THESIS

ANALYSIS AND EVALUATION OF STORMWATER QUANTITY AND QUALITY PERFORMANCE FOR THREE PERMEABLE PAVEMENT SYSTEMS IN FORT COLLINS, COLORADO

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ABSTRACT

ANALYSIS AND EVALUATION OF STORMWATER QUANTITY AND QUALITY PERFORMANCE FOR THREE PERMEABLE PAVEMENT SYSTEMS IN FORT COLLINS, COLORADO

Urbanization and the subsequent increase of effective impervious area (EIA) result in an increase in storm runoff volumes, peak flow rates and pollutant concentrations. Stormwater management has recently shifted towards a focus on site level low impact development (LID) techniques that aim to reduce the total stormwater runoff volumes in addition to attenuating peak flows and removing pollutants at or near the source of runoff. Permeable pavement systems (PPS) are a subset of LID stormwater best management practices (BMPs) of particular interest in dense urban areas because they can be installed in parking areas and low traffic roadways where the availability of land space for more traditional BMPs is not available. However, few studies have documented the performance of PPS in terms of reducing runoff volume, peak flow and pollutant loads in semi-arid environments such as Colorado. Such information is necessary to improve the selection of BMP/LIDs for stormwater management.

Three PPS in Fort Collins, Colorado were monitored between 2009 and 2011 to evaluate pollutant reduction, runoff volume reduction performance and surface infiltration rates. The Mountain and Walnut permeable inter-locking concrete paver (PICP) sites, referred to collectively as Mitchell Block, were each designed with differing "no-infiltration" sub-base designs to compare performance between a system with a sand filter layer (Walnut) and one with only gravel layers (Mountain). The third site, referred to as CTL, is a porous concrete (PC) parking lot that allows full infiltration, and was only monitored for water quality and surface infiltration rates.

Mountain, Walnut and CTL all had lower effluent median event mean concentrations (EMCs) than those found at two Fort Collins stormwater outfalls for; total suspended solids (TSS), total recoverable zinc (TR Zn), total phosphorous (TP), total nitrogen (TN), total organic nitrogen (TON), total Kjeldahl nitrogen (TKN) and ammonia (NH3). EMCs for TR copper (Cu), nitrate (NO3) and total dissolved solids (TDS) at all three sites were elevated compared to the outfall sites. The TR Cu result EMCs at the three PPS were elevated compared to effluent PPS data from the International Stormwater BMP Database, which may indicate higher source concentrations in these study areas. CTL had elevated TR chromium (Cr) concentrations, which is likely a function of the portland cement in the PC itself, leaching Cr into the exfiltrate. Walnut had lower effluent median EMCs for 10 of the 13 water quality parameters analyzed, including significantly lower concentrations for TON, TKN and TR Cu.

Recorded effluent volumes and estimated influent volumes to the PPS at the Mitchell Block sites were used to calculate runoff volume reduction on an event-based and long-term basis. Both sites provided runoff reduction for over 70% of the monitored events, with Mountain and Walnut reducing 45% and 35% of the total runoff volume monitored at each site, respectively. These results confirm that "no-infiltration" PPS designs are capable of reducing large volumes of storm runoff. Field capacity (water retention capacity) of the two sites was investigated with regard to runoff reduction. Runoff volume reduction at Mountain exceeded the field capacity for the two longest storms monitored. This suggests that runoff volume reduction potential can exceed field capacity given long intermittent rainfall events. An investigation of hydrologic storm parameters indicated a discernible trend between runoff volume reduction and antecedent dry time, showing increasing runoff volume reduction with increasing antecedent dry time.

The runoff volume reduction performance at Mountain was greater than Walnut based on 23% greater median and average volume reduction per storm in addition to 25% greater total aggregate volume reduction for common monitored events at the two sites. This study did not investigate the design characteristics that allowed Mountain to provide greater runoff volume reduction.

Surface infiltration rates at all three sites were estimated using a single infiltrometer field test. The results indicated that sections of all three sites are experiencing varying degrees of clogging. CTL had the highest degree of clogging, with two of the three tests indicating zero infiltration. Maintenance is recommended to reduce clogging for all three sites.

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Thanks to the City of Fort Collins for providing funding for this project. Basil Hamdan and the other members of stormwater group were helpful in coordinating repairs and installation of field equipment.

Thanks to the other members of the field sampling team, including; Kristina Lowthian, Jason Messamer, Chris Olson and Laurie Trifone. Jason and Chris setup and provided the framework for much of this project.

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1.0 INTRODUCTION

Urbanization shifts hydrologic processes within a watershed and often has detrimental chemical, physical and biological effects on downstream receiving waters. Expansion of urban areas is synonymous with an increase in effective impervious area (EIA), defined as impermeable surfaces directly connected hydraulically to receiving waters via storm sewers, gutters and direct runoff. The result is an increase in surface runoff volume, which is compounded by channelized runoff due to smooth conduits that route it downstream in an expedited manner. The hydrologic outcome is larger, more frequent flood flows and a decrease in groundwater recharge (Leopold 1968).

Impacts of increased EIA extend beyond the hydrologic consequences, as impervious surfaces provide a palate for pollutants to accumulate on. During runoff events these pollutants are mobilized by physical and chemical processes and carried to downstream receiving waters. Water quality problems are generally much more difficult to address than quantity issues, which led to the National Urban Runoff Program (NURP), a study conducted by the Environmental Protection Agency (EPA) between 1979 and 1983 that identified the major contaminants in urban runoff. The main contaminants identified were; heavy metals (specifically copper, zinc and lead), organics, bacteria, oxygen-demanding substances, nutrients and solids (USEPA 1983).

Due to increasing urban pressure and a larger emphasis being placed on environmental impacts, stormwater management has become a high priority for the EPA and a requirement for municipalities. Under the Clean Water Act of 1972, the EPA developed a basis for controlling the release of polluted water into receiving waters under the National Pollution Discharge Elimination System (NPDES), which specifically addressed point discharges. In 1987 the Water Quality Act (WQA) required that stormwater discharge be included as part of the NPDES. Three major groups are required to obtain a permit under these regulations: operators of large, medium and regulated small municipal stormwater systems (MS4s), operators of construction sites that are one acre or larger, and industrial sectors (U.S. EPA 2005). Stormwater permits generally require the implementation of Best Management Practices (BMPs) to reduce the impacts of discharged runoff on receiving waters.

Stormwater BMPs aim to control adverse hydrologic effects and improve water quality of runoff. Stormwater BMPs can be structural or non-structural. Detention basins are the most common stormwater BMP and are typically effective in reducing peak flows and providing improved water quality. In recent years, research has found that these structures show an inability to control the increased volume of runoff from urban areas and can actually increase flow duration exceedances (Booth and C. R. Jackson 1998; Booth et al. 2003; Finkenbine and Atwater 2001).

Recognition of this short fall has led to a different basic stormwater management philosophy, which attempts to match pre-developed hydrologic conditions through Low Impact Development (LID). Examples of common LID techniques include bio-retention basins, infiltration trenches and basins, rain gardens, green roofs and Permeable Pavement Systems (PPS). These generally have a smaller footprint than traditional BMPs by serving a purpose beyond stormwater management (i.e.: PPS often serve as parking areas).

Change in management philosophies is a continuum guided by new information from current research and changing sociological and economic conditions. Source controls and LID techniques provide reduction of runoff volume in addition to pollutant removal. Together these benefits provide a significant reduction of pollutant loads discharged downstream. PPS have become especially popular because of convenient retrofit opportunities in low traffic and parking areas, thus not requiring the designation of other land space for stormwater treatment. The increasing interest in using source treatment controls and LID techniques for stormwater management requires that we bolster our understanding of these systems and continue to improve their design and application.

This thesis presents findings from a practice-level LID monitoring study of water quantity, water quality and infiltration performance for three PPS sites in the downtown area of Fort Collins, CO. The monitoring efforts spanned two years between 2009 and 2011. Water quantity data, water quality data and surface infiltration rate data were collected and analyzed for the three different PPS sites. Two of the sites, Mountain and Walnut (referred to collectively as Mitchell Block), are diagonal street parking areas constructed with modular paving blocks that use a "no-infiltration" design with an under-drain system. Mountain and Walnut utilize different sub-base designs to compare performance based on water quantity and water quality analyses. The third site, referred to as CTL Thompson (CTL), is a commercial business parking lot constructed with porous concrete (PC) using a full-infiltration sub-base design. All monitoring and data collection efforts were performed by the Colorado State University (CSU) Civil and Environmental Engineering Department in cooperation with the City of Fort Collins Utilities Division (the City).

The four main objectives of this study were to

1. Determine if the three PPS in this study are providing treatment to urban runoff through comparison to Fort Collins stormwater outfall data and determine if their water quality is comparable to effluent values for PPS reported in the International Stormwater BMP Database

2. Determine if “no-infiltration” under-drained PPS, which have no opportunity to infiltrate water into the underlying soil, are capable of reducing runoff, quantify the runoff reduction and investigate the relationship between runoff reduction performance and the properties of the sub-base media

3. Compare alternative PPS designs at Mitchell Block with regard to runoff reduction data, water quality data and maintenance needs

4. Determine the maintenance requirements at the three sites based on field observations and surface infiltration tests

1.1 Literature Review

PPS is a general term for any pavement material that is designed to be pervious in nature and allows water to infiltrate its surface. Common types of PPS include: permeable interlocking concrete pavers, concrete grid pavers, plastic reinforcing grid pavers, porous concrete, porous asphalt and gravel. Some materials are designed to have pervious qualities (porous concrete) while others are impervious, but are placed such that pervious spaces are created (permeable interlocking pavers). Different design methodologies for the underlying aggregate layers have been developed to accommodate variable site conditions and to attempt to optimize functionality. Many sites have been developed with monitoring capabilities to attempt to understand the functionality of the varying designs and help with future applications. This literature review attempts to identify the findings of previous research on this topic.

1.1.1 Permeable Paver Water Quality

PPS have been shown to be effective in providing treatment to urban runoff and reducing the volume of surface runoff (Brattebo and Booth 2003; Fassman and Blackbourn 2010). The degree of effectiveness is dependent on many factors including, site conditions, hydrologic conditions, design and maintenance.

Available data suggests that PPS reduce the concentration of various pollutants in runoff (Booth and Leavitt 1999; Brattebo and Booth 2003; Fassman and Blackbourn 2010; C. J. Pratt et al. 1989). Permeable paver water quality research tends to focus on major stormwater constituents including metals, nutrients, solids, conductivity, hardness, alkalinity and occasionally motor oils. Brattebo and Booth (2003) showed that toxic concentrations of copper and zinc were reached in 97% of samples from traditional asphalt, while permeable paver exfiltrate EMCs were below toxic levels in 31 of 36 samples, with the majority of concentrations falling below minimum detection levels (MDLs). Bean et al. (2008) showed that concentrations of total Kjeldahl nitrogen, zinc, total phosphorous and ammonia were significantly less in exfiltrate from PPS than traditional asphalt runoff. Fassman & Blackbourn (2010) found that PPS on a roadway in New Zealand significantly reduced the concentration of total suspended solids and total recoverable and dissolved copper and zinc.

1.1.1 Permeable Paver Water Quantity

Studies have repeatedly shown that PPS are capable of reducing or potentially eliminating surface runoff (Eban Zachary Bean et al. 2008; Brattebo and Booth 2003; Fassman and Blackbourn 2010; Gilbert and Clausen 2006). Systems can be of three types: no-infiltration, partial infiltration or full infiltration (UDFCD 2010). Full infiltration systems allow all of the infiltrated runoff to infiltrate into the soil column below the system draining eventually into the groundwater. Partial infiltration systems operate on the same concept, but due to limited infiltration capacity in the subsurface soil, an under-drain is provided at some elevation above the soil layers to eliminate excessive saturation within the system. No-infiltration designs are utilized in areas where the conditions are not conducive to infiltrating water. Conditions could include nearby structures or foundations or poor draining soils. These systems are designed with an impervious lining below the aggregate layers and above the sub-surface soil layers and an under-drain at the lowest elevation of the system to collect all of the infiltrated runoff (exfiltrate) and carry it to the storm drain. It is clear that full and partial infiltration systems have the ability to reduce runoff volumes significantly. Less obvious is the ability of no-infiltration systems to reduce runoff volumes. The majority of available literature focuses on the former, but there are some studies that address the latter.

Brattebo and Booth (2003) found that full infiltration systems were capable of eliminating all surface runoff for nearly all storms. They looked at 4 different types of PPS used in a parking area application. At two of the sites no surface runoff occurred for any of the fifteen monitored events. At the other two sites minimal surface runoff occurred for six of the fifteen events, four of which were attributed to factors unassociated with the performance of the system.

Studies on runoff reduction at PPS sites with a "no-infiltration" design are rare. Pratt et. al. (1995) investigated water quantity of four PPS with different sub-base materials in the UK during the early 1990s. They found that at all four sites the pavers discharged between 34% and 47% of the rainfall depth on average. They attributed the differences between sites to higher surface area and wetting potential of the sub-base materials. They also observed significant variability in runoff reduction between individual events, which they attributed to differing antecedent hydrologic conditions.

In "no-infiltration" design systems, evaporation is the only mechanism that causes runoff reduction. The pavers and aggregate are capable of absorbing a certain amount of moisture, referred to here as wetting potential, which is dependent on the material properties. Water is also retained in the voids of the aggregate in the bedding and sub-base layers. The amount of water retained within the layers is referred to as the field capacity, which is dependent on the material type and grain size distribution. Standard field capacity values for different types of materials can be found in the literature. The field capacity of the sub-base material and the amount of water absorbed by the pavers and base layers, represent the maximum volume available for evaporation at the conclusion of a storm assuming that evaporation is able to occur through the entire depth of the material. Evaporation occurs as the water molecules increase in energy due to solar radiation to a point where there vapor pressure exceeds that of ambient pressure and the molecular bonds are broken causing a phase change. As evaporation occurs between events, the wetting potential and field capacity is restored within the material.

Andersen et al. (1999) showed that evaporation from PPS is less than evaporation from an evaporation pan subjected to identical conditions. This study looked at the hydrological characteristics from several different PPS designs under simulated rainfall events. With respect to evaporation, they found that systems with a smaller grain size distribution in the base layers resulted in the maximum evaporation, at about 27% of that from an evaporation pan. This study also indicates that another controlling factor may be the exposed permeable surface area, which acts as a wick to draw water up through the base layers for evaporation.

1.1.2 Maintenance and Surface Infiltration

Any infiltration based BMP relies on maintaining its perviousness. The first step in the design process is to select a site that is not vulnerable to clogging, due to the presence of fine materials and significant erosion potential in the contributing drainage area. It is often the case in PPS applications that clogging is unavoidable, and in these cases it is critical to be diligent with maintenance activities in order to maintain a reasonable surface infiltration rate.

Several studies have looked at surface infiltration rates in PPS and the effectiveness of maintenance efforts in recovering adequate surface infiltration capacity. A study by Bean et al. (2007) investigated surface infiltration rates of 40 different PPS sites in the North Carolina area consisting of concrete grid (CG) pavers, permeable inner-locking concrete pavers (PICP) and PC sites. They used single and double ring infiltrometers at the different sites to evaluate surface infiltration before and after "simulated maintenance", in which 13 mm to 19 mm of surface material was removed from the voids of the CG pavers. They found that 14 of the 15 CG sites tested had statistically significant higher infiltration rates after the simulated maintenance, indicating that the infiltration capacity at these sites was being inhibited due to clogging. Of the eleven PICP sites tested, seven were located in stable watersheds where deposition of fine materials was unlikely, while the other four were located in unstable watersheds and were prone to the accumulation of fine materials. The median infiltration rate for the unstable watersheds, 16 cm per hour, was about 99% less than the median from the stable watersheds, 4000 cm per hour. This demonstrates the detrimental effect of fine materials on the functionality of PPS.

Widespread use of permeable pavements is deterred due to a lack of information and available resources on the long term functionality and maintenance requirements of these systems. In 2009 the EPA constructed a test parking lot with three different types of PPS at their Edison, NJ office (Borst and Rowe 2010). The lot includes PICP, PC and permeable asphalt (PA). Infiltration tests were run every month using a slightly modified version of ASTM method C1701. After six months none of the pavers showed any significant decrease in infiltration rates. This project is ongoing and will continue monthly testing in hopes of determining life cycle costs and maintenance costs for the different types of PPS in a parking lot application.

Maintenance of PPS is a relatively new concept and requires techniques different from traditional street sweeping methods. Regenerative air sweepers and vacuum sweepers have been shown to be effective in restoring infiltration capacity. Dierkes et. al. (2002) showed that a vacuum sweeper was very effective in restoring infiltration capacity to a permeable paver site that was essentially completely clogged. Urban Drainage Flood Control District (UDFCD) recommends maintaining any type of permeable pavement site two times per year by way of dry vacuum sweepers, especially in cases where maintenance has been neglected and clogging has already occurred (UDFCD, 2010).

1.2 Research Objective and Project Background

The objective of this research was to gather flow and water quality data for three PPS sites in downtown Fort Collins and evaluate the performance of the three sites in terms of pollutant removal, reduction of runoff volume, and the consistency of performance of each site over time. The three test sites for this study were constructed as part of the City of Fort Collins LID stormwater initiative.

Fort Collins, Colorado is located about 60 miles north of the Denver area and about 5 miles west of the I-25 corridor (Figure 1). The three PPS sites in this study are located in north Fort Collins in the old town area. The Mitchell block sites, Mountain and Walnut, border Old Town Square on Mountain Avenue and Walnut Avenue, respectively. CTL Thompson is located approximately a half mile to the north of the Mitchell Block sites and just east of old town at 351 Linden Street. Figure 1 shows the locations of these three sites plus two outfall sampling sites (Howes and Udall) and their drainage areas. Since it was physically impossible to collect runoff samples from the areas draining to the PPS sites, water quality data gathered at the two outfall sites was judged to be essentially the same quality as the run-on water to the PPS sites. The

Mitchell Block PPS sites were constructed as part of the Bohemian Building Development project; a collaboration between the City and project developers. The PPS were installed with monitoring capabilities to track their performance. The CTL Thompson PC site is a converted gravel parking lot that serves several businesses; including CTL Thompson, Inc. This site was also constructed with monitoring capabilities in place. CSU was contracted by the City of Fort Collins to conduct research and monitoring to track and analyze the performance of these three sites. Data from the International Stormwater BMP Database (the Database) are used to provide comparison and context for the data obtained from this study.

1.2.1 Site Descriptions

The Mountain pavers are on the north side of Mountain Avenue in the parking area bordering the westbound traffic lanes (Figure 2). The total paver surface area at Mountain is approximately 3,265 square feet, calculated from aerial photos and plan drawings (Appendix A). The two westbound traffic lanes of Mountain Avenue total about 5,300 square feet and drain onto the pavers (Figure 3). Drainage flows in a northeasterly direction across Mountain Avenue onto the pavers. The slope varies between 3% and 4% across both the road and the paver surface, as determined from the plan drawings (Appendix A). The surface of the pavers is separated into two distinct sections by a handicap ramp for the sidewalk (Figure 4).

The Walnut pavers are located on the southwest side of Walnut Avenue in the parking area bordering the southeast bound traffic lane (Figure 5). The total paver area is 3,580 square feet and the contributing area from the southeast bound traffic lane is 3,750 square feet. Runoff drains from the crown of the road separating the two lanes of traffic and onto the pavers toward the south.

