treat the entire shallow water bench in this manner, and all pond slopes (ranging from 3:1 to 10:1). Crimp the plant litter into the pond substrate to prevent the material from being transported downstream as the pond fills.

e. Algae - High nutrient loads in wet ponds may cause algae blooms to occur. Pungent odor is often associated with these algae blooms. However, treating with an algaecide is not recommended because blooms are usually short lived and are considered desirable for nutrient removal. The use of submergents and floating-leafed aquatics can reduce the extent of algae blooms by reducing nutrient loads and shading the water.

f. Nutria - Wildlife such as nutrias has been reported to destroy the vegetated element of wet ponds in the Austin area. Evaluation of the potential of such wildlife inhabiting or being attracted to the proposed pond site is required When there is a potential for such activity, fencing (such as chain link) should be provided.

g. Mosquito Control - Mosquitoes are problematic in urban areas. Standing water in wet ponds becomes ideal breeding localities. The wet pond should be stocked with the fish species Gambusia affinis to serve as a biological control for mosquitoes. Gambusia is effective control for mosquitoes eliminating the need for chemical control. Gambusia should be stocked at the initial density of 200 individuals per surface acre.

h. Domestic Waterfowl - Domestic waterfowl can destroy vegetation and increase pollutant loading in wet pond systems. In addition, waterfowl can become nuisances to property owners near the pond. For these reasons, domestic waterfowl should not be introduced into these systems.

i. Carp and Goldfish - Carp and goldfish are bottom-feeders that can cause turbidity and other problems. They should not be introduced into a wet pond.

6 Initial filling - While the pond is in construction, it is intended that stormwater runoff, not potable water, be used to fill up the pond once the pond liner is in place.

7 Utility Lines - Utility lines may not be located within the limits of the maximum water surface elevation of a wet pond.

8. Hazardous Material Traps - Spills of hazardous liquids can severely damage or kill the biota of a wet pond. Therefore, developments where the transportation, storage, or distribution of hazardous materials is anticipated should include hazardous material traps in the drainage system immediately upstream of the wet pond inlet.

9. Aeration and Recirculation Unit (optional) - Privately maintained wet ponds may include some type of aeration device (such as a fountain) which could enhance the dissolved oxygen concentration. Increased dissolved oxygen prevents the pond from becoming anaerobic, hence minimizing problems with odor from bacterial decomposition.

10. Make-up Water Source - A nearby source for make-up water is recommended as a way to raise the level of the permanent pool, should a severe drought occur. This could include a well, a hose bibb, or a nearby fire hydrant.

11. Design Alternatives - All alternatives to these design criteria require approval by the Director of the Watershed Protection and Development Review Department. When a pond is designed to meet all volume, vegetated bench area, pond depth, length to width ratio, and outlet structure requirements, the pond will have been designed to achieve an average overflow rate of 0.42 feet per hour which will remove 20 micron and larger particles through dynamic settling. If topographic constraints, land availability, or other issues require deviating from the criteria, a check to ensure that the average overflow rate for the wet month mean storm does not exceed 0 42 feet per hour should be performed. The average overflow rate for the wet month mean storm may be estimated with the equation: $Q_{avg} = 581 * R_f * DA$, where R_f is the annual runoff coefficient and DA is the drainage area.

TABLE 1-9B
Wetland Plant List

Install Bulrush in clumps, with individual plants spaced approximately three to four feet on center. At least two of the following species shall be used:

BULRUSH	WATER DEPTH	NOTES
Scirpus validus Bulrush	1' – 3'	8' tall evergreen, resists cattail encroachment
Scirpus californicus Bulrush	1' – 3'	8' tall evergreen, resists cattail encroachment
Scirpus americanus Three-square bulrush	2" – 6"	2' to 4' tall, w/ 3 distinct edges

Install Spikerush at or near the water's edge, with individual plants spaced approximately three to six feet on center. At least two of the following species shall be used:

SPIKERUSH	WATER DEPTH	NOTES
Eleocharis montevidensis Spikerush	0" – 6"	1' tall, rhizomatous, reduces erosion at the pond edge
Eleocharis macrostachys Spikerush	0" – 6"	1' tall, rhizomatous, reduces erosion at the pond edge
Eleocharis quadrangulata Spikerush	3" – 1'	2' to 2.5' tall, rhizomatous, can accommodate deeper water, 4-angled

At least two species of the following marsh plants shall be used (additional species are encouraged) Install in clumps in shallow water, with individual plants spaced at approximately three feet on center:

MARSH DIVERSITY	WATER DEPTH	NOTES
1. Cyperus ochraeus Flatsedge	2" – 6"	1' to 2' tall, clump-forming, common to central Texas
2. Dichromena colorata White-topped Sedge	2" – 6"	1' to 2' tall, white bracts during warm season
3. Echinodorus rostratus Burhead	3' - 1'	1' to 2' tall, annual, heart-shaped leaves, flower similar to arrowhead
4. Eleocharis quadrangulata Four-square Spikerush	6" – 1'	1' to 2' tall, colonizes, inhabits deeper water than other Spikerushes
5 Iris Pseudacorus Yellow Flag Iris	1' – 2'	3' to 4' tall. can be invasive, dense growth, yellow flowers
6 Junctus effusus Soft Rush	6" – 1'	3' to 4' tall, forms a tight clump, evergreen, very attractive
7. Justicia americana Water willow	2" – 6"	2' to 3' tall, common, white flowers, herbaceous, colonizes
8 Marsilea macropoda Water Clover	2" – 6"	Looks like floating four-leaf clover, endemic to Texas
9. Najas guadalupensis Water-Naiad	1' – 4'	Submergent, valuable to fish and wildlife
10. Pontederia cordata Pickerelweed	2" – 1'	3' tall, colonizes, cosmopolitan, purple flowers
11. Rhynchospora corniculata Horned-rush	2" – 6"	2' to 3' tall, brass-colored flowers in May

Install Arrowhead in clumps in shallow water, with individual plants spaced approximately three feet on center.

ARROWHEAD	WATER DEPTH	NOTES
Saggitaria latifolia Arrowhead	2" – 1'	2' height, wildlife value, white flowers, proven water quality performer evergreen species platphylla is preferred

The following category, Aquatics, include submergents and floating-leaved aquatics. Submergents are rooted in the sediment of the pond, and are completely submerged in water. Floating-leaved aquatic plants are rooted in the sediment of the pond, and have leaves that float on the surface of the water. These leaves shade the water, which limits potential algae growth. At least two of the following species shall be used and should be placed at random locations throughout the pond:

AQUATICS	WATER DEPTH	NOTES
1. Cabomba caroliniana Fanwort	1' – 4'	Approximately 6' length underwater, submergent
2. Ceratophyllum spp. Coon-tail	1' – 4'	Maximum 8' length, tolerant of turbidity and water fluctuation, wildlife food
3. Nymphaea odorata Water-lily	6" – 2'	A native, reliably hardy, floating-leaved aquatic, with white flowers
4 Potomageton pectinatus Sago Pondweed	8" – 3'	Colonizes quickly, valuable to fish and wildlife; floating-leaved aquatic

1.6.7 Alternative Water Quality Controls

A **On-Site Dual Purpose Sedimentation-Detention Basins.** Dual purpose sedimentation-detention basins combine flood control and water quality enhancement in the same structure. The important features of these structures are the peak flow control outlet and detention outlet. References 86, 91 and 93 provide further information on the design of dual purpose basins

1. Peak Flow Control Outlet (Flood Control Outlet). The flow line of the lowest opening in this structure shall be situated at the pond elevation at which the water quality volume can be developed in the pond without flows leaving through the peak flow outlet structure.

2 Detention Outlet . The detention outlet shall be sized to provide a forty (40) hour minimum draw-down time for the water quality volume The draw-down time for dual purpose basins is defined as the period between the time at which the water surface in the pond drops below

APPENDIX B

Drainage Area Maps from the Subdivision Construction Plan

(CBD 2001a, 2001b)

Area No	Area Acres	TC Min.	C 25	C 100	I 25 In/Hr	I 100 In/Hr	Q 25 CFS	Q 100 CFS
64	1.12	10	0.62	0.71	8.40	10.22	5.8	8.1
65	1.60	10	0.62	0.71	8.40	10.22	8.3	11.6
66	1.10	12	0.62	0.71	7.94	9.69	5.4	7.6
67	1.27	10	0.62	0.71	8.40	10.22	6.6	9.2
68	1.56	15	0.62	0.71	7.36	9.01	7.1	10.0
69	0.91	8	0.67	0.75	8.91	10.81	5.4	7.4
70	1.78	15	0.67	0.75	7.36	9.01	8.8	12.0
71	0.71	5	0.77	0.85	9.84	11.88	5.4	7.2
72	0.61	6	0.62	0.71	9.51	11.50	3.6	5.0
73	1.96	12	0.62	0.71	7.94	9.69	9.7	13.5
74	0.88	6	0.67	0.75	9.51	11.50	5.6	7.6
75	1.37	12	0.62	0.71	7.94	9.69	6.7	9.4
76	1.66	12	0.62	0.71	7.94	9.69	8.2	11.4
77	1.99	12	0.62	0.71	7.94	9.69	9.8	13.7
78	1.01	6	0.67	0.75	9.51	11.50	6.4	8.7
79	1.81	12	0.62	0.71	7.94	9.69	8.9	12.5
80	0.38	5	0.77	0.85	9.84	11.88	2.9	3.8
81	0.81	6	0.67	0.75	9.51	11.50	5.2	7.0
82	0.48	5	0.77	0.85	9.84	11.88	3.6	4.8
83	2.19	10	0.62	0.71	8.40	10.22	11.4	15.9
84	0.13	5	0.77	0.85	9.84	11.88	1.0	1.3
85	1.06	10	0.62	0.71	8.40	10.22	5.5	7.7
86	0.25	5	0.77	0.85	9.84	11.88	1.9	2.5
87	1.02	8	0.67	0.75	8.91	10.81	6.1	8.3
88	0.79	6	0.67	0.75	9.51	11.50	5.0	6.8
89	0.96	10	0.62	0.71	8.40	10.22	5.0	7.0
90	1.94	10	0.62	0.71	8.40	10.22	10.1	14.1
91	0.05	10	0.62	0.71	8.40	10.22	0.3	0.4
92	0.29	6	0.72	0.80	9.51	11.50	2.0	2.7
93	0.26	5	0.77	0.85	9.84	11.88	2.0	2.6
94	5.50	10	0.48	0.58	8.40	10.22	21.4	31.4
95	19.60	15	0.48	0.58	7.36	9.01	69.2	102.4
96	3.21	20	0.48	0.56	6.58	8.08	10.1	14.5
97	7.70	15	0.58	0.66	7.36	9.01	32.9	45.8
98	0.93	12	0.62	0.71	7.94	9.69	4.6	6.4
99	0.35	5	0.67	0.75	9.84	11.88	2.3	3.1
100	1.25	12	0.62	0.71	7.94	9.69	6.2	8.6
101	0.93	12	0.62	0.71	7.94	9.69	4.6	6.4
102	1.13	15	0.65	0.75	7.36	9.01	5.4	7.6
103	0.59	15	0.61	0.70	7.36	9.01	2.6	3.7
104	1.72	10	0.60	0.65	8.40	10.22	8.7	11.4
105	0.98	10	0.60	0.65	8.40	10.22	4.9	6.5
106	0.23	5	0.67	0.75	9.84	11.88	1.5	2.0
107	0.23	5	0.67	0.75	9.84	11.88	1.5	2.0

COMBINED FLOWS FOR BERDOLL PHASE 2-1

AREAS COMBINED	TC Min.	CA 25	CA 100	I 25 In/Hr	I 100 In/Hr	Q 25 CFS	Q 100 CFS	COMMENTS
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

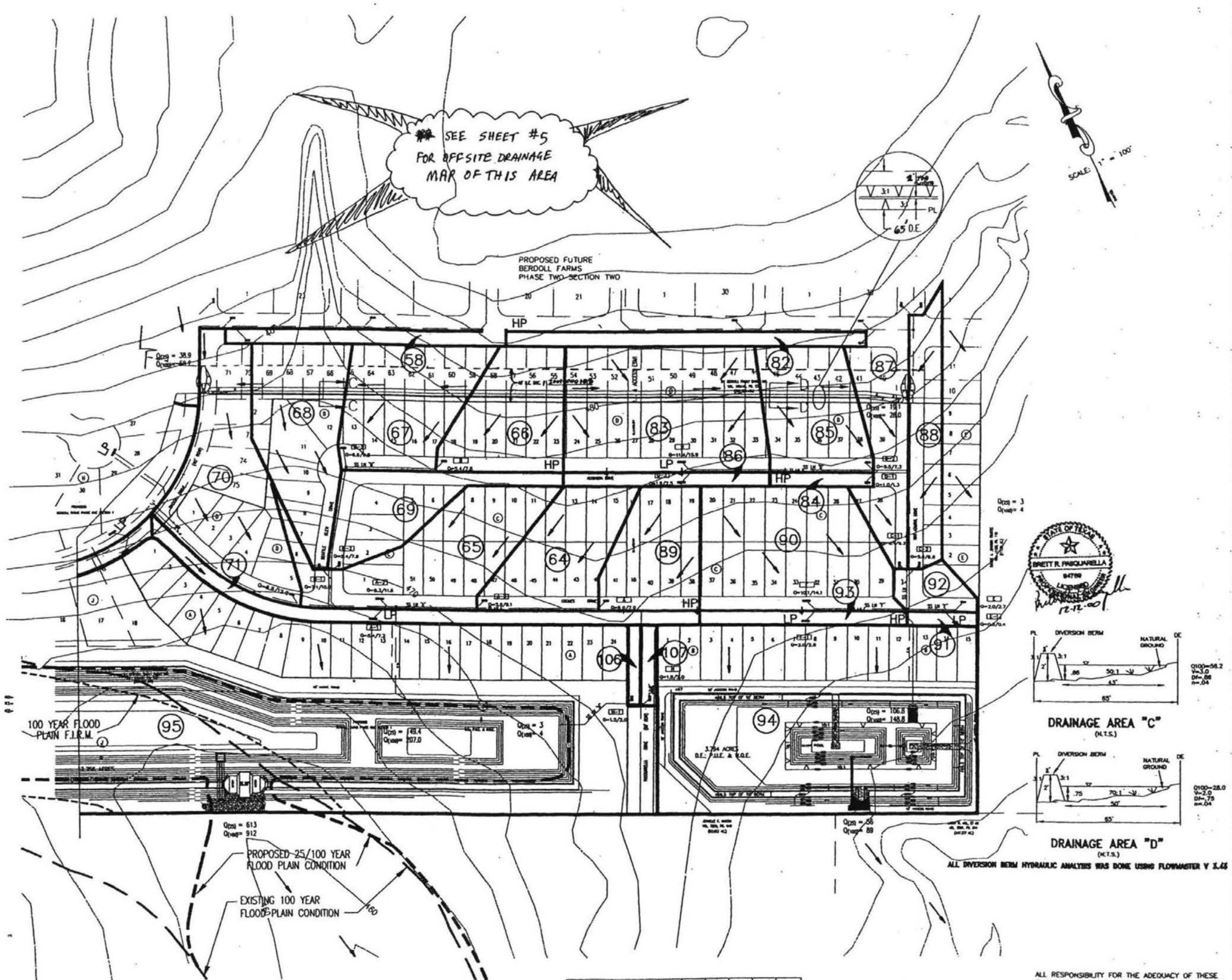
IF L/La IS GREATER THAN 1.0 THEN Q/Qa = 1