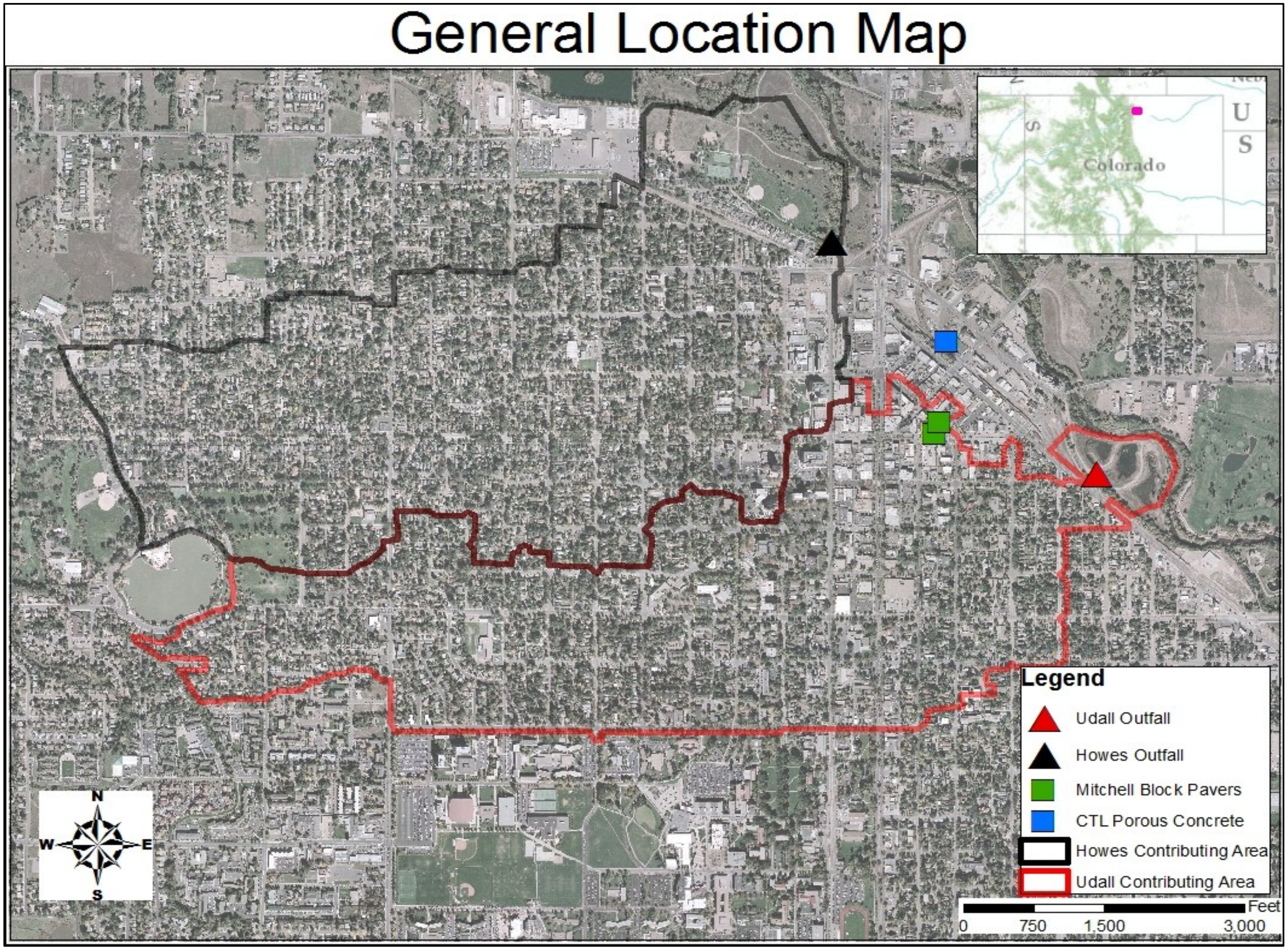

Figure 1: General Location Map of Fort Collins

Aqua-Bric Type 1 pavers, manufactured by Advanced Pavement Technology, are used at both Mitchell Block sites. The pavers themselves are impervious, but are installed with one-eighth to three-eighth inch voids between them. The voids are filled with chipped gravel which allows runoff to infiltrate between the blocks. The layers below the pavers consist of a series of bedding, base and sub-base graded-pervious-aggregate materials. Both Mountain and Walnut employ a "no-infiltration" design, by using an impervious membrane lining below the sub-base course and a perforated under-drain pipe which carries the exfiltrate from the pavers into the storm sewer. The runoff passes through a monitoring area where flow data is recorded and water quality samples are collected. The paver designs for each site are discussed in detail below.

Mountain utilizes a design specified by Advanced Pavement Technologies referred to as the Bio-Aquifer Storm System (BASS). This system specifies 3 layers of aggregate below the paver surface including: a 2-inch (No. 8) bedding layer, a 4-inch (No. 57) base layer and a 12-inch (No. 2) sub-base layer. Below the sub-base layer is a 30-mil PVC impervious membrane. The impervious liner carries a slope approximately parallel to the paver surface, draining water via a HDPE schedule 40 6-inch perforated under-drain pipe, which runs parallel to Mountain Avenue under the north side of the pavers (Figure 6). Runoff flows out of the under-drain into a monitoring box located inside of a storm drain inlet at the north east end of the pavers section. The monitoring box details are discussed in Section 2.1.1. The runoff is discharged from the monitoring box into the storm sewer.

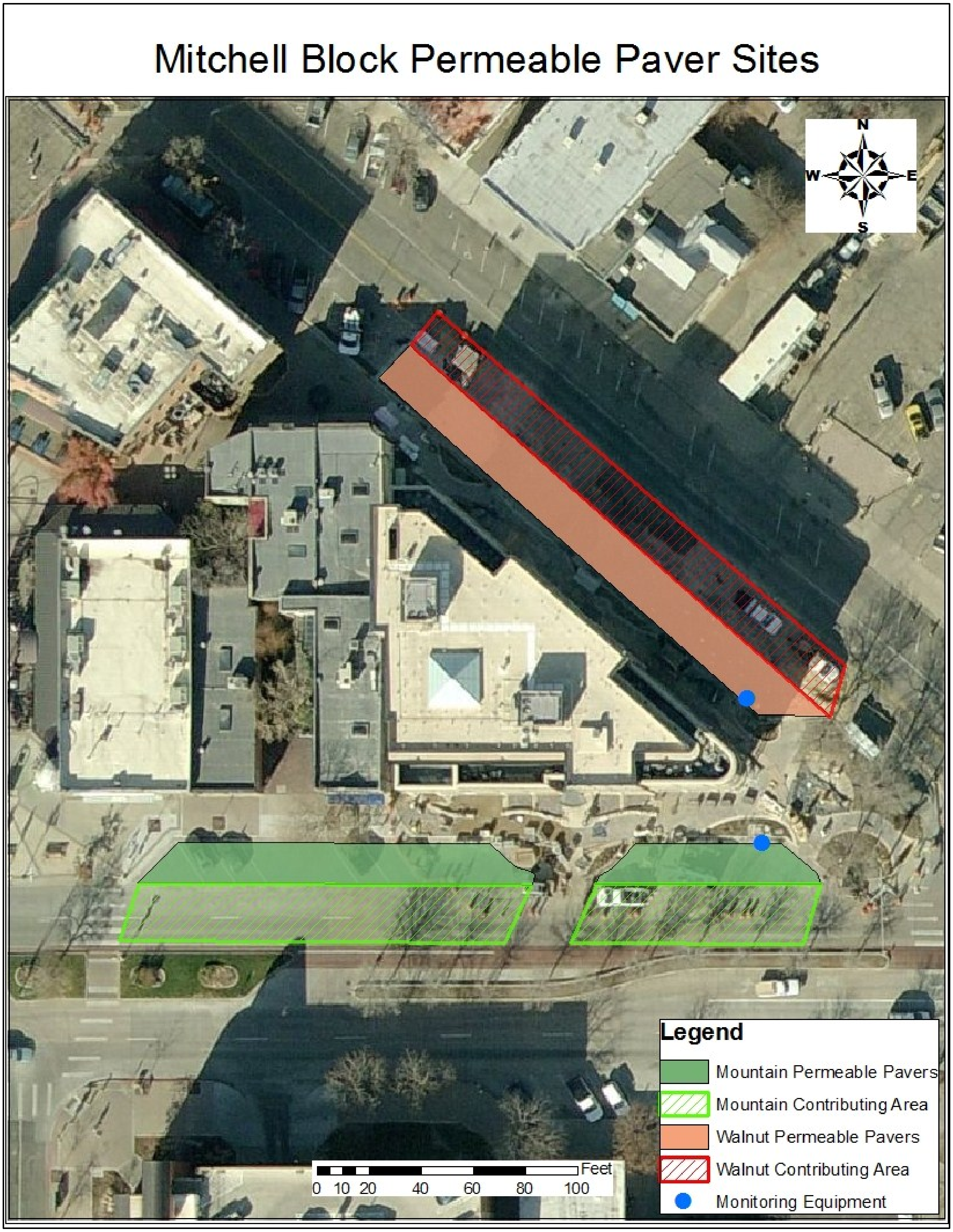

Figure 2: Mitchell block permeable paver sites and contributing areas (Fort Collins, CO)

Figure 3: Looking west at the Mountain PPS and the westbound Mountain Avenue traffic lanes

Figure 4: Looking west from the middle section of the Mountain PPS showing the separation of the two sections

Figure 5: Looking northwest at the Walnut PPS bordering Walnut Avenue

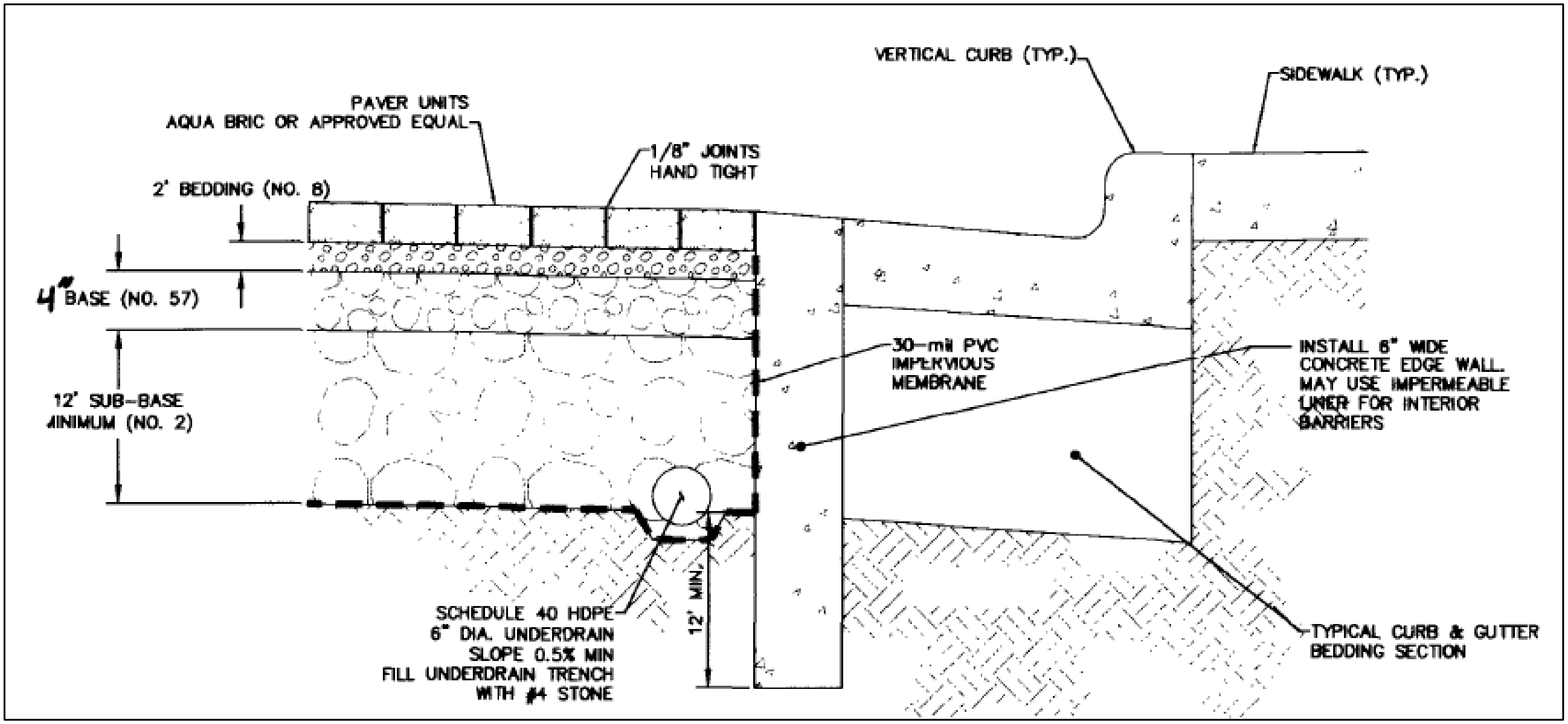

Figure 6: Advanced Pavement cross section design for the Mountain Avenue Permeable Pavers

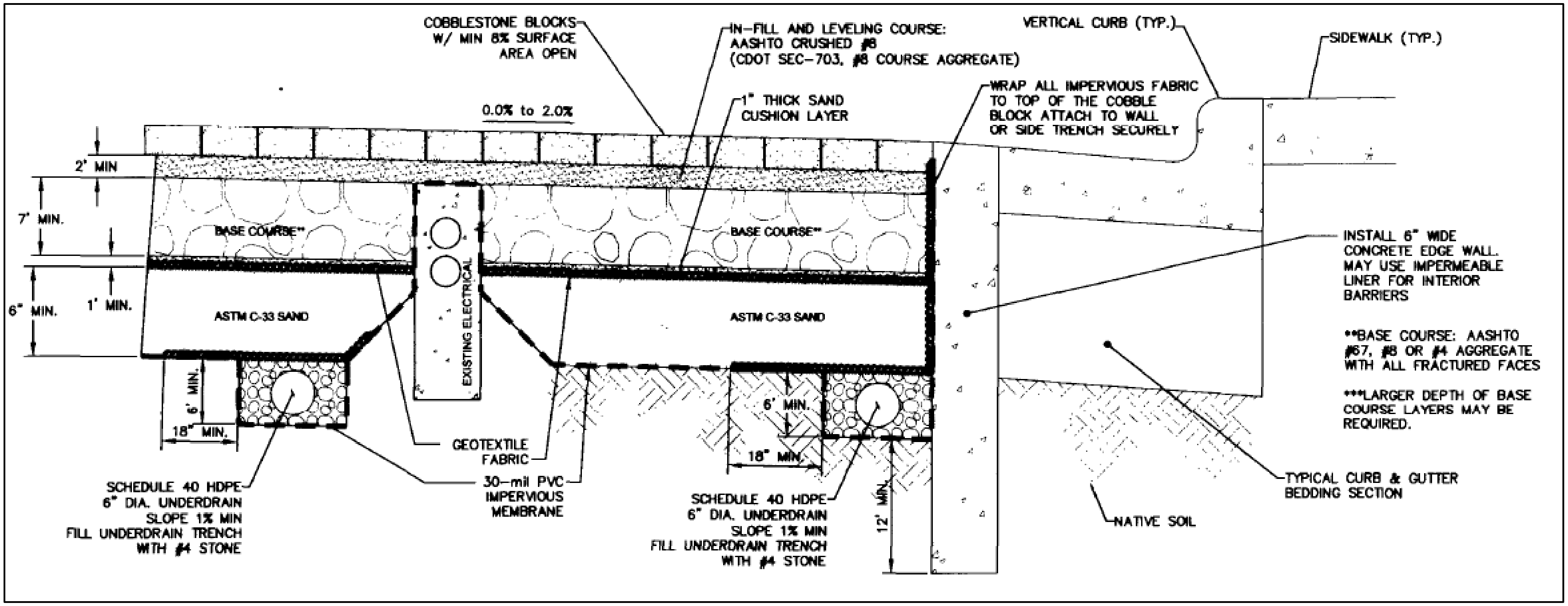

Figure 7: UDFCD cross section design for the Walnut Street Permeable Pavers

The Walnut pavers use a design from the Urban Drainage and Flood Control District's (UDFCD) criteria manual (UDFCD 2010). This design specifies four separate layers of aggregate: a 2-inch (AASTHO crushed #8) bedding layer, a 7-inch (AASHTO #67, #6 or #4) base course, a 1-inch (ASTM 33) sand cushion layer on top of a geotextile fabric and a 6-inch (ASTM 33) sand sub-base layer. As with the design at Mountain, a 30-mil PVC impervious membrane is used below the sub-base layer. Due to existing underground electrical wires running parallel to Walnut, two HDPE schedule 40 6-inch perforated under-drain pipes are used to collect exfiltrate, one on each side of the wires (Figure 7). The under-drain pipes connect to a perpendicular under-drain pipe at the southeast end of the pavers, which carries the exfiltrate through a monitoring box and into the storm sewer. The monitoring box is discussed in Section 2.1.1.

The CTL Thompson site was constructed using a PC mixture consisting of uniform graded aggregate and Type II or Type IV Portland cement with 4% to 8% air entrainment. Specifications call for a minimum compressive strength of 2,500 psi at 28 days after construction. The total parking lot area is about 13,850 square feet (Figure 8). The pavement layer is approximately 7 inches thick and sits on top of 6 inches of uniform graded coarse aggregate (3/8-inch to ¾-inch stone). This system uses a full infiltration design. Two 5 foot by 10 foot sections are lined with 30 mil impervious liner to collect exfiltrate for water quality testing (Figure 9) Each impervious section is drained by a 6 inch perforated under-drain into water quality sumps referred to as CTL parking lot (CTL PL) and CTL driveway (CTL DW) (Figure 9). The rest of the parking lot drainage infiltrates into the native soil and recharges the groundwater. Two pressure transducers were deployed at each site, one to monitor water levels within the pavement and one to monitor groundwater levels below the pavement (Figure 8). In addition, one of the groundwater monitors measures specific conductivity. The lot is designed to capture and infiltrate the 100-year storm. The PC lot is bordered by a 6-inch curb, except at the lowest point of the lot (northwest section of the unloading zone) where there is curb cut to drain any excess surface runoff should the permeable pavement matrix fill completely.

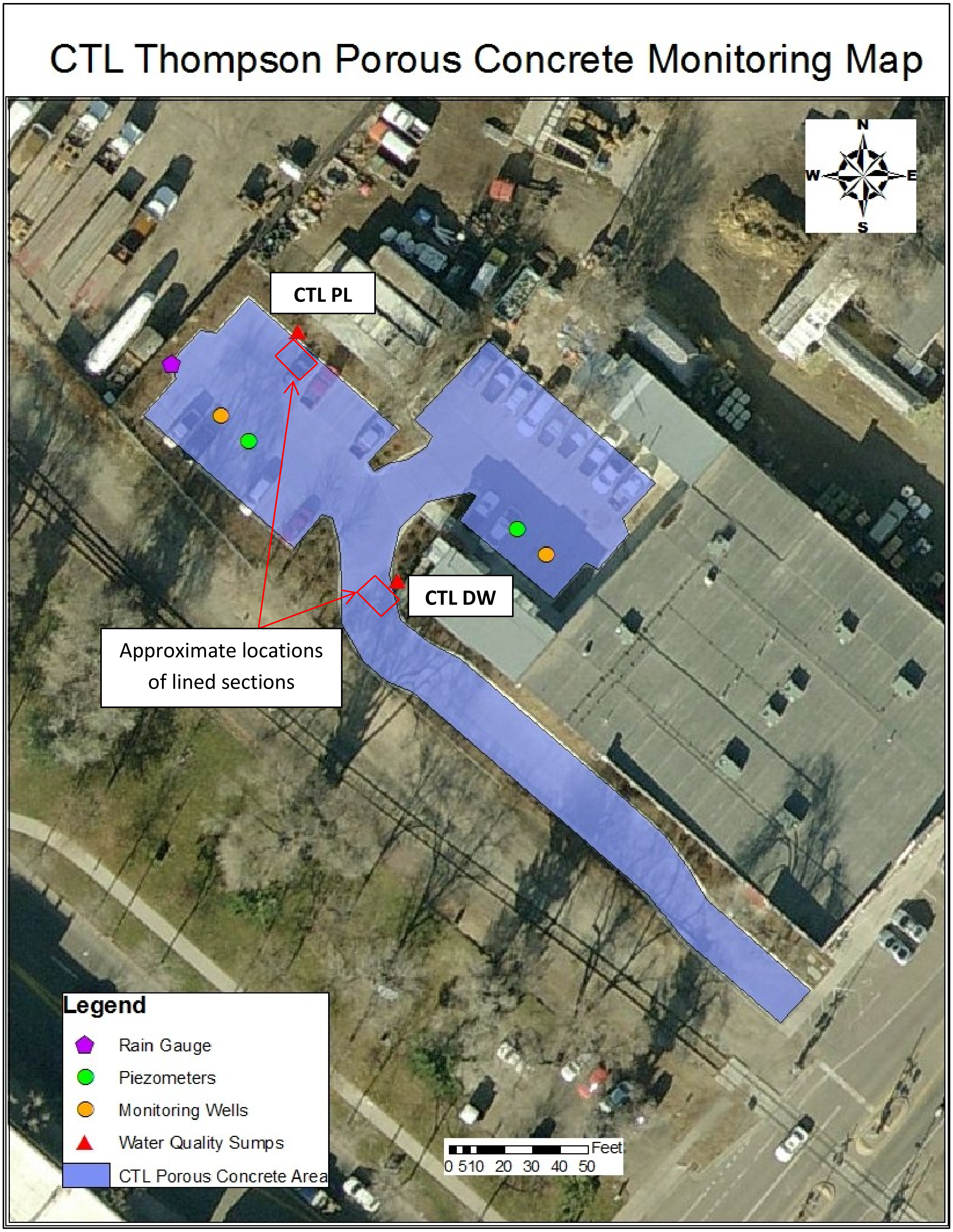

Figure 8: CTL Thompson Porous Concrete Site and Monitoring Equipment Location (Fort Collins, CO)

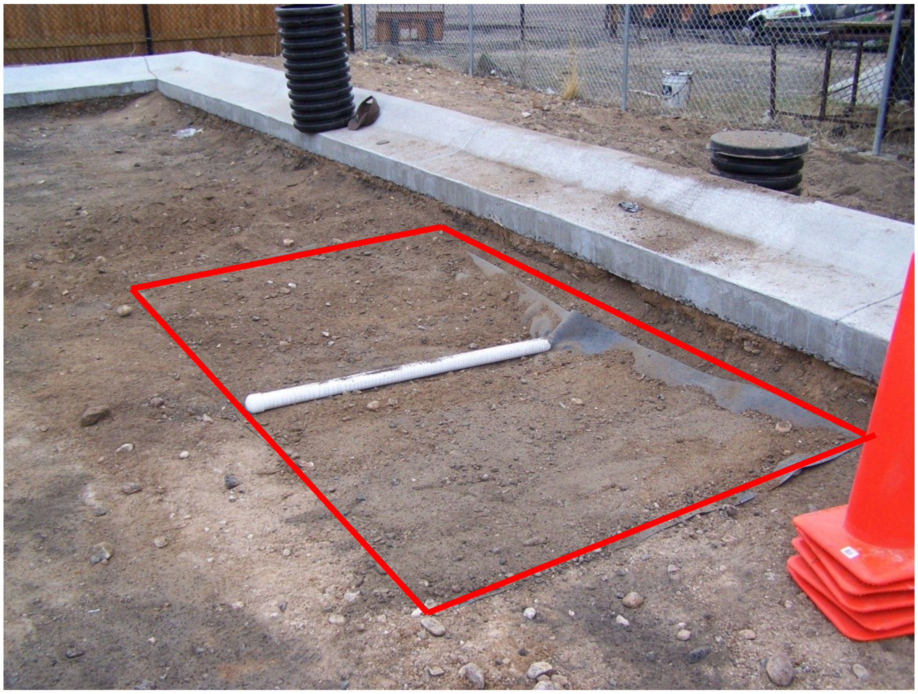

Figure 9: Impervious liner and under-drain being installed at CTL PL

Figure 10: Installation of water quality sump with under-drain connection at CTL PL

2.0 METHODS

This section details methodologies used for site monitoring, stormwater sampling, data quality control and quality assurance, data analysis and data presentation.