25-YEAR INLET FLOW CALCULATION TABLE

INLET NUMBER	DRAINAGE AREA NO.	Q (CFS)	Q PASS (CFS)	Q TOTAL (CFS)	SLOPE	a	Yo	POWERED	Qa/La	La	LENGTH	L/La	a/Yo	Q/Qa	Q	Q PASS	REMARK
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

100-YEAR INLET FLOW CALCULATION TABLE

INLET NUMBER	DRAINAGE AREA NO.	Q (CFS)	Q PASS (CFS)	Q TOTAL (CFS)	SLOPE	a	Yo	POWERED	Qa/La	La	LENGTH	L/La	a/Yo	Q/Qa	Q	Q PASS	REMARK
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]



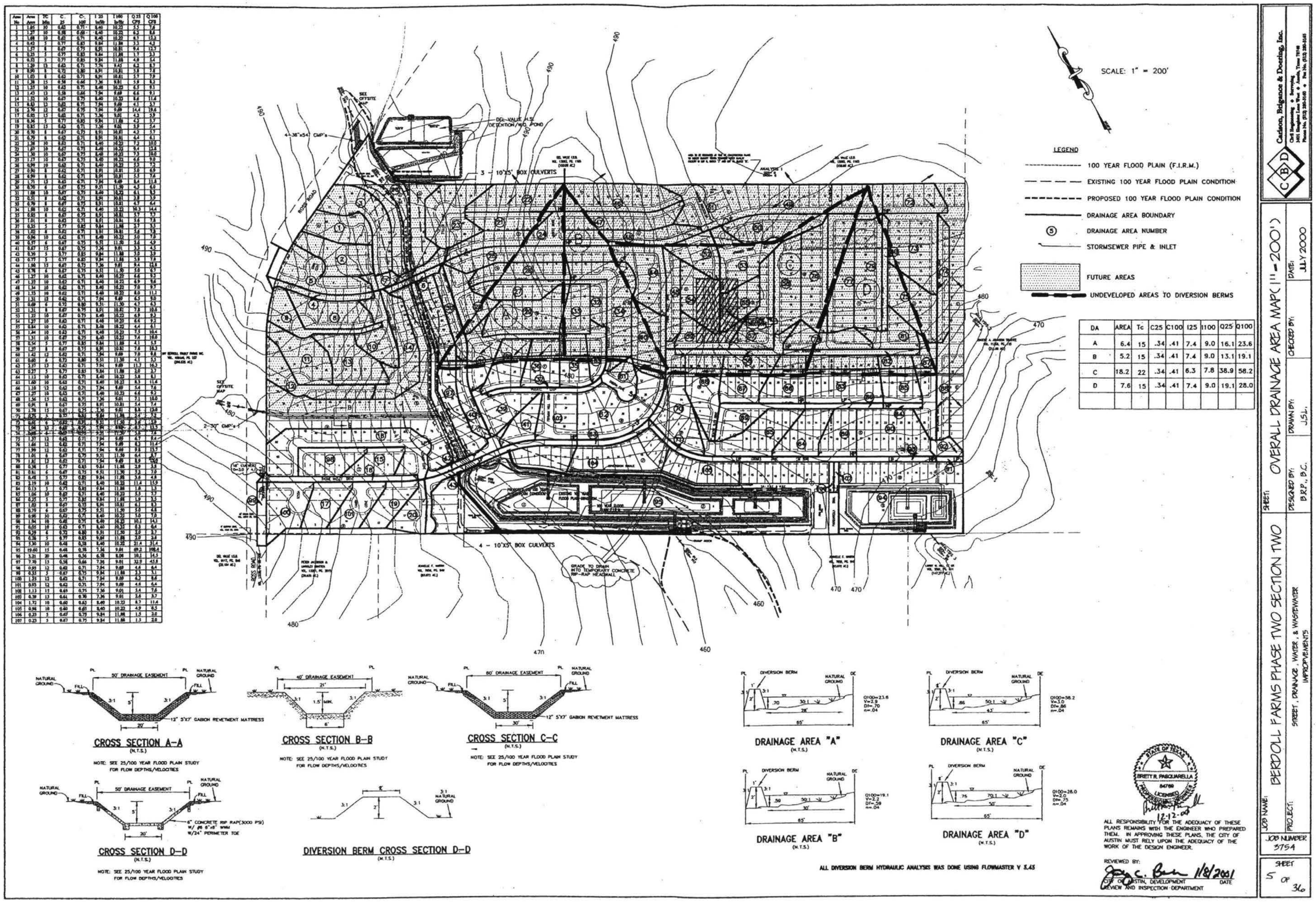
DA	AREA	Tc	C25	C100	I25	I100	Q25	Q100
C	18.2	22	.34	.41	6.3	7.8	38.9	58.2
D	7.6	15	.34	.41	7.4	9.0	19.1	28.0

ALL RESPONSIBILITY FOR THE ADEQUACY OF THESE PLANS REMAINS WITH THE ENGINEER WHO PREPARED THEM. IN APPROVING THESE PLANS, THE CITY OF AUSTIN MUST RELY UPON THE ADEQUACY OF THE WORK OF THE DESIGN ENGINEER.

REVIEWED BY: [signature] 11/8/2001
CITY OF AUSTIN, DEVELOPMENT REVIEW AND INSPECTION DEPARTMENT — DATE

Carlson, Brigance & Doering, Inc.

SHEET: DRAINAGE AREA MAP PHASE TWO SECTION ONE
DESIGNED BY: BAC | DRAWN BY: BAC | CHECKED BY: | DATE: OCT. 2000
JOB NAME: BERDOLL FARMS PHASE TWO SECTION ONE
PROJECT: STREET, DRAINAGE, WATER, & WASTEWATER IMPROVEMENTS
JOB NUMBER: 3754
SHEET 6 OF 36


SCALE: 1" = 200'
LEGEND
100 YEAR FLOOD PLAIN (F.I.R.M.)
EXISTING 100 YEAR FLOOD PLAIN CONDITION
PROPOSED 100 YEAR FLOOD PLAIN CONDITION
DRAINAGE AREA BOUNDARY
DRAINAGE AREA NUMBER
STORMSEWER PIPE & INLET
FUTURE AREAS
UNDEVELOPED AREAS TO DIVERSION BERMS
DA | AREA | Tc | C25 | C100 | I25 | I100 | Q25 | Q100
A | 6.4 | 15 | .34 | .41 | 7.4 | 9.0 | 16.1 | 23.6
B | 5.2 | 15 | .34 | .41 | 7.4 | 9.0 | 13.1 | 19.1
C | 18.2 | 22 | .34 | .41 | 6.3 | 7.8 | 38.9 | 58.2
D | 7.6 | 15 | .34 | .41 | 7.4 | 9.0 | 19.1 | 28.0
3 - 10'x5' BOX CULVERTS
4 - 10'x5' BOX CULVERTS
GRADE TO DRAIN INTO TEMPORARY CONCRETE RIP-RAP HEADWALL
CROSS SECTION A-A
(N.T.S.)
NOTE: SEE 25/100 YEAR FLOOD PLAIN STUDY FOR FLOW DEPTHS/VELOCITIES
CROSS SECTION B-B
(N.T.S.)
NOTE: SEE 25/100 YEAR FLOOD PLAIN STUDY FOR FLOW DEPTHS/VELOCITIES
CROSS SECTION C-C
(N.T.S.)
NOTE: SEE 25/100 YEAR FLOOD PLAIN STUDY FOR FLOW DEPTHS/VELOCITIES
CROSS SECTION D-D
(N.T.S.)
NOTE: SEE 25/100 YEAR FLOOD PLAIN STUDY FOR FLOW DEPTHS/VELOCITIES
DIVERSION BERM CROSS SECTION D-D
(N.T.S.)
DRAINAGE AREA "A"
(N.T.S.)
DRAINAGE AREA "B"
(N.T.S.)
DRAINAGE AREA "C"
(N.T.S.)
DRAINAGE AREA "D"
(N.T.S.)
ALL DIVERSION BERM HYDRAULIC ANALYSIS WAS DONE USING FLOWMASTER V 5.45
ALL RESPONSIBILITY FOR THE ADEQUACY OF THESE PLANS REMAINS WITH THE ENGINEER WHO PREPARED THEM. IN APPROVING THESE PLANS, THE CITY OF AUSTIN MUST RELY UPON THE ADEQUACY OF THE WORK OF THE DESIGN ENGINEER.
REVIEWED BY:
CITY OF AUSTIN, DEVELOPMENT REVIEW AND INSPECTION DEPARTMENT
DATE
1/8/2001
BRETT R. PASQUARELLA
84799
12-12-00
Carlson, Brigance & Doering, Inc.
SHEET: OVERALL DRAINAGE AREA MAP (1"=200')
DATE: JULY 2000
DESIGNED BY: B.R.P., B.C.
DRAWN BY: J.S.L.
CHECKED BY:
JOB NAME: BERDOLL FARMS PHASE TWO SECTION TWO
PROJECT: STREET, DRAINAGE, WATER, & WASTEWATER IMPROVEMENTS
JOB NUMBER 3754
SHEET 5 OF 36

APPENDIX C

Design Specifications of Berdoll Farms Wet Pond

(CBD 2001a)

DETENTION/WETPOND "B"

ELEVATION (FT.)	AREA (ACRES)	AVERAGE AREA (ACRES)	VOLUME (AC-FT)	TOTAL VOLUME (AC-FT)
456	0		0.00	0.00
456.5	0.05	0.03	0.01	0.01
457	0.1	0.08	0.04	0.05
457.5	0.115	0.11	0.05	0.10
458	0.13	0.12	0.06	0.17
458.5	0.145	0.14	0.07	0.23
459	0.16	0.15	0.08	0.31
459.5	0.205	0.18	0.09	0.40
460	0.25	0.23	0.11	0.52
460.5	0.275	0.26	0.13	0.65
461	0.3	0.29	0.14	0.79
461.5	0.33	0.32	0.16	0.95
462	0.36	0.35	0.17	1.12
462.5	0.39	0.38	0.19	1.31
463	0.42	0.41	0.20	1.51
463.5	0.53	0.47	0.24	1.75
464	0.63	0.58	0.29	2.04
464.5	1.05	0.84	0.42	2.46
465	1.47	1.26	0.63	3.09
465.5	1.68	1.57	0.79	3.87
466	1.88	1.78	0.89	4.76
466.5	1.95	1.91	0.96	5.72
467	2.01	2.04	0.99	6.71
467.5	2.07	2.10	1.02	7.73
468	2.13	2.16	1.05	8.78
468.5	2.19		1.08	9.86

POND "B" OUTLET RATING CURVE TABLE

1 - 3'X10' BOX CULVERT

	2 - 18" INVERTED RCP FL = 464.0 (CFS)	8' WEIR FL = 466.5 (CFS)	10' WEIR FL = 466.0 (CFS)	TOTAL (CFS)
464.0	0.0	0.0	0.0	0.0
464.5	10.1	0.0	0.0	10.1
465.0	14.3	0.0	0.0	14.3
465.5	17.5	0.0	0.0	17.5
466.0	22.6	0.0	0.0	22.6
466.5	26.7	0.0	10.6	37.3
467.0	30.3	0.0	30.0	60.3
467.5	33.5	24.0	55.1	112.6
468.0	40.5	44.1	84.9	169.4
468.5	43.5	67.9	118.6	230.0

AREA TO CONTROL = 28 AC. (BASIN AREA # 2)

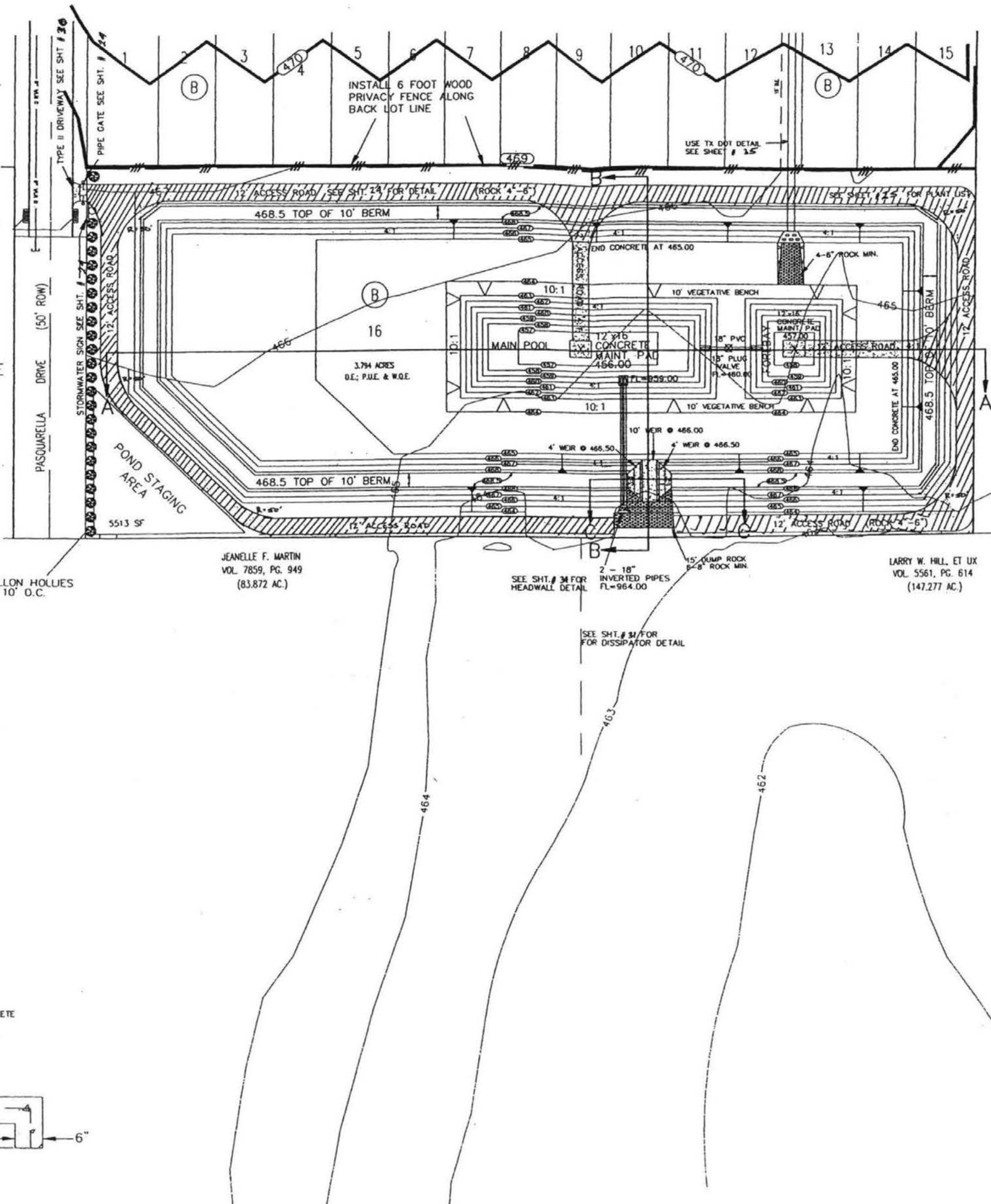
V = .162 x Rf x Da

V = (.162) x (.33) x (28ac)

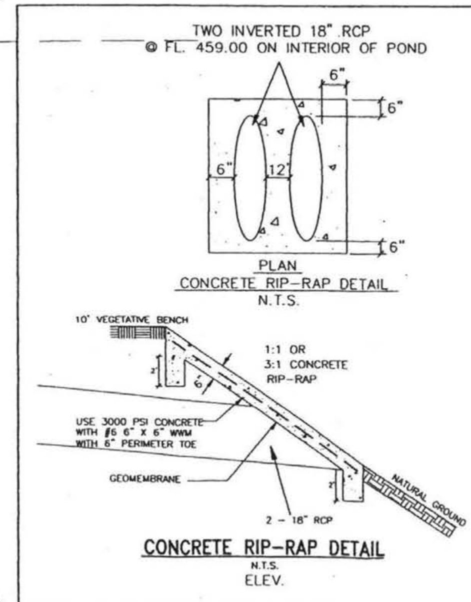
V = 1.50 AC-FT

STORM EVENT (YR)	EXISTING (CFS)	POND RELEASE (CFS)
2	21	19
10	59	51
25	64	56
100	89	89

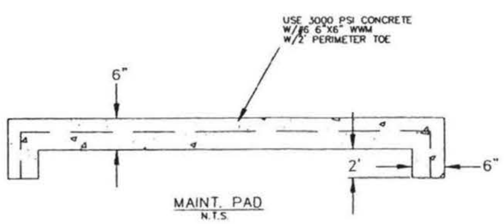
STORM YEAR	STAGE ELEV-FT	STORAGE AC-FT	DISCHARGE CFS
2	465.52	1.83	19
10	466.79	4.16	51
25	466.91	4.58	56
100	467.40	6.06	89



SCALE: 1" = 40'



CONCRETE RIP-RAP DETAIL
N.T.S.
ELEV.



MAINT. PAD
N.T.S.

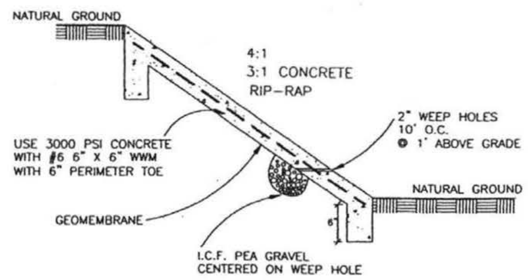
ALL RESPONSIBILITY FOR THE ADEQUACY OF THESE PLANS REMAINS WITH THE ENGINEER WHO PREPARED THEM. IN APPROVING THESE PLANS, THE CITY OF AUSTIN MUST RELY UPON THE ADEQUACY OF THE WORK OF THE DESIGN ENGINEER.

REVIEWED BY: C. B. 1/8/2001
CITY OF AUSTIN DEVELOPMENT REVIEW AND INSPECTION DEPARTMENT — DATE

G:\PROJ\3754\POND2-1.dwg
Thu Oct 26 08:12:20 2000

Carlson, Brigance & Doering, Inc.

SHEET: WET POND/ DETENTION POND # 2
DATE: JUNE 2000
CHECKED BY:
DRAWN BY: BAC
DESIGNED BY: BAC
JOB NAME: BERDOLL FARMS PHASE TWO SECTION ONE
PROJECT: STREET, DRAINAGE, WATER, & WASTEWATER IMPROVEMENTS
JOB NUMBER 3754
SHEET 23 OF 36



CONCRETE RIP-RAP DETAIL
N.T.S.

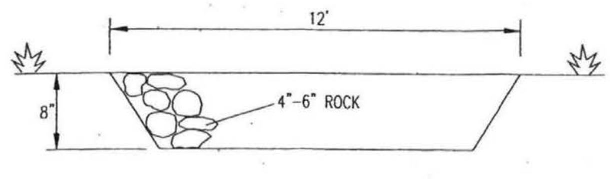
STANDARD NOTES

1. BASIN LINER (CLAY LINER REQUIRED FOR WET POND)

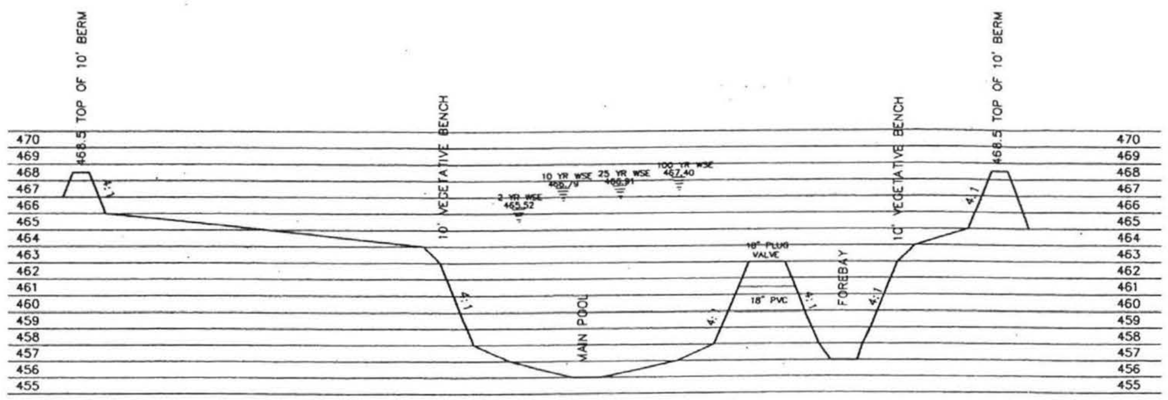
Impermeable liners must be clay.
Clay liners shall meet the following specifications.

PROPERTY	TEST METHOD	UNIT	SPECIFICATION
Permeability	ASTM D-2434	Cm/Sec	1×10^{-6}
Plasticity Index of Clay	ASTM D-423 & D-424	%	Not less than 15
Liquid Limit of Clay	ASTM D-2216	%	Not less than 30
Clay Particles Passing	ASTM D-422	%	Not less than 30
Clay Compaction	ASTM D-2216	%	95% of Standard Proctor Density

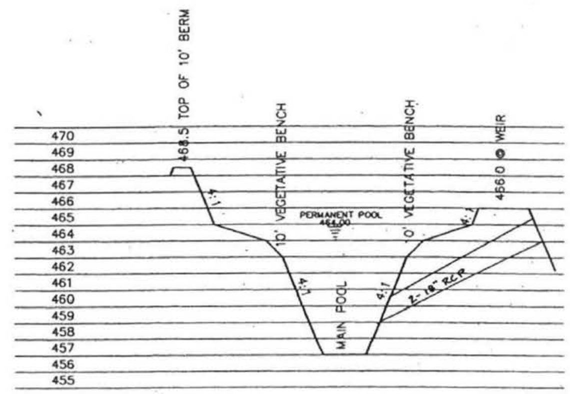
The clay liner shall have a minimum thickness of 18 inches.



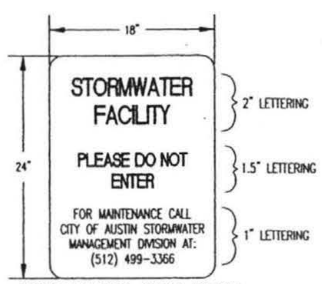
12' ACCESS DRIVE
N.T.S.



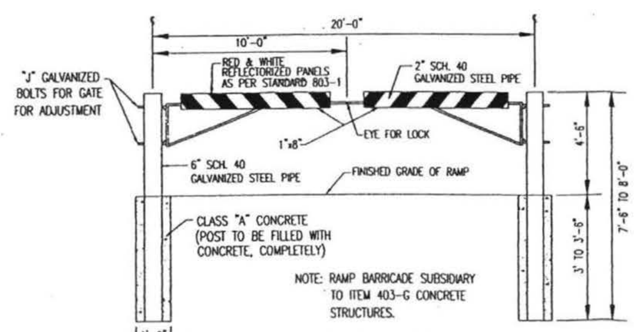
CROSS SECTION A-A



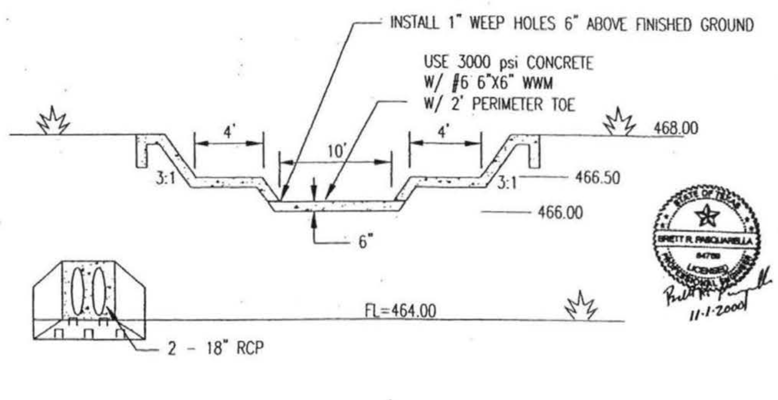
CROSS SECTION B-B



STORMWATER SIGN DETAIL
SCALE: 1" = 1'



GATE AT RAMP
N.T.S.



OUTLET STRUCTURE
CROSS SECTION C-C
N.T.S.

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REVIEWED BY: 11/8/2001
CITY OF AUSTIN DEVELOPMENT REVIEW AND INSPECTION DEPARTMENT — DATE

Carlson, Brigance & Doering, Inc.
Civil Engineering • Surveying

JOB NAME: BERDOLL FARMS PHASE TWO SECTION ONE
PROJECT: STREET, DRAINAGE, WATER, & WASTEWATER IMPROVEMENTS
SHEET: WET POND/ DETENTION POND #2 DETAILS
DESIGNED BY: BAC | DRAWN BY: BAC | CHECKED BY: | DATE: JUNE 2000
JOB NUMBER: 3754
SHEET 24 OF 36

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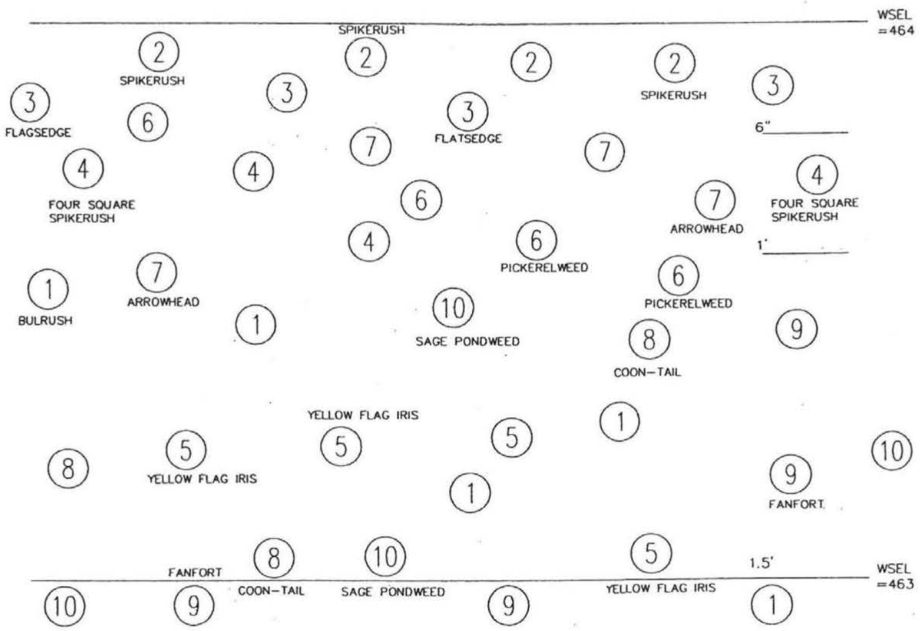
Thu Oct 26 08:12:53 2000

APPENDIX D

Berdoll Farms Wet Pond Plant List

(CBD 2001a)

TYPICAL WET POND SPACING



1. BULRUSH PLANT 3–4 FEET ON CENTER
2. SPIKERUSH PLANT 3–6 FEET ON CENTER
3. FLATSEDGE PLANT 3 FEET ON CENTER
4. FOUR SQUARE SPIKERUSH
5. YELLOW FLAG IRIS
6. PICKERELWEED
7. ARROWHEAD PLANT 3 FEET ON CENTER
8. COON–TAIL
9. FANWORT
10. SAGE PONDWEED

NOTE: ALL WETLAND PLANTS WHICH FULFILL THE MINIMUM LANDSCAPE REQUIREMENTS SHALL BE PROPAGATED OR HARVESTED FROM THE REGIONALLY ADAPTED STOCK (WHEN POSSIBLE). THESE ARE PLANT SPECIES OR GENOTYPES WHICH ARE NATIVE TO A RANGE OF WITHIN 250 MILES OF THE PROJECT SITE.

WET POND PLANTING PLAN

Surface Area of Permanent Pool =1.00 ACRES
43560 x 0.03=1307 number of plants required (minimum)

PLANT CATEGORY	RATIO	MINIMUM NO. OF PALNTS	PROVIDED NO. OF PLANTS	MINIMUM SIZE
A. BULRUSH	40%	523	523	2 GAL.
B. SPIKERUSH	20%	261	261	2.5" LINER
C. MARSH DIVERSITY	20%	261	261	1 GAL.
D. ARROWHEAD	10%	131	131	1 GAL.
E. SUBMERGENTS	5%	65	65	1 GAL.
F. FLOATING AQUATICS	5%	66	66	1 GAL.
TOTAL		1307	1307	

PLANT CATEGORY	QTY.	SPECIES
BULRUSH	261	Scirpus validus (Bullush)
BULRUSH	262	Scirpus californicus (Bullush)
SPIKERUSH	261	Eleocharis montevidensis (spikerush) Eleocharis quadrangulata (spikerush)
MARSH DIVERSITY	261	Cyperus ochraceus (Flatsedge) Eleocharis quadrangulata (Four Square Spike Rush) Iris pseudacorus (Yellow Flag Iris) Pontederia cordata (Pickerelweed)
ARROWHEAD	131	Sagittaria latifolia (Arrowhead)
SUBMERGENTS	65	Ceratophyllum ororata (Coon–tail)
FLOATING AQUATICS	33	Nymphaea odorata (Fanwort)
FLOATING AQUATICS	33	Potomageton pectinatus (Sage Pondweed)

WET POND NOTES:

1) Microbial initiation will be supplemented with leaf litter or hay. A minimum of one inch of plant litter evenly on the sides of the pond (below permanent pool level). Treat the entire shallow water bench in this manner and all pond slopes (ranging form 3:1 to 10:1). Crimp the plant litter into the pond substrate to prevent the material from being transported downstream as the pond fills.