2.1 Stormwater Monitoring and Sampling

Monitoring methodologies differed between the CTL sample sites and the Mitchell Block sample sites and will therefore be discussed separately. Whenever possible, sampling and monitoring methods were adopted from *Urban Stormwater BMP Performance Monitoring* (2009), from the International Stormwater BMP Database. In many cases, site specific limitations required improvisation of monitoring techniques. The specifics of which are discussed in detail in the following sections.

2.1.1 Mitchell Block

The monitoring setup at Mountain and Walnut were identical except for minor design details. Influent to the system was not monitored or sampled. The entire sub-base of the paver system is lined with an impermeable membrane, as discussed in Section 1.2.1. Each system captures the exfiltrate from the PPS via the under-drain that leads to the storm drain. The end of the 6-inch under-drain is equipped with a gasket seal and 3/4 inch tubing which transfers the water to a sample collection box (SCB) which is 8.5 inches tall, 12 inches long and 8.5 inches wide. The SCB acts as a flow control device to quantify the flow rate of the exfiltrate leaving the PPS. The SCB has a ¼ inch orifice drilled in the bottom corner to allow complete drainage and a 2-inch high, 90 degree weir cut into the top to measure larger discharge (Figure 11 and Figure 12).

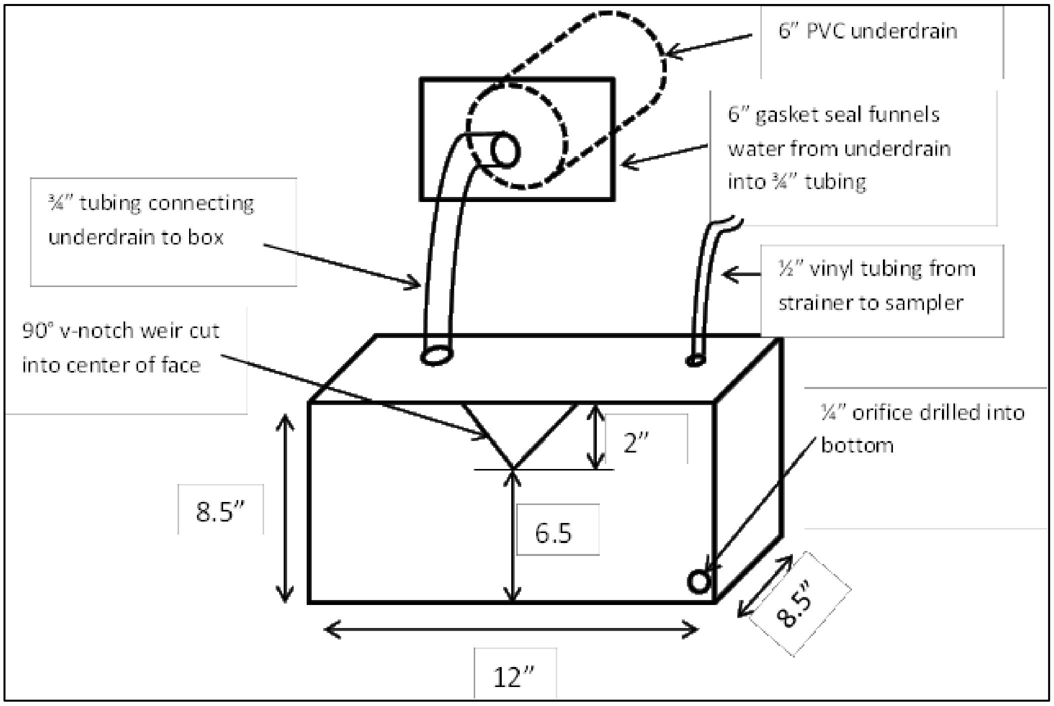

Figure 11: Flow monitoring setup at the Mitchell Block sites

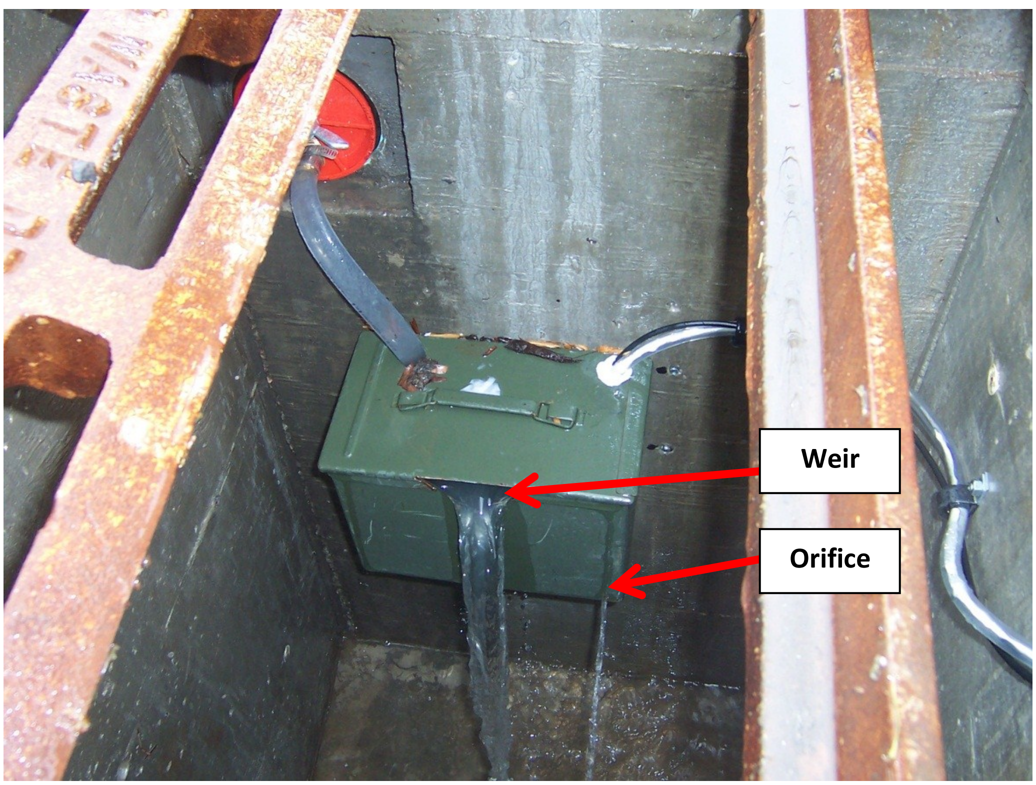

Figure 12: Flow monitoring box with both the weir and orifice discharging runoff

A vented pressure transducer (PT) rests on the bottom of the box which continuously measures pressure. The vented PT accounts for changes in barometric pressure and is capable of measuring pressure from submergence without manual barometric pressure compensation.

Data at each site was collected and stored by a programmable Campbell Scientific CR200X data logger. The program was divided into three subroutines:

1. The first subroutine initiates every minute to calculate a depth and check if the depth is greater than 0.1 inches, if so then the second subroutine is run to check if the depth is greater than 1.49 inches and calculate a flow, if so then the third subroutine is run to initiate sample collection.
2. The second subroutine calculates the flow using hydraulic equations for the SCB orifice (Equation 1) and weir (Equation 2):

$$\boldsymbol{Q_{orifice} = C_{d_{orifice}} A_{orifice} \sqrt{\frac{2gh}{12}}} \qquad \textbf{Equation 1}$$

Where: $C_{d_{orifice}}$ = Orifice discharge coefficient

$A_{orifice}$= Orifice area (ft^2)

g = Gravitational acceleration constant (ft/s^2)

h = Water depth (in)

The system was calibrated and $C_{d_{orifice}}$ was determined to be 0.62 for both Mountain and Walnut.

$$\boldsymbol{Q_{weir} = C_{d_{weir}} \sqrt{2g} \left(\frac{h-6.5}{12}\right)^{2.5}} \qquad \textbf{Equation 2}$$

Where: $C_{d_{weir}}$ = Weir discharge coefficient.

For Mountain the weir discharge coefficient was determined to be 0.54 during calibration and 0.595 for Walnut. The program only begins calculating weir flow when the depth reading exceeds 6.5 inches. The flow is then converted to an interval volume since the last scan and added to the cumulative volume since the last sample was taken.

3. The third subroutine is the sampling procedure. If the user specified cumulative volume is reached, a signal is sent to the automated ISCO sampler (see below) to collect a sample and the cumulative volume is reset to zero. It then moves to the next sample until all 24 samples are collected, or until the program is reset (Appendix B contains the code for the programs described above for both sites).

An automated ISCO 3700 sampler was used to collect flow weighted samples based on user specified volume increments passing through the SCB as specified above in the program routine. A strainer sits in the bottom of the SCB next to the PT, and is connected to the ISCO by a ½ inch vinyl tube. Samples are pumped through the tube by a peristaltic pump on the ISCO which is activated by an electrical pulse sent from the data logger.

The data logger and the sampler were powered by a deep cycle marine 12V battery, which was recharged periodically throughout the sampling season. The sampler, data logger and battery were housed in a locking above ground steel case to provide easy access and security for the monitoring equipment (Figure 5).

All sample bottles, composite bottles and beakers were cleaned with a 25% phosphoric acid-bath and rinsed with distilled water to avoid contamination of samples. During storm events ice was used to keep the collected samples at 4 degrees centigrade. This is done to preserve samples by preventing chemical degradation of constituents (Geosyntec Consultants and Wright Water Engineers, Inc., 2009). The ISCOs are capable of capturing up to 24, 1000 mL samples.

Data were downloaded from the data logger immediately following a storm event and inspected to ensure reasonable flow and volume data were obtained and the samples collected represented at least approximately 70% of the effluent hydrograph (Geosyntec Consultants and Wright Water Engineers, Inc., 2009). In some cases, events with less than 70% of the hydrograph captured were submitted for lab analysis. In such cases, a limited suite of parameters was requested from the lab, and the results were flagged for review. The limited sampling suite consisted of nutrient and solids, as these were identified as the most important parameters to this study. Spreadsheet templates were filled in with flow and sample data immediately following storms to execute the quality assurance and quality control process before sample submittal to the lab.

After confirmation that the flow and volume data were reasonable, the samples were collected and transported to CSU's Atmospheric Simulation Lab located on the Foothills Campus. At the lab the samples were composited and prepared for submittal to a professional water quality lab for water quality analysis. Equal volumes (aliquots) from each flow weighted sample were composited into one sample representing the event mean concentration (EMC) for the storm event. Equal aliquots can be used from each sample to obtain an EMC because flow weighted sampling was used which ensures that each sample represents equal volumes from the runoff hydrograph. This procedure was completed for Mountain and Walnut separately.

2.1.2 CTL PC Parking Lot

The PC site at CTL utilizes a full infiltration design, where the water is passed into the underlying native soil and is allowed to infiltrate into the groundwater. For monitoring purposes, two sections were lined with an impermeable material and drained to underground sumps as discussed in Section 1.2.1. From the sumps, the exfiltrate from the system was pumped out after storm events and submitted for lab analysis.

The site is also equipped with a tipping bucket rain gauge that records precipitation in 0.01 inch increments. The data obtained from this gauge was used for hydrological analysis at both CTL and the Mitchell Block sites. Data from the CTL sites was downloaded after storm events or every other week (whichever occurred first). The sumps were cleaned with a hose and pumped out completely between storm events to ensure that runoff was collected for individual events. Samples were assumed to be a representative EMC at each sampling site. Before sample collection, the sump was stirred with a stir stick to ensure that settling within the sump didn't bias water quality results.

2.1.3 Laboratory Sample Submission

Two different water quality labs were used over the course of this study, each with unique sample preservation guidelines. The Fort Collins Pollution Control Lab (PCL) was used between 2009 and the beginning of the 2011 sampling season. The PCL required that the samples be split into 8 bottles: one preserved with nitric acid to a pH less than 2, one preserved with sulfuric acid to a pH less than 2, one preserved with phosphoric acid to a pH less than 2, and 5 without any acid preservation (Table 1). For the remainder of the 2011 sampling season, CSU Soil and Water Testing Lab was used. This lab did not require any preservation or filtration if the samples were delivered within 24 hours of collection. For storms where this could not be achieved, they required the samples to be split into three bottles: one preserved to a pH less than 2 with sulfuric acid, one preserved to a pH less than 2 with nitric acid and one not preserved.

2.3 E. coli Grab Sample Analysis

Grab samples were collected during storm events and tested for *E. coli*. Samples were collected from the SCB discharge at the Mitchell Block sites and from the water quality sumps at the CTL sample sites, using 250 mL sample bottles. *E. coli* tests were run within two hours of sample collection using Coliscan Easygel kits. Samples were diluted with distilled water (5 mL distilled water to 1 mL of sample) during the first month of monitoring, but this was found to yield low resolution results because of low *E. coli* concentrations. Using 1 mL of sample with no dilution yielded optimal results. The sample was mixed with coliscan gel and shaken vigorously and then poured into a petri dish. The dish was left at room temperature for 48 hours to incubate. The *E. coli* growths in the petri dish were counted. The number of *E. coli* per 100 mL of sample was calculated based on the dilution ratio.

2.3 Statistical Analysis

The runoff volume reduction analysis in this project used median and average values to describe data. More in depth statistical analyses were applied to the water quality datasets. The statistical analysis began with data distribution determination. Based on the distribution (normal or lognormal), data values reported as below the detection limit (censored) were resolved using parametric statistical techniques. Actual data analysis involved a combination of parametric and non-parametric techniques, mainly location parameters. The sections below detail the statistical analyses used for data processing and analysis.

Table 1, Fort Collins Pollution Control Lab Sample Preparation Details

Constituents	Required Volume (mL)	Method	Bottle Type	Preservation	Holding Time	Notes
Total Recoverable Metals + Total Hardness	1000		1 L Poly (Acid washed in 1+1 HNO3)	Acidify with HNO_3, Cool	6 months	5mL of 35% HNO3 per 500 mL of sample for preservation
Dissolved Metals + Dissolved Minerals	500		500 mL Poly (Acid washed in 1+1 HNO3)	Cool	6 months	PCL will filter and acidify
Alkalinity	250		250 mL Poly	Cool	14 day	
TSS	250		250 mL Poly	Cool	7 days	
Nitrate + Nitrite Sulfate, Chloride	125		125 mL Poly	Cool	48 hours	Requested by PCL that these items be grouped
Total Ammonia	125		500 mL Poly	Acidify with H_2SO_4, Cool	28 days	*Take Nitrate + Nitrite test from preserved bottle if not analyzed immediately, use 1 L bottle* Approx. 2mL of 25% H2SO4 required per 500 mL of sample for preservation
TKN	250					
Total Phosphorus	125					
COD	250		250 mL Poly	Cool	28 days	
TOC	250		250 mL Poly	Acidify with H_3PO_4	28 days	Approx. 1mL of concentrated H3PO4 per 100 mL of sample for preservation
Total	**3.125**	**L**	**(8 Total)**			

*All ISCO bottles should be acid washed in 1+1 solution of reagent grade HNO3

** All bottles sent from FTC PCL are already washed

*** Preserve carefully…use pH meter to assure that final pH is between 1 and 2.

Statistical analysis began with determining the appropriate statistical method for the given dataset. With over thirty water quality parameters at six different sites, this initial step required the advent of a logical screening process to sort the datasets for each parameter at each site. Based on this process, appropriate statistical analyses could be applied. Below is a flow chart that represents the process applied to each dataset (Figure 13). The chart is color coded as follows: Input is blue, decision steps are tan, censored data replacement methods are pink and analysis techniques are green. The arrows are color coded as follows: An answer of "yes" is indicated with a black arrow and an answer of "no" is indicated with a red arrow.

2.3.1 Distribution of Datasets

The thresholds for minimum data points (n>4) and maximum censored data (40%) are based on recommended values in the literature for various statistical analyses and the context of application (Helsel and Hirsch 2002; Sibert 2006). This allowed the application of certain statistical tests without eliminating the majority of the data sets based on limited data.

After the initial screening, the remaining datasets were input into a MATLAB program that tested both log-transformed and untransformed data for normal distribution using the Lilliefors test (Lilliefors 1969). The datasets were also plotted on individual normal probability plots. These two methods (Lilliefors and graphical observation) were used in conjunction with one another to determine if the data followed either a normal or a log-normal distribution. If either a lognormal or normal distribution could reasonably be assumed for the dataset, then parametric methods could be utilized to replace censored data (Section 2.3.2). Non-parametric statistics were used for analysis to provide a common metric between all parameters and sample sites.

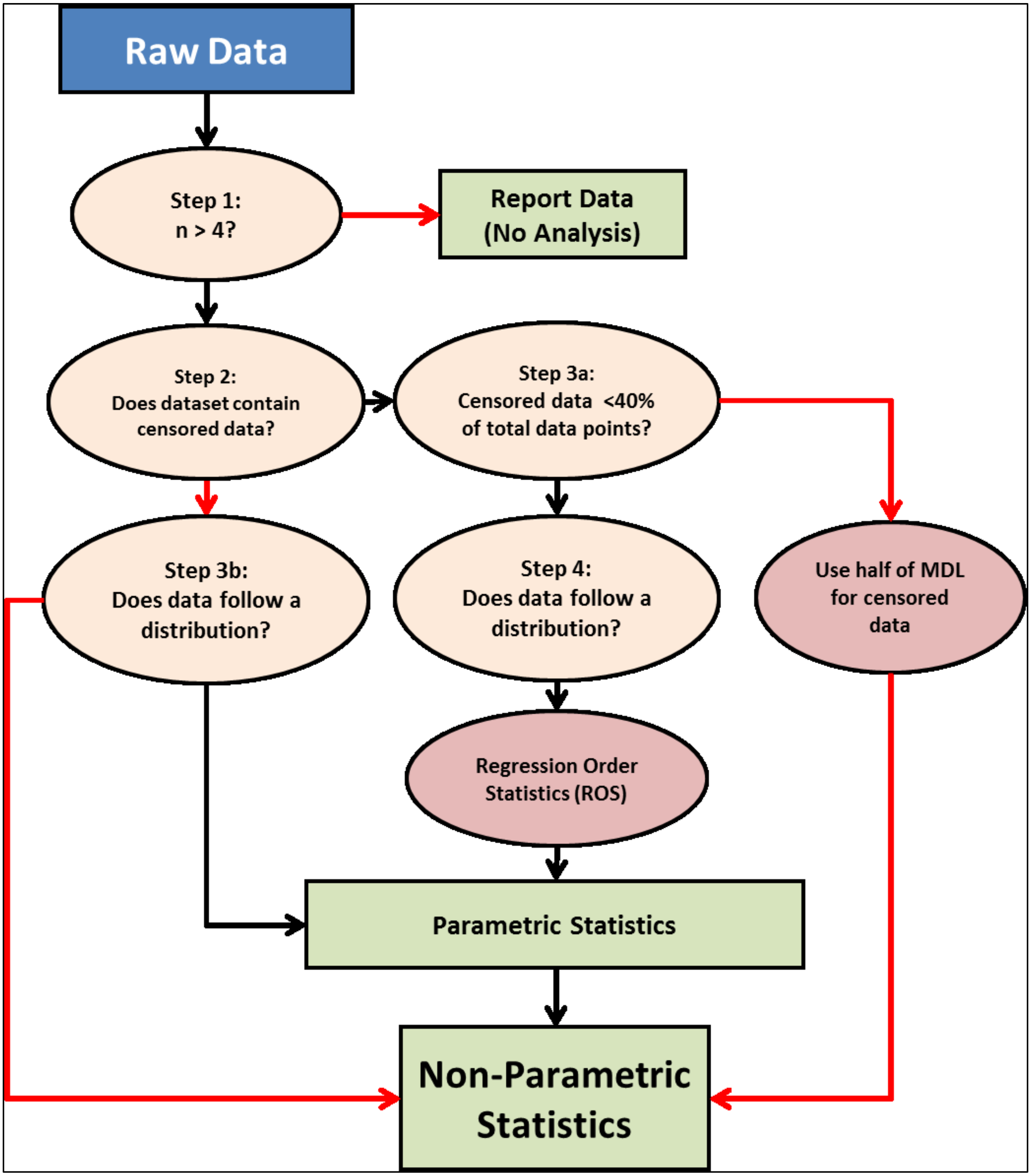

Figure 13: Flow chart for water quality analysis decision process

2.3.2 Censored Data

It is common for water quality datasets to contain data points that are below a minimum detection limit (referred to herein as censored data). These limits vary between different parameters and different laboratories. Datasets with censored data points result in additional steps in the data analysis process. There are several different statistical methods available for processing censored data, but should only be applied within the context for which they are designed. For example, these methods generally involve the assumption of an underlying distribution and are not applicable if one does not exist.