2) The use of submersed macrophytes or other aquatic vegetation will be considered for reducing algae blooms.

3) The wet pond will be stocked with the fish species Gambusia affins to serve as biological control for mosquitos. The Gambusia should be stocked at the initial density of 200 individuals per surface acre.

4) Domestic waterfowl will not be introduced into the wet pond.

5) Carp and goldfish should not be introduced into a wet pond.

WETLAND PLANT LIST

Install bullrush in clumps, with individual plants spaced appoximatley three to four feet on center. At least two of the following species shall be used.

BULRUSH	WATER DEPTH	NOTES
*Scirpus validus (Bulrush)	1'–3'	8' TALL EVERGREEN, RESISTS CATTAIL ENCROACHMENT
*Scirpus californicus (Bullush)	1'–3'	8' TALL EVERGREEN, RESISTS CATTAIL ENCROACHMENT
*Scirpus americanus (Three–square bullrush)	2"–6"	2'–4' TALL, W/3 DISTINCT EDGES

Install Spikerush at or near the water's edge, with individual plants spaced approximately three to six feet on center. At least two of the following species shall be used:

SPIKERUSH	WATER DEPTH	NOTES
*Eleocharis montevidensis (spikerush)	0"–6"	1' tall rhizomatous, reduce erosion at the pond edge
*Eleocharis macrostachys (spikerush)	0"–6"	1' tall rhizomatous, reduce erosion at the pond edge
*Eleocharis quadrangulata (spikerush)	3"–1'	2' to 2.5' tall, rhizomatous, can accomidate deeper water, 4–angled

At least two species of the following marsh plants shall be used (additional species are encouraged). Install in clumps in shallow water, with individual plants spaced at appoximatley three feet on center.

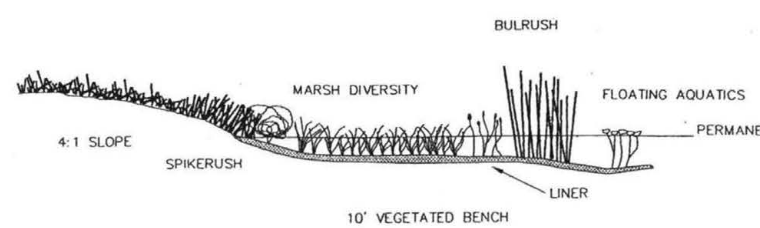
MARSH DIVERSITY	WATER DEPTH	NOTES
Cyperus ochraceus (Flatsedge)	2'–6"	1' to 2' tall, clump–forming, common to central Texas.
Dichromena colorata (white–topped Sedge)	2"–6"	1' to 2' tall, white bracts during warm season
Echinodorus rostratus (Burhead)	3'–1'	1' to 2' tall, annual, heart–shaped leaves, flowers similar to Arrowheead
Eleocharis quadranqulata (Four Square Spike Rush)	6"–1'	1' to 2' tall, colonizes, inhabits deeper water than other Spikerushes
Iris pseudacorus (Yellow Flag Iris)	1'–2'	3' to 4' tall, can be invasive, dense growth, yellow flowers
Juncus effusus (Soft Rush)	6"–1'	3' to 4' tall, forms a light clump, evergreen, very attractive
Justicia americana (Water Willow)	2"–6"	2' to 3' tall, common, white flowers. herbaceous; colonizes
Marsilea macropoda (Water Clover)	2"–6"	Looks like floating four–leaf clover, endemic to Texas
Najas guadalupensis (Water–Naiad)	1'–4'	Submergent, valuable to fish and wildlife
Pontederia cordata (Pickerelweed)	2"–1'	3' tall, colonizes, cosmopolitan, purple flowers
Rhynchospora corniculata (Horned–rush)	2'–6"	2' to 3' tall, brass–colored flowers in May

Install Arrowhead in clumps in shallow water, with individual plants spaced approximately three feet on center.

ARROWHEAD	WATER DEPTH	NOTES
Sagittaria latifolia (Arrowhead)	2"–1'	2' height, wildlife value, white flowers, proven water quality performer

The followingg category, Aquatics, include submergents and floating–leaved aquatics. Submergents are rooted in the sediment of the pond and are completely sumerged in the water. Floating–leaved aquatic plants are rooted in the sediment of the pond, and have leaves that float on the surface of the water. These leaves shade the water, which limits potential algae growth. At least two of the following species shall be used and should be placed at random locations throughout the pond.

AQUATICS	WATER DEPTH	NOTES
Carborriba caroliniaria (Fanwort)	1'–4'	Approximately 6' length underwater, submergent
Ceratophyllum spp. (Coon–tail)	1'–4'	Maximum 8' length, tolerant of turbidity and water fluctuation, wildlife food.
Nymphaea odonata (Fanwort)	6"–2'	A native, reliably hardy, floating–leaved aquatic; with white flowers
Potomageton pactinatus (Sago Pondweed)	8"–3'	Colonize quickly, valuable to fish and wildlife; floatin–leaved aquatic



CROSS SECTION OF A TYPICAL VEGETATED BENCH AREA

STATE OF TEXAS
BRETT R. PASQUARELLA
11-1-2000

ALL RESPONSIBILITY FOR THE ADEQUACY OF THESE PLANS REMAINS WITH THE ENGINEER WHO PREPARED THEM. IN APPROVING THESE PLANS, THE CITY OF AUSTIN MUST RELY UPON THE ADEQUACY OF THE WORK OF THE DESIGN ENGINEER.

REVIEWED BY: 1/8/2001
CITY OF AUSTIN DEVELOPMENT AND REVIEW AND INSPECTION DEPARTMENT — DATE

Carlson, Brigance & Doering, Inc.
Civil Engineering + Surveying
3401 Slaughter Lane West + Austin, Texas 78748
Phone No. (512) 280-5160 + Fax No. (512) 280-5165

JOB NAME: BERDOLL FARMS PHASE TWO SECTION ONE
PROJECT: STREET, DRAINAGE, WATER, & WASTEWATER IMPROVEMENTS
SHEET: WET POND PLANT LIST
DESIGNED BY: | DRAWN BY: | CHECKED BY: | DATE: DATE
JOB NUMBER 3754
SHEET 25 OF 36

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Thu Oct 26 08:48:12 2000

APPENDIX E

Water Quality and Storm Event Monitoring Data

(COA n.d.)

Event Date	Influent/Effluent	Parameter	Total Rainfall (inches)	Dry Time (hr)	Volume (liters)	Load	Load Unit	EMC	EMC UNIT
2/4/2004	Influent	CD	0.76	255.567	418571.0492	209.28439	MG	0.5	UG/L
2/4/2004	Effluent	CD	0.71	256.233	1032840.334	516.41737	MG	0.5	UG/L
2/4/2004	Influent	COD	0.76	255.567	418571.0492	43359.95789	UG	103.590998	MG/L
2/4/2004	Effluent	COD	0.71	256.233	1032840.334	35532.95451	UG	34.40333	MG/L
2/4/2004	Influent	CU	0.76	255.567	418571.0492	2655.70274	MG	6.344722	UG/L
2/4/2004	Effluent	CU	0.71	256.233	1032840.334	5164.17369	MG	5	UG/L
2/4/2004	Influent	DP	0.76	255.567	418571.0492	101595.4459	MG	0.242721	MG/L
2/4/2004	Effluent	DP	0.71	256.233	1032840.334	31046.84439	MG	0.03006	MG/L
2/4/2004	Influent	NH3	0.76	255.567	418571.0492	106975.4617	MG	0.255574	MG/L
2/4/2004	Effluent	NH3	0.71	256.233	1032840.334	64308.34818	MG	0.062264	MG/L
2/4/2004	Influent	NO23	0.76	255.567	418571.0492	387053.318	MG	0.924707	MG/L
2/4/2004	Effluent	NO23	0.71	256.233	1032840.334	73795.08471	MG	0.071449	MG/L
2/4/2004	Influent	PB	0.76	255.567	418571.0492	735.611	MG	1.757444	UG/L
2/4/2004	Effluent	PB	0.71	256.233	1032840.334	1549.25211	MG	1.5	UG/L
2/4/2004	Influent	TKN	0.76	255.567	418571.0492	276843.0683	MG	0.661404	MG/L
2/4/2004	Effluent	TKN	0.71	256.233	1032840.334	603930.4568	MG	0.584731	MG/L
2/4/2004	Influent	TN	0.76	255.567	418571.0492	663896.3863	MG	1.586111	MG/L
2/4/2004	Effluent	TN	0.71	256.233	1032840.334	677725.5415	MG	0.65618	MG/L
2/4/2004	Influent	TOC	0.76	255.567	418571.0492	4627.11291	UG	11.054606	MG/L
2/4/2004	Effluent	TOC	0.71	256.233	1032840.334	8542.04071	UG	8.270482	MG/L
2/4/2004	Influent	TP	0.76	255.567	418571.0492	143795.0256	MG	0.34354	MG/L
2/4/2004	Effluent	TP	0.71	256.233	1032840.334	76655.66001	MG	0.074219	MG/L
2/4/2004	Influent	TSS	0.76	255.567	418571.0492	185946.3025	UG	444.243122	MG/L
2/4/2004	Effluent	TSS	0.71	256.233	1032840.334	21546.09539	UG	20.861126	MG/L
2/4/2004	Influent	VSS	0.76	255.567	418571.0492	21480.6481	UG	51.319279	MG/L
2/4/2004	Effluent	VSS	0.71	256.233	1032840.334	10922.24771	UG	10.57502	MG/L

Event Date	Influent/Effluent	Parameter	Total Rainfall (inches)	Dry Time (hr)	Volume (liters)	Load	Load Unit	EMC	EMC UNIT
2/4/2004	Influent	ZN	0.76	255.567	418571.0492	23968.32418	MG	57.26257	UG/L
2/4/2004	Effluent	ZN	0.71	256.233	1032840.334	10235.62599	MG	9.910226	UG/L
2/10/2004	Influent	CD	1.83	6.533	2137871.036	1068.84474	MG	0.5	UG/L
2/10/2004	Effluent	CD	1.61	6.467	11950574.64	5975.25495	MG	0.5	UG/L
2/10/2004	Influent	COD	1.83	6.533	2137871.036	61575.71603	UG	28.804799	MG/L
2/10/2004	Effluent	COD	1.61	6.467	11950574.64	375884.9468	UG	31.453465	MG/L
2/10/2004	Influent	CU	1.83	6.533	2137871.036	7295.33998	MG	3.412722	UG/L
2/10/2004	Effluent	CU	1.61	6.467	11950574.64	48789.01409	MG	4.082588	UG/L
2/10/2004	Influent	DP	1.83	6.533	2137871.036	232379.8853	MG	0.108706	MG/L
2/10/2004	Effluent	DP	1.61	6.467	11950574.64	943177.2707	MG	0.078924	MG/L
2/10/2004	Influent	NH3	1.83	6.533	2137871.036	171755.248	MG	0.080346	MG/L
2/10/2004	Effluent	NH3	1.61	6.467	11950574.64	1190341.45	MG	0.099606	MG/L
2/10/2004	Influent	NO23	1.83	6.533	2137871.036	2460680.659	MG	1.151094	MG/L
2/10/2004	Effluent	NO23	1.61	6.467	11950574.64	3189859.886	MG	0.266922	MG/L
2/10/2004	Influent	PB	1.83	6.533	2137871.036	3206.53423	MG	1.5	UG/L
2/10/2004	Effluent	PB	1.61	6.467	11950574.64	17925.76484	MG	1.5	UG/L
2/10/2004	Influent	TKN	1.83	6.533	2137871.036	753474.835	MG	0.352472	MG/L
2/10/2004	Effluent	TKN	1.61	6.467	11950574.64	6355531.174	MG	0.531821	MG/L
2/10/2004	Influent	TN	1.83	6.533	2137871.036	3214155.494	MG	1.503566	MG/L
2/10/2004	Effluent	TN	1.61	6.467	11950574.64	9545391.06	MG	0.798743	MG/L
2/10/2004	Influent	TOC	1.83	6.533	2137871.036	10321.36283	UG	4.82828	MG/L
2/10/2004	Effluent	TOC	1.61	6.467	11950574.64	74389.52038	UG	6.224799	MG/L
2/10/2004	Influent	TP	1.83	6.533	2137871.036	363018.14	MG	0.169818	MG/L
2/10/2004	Effluent	TP	1.61	6.467	11950574.64	1971592.832	MG	0.16498	MG/L
2/10/2004	Influent	TSS	1.83	6.533	2137871.036	248241.4611	UG	116.126062	MG/L
2/10/2004	Effluent	TSS	1.61	6.467	11950574.64	880587.7131	UG	73.686204	MG/L