A commonly applied method is substitution, which is non-parametric and involves substituting a single value (commonly the detection limit, half the detection limit or zero) in for all non-detect data points. Substitution techniques obscure actual patterns in the data because the reporting limit depends on the laboratory method and calibration (Helsel, 2005; Sibert 2006). Therefore, substituting a value in based on the detection limit reflects laboratory conditions rather than the data itself. The three common alternative methods are; maximum likelihood estimation (MLE), Kaplan-Meir (K-M) method, and regression on order statistics (ROS) methods (Helsel, 2005).

The MLE method requires the assumption of a distribution to the data and is most suitable for samples with more than 50 data points (Helsel, 2005). The datasets addressed in this paper are significantly smaller than 50 data points; therefore this method is not addressed in detail here. The K-M method is most often applied to datasets with multiple reporting limits (Helsel, 2005). If the data contain only one reporting limit, then the K-M method is equivalent to substitution and should be avoided because of the previously mentioned issues associated with substitution methods. The K-M method is however advantageous in the fact that it is non-parametric in nature and doesn't require a distribution be assumed for the dataset. The ROS method is parametric and is applicable to smaller datasets. Kayhanian et al. (2001) recommends application of this method to datasets with at least 10 detected points when censored data comprises less than 40% of the dataset. As the censored percentage increases, so to should the number of detected values with which the method is applied to.

The most appropriate method found for this analysis was the ROS method, which uses an ordinary least squares (OLS) regression of x_i and q_i pairs, where x is the dataset and q is the corresponding quantile. The analysis is performed on the detected data points and the censored data points are extrapolated based on the OLS regression equation obtained. Often this method results in calculated data points that are greater than the detection limit or even some of the detected values (Kayhanian et al. 2001).

After the above methodology is performed on log-transformed data, there are several different methods for obtaining parametric location parameters from the resulting dataset. Many statistical software packages use the fully parametric ROS method on log transformed data, in which the mean and standard deviation are calculated in log units and then transformed back to original units using Equation 3 and Equation 4 below:

$$\boldsymbol{\mu_{org} = 10^{\left(\mu_{log} + \frac{\sigma_{log}^2}{2}\right)}} \qquad \textbf{Equation 3}$$

$$\boldsymbol{\sigma_{org}^2 = \mu_{log}^2 \left(10^{\left(\sigma_{log}^2\right)} - 1\right)} \qquad \textbf{Equation 4}$$

Where: μ = Mean

σ = Standard deviation

Kayhanian et al. (2001) showed that this method suffers from a significant amount of transformation bias and should be avoided. Instead the Robust ROS Method (or Helsel's Robust Method) should be used, in which the extrapolated data points are transformed back to original units and the resulting full dataset in original units is used to calculate the mean and standard deviation (Helsel, 2005; Sibert 2006). This was the method selected for analyzing censored data herein.

2.3.3 Graphical Methods

Graphical methods are an effective way to present large datasets in a concise meaningful manner. This study used several different plotting methods including, boxplots, scatter plots and bar charts. All of these plotting methods are non-parametric in nature.

Boxplots display several non-parametric measurements of the dataset on one plot. This study used notched boxplots (Figure 14). The boxplot shows, the first (Q1), second (Q2) and third (Q3) quartiles, represented by the top, middle and bottom of the box, respectively. The notches in the boxplot represent the 95% confidence interval for the dataset. The difference between Q1 and Q3 is known as the interquartile range (IQR). The whiskers that extend out from the box represent 1.5 IQRs from the end of the box (Q1 and Q3). Outliers are points outside of that range and are displayed as a point. Boxplots can be used to determine if two datasets have statistically significant differences by comparing the upper and lower 95% confidence intervals for two plots. If the upper and lower 95% confidence intervals of two boxplots do not overlap, then the two datasets can be considered statistically different. Boxplots were used for all water quality datasets with more than four points and less than 40% non-detect values.

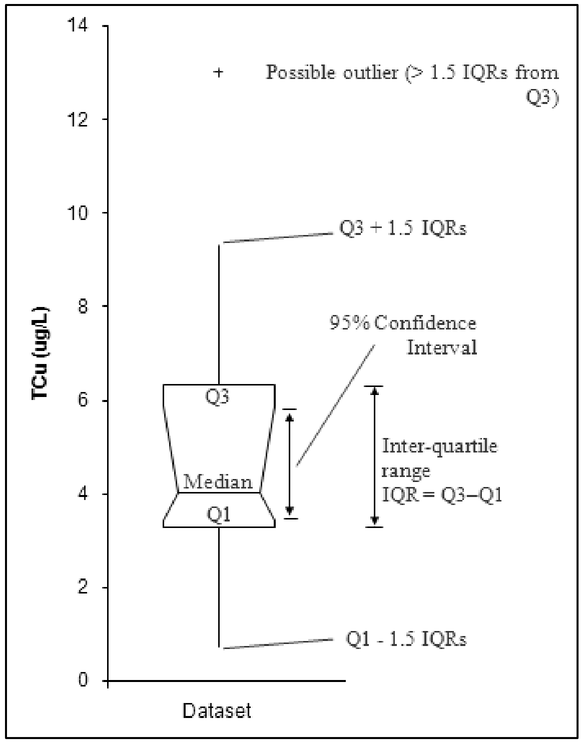

Figure 14: Boxplot example and description

2.4 Determination of Runoff Volume Reduction

2.4.1 Rainfall Calculations

Precipitation data were analyzed in order to understand the hydrologic parameters affecting the amount of runoff volume reduction. Precipitation depths were taken from the CTL Thompson rain gauge for most events, but the Lincoln Rain Gauge was used for several events in which the CTL gauge was being repaired. A six-hour inter-event time was used to separate individual storm events. Every 0.01 inches of precipitation was recorded at the CTL site. Fifteen-minute rainfall intensity values were calculated using this data by finding the fifteen minute span with the maximum precipitation during the event. Average intensities were calculated by dividing the precipitation depth by the storm duration. The Antecedent dry time was calculated by subtracting the end time and date of the most recent storm (greater than 0.1 inch) from the start time and date of the storm event being evaluated.

2.4.2 Field Capacity Determination

Field capacity was estimated at each site to provide context to the runoff reduction results. Field capacity can be defined as the amount of water retained within soil media or aggregate void space due to the balance of capillary and gravitational forces (Brouwer *et al.* 1985). Literature values were obtained for both sand and gravel and used to estimate field capacities for both materials (Table 2).

Table 2, Porosity Values and Field Capacity for Sand and Gravel

	Total Porosity	Effective Porosity	Field Capacity Values (in/in)
Gravel (coarse)	0.28	0.21	0.07
Sand (coarse)	0.39	0.3	0.09

Note: Values obtained from the Argonne National Laboratory, Environmental Science Division

The site field capacity was estimated by multiplying the literature field capacity value by the depth of the corresponding layer (gravel or sand), converting it to feet and multiplying by the paver area to give a volume in cubic feet. This value was then normalized to the watershed by dividing by the total watershed area and converting back to inches. The normalized field capacity is useful for comparison with precipitation and normalized runoff reduction values. The available field capacity values at the Mitchell Block sites are shown in Table 3.

Table 3, Mountain and Walnut Permeable Paver Dimensions and Calculated Field Capacity

	Total Watershed Area	Depth of Gravel Baselayer	Depth of Sand Baselayer	Estimated Field Capacity in Baselayer	Estimated Normalized Field Capacity in Baselayer
Site	(ft^2)	(in)	(in)	(ft^3)	(in)
Mountain	8565	18	0	342.8	0.48
Walnut	7330	9	7	375.9	0.62

2.4.3 Calculation of Runoff Reduction

Flow data at the Mitchell Block sites were collected for the effluent leaving the pavers as discussed in Section 2.1.1. Influent storm runoff volume estimates were calculated by applying various assumptions. The main assumption was that the surface infiltration rates were high enough to infiltrate all runoff through the pavers. In addition, we assumed all runoff that infiltrated through the pavers was; drained through the under-drain system, evaporated back through the surface or stored in the bedding, base and sub-base layers. Applying these assumptions allowed quantification of inflow to the system using the surface area of the pavers, area of the contributing watershed to the pavers, precipitation depth and surface depression storage. Using this as a surrogate for inflow volume allowed for an estimate of volume reduction using the simple system mass balance described below

$$\boldsymbol{RR = I - O} \quad \textbf{Equation 5}$$

Where: RR = Runoff reduction (ft^3)

I = Inflow (ft^3)

O = Outflow (ft^3)

Outflow was recorded from the monitoring efforts previously described. System inflow was estimated using the following equation

$$\boldsymbol{I = \left[A_{pavers} * \left(P - DS_{pavers}\right)\right] + \left[A_{runon} * \left(P - DS_{runon}\right)\right]} \qquad \textbf{Equation 6}$$

Where: A = Area (ft^2)

P = Precipitation (ft)

DS = Depression storage (ft)

ArcMap was used to estimate the surface area of the permeable pavers and contributing watersheds at each site. Figure 2 shows these areas for both Mitchell Block sites. The most recent aerial photo available was from 2009 during the paver installation. Outlining the pavers was accomplished from examination of the aerial photo, plan drawings (Appendix A) and field observations. The contributing areas were determined based on field observations of drainage patterns during precipitation events and topographic lines provided on site plan drawings from a survey conducted by Northern Engineering (Appendix A). Depression storage values were estimated based on field observations and literature values for asphalt (0.1 in) at Walnut and concrete (0.05 in.) at Mountain (Gironas 2009).

2.4.4 Validation of Assumptions

Equation 5 and Equation 6 require that the key assumption mentioned in the above section is valid. It states that all effective run-on and runoff at the PPS sites infiltrates the surface of the pavers. To validate this assumption, a SWWM 5 model was developed and applied to both Mitchell Block sites.

The SWMM 5 model was developed to estimate the minimum precipitation intensity that caused surface runoff from both Mountain and Walnut. The important input parameters were the paver areas and slopes, contributing watershed areas and slopes, width of catchments, depression storage and infiltration rates. The slopes and catchment widths were determined from site plan drawings (Appendix A). The area and depression storage values used in the inflow volume calculations were applied to the model. The infiltration rate was the controlling parameter for the model and was determined based on field data obtained from the infiltration rate portion of this study, discussed in Section 3.1 of this document. Two model simulations were run. The first used the median and the second used the average infiltration rates determined from the field tests at each site. The Horton infiltration model was used, but the initial and final infiltration rates were set equal to each other such that the infiltration rate at each catchment remained constant throughout the simulations. Conservative infiltration rates representing saturated conditions were applied in the model to justify the constant non-decay infiltration input. A complete list of model input parameters for all scenarios can be found in Table 4.

The Mountain pavers were separated into two different catchments, each with its own run-on area (Figure 15). In total there were six catchments, three paver areas and three run-on areas. The three run-on areas were all 100% impervious as they represented the street areas on Mountain and Walnut. The three catchments representing the paver areas were set at 0% impervious, representing the pavers themselves. The run-on areas were setup to drain onto their respective paver area and the paver areas were then set to drain any surface runoff to adjacent nodes, which represent surface runoff into the curb gutter. The water that infiltrates the paver catchments in effect disappears from the model and is treated as a loss to the system.

The precipitation intensity was steadily increased throughout the simulation from 0 in/hr up to 3 in/hr over a period of 20 hours. This type of simulation is referred to as a ramp model. The results were examined to determine at approximately what intensity the catchments began producing surface runoff. The model was then run again minimizing the incremental increase in precipitation intensity near the point where surface runoff was produced for each catchment, thus increasing the resolution of the results. The results are presented in Section 4.2 of this document.

Table 4, SWMM Model Parameters and Inputs

Parameter	Units	Mountain 1	Mountain 1 Runon	Mountain 2	Mountain 2 Runon	Walnut	Walnut Runon
Area	(ft3)	1632.5	2650	1632.5	2650	3580	3750
Width	(ft)	233	379	233	379	511	536
% Impervious	(%)	100%	0%	100%	0%	100%	0%
Depression Storage	(in)	0	0.05	0	0.05	0	0.1
Mannings n		0.03	0.03	0.03	0.03	0.03	0.03
Slope	(%)	3%	3%	2%	2%	2%	2%
Infiltration Rate (Simulation 1)	(in/hr)	5.62	0	5.62	0	2.64	0
Infiltration Rate (Simulation 2)	(in/hr)	1.96	0	1.96	0	2.07	0

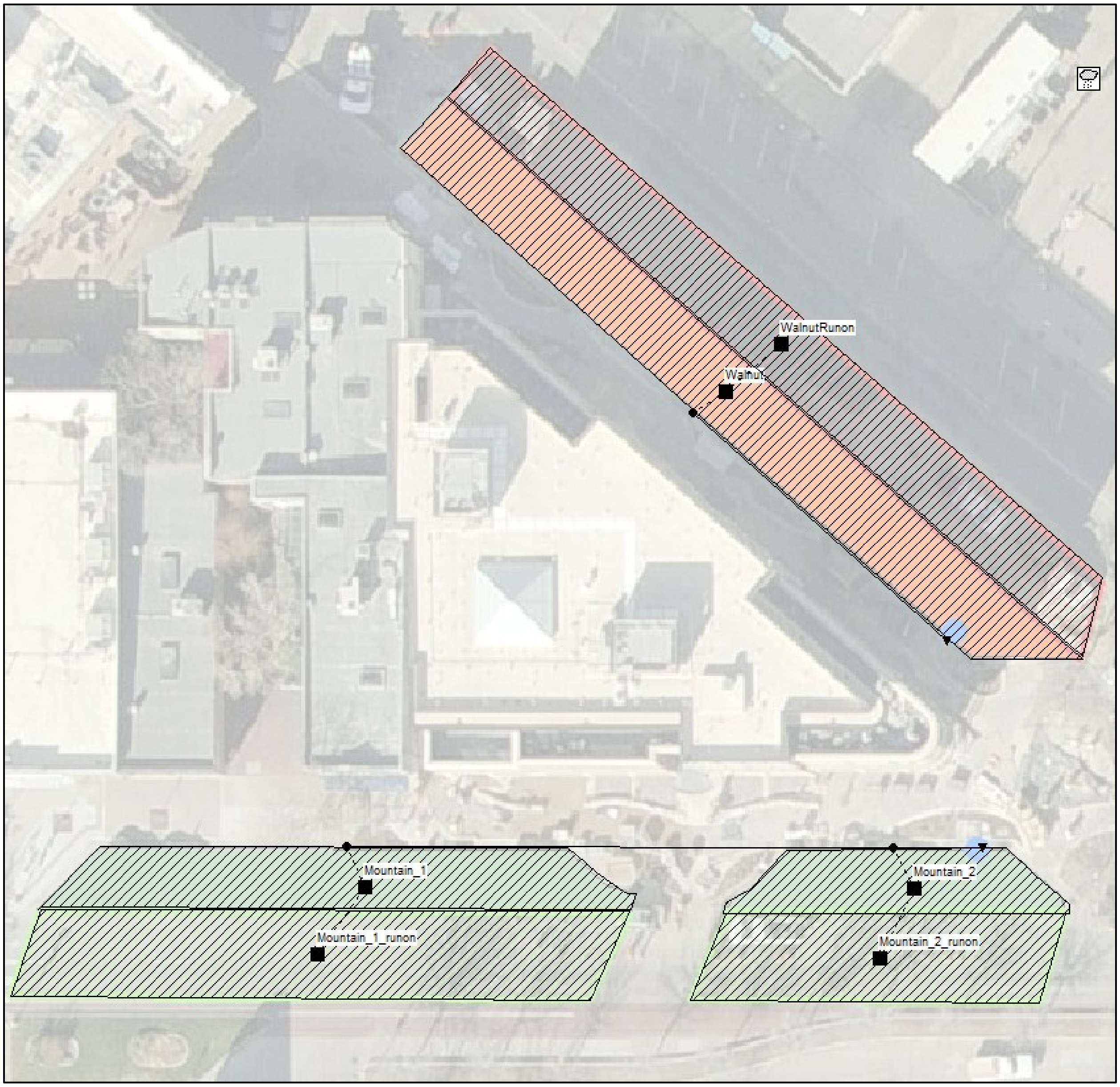

Figure 15: Mitchell Block runoff SWWM 5 model schematic

2.5 Surface Infiltration Analysis

During the summer of 2011 several areas were identified at both the Mitchell Block and CTL sites that had significant clogging and reduced infiltration rates. Preliminary infiltration tests were run to determine the need for further analysis. It was found that the sites had high spatial variability in infiltration rates. After a review of literature and discussions with stormwater professionals at UDFCD, a proper surface infiltration testing method was identified. The method used follows ASTM C1701. A similar method is used by researchers for the EPA pilot study site in the parking lot of their Edison facility, with the only difference being the sealing technique (Borst and Rowe 2010).

A 12-inch diameter PVC pipe was used as the infiltrometer in this method. Two parallel lines were marked on the inside of pipe at 10 and 15 mm above the bottom, between which the water height was maintained during tests. A bead of plumber's putty was used to seal the infiltrometer to the ground to prevent leaks. Weight was applied to the top of the infiltrometer using tie-down straps connected to hooks on a square wooden frame. The frame was held down on each corner using five-gallon buckets filled with rocks. The tie-down straps were cranked as tight as possible to compress the plumber's putty, improving the seal. Figure 16 shows the set up before the test.

Figure 16: Infiltration test setup at CTL on 5/03/2012

The test was completed in two stages. The first was a pre-wet test, in which 3.6 kg or 3.6 liters of water was measured out in a 5-gallon bucket. At the Mitchell Block sites only 2.8 kg or 2.8 liters of water was used because the weight of the 5-gallon bucket was not accounted for (0.8 kg). Any effect on the results was assumed to be negligible. The water was poured into the infiltrometer up to a point between the 10 and 15 mm lines in the infiltrometer. A stop watch was started as soon as water was applied. During the test, water was poured as needed to maintain the head between 10 and 15 mm. This process was carried out until all of the water had infiltrated through the surface, at which point the timer was stopped. If the pre-wet test was completed in less than 30 seconds, then the infiltration test was run using 18 kg or 18 liters of water. If the pre-wet test exceeded 30 seconds then the infiltration test was run with 3.6 kg or 3.6 liters of water. The infiltration test was run within two minutes of the completion of the pre-wet test. The infiltration test was conducted in the exact same manner as the pre-wet test, and the time to infiltrate all of the water was recorded. Testing was not conducted within 24 hours of any measurable rainfall. The pre-wet tests were used only as a means to saturate the sub-base media and determine the appropriate volume of water to use for the infiltration test.

Test sites were selected using a 5 foot by 5 foot numbered grid system designed in GIS. Figure 17 and Figure 18 show the test locations for the Mitchell Block sites and CTL, respectively, and can be found in Section 3.1 of this document. The ASTM method requires three test locations for sites with areas up to 2,500 square meters and an additional location for each additional 100 square meters. Grid numbers were selected using MS EXCEL's random number generator. Three tests were conducted at each site, with five grid numbers generated for each site, in case some sites were inaccessible due to parked vehicles or other obstructions at the time of the test.