Event Date	Influent/Effluent	Parameter	Total Rainfall (inches)	Dry Time (hr)	Volume (liters)	Load	Load Unit	EMC	EMC UNIT
2/10/2004	Influent	VSS	1.83	6.533	2137871.036	27173.24427	UG	12.711502	MG/L
2/10/2004	Effluent	VSS	1.61	6.467	11950574.64	100524.068	UG	8.411697	MG/L
2/10/2004	Influent	ZN	1.83	6.533	2137871.036	37886.48416	MG	17.7231	UG/L
2/10/2004	Effluent	ZN	1.61	6.467	11950574.64	124987.5019	MG	10.458759	UG/L
2/23/2004	Influent	CD	0.05	224.717	24669.79758	12.33483	MG	0.5	UG/L
2/24/2004	Effluent	CD	0.05	8.783	1782417.54	891.20394	MG	0.5	UG/L
2/23/2004	Influent	COD	0.81	224.717	24669.79758	7225.78972	UG	292.90183	MG/L
2/24/2004	Effluent	COD	0.81	8.783	1782417.54	68652.14849	UG	38.51652	MG/L
2/23/2004	Influent	CU	0.05	224.717	24669.79758	74.00899	MG	3	UG/L
2/24/2004	Effluent	CU	0.05	8.783	1782417.54	5347.22365	MG	3	UG/L
2/23/2004	Influent	DP	0.81	224.717	24669.79758	2542.95718	MG	0.10308	MG/L
2/24/2004	Effluent	DP	0.81	8.783	1782417.54	29395.97781	MG	0.016492	MG/L
2/23/2004	Influent	NH3	0.05	224.717	24669.79758	9791.68656	MG	0.396912	MG/L
2/24/2004	Effluent	NH3	0.05	8.783	1782417.54	71297.94718	MG	0.040001	MG/L
2/23/2004	Influent	NO23	0.81	224.717	24669.79758	36550.41652	MG	1.481594	MG/L
2/24/2004	Effluent	NO23	0.81	8.783	1782417.54	185518.1004	MG	0.104083	MG/L
2/23/2004	Influent	PB	0.05	224.717	24669.79758	37.0045	MG	1.5	UG/L
2/24/2004	Effluent	PB	0.05	8.783	1782417.54	2673.61183	MG	1.5	UG/L
2/23/2004	Influent	TKN	0.81	224.717	24669.79758	46613.80179	MG	1.889519	MG/L
2/24/2004	Effluent	TKN	0.81	8.783	1782417.54	611476.8836	MG	0.343062	MG/L
2/23/2004	Influent	TN	0.05	224.717	24669.79758	83164.21831	MG	3.371113	MG/L
2/24/2004	Effluent	TN	0.05	8.783	1782417.54	796994.984	MG	0.447145	MG/L
2/23/2004	Influent	TOC	0.81	224.717	24669.79758	798.8926	UG	32.383603	MG/L
2/24/2004	Effluent	TOC	0.81	8.783	1782417.54	18168.93377	UG	10.193477	MG/L
2/23/2004	Influent	TP	0.05	224.717	24669.79758	23751.96681	MG	0.962801	MG/L
2/24/2004	Effluent	TP	0.05	8.783	1782417.54	178941.5005	MG	0.100393	MG/L

Event Date	Influent/Effluent	Parameter	Total Rainfall (inches)	Dry Time (hr)	Volume (liters)	Load	Load Unit	EMC	EMC UNIT
2/23/2004	Influent	TSS	0.81	224.717	24669.79758	21403.05741	UG	867.586095	MG/L
2/24/2004	Effluent	TSS	0.81	8.783	1782417.54	108901.9511	UG	61.098221	MG/L
2/23/2004	Influent	VSS	0.05	224.717	24669.79758	2561.55023	UG	103.834014	MG/L
2/24/2004	Effluent	VSS	0.05	8.783	1782417.54	14390.42459	UG	8.073587	MG/L
2/23/2004	Influent	ZN	0.81	224.717	24669.79758	630.90166	MG	25.573987	UG/L
2/24/2004	Effluent	ZN	0.81	8.783	1782417.54	8346.8871	MG	4.682928	UG/L
5/13/2004	Influent	CD	0.76	55.783	362265.0702	181.12295	MG	0.5	UG/L
5/13/2004	Effluent	CD	0.71	55.433	356787.8964	178.39298	MG	0.5	UG/L
5/13/2004	Influent	COD	0.76	55.783	362265.0702	19552.63306	UG	53.976133	MG/L
5/13/2004	Effluent	COD	0.71	55.433	356787.8964	15117.06903	UG	42.370134	MG/L
5/13/2004	Influent	CU	0.76	55.783	362265.0702	1403.9815	MG	3.875769	UG/L
5/13/2004	Effluent	CU	0.71	55.433	356787.8964	1070.35789	MG	3	UG/L
5/13/2004	Influent	DP	0.76	55.783	362265.0702	168128.9046	MG	0.464129	MG/L
5/13/2004	Effluent	DP	0.71	55.433	356787.8964	43824.45635	MG	0.122831	MG/L
5/13/2004	Influent	NH3	0.76	55.783	362265.0702	104301.2434	MG	0.287929	MG/L
5/13/2004	Effluent	NH3	0.71	55.433	356787.8964	79542.61423	MG	0.222942	MG/L
5/13/2004	Influent	NO23	0.76	55.783	362265.0702	208198.7032	MG	0.574744	MG/L
5/13/2004	Effluent	NO23	0.71	55.433	356787.8964	14715.14096	MG	0.041244	MG/L
5/13/2004	Influent	PB	0.76	55.783	362265.0702	765.66405	MG	2.113658	UG/L
5/13/2004	Effluent	PB	0.71	55.433	356787.8964	535.17894	MG	1.5	UG/L
5/13/2004	Influent	TKN	0.76	55.783	362265.0702	242188.5482	MG	0.668575	MG/L
5/13/2004	Effluent	TKN	0.71	55.433	356787.8964	219506.9597	MG	0.615234	MG/L
5/13/2004	Influent	TN	0.76	55.783	362265.0702	450387.2514	MG	1.243319	MG/L
5/13/2004	Effluent	TN	0.71	55.433	356787.8964	234222.1007	MG	0.656478	MG/L
5/13/2004	Influent	TOC	0.76	55.783	362265.0702	2680.42663	UG	7.399467	MG/L
5/13/2004	Effluent	TOC	0.71	55.433	356787.8964	2650.38229	UG	7.428494	MG/L

Event Date	Influent/Effluent	Parameter	Total Rainfall (inches)	Dry Time (hr)	Volume (liters)	Load	Load Unit	EMC	EMC UNIT
5/13/2004	Influent	TP	0.76	55.783	362265.0702	242755.4438	MG	0.67014	MG/L
5/13/2004	Effluent	TP	0.71	55.433	356787.8964	62569.09883	MG	0.175369	MG/L
5/13/2004	Influent	TSS	0.76	55.783	362265.0702	99744.96165	UG	275.351526	MG/L
5/13/2004	Effluent	TSS	0.71	55.433	356787.8964	10559.5543	UG	29.596328	MG/L
5/13/2004	Influent	VSS	0.76	55.783	362265.0702	8962.04115	UG	24.740214	MG/L
5/13/2004	Effluent	VSS	0.71	55.433	356787.8964	2953.74311	UG	8.278754	MG/L
5/13/2004	Influent	ZN	0.76	55.783	362265.0702	9504.63942	MG	26.238087	UG/L
5/13/2004	Effluent	ZN	0.71	55.433	356787.8964	1070.35789	MG	3	UG/L
6/27/2004	Influent	CD	0.09	8.6	178327.9972	22.03615	MG	0.5	UG/L
6/27/2004	Effluent	CD	0.09	5.733	94618.86979	47.30918	MG	0.5	UG/L
6/27/2004	Influent	COD	0.08	8.6	44072.53434	1053.19031	UG	23.896879	MG/L
6/27/2004	Effluent	COD	0.08	5.733	94618.86979	1069.43695	UG	11.302637	MG/L
6/27/2004	Influent	CU	0.09	8.6	44072.53434	306.90211	UG	6.963606	UG/L
6/27/2004	Effluent	CU	0.09	5.733	94618.86979	283.85507	MG	3	UG/L
6/27/2004	Influent	DP	0.08	8.6	44072.53434	2248.19352	MG	0.051011	MG/L
6/27/2004	Effluent	DP	0.08	5.733	94618.86979	946.20523	MG	0.01	MG/L
6/27/2004	Influent	NH3	0.09	8.6	44072.53434	5110.196	MG	0.11595	MG/L
6/27/2004	Effluent	NH3	0.09	5.733	94618.86979	4876.08624	MG	0.051534	MG/L
6/27/2004	Influent	NO23	0.08	8.6	44072.53434	10052.85771	MG	0.228099	MG/L
6/27/2004	Effluent	NO23	0.08	5.733	94618.86979	1535.35423	MG	0.016227	MG/L
6/27/2004	Influent	PB	0.09	8.6	44072.53434	130.27852	MG	2.956018	UG/L
6/27/2004	Effluent	PB	0.09	5.733	94618.86979	186.73228	MG	1.973531	UG/L
6/27/2004	Influent	TKN	0.08	8.6	44072.53434	8145.16481	MG	0.184814	MG/L
6/27/2004	Effluent	TKN	0.08	5.733	94618.86979	29182.33733	MG	0.308422	MG/L
6/27/2004	Influent	TN	0.09	8.6	44072.53434	18198.02252	MG	0.412913	MG/L
6/27/2004	Effluent	TN	0.09	5.733	94618.86979	30717.69156	MG	0.324649	MG/L

Event Date	Influent/Effluent	Parameter	Total Rainfall (inches)	Dry Time (hr)	Volume (liters)	Load	Load Unit	EMC	EMC UNIT
6/27/2004	Influent	TOC	0.08	8.6	44072.53434	199.99603	UG	4.537908	MG/L
6/27/2004	Effluent	TOC	0.08	5.733	94618.86979	494.06207	UG	5.22163	MG/L
6/27/2004	Influent	TP	0.09	8.6	44072.53434	5294.08598	MG	0.120123	MG/L
6/27/2004	Effluent	TP	0.09	5.733	94618.86979	7980.30784	MG	0.084342	MG/L
6/27/2004	Influent	TSS	0.08	8.6	44072.53434	4757.50297	UG	107.947701	MG/L
6/27/2004	Effluent	TSS	0.08	5.733	94618.86979	3186.49679	UG	33.677363	MG/L
6/27/2004	Influent	VSS	0.09	8.6	44072.53434	484.79525	UG	11	MG/L
6/27/2004	Effluent	VSS	0.09	5.733	94618.86979	756.83994	UG	7.99887	MG/L
6/27/2004	Influent	ZN	0.08	8.6	44072.53434	1118.43622	MG	25.377308	UG/L
6/27/2004	Effluent	ZN	0.08	5.733	94618.86979	1284.68571	MG	13.577552	UG/L
11/1/2004	Influent	CD	1.18	106.7	1297372.815	648.68289	MG	0.5	UG/L
11/1/2004	Effluent	CD	1.17	106.75	1317942.023	658.96744	MG	0.5	UG/L
11/1/2004	Influent	COD	1.18	106.7	1297372.815	43807.77769	UG	33.766713	MG/L
11/1/2004	Effluent	COD	1.17	106.75	1317942.023	26614.18955	UG	20.193858	MG/L
11/1/2004	Influent	CU	1.18	106.7	1297372.815	4428.77778	MG	3.413669	UG/L
11/1/2004	Effluent	CU	1.17	106.75	1317942.023	3953.80465	MG	3	UG/L
11/1/2004	Influent	DP	1.18	106.7	1297372.815	500216.2883	MG	0.385563	MG/L
11/1/2004	Effluent	DP	1.17	106.75	1317942.023	163694.2729	MG	0.124205	MG/L
11/1/2004	Influent	NH3	1.18	106.7	1297372.815	159169.7426	MG	0.122687	MG/L
11/1/2004	Effluent	NH3	1.17	106.75	1317942.023	46818.71694	MG	0.035524	MG/L
11/1/2004	Influent	NO23	1.18	106.7	1297372.815	737828.1447	MG	0.568713	MG/L
11/1/2004	Effluent	NO23	1.17	106.75	1317942.023	102938.7344	MG	0.078106	MG/L
11/1/2004	Influent	PB	1.18	106.7	1297372.815	2105.4587	MG	1.622872	UG/L
11/1/2004	Effluent	PB	1.17	106.75	1317942.023	1976.90232	MG	1.5	UG/L
11/1/2004	Influent	TKN	1.18	106.7	1297372.815	664811.8897	MG	0.512432	MG/L
11/1/2004	Effluent	TKN	1.17	106.75	1317942.023	382477.3852	MG	0.29021	MG/L

Event Date	Influent/Effluent	Parameter	Total Rainfall (inches)	Dry Time (hr)	Volume (liters)	Load	Load Unit	EMC	EMC UNIT
11/1/2004	Influent	TN	1.18	106.7	1297372.815	1402640.034	MG	1.081145	MG/L
11/1/2004	Effluent	TN	1.17	106.75	1317942.023	485416.1196	MG	0.368316	MG/L
11/1/2004	Influent	TOC	1.18	106.7	1297372.815	12452.19328	UG	9.598059	MG/L
11/1/2004	Effluent	TOC	1.17	106.75	1317942.023	9064.7815	UG	6.878019	MG/L
11/1/2004	Influent	TP	1.18	106.7	1297372.815	684499.7404	MG	0.527607	MG/L
11/1/2004	Effluent	TP	1.17	106.75	1317942.023	210850.2264	MG	0.159985	MG/L
11/1/2004	Influent	TSS	1.18	106.7	1297372.815	142337.4404	UG	109.712651	MG/L
11/1/2004	Effluent	TSS	1.17	106.75	1317942.023	30024.11324	UG	22.781181	MG/L
11/1/2004	Influent	VSS	1.18	106.7	1297372.815	19692.39425	UG	15.178753	MG/L
11/1/2004	Effluent	VSS	1.17	106.75	1317942.023	7795.24659	UG	5.914743	MG/L
11/1/2004	Influent	ZN	1.18	106.7	1297372.815	40958.00417	MG	31.570128	UG/L
11/1/2004	Effluent	ZN	1.17	106.75	1317942.023	19123.99961	MG	14.51058	UG/L
11/20/2004	Influent	CD	0.39	62.683	471811.274	235.90441	MG	0.5	UG/L
11/20/2004	Effluent	CD	0.39	62.683	91206.50649	45.60301	MG	0.5	UG/L
11/20/2004	Influent	COD	0.39	62.683	471811.274	22420.21891	UG	47.519711	MG/L
11/20/2004	Effluent	COD	0.39	62.683	91206.50649	1568.74348	UG	17.200001	MG/L
11/20/2004	Influent	CU	0.39	62.683	471811.274	1442.11135	MG	3.056559	UG/L
11/20/2004	Effluent	CU	0.39	62.683	91206.50649	273.61804	MG	3	UG/L
11/20/2004	Influent	DP	0.39	62.683	471811.274	146934.3892	MG	0.311428	MG/L
11/20/2004	Effluent	DP	0.39	62.683	91206.50649	10944.62407	MG	0.119999	MG/L
11/20/2004	Influent	NH3	0.39	62.683	471811.274	64374.4114	MG	0.136442	MG/L
11/20/2004	Effluent	NH3	0.39	62.683	91206.50649	11856.53111	MG	0.129997	MG/L
11/20/2004	Influent	NO23	0.39	62.683	471811.274	260877.3538	MG	0.55293	MG/L
11/20/2004	Effluent	NO23	0.39	62.683	91206.50649	19153.17911	MG	0.209999	MG/L
11/20/2004	Influent	PB	0.39	62.683	471811.274	818.7251	MG	1.73529	UG/L
11/20/2004	Effluent	PB	0.39	62.683	91206.50649	136.80902	MG	1.5	UG/L

Event Date	Influent/Effluent	Parameter	Total Rainfall (inches)	Dry Time (hr)	Volume (liters)	Load	Load Unit	EMC	EMC UNIT
11/20/2004	Influent	TKN	0.39	62.683	471811.274	311877.4713	MG	0.661025	MG/L
11/20/2004	Effluent	TKN	0.39	62.683	91206.50649	47426.12444	MG	0.519989	MG/L
11/20/2004	Influent	TN	0.39	62.683	471811.274	572754.8251	MG	1.213955	MG/L
11/20/2004	Effluent	TN	0.39	62.683	91206.50649	66579.30355	MG	0.729988	MG/L
11/20/2004	Influent	TOC	0.39	62.683	471811.274	6830.09015	UG	14.476393	MG/L
11/20/2004	Effluent	TOC	0.39	62.683	91206.50649	615.64058	UG	6.75	MG/L
11/20/2004	Influent	TP	0.39	62.683	471811.274	231942.0519	MG	0.491602	MG/L
11/20/2004	Effluent	TP	0.39	62.683	91206.50649	16417.11007	MG	0.18	MG/L
11/20/2004	Influent	TSS	0.39	62.683	471811.274	64402.71204	UG	136.501711	MG/L
11/20/2004	Effluent	TSS	0.39	62.683	91206.50649	1915.32626	UG	21	MG/L
11/20/2004	Influent	VSS	0.39	62.683	471811.274	5775.88229	UG	12.241997	MG/L
11/20/2004	Effluent	VSS	0.39	62.683	91206.50649	182.41202	UG	2	MG/L
11/20/2004	Influent	ZN	0.39	62.683	471811.274	17743.92447	MG	37.608293	UG/L
11/20/2004	Effluent	ZN	0.39	62.683	91206.50649	1395.45192	MG	15.299999	UG/L
2/7/2005	Influent	CD	0.74	19.45	1110862.839	555.41987	MG	0.5	UG/L
2/7/2005	Effluent	CD	0.72	19.583	3485642.422	1742.81177	MG	0.5	UG/L
2/7/2005	Influent	COD	0.74	19.45	1110862.839	40666.4084	UG	36.608708	MG/L
2/7/2005	Effluent	COD	0.72	19.583	3485642.422	118217.0348	UG	33.915606	MG/L
2/7/2005	Influent	CU	0.74	19.45	1110862.839	5289.32964	MG	4.76156	UG/L
2/7/2005	Effluent	CU	0.72	19.583	3485642.422	10456.87062	MG	3	UG/L
2/7/2005	Influent	DP	0.74	19.45	1110862.839	329396.5337	MG	0.296529	MG/L
2/7/2005	Effluent	DP	0.72	19.583	3485642.422	536094.6531	MG	0.153802	MG/L
2/7/2005	Influent	NH3	0.74	19.45	1110862.839	478795.1954	MG	0.431021	MG/L
2/7/2005	Effluent	NH3	0.72	19.583	3485642.422	908494.1543	MG	0.26064	MG/L
2/7/2005	Influent	NO23	0.74	19.45	1110862.839	456440.4696	MG	0.410897	MG/L
2/7/2005	Effluent	NO23	0.72	19.583	3485642.422	1411472.296	MG	0.404941	MG/L

Event Date	Influent/Effluent	Parameter	Total Rainfall (inches)	Dry Time (hr)	Volume (liters)	Load	Load Unit	EMC	EMC UNIT
2/7/2005	Influent	PB	0.74	19.45	1110862.839	1666.25962	MG	1.5	UG/L
2/7/2005	Effluent	PB	0.72	19.583	3485642.422	5228.43531	MG	1.5	UG/L
2/7/2005	Influent	TKN	0.74	19.45	1110862.839	2391409.474	MG	2.152794	MG/L
2/7/2005	Effluent	TKN	0.72	19.583	3485642.422	2872589.975	MG	0.824125	MG/L
2/7/2005	Influent	TN	0.74	19.45	1110862.839	2847849.944	MG	2.563691	MG/L
2/7/2005	Effluent	TN	0.72	19.583	3485642.422	4284062.271	MG	1.229066	MG/L
2/7/2005	Influent	TOC	0.74	19.45	1110862.839	7677.72176	UG	6.911638	MG/L
2/7/2005	Effluent	TOC	0.72	19.583	3485642.422	19190.39483	UG	5.505584	MG/L
2/7/2005	Influent	TP	0.74	19.45	1110862.839	559431.3601	MG	0.503611	MG/L
2/7/2005	Effluent	TP	0.72	19.583	3485642.422	1080012.706	MG	0.309848	MG/L
2/7/2005	Influent	TSS	0.74	19.45	1110862.839	265533.747	UG	239.038752	MG/L
2/7/2005	Effluent	TSS	0.72	19.583	3485642.422	490544.3622	UG	140.733604	MG/L
2/7/2005	Influent	VSS	0.74	19.45	1110862.839	30554.56342	UG	27.505825	MG/L
2/7/2005	Effluent	VSS	0.72	19.583	3485642.422	59650.19968	UG	17.113208	MG/L
2/7/2005	Influent	ZN	0.74	19.45	1110862.839	48887.39687	MG	44.009405	UG/L
2/7/2005	Effluent	ZN	0.72	19.583	3485642.422	99403.24534	MG	28.518067	UG/L
2/24/2005	Influent	CD	0.45	105.617	435790.968	217.8943	MG	0.5	UG/L
2/24/2005	Effluent	CD	0.47	142.267	78969.5327	39.48455	MG	0.5	UG/L
2/24/2005	Influent	COD	0.45	105.617	435790.968	21506.96829	UG	49.351837	MG/L
2/24/2005	Effluent	COD	0.47	142.267	78969.5327	2366.56244	UG	29.968206	MG/L
2/24/2005	Influent	CU	0.45	105.617	435790.968	2277.05405	MG	5.225134	UG/L
2/24/2005	Effluent	CU	0.47	142.267	78969.5327	236.90731	MG	3	UG/L
2/24/2005	Influent	DP	0.45	105.617	435790.968	71807.4741	MG	0.164776	MG/L
2/24/2005	Effluent	DP	0.47	142.267	78969.5327	7107.23148	MG	0.09	MG/L
2/24/2005	Influent	NH3	0.45	105.617	435790.968	71146.80003	MG	0.16326	MG/L
2/24/2005	Effluent	NH3	0.47	142.267	78969.5327	3698.97662	MG	0.046841	MG/L

Event Date	Influent/Effluent	Parameter	Total Rainfall (inches)	Dry Time (hr)	Volume (liters)	Load	Load Unit	EMC	EMC UNIT
2/24/2005	Influent	NO23	0.45	105.617	435790.968	364263.0642	MG	0.835871	MG/L
2/24/2005	Effluent	NO23	0.47	142.267	78969.5327	1787.52533	MG	0.022636	MG/L
2/24/2005	Influent	PB	0.45	105.617	435790.968	995.14867	MG	2.283558	UG/L
2/24/2005	Effluent	PB	0.47	142.267	78969.5327	118.45366	MG	1.5	UG/L
2/24/2005	Influent	TKN	0.45	105.617	435790.968	252274.4661	MG	0.578892	MG/L
2/24/2005	Effluent	TKN	0.47	142.267	78969.5327	51245.7702	MG	0.648934	MG/L
2/24/2005	Influent	TN	0.45	105.617	435790.968	616537.5303	MG	1.414763	MG/L
2/24/2005	Effluent	TN	0.47	142.267	78969.5327	53033.29553	MG	0.67157	MG/L
2/24/2005	Influent	TOC	0.45	105.617	435790.968	4086.43872	UG	9.377112	MG/L
2/24/2005	Effluent	TOC	0.47	142.267	78969.5327	466.50084	UG	5.907384	MG/L
2/24/2005	Influent	TP	0.45	105.617	435790.968	205896.5224	MG	0.472469	MG/L
2/24/2005	Effluent	TP	0.47	142.267	78969.5327	18163.15919	MG	0.230003	MG/L
2/24/2005	Influent	TSS	0.45	105.617	435790.968	50868.03329	UG	116.726395	MG/L
2/24/2005	Effluent	TSS	0.47	142.267	78969.5327	1026.59836	UG	13	MG/L
2/24/2005	Influent	VSS	0.45	105.617	435790.968	6836.21571	UG	15.687	MG/L
2/24/2005	Effluent	VSS	0.47	142.267	78969.5327	448.8686	UG	5.684104	MG/L
2/24/2005	Influent	ZN	0.45	105.617	435790.968	17430.06099	MG	39.996596	UG/L
2/24/2005	Effluent	ZN	0.47	142.267	78969.5327	1095.51718	MG	13.872731	UG/L
3/2/2005	Influent	CD	0.72	76.333	1316600.246	2263.25305	MG	1.719022	UG/L
3/2/2005	Effluent	CD	0.67	76.867	2485278.075	1242.63231	MG	0.5	UG/L
3/2/2005	Influent	COD	0.72	76.333	1316600.246	30110.8328	UG	22.870265	MG/L
3/2/2005	Effluent	COD	0.67	76.867	2485278.075	60659.89356	UG	24.407821	MG/L
3/2/2005	Influent	CU	0.72	76.333	1316600.246	9110.61375	MG	6.91984	UG/L
3/2/2005	Effluent	CU	0.67	76.867	2485278.075	8259.14399	MG	3.323245	UG/L
3/2/2005	Influent	DP	0.72	76.333	1316600.246	223683.1979	MG	0.169895	MG/L
3/2/2005	Effluent	DP	0.67	76.867	2485278.075	373925.6175	MG	0.150457	MG/L

Event Date	Influent/Effluent	Parameter	Total Rainfall (inches)	Dry Time (hr)	Volume (liters)	Load	Load Unit	EMC	EMC UNIT
3/2/2005	Influent	NH3	0.72	76.333	1316600.246	185815.171	MG	0.141133	MG/L
3/2/2005	Effluent	NH3	0.67	76.867	2485278.075	415811.2928	MG	0.167311	MG/L
3/2/2005	Influent	NO23	0.72	76.333	1316600.246	517901.7318	MG	0.393365	MG/L
3/2/2005	Effluent	NO23	0.67	76.867	2485278.075	438459.5062	MG	0.176424	MG/L
3/2/2005	Influent	PB	0.72	76.333	1316600.246	1974.88967	MG	1.5	UG/L
3/2/2005	Effluent	PB	0.67	76.867	2485278.075	4427.10906	MG	1.781343	UG/L
3/2/2005	Influent	TKN	0.72	76.333	1316600.246	695803.5792	MG	0.528488	MG/L
3/2/2005	Effluent	TKN	0.67	76.867	2485278.075	1611539.149	MG	0.648438	MG/L
3/2/2005	Influent	TN	0.72	76.333	1316600.246	1213705.311	MG	0.921853	MG/L
3/2/2005	Effluent	TN	0.67	76.867	2485278.075	2049998.656	MG	0.824862	MG/L
3/2/2005	Influent	TOC	0.72	76.333	1316600.246	7762.37797	UG	5.895806	MG/L
3/2/2005	Effluent	TOC	0.67	76.867	2485278.075	16236.57542	UG	6.533137	MG/L
3/2/2005	Influent	TP	0.72	76.333	1316600.246	416317.5515	MG	0.316208	MG/L
3/2/2005	Effluent	TP	0.67	76.867	2485278.075	746331.3737	MG	0.300303	MG/L
3/2/2005	Influent	TSS	0.72	76.333	1316600.246	131088.7044	UG	99.566603	MG/L
3/2/2005	Effluent	TSS	0.67	76.867	2485278.075	99287.97319	UG	39.950665	MG/L
3/2/2005	Influent	VSS	0.72	76.333	1316600.246	16540.00455	UG	12.562731	MG/L
3/2/2005	Effluent	VSS	0.67	76.867	2485278.075	22957.41533	UG	9.237413	MG/L
3/2/2005	Influent	ZN	0.72	76.333	1316600.246	51638.96412	MG	39.221657	UG/L
3/2/2005	Effluent	ZN	0.67	76.867	2485278.075	56465.78973	MG	22.720232	UG/L
5/29/2005	Influent	CD	0.61	14.833	391019.4005	195.50864	MG	0.5	UG/L
6/1/2005	Effluent	CD	0.54	49.167	941197.1026	470.54491	MG	0.5	UG/L
5/29/2005	Influent	COD	0.61	14.833	391019.4005	12120.14687	UG	30.996448	MG/L
6/1/2005	Effluent	COD	0.54	49.167	941197.1026	21648.52996	UG	23.003681	MG/L
5/29/2005	Influent	CU	0.61	14.833	391019.4005	1673.54822	MG	4.279985	UG/L
6/1/2005	Effluent	CU	0.54	49.167	941197.1026	2823.26946	MG	3	UG/L

Event Date	Influent/Effluent	Parameter	Total Rainfall (inches)	Dry Time (hr)	Volume (liters)	Load	Load Unit	EMC	EMC UNIT
5/29/2005	Influent	DP	0.61	14.833	391019.4005	28775.0469	MG	0.07359	MG/L
6/1/2005	Effluent	DP	0.54	49.167	941197.1026	98143.85687	MG	0.104287	MG/L
5/29/2005	Influent	NH3	0.61	14.833	391019.4005	67042.22554	MG	0.171456	MG/L
6/1/2005	Effluent	NH3	0.54	49.167	941197.1026	35123.82187	MG	0.037322	MG/L
5/29/2005	Influent	NO23	0.61	14.833	391019.4005	125192.7169	MG	0.320172	MG/L
6/1/2005	Effluent	NO23	0.54	49.167	941197.1026	16974.36519	MG	0.018037	MG/L
5/29/2005	Influent	PB	0.61	14.833	391019.4005	836.20205	MG	2.13853	UG/L
6/1/2005	Effluent	PB	0.54	49.167	941197.1026	1411.63473	MG	1.5	UG/L
5/29/2005	Influent	TKN	0.61	14.833	391019.4005	285542.0832	MG	0.730254	MG/L
6/1/2005	Effluent	TKN	0.54	49.167	941197.1026	767650.5313	MG	0.815704	MG/L
5/29/2005	Influent	TN	0.61	14.833	391019.4005	410734.8001	MG	1.050426	MG/L
6/1/2005	Effluent	TN	0.54	49.167	941197.1026	784624.8964	MG	0.833741	MG/L
5/29/2005	Influent	TOC	0.61	14.833	391019.4005	2829.08689	UG	7.235197	MG/L
6/1/2005	Effluent	TOC	0.54	49.167	941197.1026	8988.3355	UG	9.550986	MG/L
5/29/2005	Influent	TP	0.61	14.833	391019.4005	49982.82047	MG	0.127828	MG/L
6/1/2005	Effluent	TP	0.54	49.167	941197.1026	224131.1544	MG	0.238161	MG/L
5/29/2005	Influent	TSS	0.61	14.833	391019.4005	19099.26665	UG	48.84507	MG/L
6/1/2005	Effluent	TSS	0.54	49.167	941197.1026	17817.37806	UG	18.932707	MG/L
5/29/2005	Influent	ZN	0.61	14.833	391019.4005	13248.12922	MG	33.881186	UG/L
6/1/2005	Effluent	ZN	0.54	49.167	941197.1026	2823.26946	MG	3	UG/L
3/28/2006	Influent	COD	1.12	125.767	428518.3236	26779.15514	UG	62.492777	MG/L
3/28/2006	Effluent	COD	1.08	125.883	1789156.705	67005.6074	UG	37.451147	MG/L
3/28/2006	Influent	CU	1.12	125.767	428518.3236	12325.64466	MG	28.763557	UG/L
3/28/2006	Effluent	CU	1.08	125.883	1789156.705	28009.02837	MG	15.654962	UG/L
3/28/2006	Influent	DP	1.12	125.767	428518.3236	188160.9797	MG	0.439099	MG/L
3/28/2006	Effluent	DP	1.08	125.883	1789156.705	721681.8311	MG	0.403366	MG/L