Figure 17: Infiltration test locations at the Mitchell Block sites

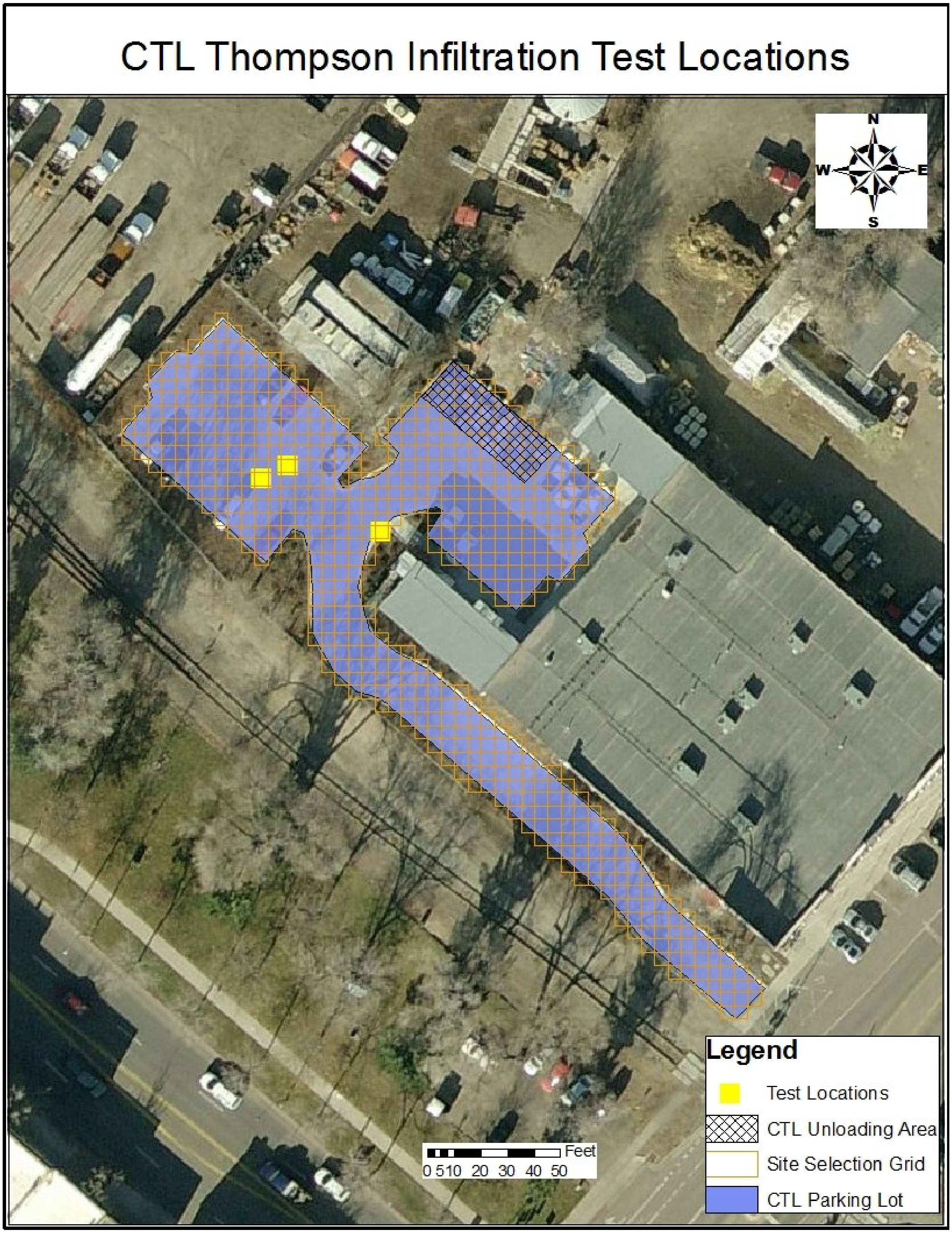

Figure 18: Infiltration test locations for the CTL Thompson parking lot

3.0 SURFACE INFILTRATION AND MAINTENANCE ANALYSIS

3.1 Test Location Results

Three tests were carried out at each PPS site in this study: Mountain, Walnut and CTL. The selected test locations at each site are shown in the previous section in Figure 17 and Figure 18.

3.2 Initial Infiltration Tests

3.2.1 Results

All of the tests at the two Mitchell Block sites were carried out on 5/14/2012. The three tests at CTL were carried out on 5/03/2012, 5/22/2012 and 7/02/2012. The pre-wet times for all test sites were greater than 30 seconds.

Mountain had the highest average infiltration rate at 5.62 in/hr. Walnut's average infiltration rate was less than half of Mountain's at 2.64 in/hr. CTL had the lowest infiltration rate at 1.04 in/hr. The results for both the pre-wet and infiltration tests at all three sites are presented in Table 5.

Table 5, Field Infiltration test results for Mountain, Walnut and CTL

	Walnut		Mountain		CTL[1]	
Test Site	**Pre-Wet (in/hr)**	**Test (in/hr)**	**Pre-Wet (in/hr)**	**Test (in/hr)**	**Pre-Wet (in/hr)**	**Test (in/hr)**
1	7.39	2.07	4.91	1.78	3.31	3.12
2	4.73	1.57	7.19	1.96	0	0
3	11.68	4.27	33.73	13.12	0	0
Average	**7.93**	**2.64**	**15.28**	**5.62**	**1.10**	**1.04**
Median	**7.39**	**2.07**	**7.19**	**1.96**	**0**	**0.00**

Notes

1. The second two CTL test sites stopped infiltrating shortly after testing began, and are designated as a zero infiltration rate.

The pre-wet results aren't meant to be representative of infiltration rates at the sites. The average rate at all three sites was greater than the median, particularly at Mountain were the average was nearly three times the median. This was due to the large infiltration rate at the third test location of over 13 in/hr, compared to sites one and two where both were less than 2 in/hr. The three infiltration rates at Walnut were less variable ranging from 1.57 to 4.27 in/hr. At CTL, the second and third test sites did not infiltrate completely during the pre-wet tests; therefore the sites were assigned an infiltration rate of zero.

3.2.2 Discussion

The infiltration test results obtained are useful to understand maintenance needs at each PPS site and the general condition of each site. The ability of these sites to infiltrate water is critical for proper functionality of the system. The Mitchell Block infiltration results were used in the runoff volume reduction analysis assumption validation model (Section 2.4.3). The three test sites at Mountain represent the spectrum of variability in infiltration rates. The third test site had an infiltration rate over 13 in/hr, which represents a relatively unclogged portion of the pavers. In contrast, the first two test sites had infiltration rates less than 2 in/hr and were notably clogged with debris and sediment at the time the tests were performed. Field observations of the site as a whole indicated that approximately 70% of the pavers at Mountain were clogged to some degree. Some areas appeared to be completely clogged, particularly the eastern portion of the west pavers section (just above the handicap ramp area). The highest tested infiltration rate at Walnut was 4.27 in/hr, over three times less than the highest infiltration rate at Mountain. Based on field observations at Walnut, it was estimated that about 70% of the pavers were at least slightly clogged, although none of the areas appeared to be fully clogged. The CTL results indicate that sections of the PC parking lot are completely clogged. The second and third test sites stopped infiltrating water during the pre-wet portion of the test. Although some water infiltrated, the infiltration rate was effectively zero at these sites. There appeared to be two factors that affected the infiltration rates at the site:

1. Over-smoothing during the construction of the lot
2. Presence of excessive debris and sediment

The over smoothing factor reduced the as built surface infiltration rate for a few small sections of the parking lot. The second factor appeared to be the more significant problem. Based on field observations, approximately 70% of the parking lot was clogged with sediment and debris. About 30% of the parking appeared to be completely clogged (Figure 19).

Figure 19: Clogged section of the CTL PC Parking Lot

3.4 Conclusions

These results clearly indicate the need for maintenance at all three of the study sites. The highest infiltration rate at Mountain of over 13 in/hr indicates the potential of these sights to have very high infiltration rates. The other two tests at Mountain were less than 2 in/hr and were visually clogged with sediment. The highest infiltration rate at Walnut was about 4 in/hr and the other two test sites were at or below 2 in/hr. All of the test locations at Walnut appeared to be clogged to varying degrees. CTL had the lowest infiltration rates, with two of the test sites having infiltration rates of zero in/hr. It is clear that CTL requires immediate maintenance. After storms it was observed that the lowest section of the parking lot would become inundated with water for several days due to clogging.

Based on discussions with UDFCD, it is recommended that the Mitchell Block pavers be swept with a non-brush dry vacuum street sweeper and the CTL site be swept with a non-brush wet vacuum sweeper. After maintenance is performed infiltration tests should be carried out in the same locations as the ones described in the above sections to determine the effectiveness of the maintenance activities. In addition, it is recommended that regular maintenance be performed at least once a year to extend the effective life of the pavers.

4.0 WATER QUANTITY ANALYSIS

Water quantity data is important to evaluating the performance of PPS, particularly those that utilize a "no-infiltration" design. This section presents the water quantity data and analysis for the Mitchell Block sites.

4.1 Hydrologic Summary

From 2010 through 2011, a total of nineteen precipitation events were monitored at the Mitchell Block sites. Precipitation data was collected at the CTL rain gauge for all of the events except storms 4, 5 and 6 due to equipment maintenance (Table 7). For these storms, data from a City of Fort Collins rain gauge at Lincoln Street was used. Eight of the storms occurred in 2010 totaling 6.81 inches and the other eleven occurred in 2011 totaling 6.82 inches. The median storm depth for all nineteen events was about 0.63 inches, with individual events ranging from 0.12 inches to 1.57 inches, based on a six hour inter-event time. The monitoring season covered approximately 6 months from April through September. Between September and April the majority of precipitation in the Fort Collins area occurred as snow, making stormwater monitoring unfeasible. Table 7 and Table 8 in the following sections detail important hydrologic parameters for each monitored storm including: Precipitation depth, storm duration, peak 15-minute precipitation intensity and antecedent dry period. These parameters were used to help analyze and describe various data trends and observations. Storm durations ranged from 2 hours to 45 hours with a median of 15 hours. Peak 15-minute storm intensities ranged from 0.12 to 1.08 in/hr with a median of 0.36 in/hr. Antecedent dry times ranged from approximately 7 hours to approximately 1,250 hours.

.2 SWMM Model Runoff Validation

4.2.1 Results

Estimating influent to the PPS hinged on the assumption that no surface runoff occurred from the paver sections during storm events. This was verified using a model of the system constructed in the EPA's SWMM 5, described in detail in Section 2.4.3. Table 6 summarizes the results from the two model simulations using average (Simulation 1) and the median (Simulation 2) infiltration rates, based on measured infiltration rates described in Section 3.2.1. Simulation 1 showed that surface runoff occurred from Walnut at a precipitation intensity of 1.35 in/hr, while both of the Mountain catchments produced surface runoff at a precipitation intensity greater than 2 in/hr. Simulation 2 was more conservative, with Mountain catchments 1 and 2 producing surface runoff at 0.85 and 0.8 in/hr, respectively. Surface runoff began at the Walnut catchment at a precipitation intensity of 1.05 in/hr.

Table 6, Mitchell Block SWMM Model Infiltration Parameters and Runoff Results

	Average Infiltration Rate		Median Infiltration Rate	
Subcatchment	Infiltration Rate (in/hr)	Precipitation Intensity of Runoff Initiation (in/hr)	Infiltration Rate (in/hr)	Precipitation Intensity of Runoff Initiation (in/hr)
Walnut	2.64	1.35	2.07	1.05
Mountain 1	5.62	2.25	1.96	0.85
Mountain 2	5.62	2.1	1.96	0.8

4.2.2 Discussion

The results from these simulations, coupled with the hydrologic data from the monitored storms suggest that minimal surface runoff occurred during any events. The minimum intensity which caused surface runoff was 0.8 in/hr at the Mountain 2 catchment during Simulation 2. The hydrologic data shows the two most intense storms, storms 7 and 8, had peak 15-minute intensities of 0.92 and 1.08 in/hr, respectively (Table 7). This suggests that during these storms some surface runoff would have occurred in areas with infiltration rates at or below the median. However, this simulation used highly conservative infiltration values and the peak 15-minute intensities that exceeded the simulation intensities were not sustained for more than 15-minutes during the event. It is reasonable to conclude that the amount of surface runoff that occurred was negligible compared to the total event runoff. The Simulation 1 results are less conservative, but represent average site conditions. These results showed surface runoff occurring at a precipitation intensity of 1.35 in/hr, greater than the two largest peak storm 15-minute intensities listed previously.

It should be noted that infiltration rates at different areas of the paver sections are spatially variable across the pavers and the infiltration rates used in the model are meant to represent the average conditions at the sites. The infiltration rates presented in Section 3.2.1 and applied to this model represent saturated conditions.

4.3 Runoff Volume Reduction

4.3.1 Initial Observations and Quality Control

It was observed during several monitoring events that the culvert drainage under the handicap ramp separating the two paver sections at Mountain caused flooding upstream. This area utilizes a small square culvert to pass upstream drainage from the curb gutter under the sidewalk to the curb gutter downstream (Figure 20). The culvert does not have the hydraulic capacity or gradient to drain the upstream flow during high intensity storms and was often inhibited by debris and sediment, which caused the area of pavers immediately upstream to flood from backwater effects (Figure 21 and Figure 22).

The standing water was free to infiltrate through the pavers, causing additional runoff to pass through the under-drain and eventually the monitoring box. This made it appear that less runoff volume reduction was occurring and in some cases negative runoff volume reduction. Three events that indicated large negative runoff reduction were assumed to be biased because of the flooded area and were eliminated from the analysis. These three events (storm numbers 1, 8 and 9) are highlighted in grey in Table 7. The effect that the flooding had on the data for the storms used in the analysis is unknown. Several other storms at Mountain indicated slightly negative or zero runoff reduction, which may have been influenced by the flooded pavers, but for the purpose of this report those results were assumed to be unbiased. The inundated area of pavers was investigated several times during the 2011 sampling season and it appeared that an abundance of fine materials had accumulated due to settling when the area had been flooded. This most likely inhibited the ability of this area to infiltrate water and may have reduced the effect of additional runoff infiltrating the pavers.

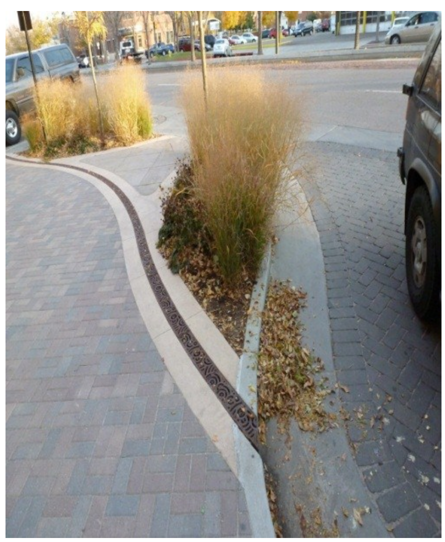

Figure 20, Sidewalk culvert for curb drainage (dry)

Figure 21: Flooded pavers during storm due to clogged culvert

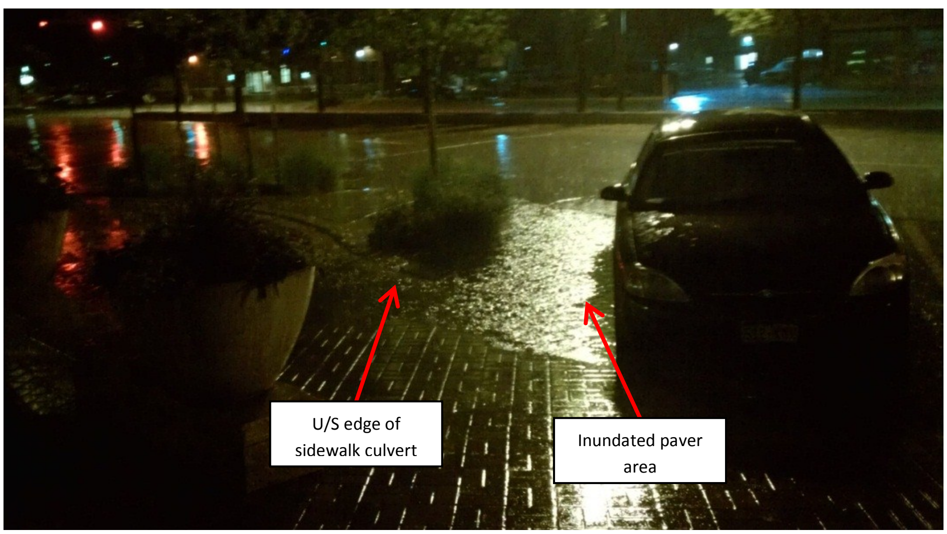

Figure 22: Severely inundated pavers during storm due to clogged culvert

Two other storms (storm 13 at Mountain and storm 19 at Walnut) were eliminated due to the storm sewers at each respective site flooding and submerging the flow monitoring equipment, which caused erroneous depth measurements. These storms are also highlighted in gray in Table 7 and Table 8 for each respective site.

4.3.2 Runoff Volume Reduction Summary

4.3.2.1 Results

Runoff volume reduction was calculated using a simple mass balance equation described in the Methods section and is reported for each site in Table 7 and Table 8 using three different metrics: A percentage of the total storm runoff, an absolute volume reduction in cubic feet and a normalized volume reduction in inches. The normalized volume was calculated by dividing the volume reduction by the total contributing area (paver area and run-on area).

Normalized runoff volume reduction at Mountain ranged from approximately zero inches to about 1 inch per storm, with a median of 0.24 inches and an average of 0.28 inches. In total, 4.13 inches of runoff volume was eliminated in the fifteen monitored events, equating to about 45% of the total runoff for those monitored events. Storms 2, 5, 10 and 14 at Mountain had runoff reduction between 0% and negative 10% of the total storm, while five storms reduced storm runoff by 70%.

Runoff volume reduction at Walnut ranged from -0.13 inches to 0.66 inches, with a median of 0.17 inches and an average of 0.23 inches. The total runoff volume reduction at Walnut for the eighteen events monitored was about 4.09 inches or 35% of the total runoff for those events. Sixteen of the eighteen monitored events exceeded 28% runoff reduction and only storm 13 had negative runoff reduction.

Table 7, Hydrologic and Runoff Reduction Results for the Mountain Permeable Pavers

Mountain											
Hydrology Data						Response Data			Runoff Reduction Calculations		
Storm Number	Storm Date	Total Precipitation (in)	Storm Duration (hrs)	Peak 15 Minute Storm Intensity (in/hr)	Antecedent Dry Period (hrs)	Response Duration (hrs)	Observed Effluent Runoff Volume (ft^3)	Calculated Influent Runoff Volume (ft^3)	Runoff Reduction (ft^3)	Runoff Reduction (%)	Normalized Runoff Reduction (in)
1*	4/21/2010	--	--	--	--	--	--	--	--	--	--
2	4/23/2010	0.79	14.0	0.4	8	11	570	542	-28	-5%	-0.04
3	4/28/2010	0.68	11.1	0.36	123	9	240	463	223	48%	0.31
4	5/11/2010	1.38	27.0	0.46	292	23	795	963	168	17%	0.24
5	5/26/2010	0.12	2.0	0.3	170	2	71	64	-7	-10%	-0.01
6	6/11/2010	1.57	45.0	0.47	376	41	494	1,099	605	55%	0.85
7	7/4/2010	0.63	4.7	1.08	479	7	252	428	176	41%	0.25
8*	8/8/2010	--	--	--	--	--	--	--	--	--	--
9*	4/13/2011	--	--	--	--	--	--	--	--	--	--
10	4/20/2011	0.34	24.5	0.36	46	3	221	221	0	0%	0
11	4/24/2011	0.37	41.1	0.16	70	6	49	242	193	80%	0.27
12	5/10/2011	1.39	39.2	0.24	379	39	267	970	703	72%	0.98
13**	5/18/2011	--	--	--	--	--	--	--	--	--	--
14	5/24/2011	0.19	8.5	0.56	93	2	116	114	-2	-2%	0.00
15	6/16/2011	0.35	9.6	0.64	182	14	105	228	123	54%	0.17
16	6/29/2011	0.24	28.6	0.36	250	2	15	149	134	90%	0.19
17	7/6/2011	0.28	10.8	0.32	20	2	46	178	132	74%	0.18
18	9/7/2011	0.39	12.9	0.12	1,253	4	28	256	228	89%	0.32
19	9/14/2011	0.88	18.2	0.36	170	11	308	606	298	49%	0.42
Median		**0.39**	**14.0**	**0.36**	**170**	**7**	**221**	**256**	**168**	**49%**	**0.24**
Total		**9.6**	**--**	**--**	**--**	**--**	**3,576**	**6,521**	**2,945**	**45%**	**4.13**
Average		**0.64**	**20**	**0.41**	**261**	**12**	**238**	**435**	**196**	**44%**	**0.28**

***Eliminated from analysis due to large negative runoff reduction attributed to flooding of a paver section**

****Eliminated from analysis due to flooding of monitoring equipment, causing erroneous depth readings**

Blue rows indicate storms that used precipitation data from the Lincoln Rain Gauge; Grey rows indicate storms eliminated from analysis

Table 8, Hydrologic and Runoff Reduction Results for the Walnut Permeable Pavers

Walnut											
Hydrology Data						Response Data			Runoff Reduction Calculations		
Storm Number	Storm Date	Total Precipitation (in)	Storm Duration (hrs)	Peak 15 Minute Storm Intensity (in/hr)	Antecedent Dry Period (hrs)	Response Duration (hrs)	Observed Effluent Runoff Volume (ft^3)	Calculated Influent Runoff Volume (ft^3)	Runoff Reduction (ft^3)	Runoff Reduction (%)	Normalized Runoff Reduction (in)
1	4/21/2010	1.38	26.7	0.56	101	26	552	812	259	32%	0.42
2	4/23/2010	0.79	14.0	0.4	8	10	301	451	150	33%	0.25
3	4/28/2010	0.68	11.1	0.36	123	18	204	384	180	47%	0.29
4	5/11/2010	1.38	27.0	0.46	292	26	704	812	108	13%	0.18
5	5/26/2010	0.12	2.0	0.3	170	7	16	42	26	62%	0.04
6	6/11/2010	1.57	45.0	0.47	376	57	522	928	406	44%	0.66
7	7/4/2010	0.63	4.7	1.08	479	11	104	354	250	71%	0.41
8	8/8/2010	0.26	2.2	0.92	16	8	36	128	92	72%	0.15
9	4/13/2011	0.82	15.1	0.28	758	31	368	470	102	28%	0.17
10	4/20/2011	0.34	24.5	0.36	46	15	125	176	51	29%	0.08
11	4/24/2011	0.37	41.1	0.16	70	27	123	195	72	37%	0.12
12	5/10/2011	1.39	39.2	0.24	379	42	458	818	360	44%	0.59
13	5/18/2011	1.57	19.5	0.36	146	21	1,010	928	-82	-8%	-0.13
14	5/24/2011	0.19	8.5	0.56	93	9	26	85	59	69%	0.10
15	6/16/2011	0.35	9.6	0.64	182	13	44	183	139	76%	0.23
16	6/29/2011	0.24	28.6	0.36	250	7	30	115	85	74%	0.14
17	7/6/2011	0.28	10.8	0.32	20	13	72	140	68	48%	0.11
18	9/7/2011	0.39	12.9	0.12	1,253	13	35	207	172	83%	0.28
19**	9/14/2011	--	--	--	--	--	--	--	--	--	--
Median		**0.51**	**14.6**	**0.36**	**158**	**14**	**124**	**280**	**105**	**45%**	**0.17**
Total		**12.75**	**--**	**--**	**--**	**--**	**4,730**	**7,226**	**2,496**	**35%**	**4.09**
Average		**0.71**	**19**	**0.44**	**264**	**20**	**263**	**401**	**139**	**47%**	**0.23**

****Eliminated from analysis due to flooding of monitoring equipment, causing erroneous depth readings**

***Depth values larger than possible depth within box, indicates flooding of equipment**

Blue rows indicate storms that used precipitation data from the Lincoln Rain Gauge; Grey rows indicate storms eliminated from analysis

4.3.2.2 Discussion

The data from Table 7 and Table 8 indicate that both the Mountain and Walnut PPS are reducing storm runoff volume. Runoff volume was reduced in 73% (11/15) and 95% (18/19) of the monitored storms at Mountain and Walnut, respectively. The data show that the sites were able to reduce runoff volume on a per storm basis as well as an aggregate total over two monitoring periods. Runoff volume reduction was variable at both sites for individual storms. The next section evaluates various factors that influenced performance on a per storm basis.

4.3.3 Field Capacity Analysis

4.3.3.1 Results

Figure 23 compares calculated field capacity values to runoff volume reduction at each site. All runoff volume reduction values were less than the field capacity values for the respective sites, except for storm 6 at both sites and storm 12 at Mountain. Runoff volume reduction at Walnut exceeded the field capacity for storm 6 by only about 0.05 inches. Runoff volume reduction at Mountain for both storms 6 and 12 was about twice the calculated field capacity.

4.3.3.2 Discussion

The field capacity of the sub-base media plus the wetting potential of the PPS surface are equivalent to the maximum water retention at the Mitchell Block sites before any evaporation occurs. The runoff volume reduction at Mountain for storms 6 and 12 suggests either, an error in measurement or restoration of the available field capacity during the event. Storms 6 and 12 had storm durations of 45 hours and 39 hours, respectively (Table 7). Further evaluation of the data indicated "dry periods" of up to 5 hours during the storms where no rainfall occurred. During these "dry periods", evaporation may have occurred helping to produce large runoff volume reduction. This explanation would require that ambient air moisture be reduced enough in the dry periods to produce an evaporative gradient between the saturated sub-base layers in the PPS and the surrounding air. Without collecting additional data this explanation cannot be validated.

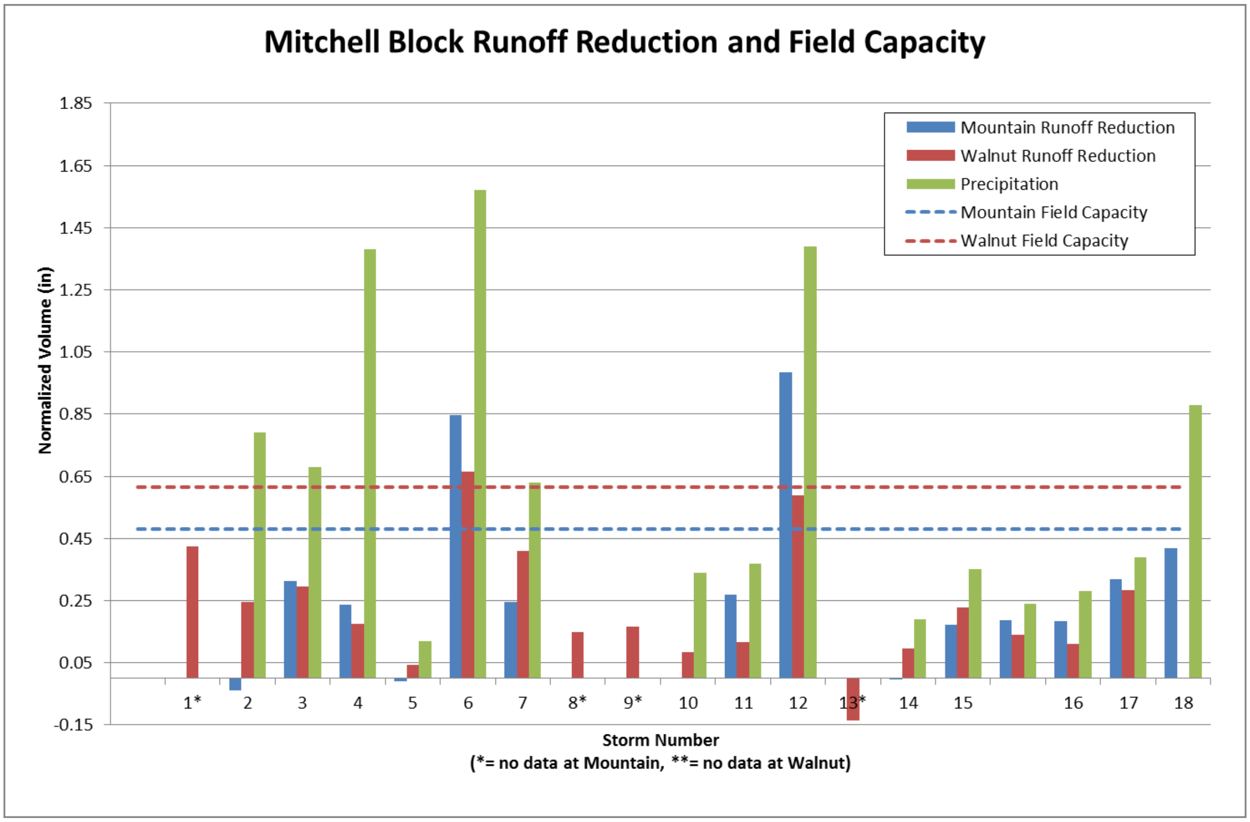

Figure 23: Runoff reduction and precipitation bar chart for Mountain and Walnut

Walnut produced the highest runoff volume reduction over the monitoring period for these two storms, which were approximately 30% and 50% less than those at Mountain for storms 6 and 12, respectively, despite Walnuts higher field capacity. Two potential explanations for larger reduction in runoff volumes at Mountain during these storms are:

1. The Mountain sub-base design is more efficient at restoring field capacity
2. The surface of the Mountain site is oriented in a manner that recieves more direct sunlight providing more favorable evaporative conditions, thus restoring field capacity more efficiently

This study did not explore these factors in detail, but both shoud be addressed by future research to help optimized PPS application and design.

It should be noted that both of these storms had antecedent dry times of nearly 400 hours, which plays an important role in allowing the field capacity to be restored between events via evaporation. Available field capacity, and in turn runoff volume reduction potential, at the start of the storm are related to the antecedent hydrologic conditions. Longer antecedent dry times and favorable evaporation conditions should increase the available field capacity at the start of the storm and lead to larger runoff volume reduction. Figure 24 below plots the normalized runoff volume reduction as a function of antecedent dry time.

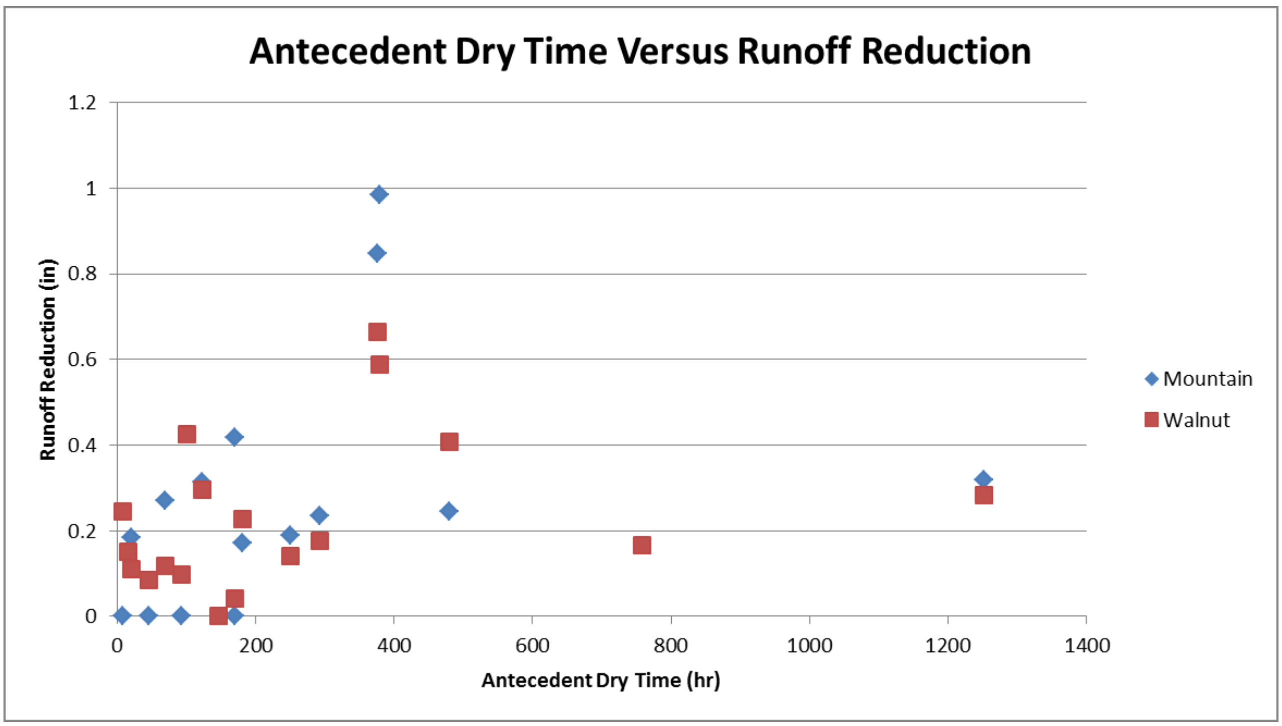

Figure 24: Normalized runoff reduction as a function of antecedent dry time for Mountain and Walnut

There does appear to be an upward trend in runoff volume reduction with increasing antecedent dry time at both sites. Other hydrologic factors play an important role in runoff volume reduction results for individual storms which would help explain the variable nature of the results indicated in Figure 24.

4.3.4 Alternative Designs at Mountain and Walnut

4.3.4.1 Results

For a better comparison of performance at the two sites; absolute volumes in cubic feet were used to compare the; medians, averages and totals from each site (Table 9). In addition, the statistics and totals are based only on storms that were monitored at both sites, 14 in total. These results indicate that Mountain reduced runoff volume by 23% more per storm based on average and median values for the two sites. In addition, Mountain reduced the total aggregate runoff volume by 25% more than Walnut, for common monitored storms at each site.

Table 9, Runoff reduction summary for common monitored events at Mountain and Walnut

Site	No. of Storms	Runoff Reduction Average (ft^3)	Median (ft^3)	Total (ft^3)
Mountain	14	187	151	2,647
Walnut	14	152	123	2,125

4.3.4.2 Discussion

Based on the field capacity values calculated for Mountain and Walnut, the Walnut design (UDFCD design with a sand filter layer) has a greater physical potential to retain water. These data show that Mountain provided greater runoff volume reduction, indicating that field capacity is not a good predictor of runoff volume reduction potential. The individual storm data shows that Walnut eliminated runoff in a higher percentage of storms, but Mountain was able to eliminate a greater volume of runoff on average per storm.

Gobel et. al. (2010) found that PPS with sub-base designs composed of “twin layers”, with a coarse grained material on top of fine grained material, had lower average evaporation rates. This finding suggests that a more homogenous sub-base material will likely result in greater runoff volume reduction, due to greater evaporation potential. This offers a potential explanation for the runoff volume reduction performance at Mountain.

4.4 Conclusions

This portion of the study set out to determine if “no-infiltration” PPS are able to reduce stormwater runoff volumes and quantify the amount of reduction provided by different sub-base designs. The data and analyses presented in this section clearly indicate that both PPS are capable of reducing storm runoff volumes. Quantification of the runoff volume reduction was accomplished by utilizing calculated inflow volumes for each site based on the watershed characteristics, storm hydrology and the assumption of negligible surface runoff during any of the analyzed storm events. Some of the important results and notable observations are listed below

- Both systems were able to provide runoff volume reduction for a high percentage of the monitored events, with Mountain reducing runoff for 73% of events and Walnut for 94% of events
- Mountain eliminated 45% of the total storm runoff volume from the monitored events
- Walnut eliminated 35% of the total storm runoff volume from the monitored events

- Mountain was able to provide large runoff volume reduction for several storms, including one storm in which 0.98 inches of runoff was eliminated, nearly double the site's field capacity
- A discernible trend was noted of increasing runoff volume reduction with increased antecedent dry time, future research should bolster datasets and determine if a statistically significant correlation exists between the two parameters
- Runoff volume reduction exceeded the calculated field capacity at Mountain for two events
- Mountain eliminated 23% more runoff volume on average than Walnut and eliminated 25% more total runoff volume for commonly monitored storms at the two sites
- The higher runoff volume reduction at Mountain may be related to more homogenous grain sizes of the sub-base design used at Mountain

Both Mitchell Block PPS are viable LID designs and should be considered for more widespread application in urban settings. The data presented should be used in addition to data from the Database to help guide decisions and designs for future PPS applications. Monitoring should continue at these sites to improve the dataset and attempt to identify more data trends. It is recommended that if monitoring efforts continue, the flooding problem should be addressed at the Mountain site by increasing the hydraulic capacity of the culvert passing water under the sidewalk.

5.0 WATER QUALITY ANALYSIS

The ability of different types of PPS to remove pollutants from urban runoff is not well documented and is important for planning agencies and municipalities using LID practices. This section presents the water quality results and analysis from the Mitchell Block PICP sites and the CTL Thompson PC site (two sampling locations). No influent data was collected for any of the PPS sample sites, so data from two outfall sites in Fort Collins was used as a surrogate for inflow. As a comparison metric, effluent data for PPS from the International Stormwater BMP Database (the Database) was used to help provide context to the results.

5.1 Dataset Characterization

Table 10 below details all of the events monitored for the water quality portion of this study. The percent of the hydrograph captured during sampling is provided for Mountain and Walnut. The sampled storms are indicated for the CTL sample sites with a "Yes."

The total sampled storms at Mountain, Walnut, CTL DW and CTL PL were 11, 12, 14 and 19, respectively. Water quality samples were collected in underground sumps in two different areas of the CTL site at the conclusion of drainage, therefore it was assumed that each sample collected was representative of 100% of the hydrograph from the storm event. All sampled events at Mountain and Walnut exceeded 60% hydrograph capture and were assumed to be representative EMCs. As noted in the table, the storm on 5/11/2011 occurred in two distinct pulses. Although the inter-event time was slightly less than 6 hours, the samples were composited after the first pulse to meet laboratory holding times for sample submittal. The second pulse was not sampled.

Table 10, Hydrologic Summary for Water Quality Events

Storm Date	Storm Depth (in)	Percent Hydrograph Captured		Storms Sampled	
		Mountain	Walnut	CTL DW	CTL PL
10/27/2009	1.32	--	--	Yes	Yes
3/20/2010	0.19	--	--	-	Yes
4/21/2010	1.38	80%	89%	Yes	Yes
4/28/2010	0.68	96%	87%	Yes	Yes
5/11/2010	1.38	77%	99%	Yes	Yes
5/26/2010	0.12	--	--	-	Yes
6/11/2010	1.57	79%	71%	-	-
7/4/2010	0.64	97%	77%	Yes	Yes
8/8/2010	0.26	100%	64%	-	-
10/22/2010	0.31	--	--	Yes	Yes
11/9/2010	0.21	--	--	-	Yes
4/13/2011	0.82	68%	77%	Yes	Yes
4/20/2011	0.57	99%	82%	Yes	Yes
4/24/2011	0.37	--	77%	Yes	Yes
5/11/2011*	0.8*	90%	84%	Yes	Yes
5/18/2011	1.57	--	94%	Yes	Yes
5/24/2011	0.19	89%	--	-	Yes
6/30/2011	0.25	--	--	-	Yes
7/7/2011	0.4	--	91%	Yes	Yes
7/13/2011	0.66	--	--	Yes	Yes
9/8/2011	0.39	--	--	Yes	Yes
9/14/2011	0.91	97%	--	-	-
Total Sampled Events		11	12	14	19

*Storm occurred in two distinct pulses, only the first was sampled.The storm depth and hydrograph capture percent is representative of the first pulse only.

5.1.1 QA/QC of Laboratory Results

All water quality results received from the labs were examined to eliminate any erroneous results from the analysis. Results for five constituents obtained from CSU's lab were notably different from results obtained from the PCL and outside the range of typical stormwater concentrations. The constituents in question were: Total phosphorous (TP), dissolved cadmium (Cd), dissolved copper (Cu), dissolved lead (Pb) and dissolved zinc (Zn). The results for these constituents were consistently unreasonable and were therefore removed from the analysis.

5.1.2 Dataset Distribution and Censored Data Points

Datasets that contained censored values were tested for both a lognormal and normal distribution. The results are detailed in Table 11 below. The normal probability plots and Lilliefors Test results can be found in Appendix C for the datasets listed in the table below. These results allowed for the correct application of the ROS method to the eligible censored datasets (see Section 2.3.2 for details on application of the ROS method). The calculated values for the censored data points were used in all of the water quality data analyses. The regression plots and calculated regression values from the ROS method for the censored data can be found in Appendix C. Datasets that were not eligible for the ROS method used substitution of half of the minimum detection limit (MDL) as described in the methods section.

Table 11, Data Distribution of Eligible Censored Datasets

Constituent	Mountain	Walnut	CTL DW	CTL PL	Howes	Udall
TSS	L	L*	L	L*	L	L
NH3-N	--	--	L*	L*	L	L*
TKN	L*	L*	L*	L*	L	L*

L=Lognormal; -- = ineligible dataset
*Indicates datasets with censored data

5.2 Water Quality Results

5.2.1 Summary

In total, 39 parameters were tested for, consisting of 33 laboratory parameters and 6 field parameters. This report focuses on 4 main water quality categories consisting of 13 parameters:

1. **Solids**: Total Suspended Solids (TSS) and Total Dissolved Solids (TDS)
2. **Nutrients**: TP, Total Kjeldahl Nitrogen (TKN), Ammonia (NH_3), Nitrate (NO_3), Total Nitrogen (TN), Total Inorganic Nitrogen (TIN) and Total Organic Nitrogen (TON)

3. **Metals**: Total Recoverable (TR) Cu, TR Chromium (Cr) and TR Zinc (Zn)

4. **Bacteria**: E. coli

These parameters were chosen because of their known prevalence in stormwater and their practical importance for future LID design. Table 12 gives a concise summary of data from this study along with available data from the Database and the Udall and Howes outfalls. Median EMCs are reported for all of the parameters, except bacteria which are reported as geometric means. The high and low medians for the four PPS sample sites in this study for each parameter are identified in the table with red and blue text, respectively. Consult Appendix D for a complete summary table of all 39 parameters.