Event Date	Influent/Effluent	Parameter	Total Rainfall (inches)	Dry Time (hr)	Volume (liters)	Load	Load Unit	EMC	EMC UNIT
3/28/2006	Influent	NH3	1.12	125.767	428518.3236	274198.1661	MG	0.639878	MG/L
3/28/2006	Effluent	NH3	1.08	125.883	1789156.705	708023.4628	MG	0.395732	MG/L
3/28/2006	Influent	NO23	1.12	125.767	428518.3236	1022995.963	MG	2.387299	MG/L
3/28/2006	Effluent	NO23	1.08	125.883	1789156.705	643354.0335	MG	0.359587	MG/L
3/28/2006	Influent	PB	1.12	125.767	428518.3236	2142.58001	MG	5	UG/L
3/28/2006	Effluent	PB	1.08	125.883	1789156.705	8945.73506	MG	5	UG/L
3/28/2006	Influent	TKN	1.12	125.767	428518.3236	1144492.408	MG	2.670828	MG/L
3/28/2006	Effluent	TKN	1.08	125.883	1789156.705	6019903.384	MG	3.364678	MG/L
3/28/2006	Influent	TN	1.12	125.767	428518.3236	2167488.371	MG	5.058127	MG/L
3/28/2006	Effluent	TN	1.08	125.883	1789156.705	6663257.418	MG	3.724265	MG/L
3/28/2006	Influent	TOC	1.12	125.767	428518.3236	5093.09171	UG	11.885418	MG/L
3/28/2006	Effluent	TOC	1.08	125.883	1789156.705	21134.56192	UG	11.812647	MG/L
3/28/2006	Influent	TP	1.12	125.767	428518.3236	214718.4053	MG	0.501074	MG/L
3/28/2006	Effluent	TP	1.08	125.883	1789156.705	948601.784	MG	0.530198	MG/L
3/28/2006	Influent	TSS	1.12	125.767	428518.3236	87935.32588	UG	205.208966	MG/L
3/28/2006	Effluent	TSS	1.08	125.883	1789156.705	35241.50116	UG	19.697376	MG/L
3/28/2006	Influent	VSS	1.12	125.767	428518.3236	8393.03859	UG	19.58629	MG/L
3/28/2006	Effluent	VSS	1.08	125.883	1789156.705	15222.98193	UG	8.508514	MG/L
3/28/2006	Influent	ZN	1.12	125.767	428518.3236	63118.31866	MG	147.295126	UG/L
3/28/2006	Effluent	ZN	1.08	125.883	1789156.705	8945.73506	MG	5	UG/L

APPENDIX F

Statistical Analysis of Paired Data

Cadmium

T-Test

Paired Samples Statistics

		Mean	N	Std. Deviation	Std. Error Mean
Pair 1	InfluentLn	-.5809	11	.37234	.11226
	EffluentLn	-.6931	11	.00000	.00000

Paired Samples Test

		Paired Differences					t	df	Sig. (2-tailed)
		Mean	Std. Deviation	Std. Error Mean	95% Confidence Interval of the Difference				
					Lower	Upper			
Pair 1	InfluentLn - EffluentLn	.11226	.37234	.11226	-.13788	.36240	1.000	10	.341

Chemical oxygen demand

T-Test

Paired Samples Statistics

		Mean	N	Std. Deviation	Std Error Mean
Pair 1	InfluentLn	3.8685	12	.71556	20656
	EffluentLn	3.2948	12	.38883	.11224

Paired Samples Test

		Paired Differences					t	df	Sig. (2-tailed)
		Mean	Std. Deviation	Std. Error Mean	95% Confidence Interval of the Difference Lower	95% Confidence Interval of the Difference Upper			
Pair 1	InfluentLn - EffluentLn	57372	.59658	.17222	.19467	.95277	3.331	11	.007

Correlations

Correlations

		InfluentLn	DryHoursInfluent
InfluentLn	Pearson Correlation	1	.827(**)
	Sig. (2-tailed)		.001
	N	12	12
DryHoursInfluent	Pearson Correlation	.827(**)	1
	Sig. (2-tailed)	.001	
	N	12	12

** Correlation is significant at the 0.01 level (2-tailed).

Nonparametric Correlations

Correlations

			InfluentLn	DryHoursInfluent
Spearman's rho	InfluentLn	Correlation Coefficient	1.000	.720(**)
		Sig. (2-tailed)	.	.008
		N	12	12
	DryHoursInfluent	Correlation Coefficient	.720(**)	1.000
		Sig. (2-tailed)	.008	.
		N	12	12

** Correlation is significant at the 0.01 level (2-tailed).

Correlations

Correlations

		EffluentLn	DryHoursEffluent
EffluentLn	Pearson Correlation	1	.232
	Sig. (2-tailed)		.468
	N	12	12
DryHoursEffluent	Pearson Correlation	.232	1
	Sig. (2-tailed)	.468	
	N	12	12

Nonparametric Correlations

Correlations

			EffluentLn	DryHoursEffluent
Spearman's rho	EffluentLn	Correlation Coefficient	1.000	140
		Sig. (2-tailed)	.	.665
		N	12	12
	DryHoursEffluent	Correlation Coefficient	.140	1.000
		Sig. (2-tailed)	.665	.
		N	12	12

Regression

Variables Entered/Removed(b)

Model	Variables Entered	Variables Removed	Method
1	DryHoursInfluent(a)	.	Enter

a All requested variables entered.
b Dependent Variable: InfluentLn

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.827(a)	.684	.652	.42183

a Predictors: (Constant), DryHoursInfluent

ANOVA(b)

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	3.853	1	3.853	21 653	.001(a)
	Residual	1.779	10	.178		
	Total	5.632	11			

a Predictors: (Constant), DryHoursInfluent
b Dependent Variable: InfluentLn

Coefficients(a)

		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
Model		B	Std. Error	Beta	B	Std. Error
1	(Constant)	3.228	.184		17.567	.000
	DryHoursInfluent	.007	.002	.827	4.653	.001

a Dependent Variable: InfluentLn

Copper

T-Test

Paired Samples Statistics

		Mean	N	Std. Deviation	Std. Error Mean
Pair 1	InfluentLn	1.6480	12	.61818	.17845
	EffluentLn	1.3131	12	.48092	.13883

Paired Samples Test

		Paired Differences					t	df	Sig. (2-tailed)
		Mean	Std. Deviation	Std. Error Mean	95% Confidence Interval of the Difference Lower	95% Confidence Interval of the Difference Upper			
Pair 1	InfluentLn - EffluentLn	.33491	.31481	.09088	.13490	.53493	3.685	11	.004

Correlations

Correlations

		InfluentLn	DryHoursInfluent
InfluentLn	Pearson Correlation	1	.119
	Sig. (2-tailed)		.714
	N	12	12
DryHoursInfluent	Pearson Correlation	.119	1
	Sig. (2-tailed)	.714	
	N	12	12

Nonparametric Correlations

Correlations

			InfluentLn	DryHoursInfluent
Spearman's rho	InfluentLn	Correlation Coefficient	1.000	.070
		Sig. (2-tailed)	.	.829
		N	12	12
	DryHoursInfluent	Correlation Coefficient	.070	1.000
		Sig. (2-tailed)	.829	.
		N	12	12

Correlations

Correlations

		EffluentLn	DryHoursEffluent
EffluentLn	Pearson Correlation	1	.393
	Sig. (2-tailed)		.207
	N	12	12
DryHoursEffluent	Pearson Correlation	.393	1
	Sig. (2-tailed)	.207	
	N	12	12

Nonparametric Correlations

Correlations

			EffluentLn	DryHoursEffluent
Spearman's rho	EffluentLn	Correlation Coefficient	1.000	.366
		Sig. (2-tailed)	.	.242
		N	12	12
	DryHoursEffluent	Correlation Coefficient	.366	1.000
		Sig. (2-tailed)	.242	.
		N	12	12

Dissolved phosphorus

T-Test

Paired Samples Statistics

		Mean	N	Std. Deviation	Std. Error Mean
Pair 1	InfluentLn	-1.6661	12	.72923	.21051
	EffluentLn	-2.5333	12	1.03769	.29956

Paired Samples Test

		Paired Differences					t	df	Sig. (2-tailed)
		Mean	Std. Deviation	Std. Error Mean	95% Confidence Interval of the Difference				
					Lower	Upper			
Pair 1	InfluentLn - EffluentLn	.86715	.76012	.21943	.38419	1.35011	3.952	11	.002

Correlations

Correlations

		InfluentLn	DryHoursInfluent
InfluentLn	Pearson Correlation	1	.226
	Sig. (2-tailed)		.479
	N	12	12
DryHoursInfluent	Pearson Correlation	.226	1
	Sig. (2-tailed)	.479	
	N	12	12

Nonparametric Correlations

Correlations

			InfluentLn	DryHoursInfluent
Spearman's rho	InfluentLn	Correlation Coefficient	1.000	.350
		Sig. (2-tailed)	.	.265
		N	12	12
	DryHoursInfluent	Correlation Coefficient	.350	1.000
		Sig. (2-tailed)	.265	.
		N	12	12

Correlations

Correlations

		EffluentLn	DryHoursEffluent
EffluentLn	Pearson Correlation	1	.145
	Sig. (2-tailed)		.653
	N	12	12
DryHoursEffluent	Pearson Correlation	.145	1
	Sig. (2-tailed)	.653	
	N	12	12

Nonparametric Correlations

Correlations

			EffluentLn	DryHoursEffluent
Spearman's rho	EffluentLn	Correlation Coefficient	1.000	.357
		Sig. (2-tailed)	.	.255
		N	12	12
	DryHoursEffluent	Correlation Coefficient	.357	1.000
		Sig. (2-tailed)	.255	.
		N	12	12

Nitrate + nitrite

T-Test

Paired Samples Statistics

		Mean	N	Std. Deviation	Std. Error Mean
Pair 1	InfluentLn	-.4170	12	.67439	.19468
	EffluentLn	-2.4256	12	1.15430	.33322

Paired Samples Test

	Paired Differences					t	df	Sig. (2-tailed)
	Mean	Std. Deviation	Std. Error Mean	95% Confidence Interval of the Difference Lower	95% Confidence Interval of the Difference Upper			
Pair 1 InfluentLn - EffluentLn	2.00862	1.02956	.29721	1.35447	2.66277	6.758	11	.000

Correlations

Correlations

		InfluentLn	DryHoursInfluent
InfluentLn	Pearson Correlation	1	.587(*)
	Sig. (2-tailed)		.045
	N	12	12
DryHoursInfluent	Pearson Correlation	.587(*)	1
	Sig (2-tailed)	.045	
	N	12	12

* Correlation is significant at the 0.05 level (2-tailed).

Nonparametric Correlations

Correlations

			InfluentLn	DryHoursInfluent
Spearman's rho	InfluentLn	Correlation Coefficient	1.000	.559
		Sig (2-tailed)	.	.059
		N	12	12
	DryHoursInfluent	Correlation Coefficient	.559	1.000
		Sig. (2-tailed)	.059	.
		N	12	12

Correlations

Correlations

		EffluentLn	DryHoursEffluent
EffluentLn	Pearson Correlation	1	-.075
	Sig. (2-tailed)		.817
	N	12	12
DryHoursEffluent	Pearson Correlation	-.075	1
	Sig. (2-tailed)	.817	
	N	12	12

Nonparametric Correlations

Correlations

			EffluentLn	DryHoursEffluent
Spearman's rho	EffluentLn	Correlation Coefficient	1.000	.007
		Sig. (2-tailed)	.	.983
		N	12	12
	DryHoursEffluent	Correlation Coefficient	.007	1.000
		Sig. (2-tailed)	.983	.
		N	12	12

Regression

Variables Entered/Removed(b)

Model	Variables Entered	Variables Removed	Method
1	DryHoursInfluent(a)	.	Enter

a All requested variables entered.
b Dependent Variable: InfluentLn

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.587(a)	.345	.279	.57255

a Predictors: (Constant), DryHoursInfluent

ANOVA(b)

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	1.725	1	1.725	5.262	.045(a)
	Residual	3.278	10	.328		
	Total	5.003	11			

a Predictors (Constant), DryHoursInfluent
b Dependent Variable: InfluentLn

Coefficients(a)

		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
Model		B	Std. Error	Beta	B	Std. Error
1	(Constant)	-.845	.249		-3.390	.007
	DryHoursInfluent	.005	.002	.587	2.294	.045

a Dependent Variable: InfluentLn

Ammonia

T-Test

Paired Samples Statistics

		Mean	N	Std. Deviation	Std. Error Mean
Pair 1	InfluentLn	-1.5934	12	.62538	.18053
	EffluentLn	-2.3796	12	.84405	.24365

Paired Samples Test

		Paired Differences					t	df	Sig. (2-tailed)
		Mean	Std. Deviation	Std. Error Mean	95% Confidence Interval of the Difference Lower	95% Confidence Interval of the Difference Upper			
Pair 1	InfluentLn - EffluentLn	.78611	.77160	22274	.29586	1.27636	3.529	11	.005

Correlations

Correlations

		InfluentLn	DryHoursInfluent
InfluentLn	Pearson Correlation	1	.434
	Sig. (2-tailed)		.159
	N	12	12
DryHoursInfluent	Pearson Correlation	.434	1
	Sig. (2-tailed)	.159	
	N	12	12

Nonparametric Correlations

Correlations

			InfluentLn	DryHoursInfluent
Spearman's rho	InfluentLn	Correlation Coefficient	1.000	.476
		Sig. (2-tailed)	.	.118
		N	12	12
	DryHoursInfluent	Correlation Coefficient	.476	1.000
		Sig. (2-tailed)	.118	.
		N	12	12

Correlations

Correlations

		EffluentLn	DryHoursEffluent
EffluentLn	Pearson Correlation	1	-.055
	Sig. (2-tailed)		.866
	N	12	12
DryHoursEffluent	Pearson Correlation	-.055	1
	Sig. (2-tailed)	.866	
	N	12	12

Nonparametric Correlations

Correlations

			EffluentLn	DryHoursEffluent
Spearman's rho	EffluentLn	Correlation Coefficient	1.000	.056
		Sig. (2-tailed)	.	.863
		N	12	12
	DryHoursEffluent	Correlation Coefficient	.056	1.000
		Sig. (2-tailed)	.863	.
		N	12	12