5.2.2 Boxplots

Boxplots are presented below for all of the parameters except E. coli. These plots provide characteristics of the datasets as a whole, rather than just a descriptive statistic. Boxplots show if two datasets have statistically significant differences by whether the upper 95% confidence interval is less than the lower 95% confidence interval. The confidence intervals are indicated by the notches on the side of the box. The number of data points in each dataset appears on each figure above the plot for the corresponding site.

Table 12, Water Quality Data Summary

	Constituent	Units	CTL Parking Lot	CTL Driveway	Mountain Ave	Walnut Ave	BMP Database - Porous Pavement Effluent Concentrations[1]	Howes Outfall	Udall Outfall
			PP Systems - Median EMC					Outfall/street - Median EMC	
Solids	TSS	mg/L	16	26	15	9.5	14	109	126
	TDS	mg/L	653	387	233	260	NR[3]	66	74
Nutrients	Tphos	mg/L	0.229	0.089	0.079	0.151	0.1	0.360	0.250
	TKN	mg/L	1.126	1.243	0.816	0.478	1.15	2.87	2.85
	NH3 as N	mg/L	0.090	0.060	0.050	0.050	NR[3]	0.88	0.91
	NO3 as N	mg/L	1.31	0.79	1.99	2.02	1	0.75	0.72
	Total N	mg/l	2.77	2.17	2.97	2.37	NR[3]	3.5	3.21
	Total Inorganic N	mg/l	1.54	0.915	1.83	1.94	NR[3]	0.77	0.97
	Total Organic N	mg/l	1.05	1.07	0.96	0.42	NR[3]	2.85	2.6
Metals	Cu (TR)	ug/L	17.65	14.61	19.39	10.37	6	8.6	21.7
	Cr (TR)	ug/L	23.995	17.57	2.5	4.195	DL[2]	4.49	5.74
	Zn (TR)	ug/L	21.7	14.2	18.4	17.3	18	62.8	126.5
			PP Systems - Geometric Mean					Outfall/street - Geometric Mean	
Bateria	Ecoli	#/100ml	29	28	125	62	NR[3]	3150	3661

[1]From International Stormwater Database BMP perfromance summary November 2011

[2]DL - More than 80% of the values were reported as non-detect; results excluded from analysis