Lead

T-Test

Paired Samples Statistics

		Mean	N	Std Deviation	Std. Error Mean
Pair 1	InfluentLn	.6874	12	.36037	.10403
	EffluentLn	.5430	12	.34738	.10028

Paired Samples Test

		Paired Differences					t	df	Sig. (2-tailed)
		Mean	Std. Deviation	Std. Error Mean	95% Confidence Interval of the Difference Lower	95% Confidence Interval of the Difference Upper			
Pair 1	InfluentLn - EffluentLn	.14440	.19426	.05608	.02098	.26783	2.575	11	.026

Correlations

Correlations

		InfluentLn	DryHoursInfluent
InfluentLn	Pearson Correlation	1	-.048
	Sig. (2-tailed)		.883
	N	12	12
DryHoursInfluent	Pearson Correlation	-.048	1
	Sig. (2-tailed)	.883	
	N	12	12

Nonparametric Correlations

Correlations

			InfluentLn	DryHoursInfluent
Spearman's rho	InfluentLn	Correlation Coefficient	1.000	.032
		Sig. (2-tailed)	.	.921
		N	12	12
	DryHoursInfluent	Correlation Coefficient	.032	1.000
		Sig. (2-tailed)	.921	.
		N	12	12

Correlations

Correlations

		EffluentLn	DryHoursEffluent
EffluentLn	Pearson Correlation	1	.144
	Sig. (2-tailed)		.655
	N	12	12
DryHoursEffluent	Pearson Correlation	.144	1
	Sig. (2-tailed)	.655	
	N	12	12

Nonparametric Correlations

Correlations

			EffluentLn	DryHoursEffluent
Spearman's rho	EffluentLn	Correlation Coefficient	1.000	-.009
		Sig. (2-tailed)	.	.977
		N	12	12
	DryHoursEffluent	Correlation Coefficient	-.009	1.000
		Sig. (2-tailed)	.977	.
		N	12	12

Total Kjedhal Nitrogen

T-Test

Paired Samples Statistics

		Mean	N	Std. Deviation	Std. Error Mean
Pair 1	InfluentLn	-.3119	12	.76777	.22164
	EffluentLn	-.4867	12	63905	.18448

Paired Samples Test

	Paired Differences					t	df	Sig. (2-tailed)
	Mean	Std. Deviation	Std. Error Mean	95% Confidence Interval of the Difference Lower	Upper			
Pair 1 InfluentLn - EffluentLn	.17479	.63398	.18301	-.22802	.57760	.955	11	.360

Total nitrogen

T-Test

Paired Samples Statistics

		Mean	N	Std. Deviation	Std. Error Mean
Pair 1	InfluentLn	.3806	12	64911	.18738
	EffluentLn	-.2968	12	62944	18171

Paired Samples Test

		Paired Differences					t	df	Sig. (2-tailed)
		Mean	Std. Deviation	Std. Error Mean	95% Confidence Interval of the Difference				
					Lower	Upper			
Pair 1	InfluentLn - EffluentLn	.67737	.51284	.14804	.35153	1.00322	4.575	11	.001

Correlations

Correlations

		InfluentLn	DryHoursInfluent
InfluentLn	Pearson Correlation	1	.458
	Sig. (2-tailed)		.134
	N	12	12
DryHoursInfluent	Pearson Correlation	.458	1
	Sig. (2-tailed)	.134	
	N	12	12

Nonparametric Correlations

Correlations

			InfluentLn	DryHoursInfluent
Spearman's rho	InfluentLn	Correlation Coefficient	1.000	.469
		Sig. (2-tailed)	.	.124
		N	12	12
	DryHoursInfluent	Correlation Coefficient	.469	1.000
		Sig. (2-tailed)	124	.
		N	12	12

Correlations

Correlations

		EffluentLn	DryHoursEffluent
EffluentLn	Pearson Correlation	1	.174
	Sig. (2-tailed)		.588
	N	12	12
DryHoursEffluent	Pearson Correlation	.174	1
	Sig. (2-tailed)	.588	
	N	12	12

Nonparametric Correlations

Correlations

			EffluentLn	DryHoursEffluent
Spearman's rho	EffluentLn	Correlation Coefficient	1.000	.112
		Sig. (2-tailed)	.	.729
		N	12	12
	DryHoursEffluent	Correlation Coefficient	.112	1.000
		Sig. (2-tailed)	729	.
		N	12	12

Total organic carbon

T-Test

Paired Samples Statistics

		Mean	N	Std. Deviation	Std. Error Mean
Pair 1	InfluentLn	2.1919	12	.53782	.15526
	EffluentLn	1.9870	12	.25500	.07361

Paired Samples Test

	Paired Differences					t	df	Sig. (2-tailed)
	Mean	Std. Deviation	Std. Error Mean	95% Confidence Interval of the Difference Lower	95% Confidence Interval of the Difference Upper			
Pair 1 InfluentLn - EffluentLn	.20494	.43135	.12452	-.06912	.47900	1.646	11	.128

Total phosphorus

T-Test

Paired Samples Statistics

		Mean	N	Std. Deviation	Std. Error Mean
Pair 1	InfluentLn	-1.0070	12	.65718	.18971
	EffluentLn	-1.6980	12	.57062	.16472

Paired Samples Test

		Paired Differences					t	df	Sig. (2-tailed)
		Mean	Std. Deviation	Std. Error Mean	95% Confidence Interval of the Difference Lower	95% Confidence Interval of the Difference Upper			
Pair 1	InfluentLn - EffluentLn	.69105	.81083	.23407	.17587	1.20623	2.952	11	.013

Correlations

Correlations

		InfluentLn	DryHoursInfluent
InfluentLn	Pearson Correlation	1	.551
	Sig. (2-tailed)		.064
	N	12	12
DryHoursInfluent	Pearson Correlation	.551	1
	Sig. (2-tailed)	.064	
	N	12	12

Nonparametric Correlations

Correlations

			InfluentLn	DryHoursInfluent
Spearman's rho	InfluentLn	Correlation Coefficient	1.000	.524
		Sig. (2-tailed)	.	.080
		N	12	12
	DryHoursInfluent	Correlation Coefficient	.524	1.000
		Sig. (2-tailed)	.080	.
		N	12	12

Correlations

Correlations

		EffluentLn	DryHoursEffluent
EffluentLn	Pearson Correlation	1	-.069
	Sig. (2-tailed)		.831
	N	12	12
DryHoursEffluent	Pearson Correlation	-.069	1
	Sig. (2-tailed)	.831	
	N	12	12

Nonparametric Correlations

Correlations

			EffluentLn	DryHoursEffluent
Spearman's rho	EffluentLn	Correlation Coefficient	1.000	.168
		Sig. (2-tailed)	.	.602
		N	12	12
	DryHoursEffluent	Correlation Coefficient	.168	1.000
		Sig. (2-tailed)	.602	.
		N	12	12

Total suspended solids

T-Test

Paired Samples Statistics

		Mean	N	Std. Deviation	Std. Error Mean
Pair 1	InfluentLn	5.1317	12	.77093	.22255
	EffluentLn	3.4705	12	.68515	.19778

Paired Samples Test

	Paired Differences					t	df	Sig. (2-tailed)
	Mean	Std. Deviation	Std. Error Mean	95% Confidence Interval of the Difference Lower	Upper			
Pair 1 InfluentLn - EffluentLn	1 66122	.85884	.24792	1.11554	2.20690	6.701	11	.000

Correlations

Correlations

		InfluentLn	DryHoursInfluent
InfluentLn	Pearson Correlation	1	.731(**)
	Sig. (2-tailed)		.007
	N	12	12
DryHoursInfluent	Pearson Correlation	.731(**)	1
	Sig. (2-tailed)	.007	
	N	12	12

** Correlation is significant at the 0.01 level (2-tailed).

Nonparametric Correlations

Correlations

			InfluentLn	DryHoursInfluent
Spearman's rho	InfluentLn	Correlation Coefficient	1.000	.538
		Sig. (2-tailed)	.	.071
		N	12	12
	DryHoursInfluent	Correlation Coefficient	.538	1.000
		Sig. (2-tailed)	.071	.
		N	12	12

Correlations

Correlations

		EffluentLn	DryHoursEffluent
EffluentLn	Pearson Correlation	1	-.613(*)
	Sig. (2-tailed)		.034
	N	12	12
DryHoursEffluent	Pearson Correlation	-.613(*)	1
	Sig. (2-tailed)	.034	
	N	12	12

* Correlation is significant at the 0.05 level (2-tailed).

Nonparametric Correlations

Correlations

			EffluentLn	DryHoursEffluent
Spearman's rho	EffluentLn	Correlation Coefficient	1.000	-.678(*)
		Sig. (2-tailed)	.	.015
		N	12	12
	DryHoursEffluent	Correlation Coefficient	-.678(*)	1.000
		Sig. (2-tailed)	.015	.
		N	12	12

* Correlation is significant at the 0.05 level (2-tailed)

Regression

Variables Entered/Removed(b)

Model	Variables Entered	Variables Removed	Method
1	DryHoursInfluent(a)	.	Enter

a All requested variables entered.
b Dependent Variable: InfluentLn

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.731(a)	.535	.488	.55162

a Predictors: (Constant), DryHoursInfluent

ANOVA(b)

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	3.495	1	3.495	11.485	.007(a)
	Residual	3.043	10	.304		
	Total	6.538	11			

a Predictors: (Constant), DryHoursInfluent
b Dependent Variable: InfluentLn

Coefficients(a)

		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
Model		B	Std. Error	Beta	B	Std. Error
1	(Constant)	4.522	.240		18.817	.000
	DryHoursInfluent	.007	.002	.731	3.389	.007

a Dependent Variable: InfluentLn

Regression

Variables Entered/Removed(b)

Model	Variables Entered	Variables Removed	Method
1	DryHoursEffluent(a)	.	Enter

a All requested variables entered
b Dependent Variable: EffluentLn

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.613(a)	.376	.313	.56778

a Predictors: (Constant), DryHoursEffluent

ANOVA(b)

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	1.940	1	1.940	6.018	.034(a)
	Residual	3.224	10	.322		
	Total	5 164	11			

a Predictors: (Constant), DryHoursEffluent
b Dependent Variable: EffluentLn

Coefficients(a)

		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
Model		B	Std. Error	Beta	B	Std. Error
1	(Constant)	3.908	.242		16.138	.000
	DryHoursEffluent	-.006	.002	-.613	-2.453	.034

a Dependent Variable: EffluentLn

Volatile Suspended Solids

T-Test

Paired Samples Statistics

		Mean	N	Std. Deviation	Std. Error Mean
Pair 1	InfluentLn	3.0479	11	.69808	21048
	EffluentLn	2.0166	11	.52637	.15871

Paired Samples Test

		Paired Differences					t	df	Sig. (2-tailed)
		Mean	Std. Deviation	Std. Error Mean	95% Confidence Interval of the Difference				
					Lower	Upper			
Pair 1	InfluentLn - EffluentLn	1.03137	.70742	.21329	.55612	1.50662	4.835	10	.001

Correlations

Correlations

		InfluentLn	DryHoursInfluent
InfluentLn	Pearson Correlation	1	.770(**)
	Sig. (2-tailed)		.006
	N	11	11
DryHoursInfluent	Pearson Correlation	.770(**)	1
	Sig. (2-tailed)	.006	
	N	11	11

** Correlation is significant at the 0.01 level (2-tailed).

Nonparametric Correlations

Correlations

			InfluentLn	DryHoursInfluent
Spearman's rho	InfluentLn	Correlation Coefficient	1.000	.564
		Sig. (2-tailed)	.	.071
		N	11	11
	DryHoursInfluent	Correlation Coefficient	.564	1.000
		Sig. (2-tailed)	.071	.
		N	11	11

Correlations

Correlations

		EffluentLn	DryHoursEffluent
EffluentLn	Pearson Correlation	1	-.014
	Sig. (2-tailed)		.967
	N	11	11
DryHoursEffluent	Pearson Correlation	-.014	1
	Sig. (2-tailed)	.967	
	N	11	11

Nonparametric Correlations

Correlations

			EffluentLn	DryHoursEffluent
Spearman's rho	EffluentLn	Correlation Coefficient	1.000	.055
		Sig. (2-tailed)	.	.873
		N	11	11
	DryHoursEffluent	Correlation Coefficient	.055	1.000
		Sig. (2-tailed)	.873	.
		N	11	11

Regression

Variables Entered/Removed(b)

Model	Variables Entered	Variables Removed	Method
1	DryHoursInfluent(a)	.	Enter

a All requested variables entered.
b Dependent Variable: InfluentLn

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.350(a)	.123	.035	.52236

a Predictors: (Constant), DryHoursInfluent

ANOVA(b)

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	.382	1	.382	1.399	.264(a)
	Residual	2.729	10	.273		
	Total	3.110	11			

a Predictors: (Constant), DryHoursInfluent
b Dependent Variable: InfluentLn

Coefficients(a)

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta	B	Std. Error
1	(Constant)	3.415	.228		15.009	.000
	DryHoursInfluent	.002	.002	.350	1.183	.264

a Dependent Variable: InfluentLn

Zinc

T-Test

Paired Samples Statistics

		Mean	N	Std. Deviation	Std Error Mean
Pair 1	InfluentLn	3.6169	12	.53174	.15350
	EffluentLn	2.2163	12	.80503	.23239

Paired Samples Test

		Paired Differences					t	df	Sig. (2-tailed)
		Mean	Std. Deviation	Std. Error Mean	95% Confidence Interval of the Difference: Lower	95% Confidence Interval of the Difference: Upper			
Pair 1	InfluentLn - EffluentLn	1.40056	1 03800	.29964	.74105	2.06007	4.674	11	.001

Correlations

Correlations

		InfluentLn	DryHoursInfluent
InfluentLn	Pearson Correlation	1	.350
	Sig. (2-tailed)		.264
	N	12	12
DryHoursInfluent	Pearson Correlation	.350	1
	Sig. (2-tailed)	.264	
	N	12	12

Nonparametric Correlations

Correlations

			InfluentLn	DryHoursInfluent
Spearman's rho	InfluentLn	Correlation Coefficient	1.000	.531
		Sig. (2-tailed)	.	.075
		N	12	12
	DryHoursInfluent	Correlation Coefficient	.531	1.000
		Sig. (2-tailed)	.075	.
		N	12	12

Correlations

Correlations

		EffluentLn	DryHoursEffluent
EffluentLn	Pearson Correlation	1	- 014
	Sig. (2-tailed)		.967
	N	12	12
DryHoursEffluent	Pearson Correlation	-.014	1
	Sig. (2-tailed)	.967	
	N	12	12

Nonparametric Correlations

Correlations

			EffluentLn	DryHoursEffluent
Spearman's rho	EffluentLn	Correlation Coefficient	1.000	.007
		Sig. (2-tailed)	.	.983
		N	12	12
	DryHoursEffluent	Correlation Coefficient	.007	1.000
		Sig. (2-tailed)	983	.
		N	12	12

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