[3]Not reported in the BMP perfromance summary for PP sites

High Value; Low Value

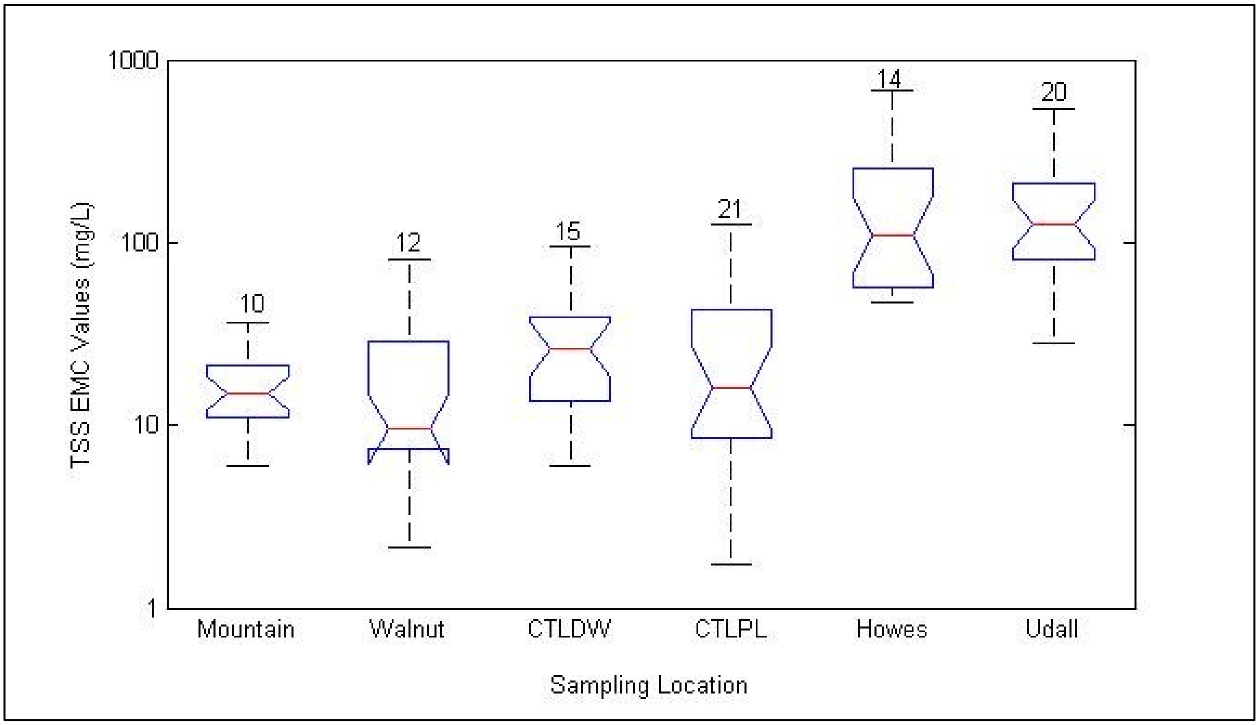

Figure 25: Boxplot for TSS

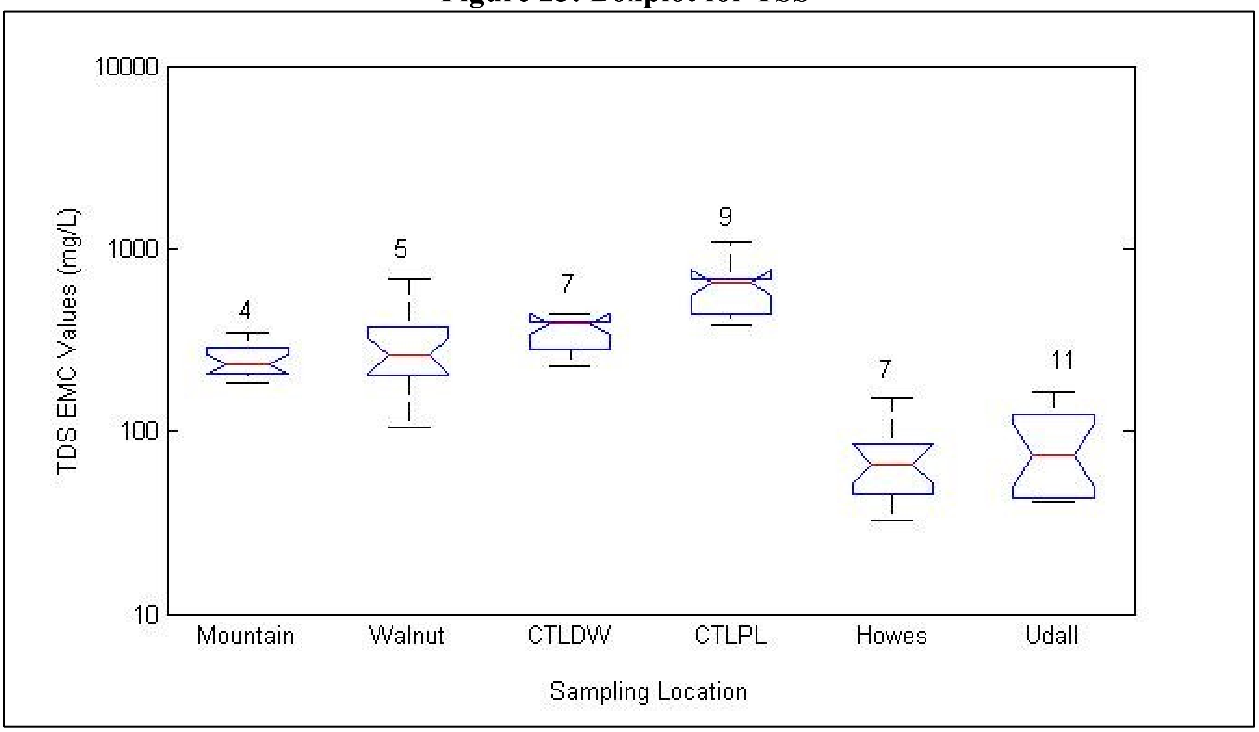

Figure 26: Boxplot for TDS

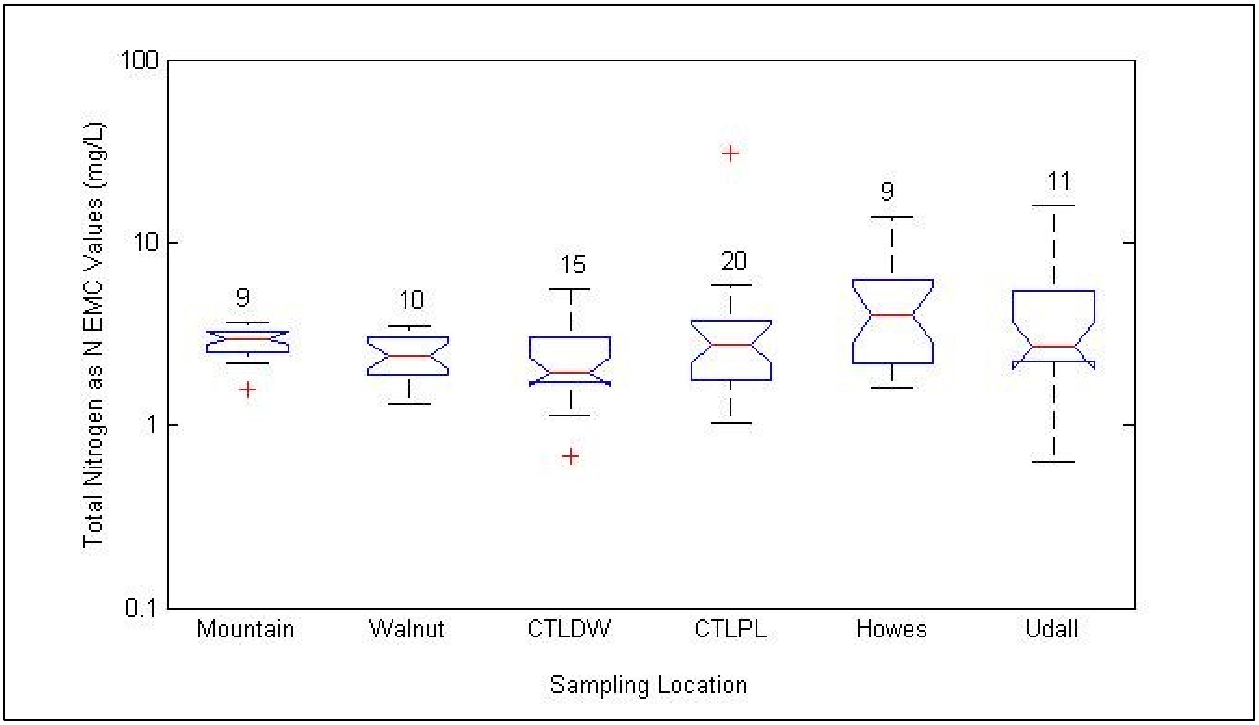

Figure 27: Boxplot for TN

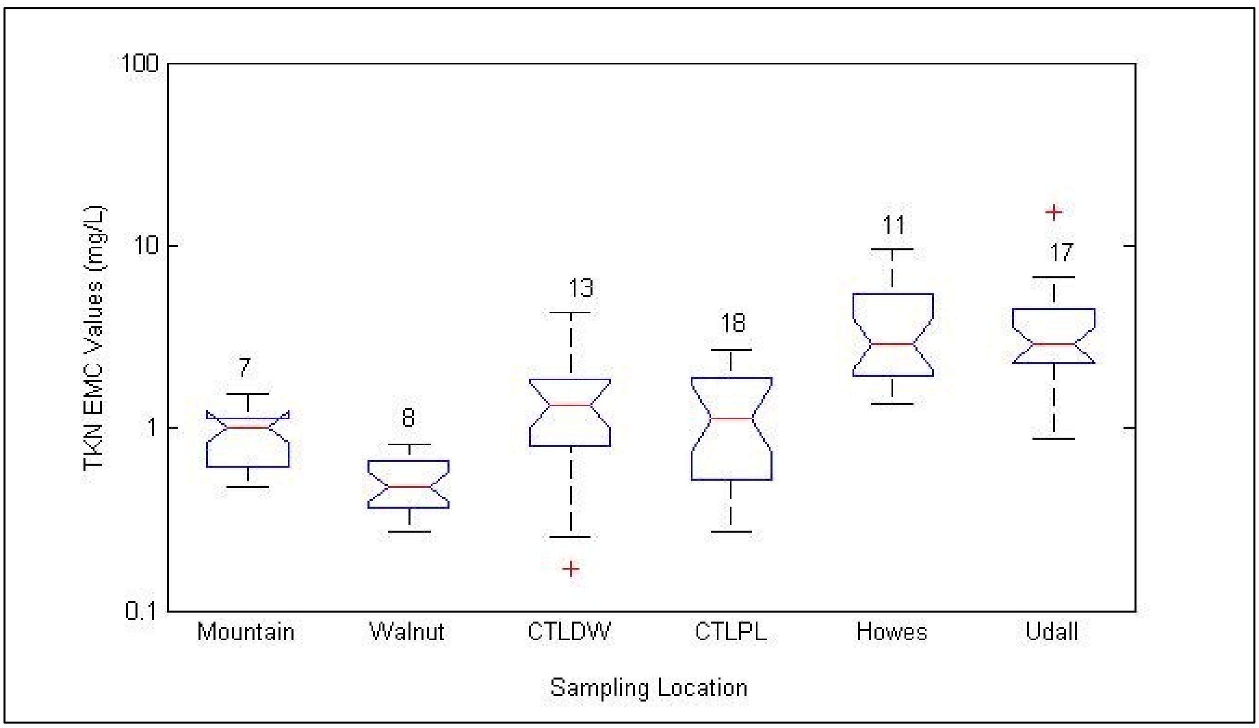

Figure 28: Boxplot for TKN

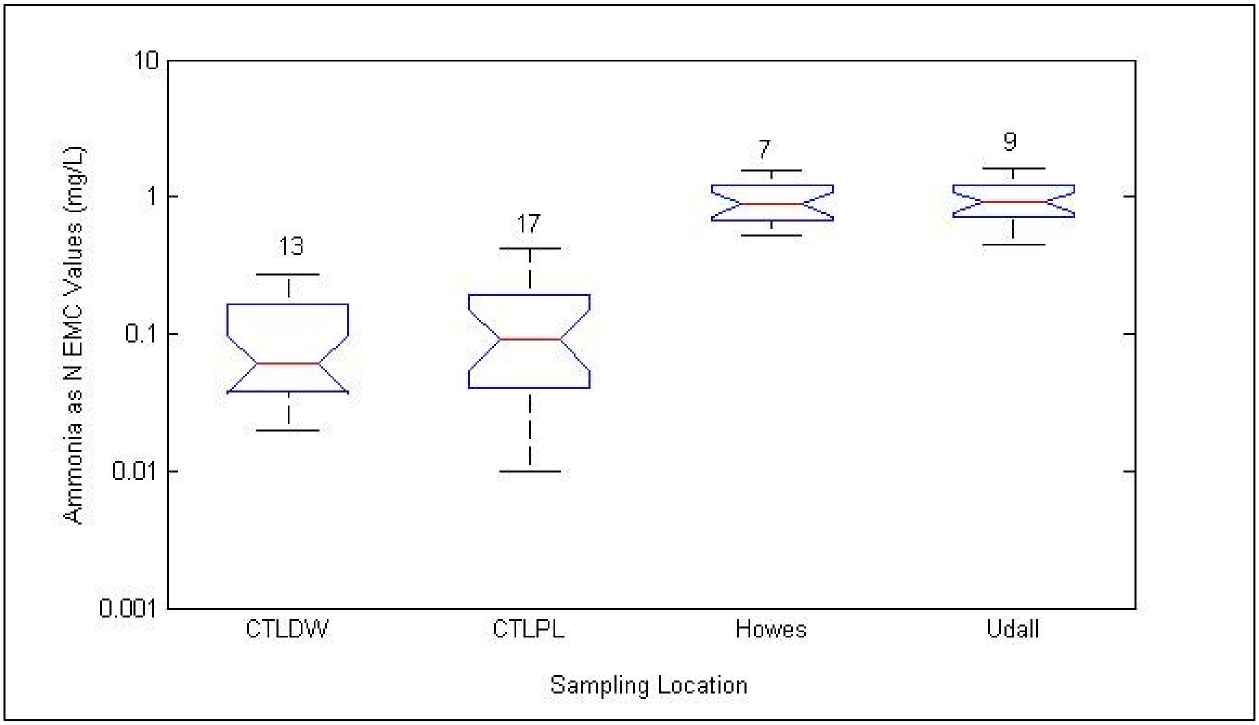

Figure 29: Boxplot for NH3 as N

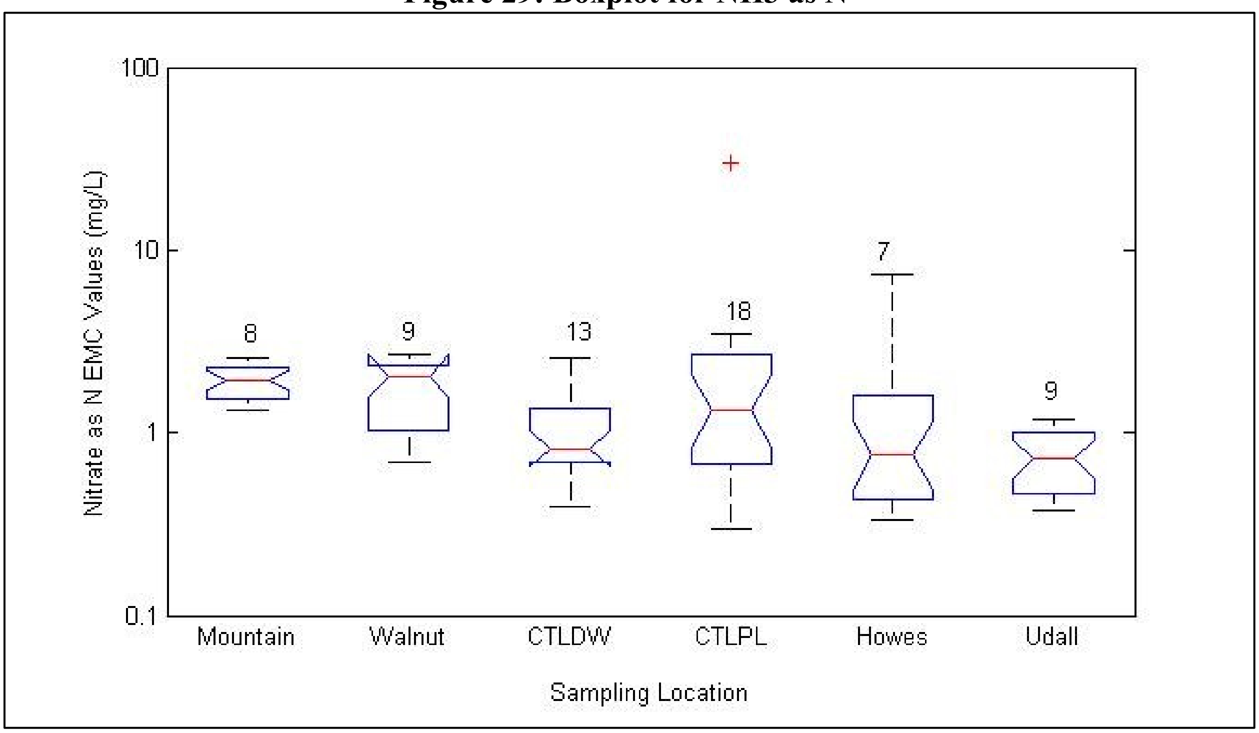

Figure 30: Boxplot for NO3 as N

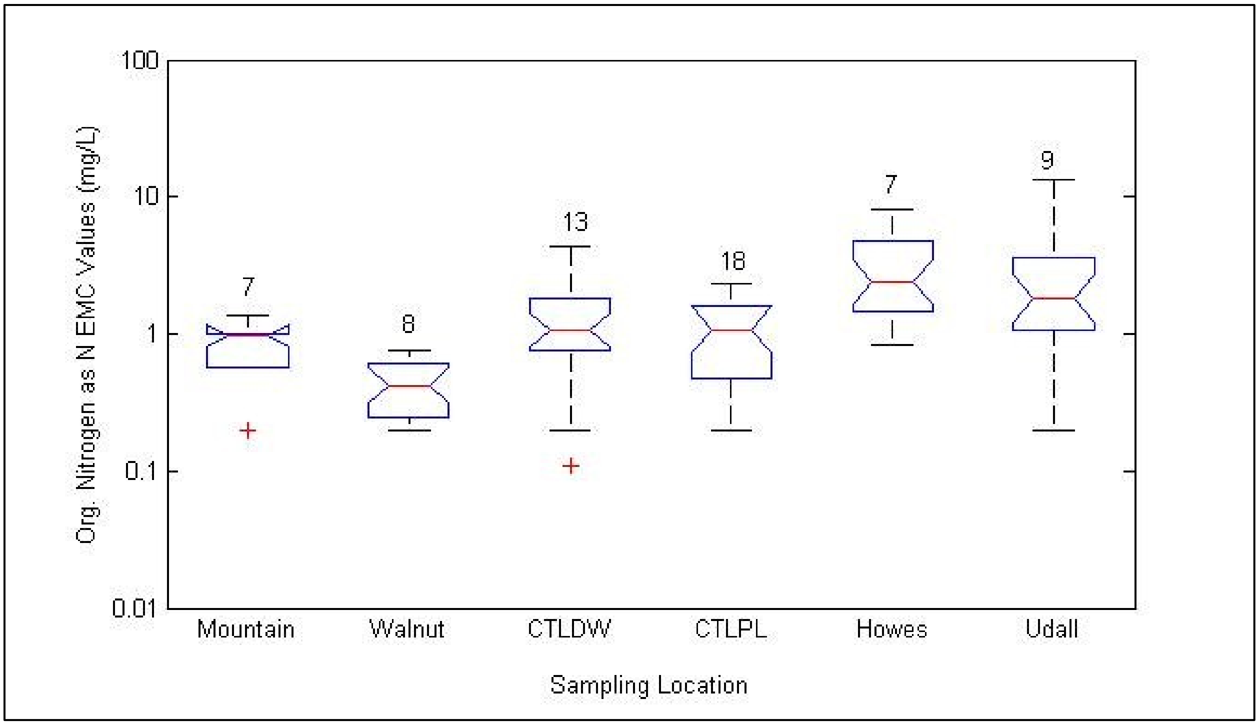

Figure 31: Boxplot for TON as N

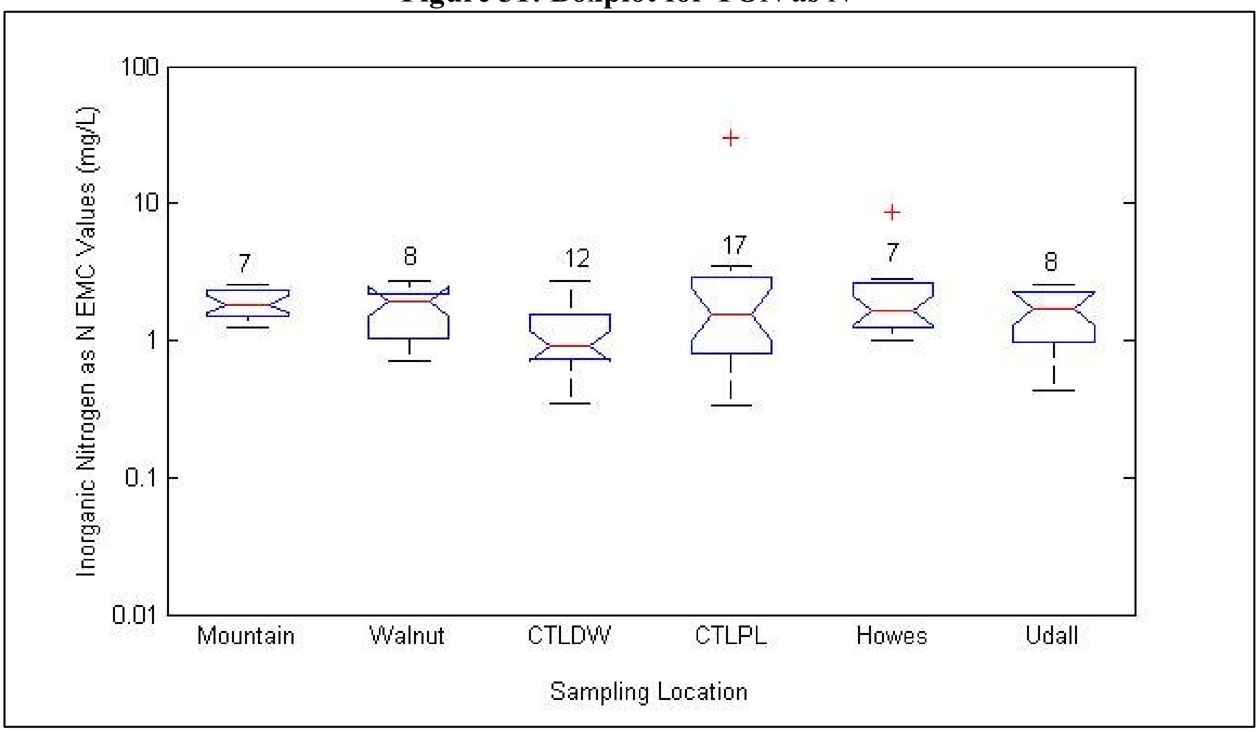

Figure 32: Boxplot for TIN as N

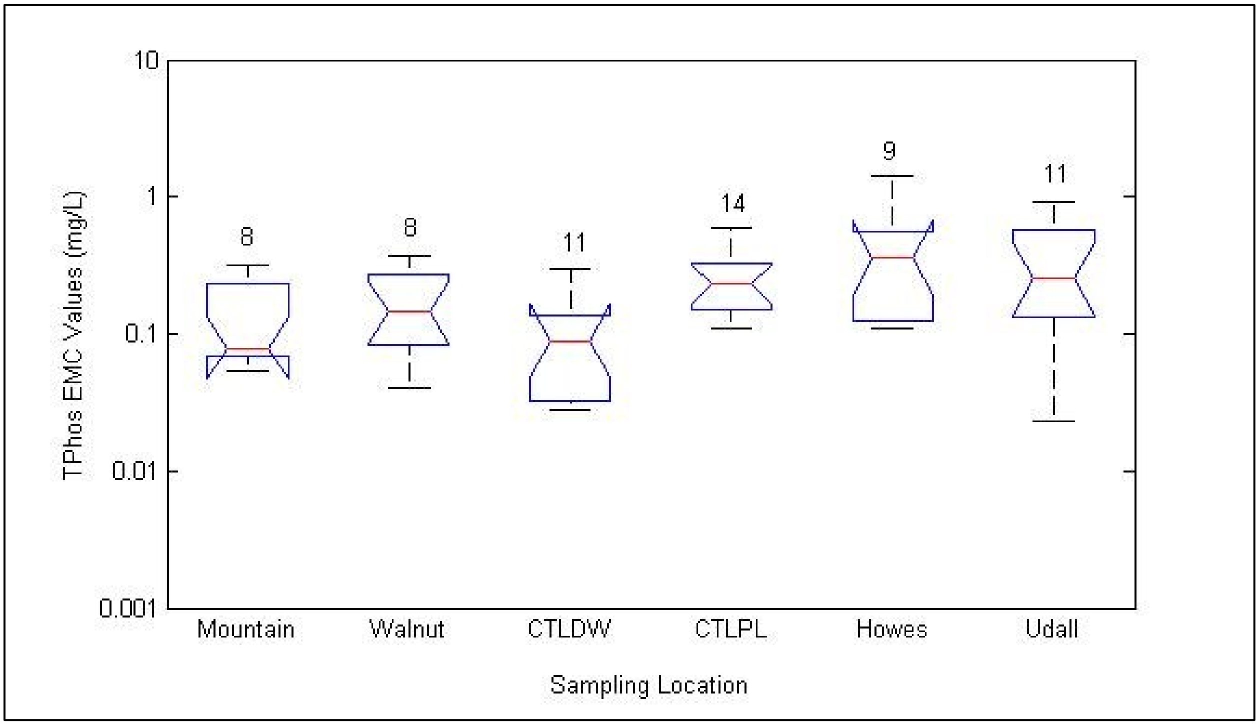

Figure 33: Boxplot for TP

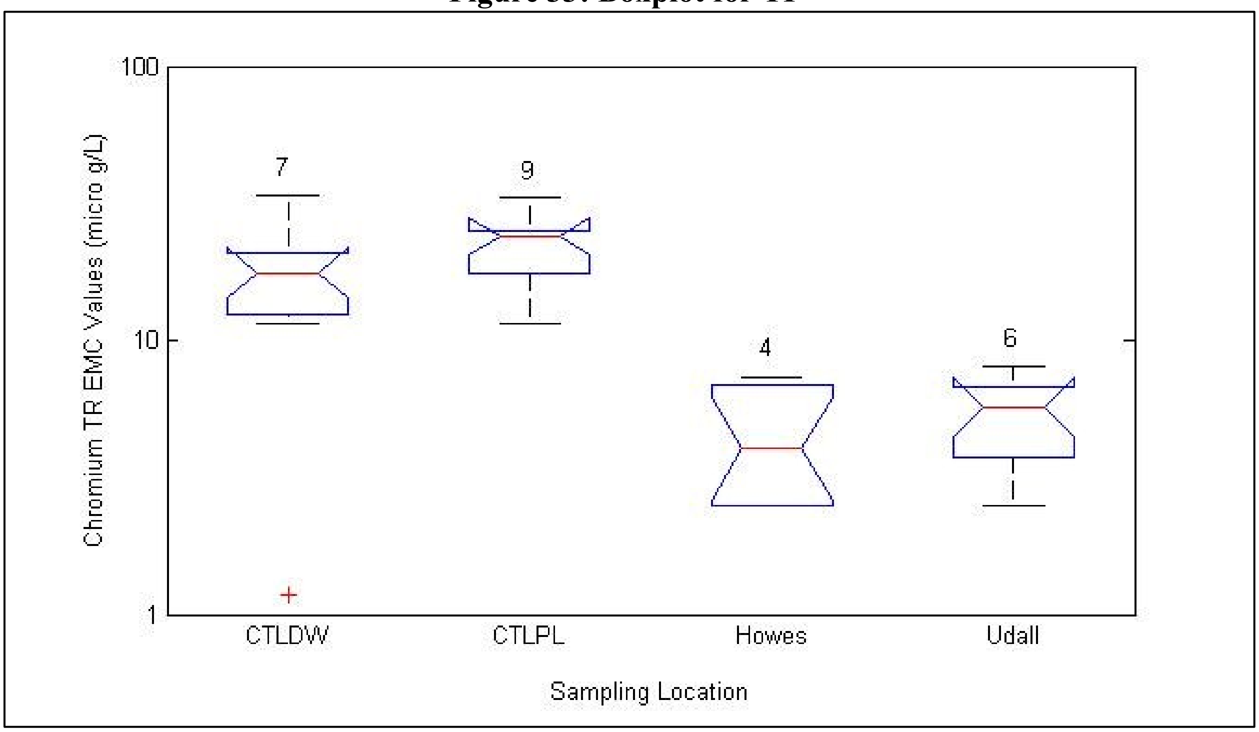

Figure 34: Boxplot for TR Cr

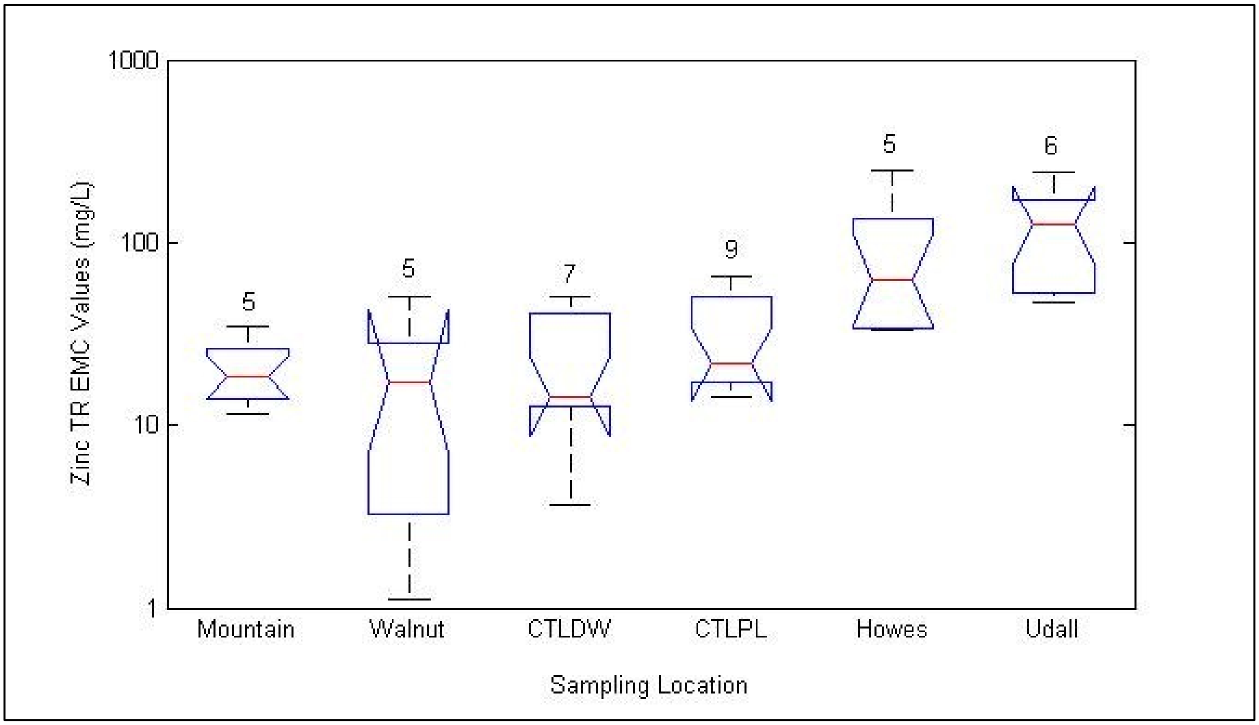

Figure 35: Boxplot for TR Zn

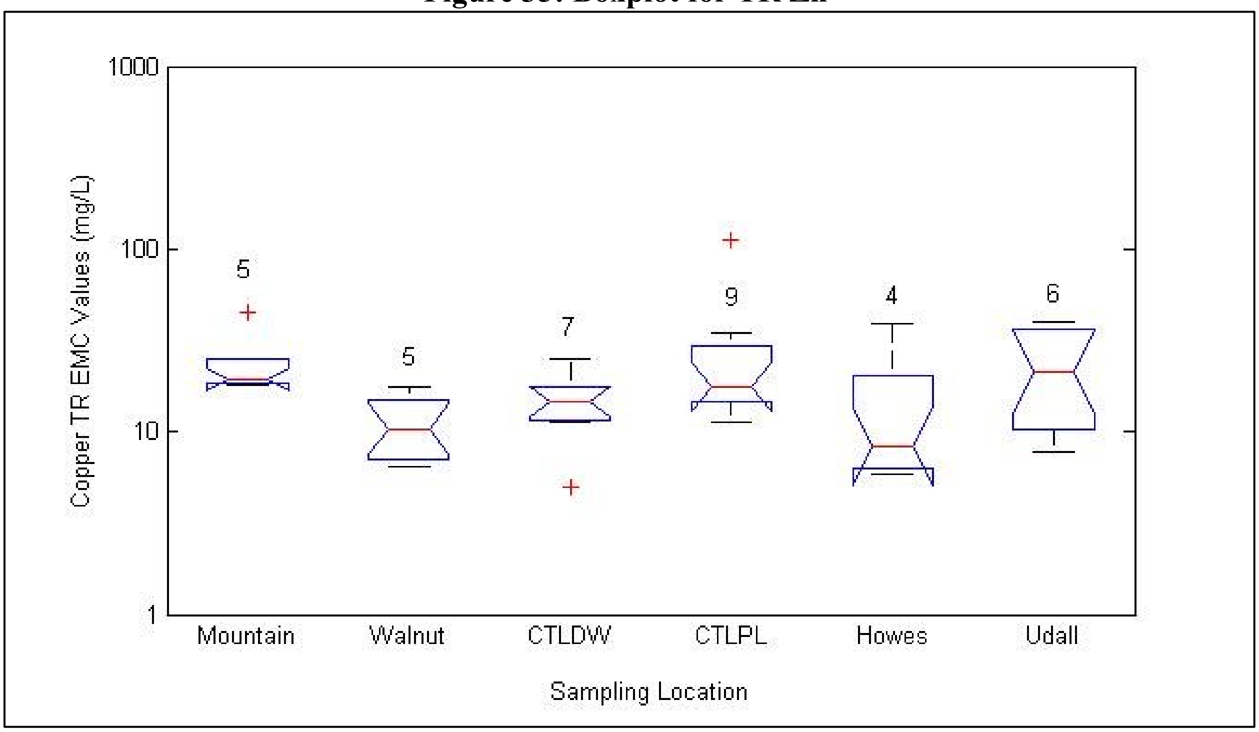

Figure 36: Boxplot for TR Cu

5.3 Water Quality Discussion

The following sections reference the outfall water quality data as a surrogate for influent concentrations to the PPS in this study. It is important to note that the Howes and Udall outfalls drain 524 acres and 517 acres, respectively, consisting of approximately 20% commercial, 70% residential and 10% open space. The PPS contributing areas consist of nearly all commercial area. Storm runoff water quality is often a function of the land use that comprises the drainage area. Correct interpretation of the results discussed below must fully consider the difference in the influent to the pavers and the surrogate data from the outfall sites used to calculate removal rates. Using traditional asphalt runoff data from the Database as a surrogate to calculate removal rates was considered, but the outfall data was used because the Mitchell Block sites and CTL are contained within the drainage area for Udall and are adjacent to the drainage area for Howes. In addition, this allowed for consistent climate and hydrology between the source surrogate and the PPS sites.

5.3.1 Solids

TSS is a common surrogate for pollutant removal efficiency, and is often used as a benchmark by regulatory agencies. The outfall data indicates that the PPS sample sites are providing markedly lower TSS values than what exists in typical Fort Collins stormwater. Figure 25 shows that TSS data for all four PPS sample sites are statistically less than the two outfall sites. Median EMC TSS concentrations at the four PPS sample sites in this study indicated between 76% and 92% removal based on the median EMCs at the outfall sites. The Mountain and Walnut plots do not offer evidence that the TSS datasets for the two sites are statistically different from one another. The sample sizes for TSS were among the largest for any parameter in this study. The lower median value for TSS at Walnut could be a result of the sand filter layer contained within the sub-base. The UDFCD criteria manual suggests that this layer is necessary for effective pollutant removal in PPS (UDFCD 2009).

The TDS median EMC at CTL PL is approximately 650 mg/l, which is more than double both of the Mitchell Block sites and almost double CTL DW. Figure 26 shows that TDS concentrations at the four PPS sample sites are significantly greater than the two outfall sites. In addition, CTL DW is significantly greater than the other three PPS sample sites. Mountain and Walnut do not appear statistically different from one another. It is important to note the small sample sizes at all of the sites, in particular the Mitchell Block sites. PPS are able to filter out suspended particles, but this data indicates a potential increase in dissolved solids in the exfiltrate.

5.3.2 Total Phosphorous

The median TP EMCs for the four sample sites in this study show that these systems are in the range of PPS represented by the Database median value of 0.1 mg/l. Figure 33 shows that CTL PL is statistically greater than Mountain and CTL DW. Mountain, Walnut and CTL DW are not statistically different from each other. Based on the median EMCs from the four PPS sample sites and the Udall TP data; TP removal ranged from 36% at CTL PL to 78% at Mountain. CTL DW and Walnut fell in between at 75% and 57% removal, respectively.

TP removal occurs in PPS mainly by removal of the particulate portion of TP through filtration and adsorption of dissolved P by the sub-base media. Dissolved P is effectively adsorbed by metal hydroxides, which aren't prevalent in the sand and gravel sub-base materials at the sites in this study. Adsorption is a reversible process, that is, once the adsorbed concentration in the sub-base media exceeds the dissolved aqueous concentration, it is possible for desorption of phosphorous to occur. Dissolved P data was collected for a small number of storms and thus was not analyzed for this study. Future research should continue collecting this data, along with TP data, to identify long-term trends and help understand the potential for desorption of P in these systems.

5.3.3 Nitrogen Species

TKN is a measure of nitrogen bound to organics and nitrogen in the form of NH_3. It represents the portion of nitrogen that is unavailable for biological uptake. The TKN boxplot in Figure 28 indicates that all four PPS sample sites appear to be statistically less than the two outfall sites.

NH_3 concentrations were low at all four PPS sample sites in this study. Mountain and Walnut both have median values of half the MDL, because 57% and 75% of the datasets were below the MDL, respectively, and substitution of half the MDL was used for the censored data. They were not included on the boxplot. Figure 29 shows that both of the CTL sites were statistically less than both of the outfalls. The median EMCs at the CTL sites were a full order of magnitude less than the outfall median EMCs. As discussed previously, some of this difference may be attributable to differing land uses in the outfall drainage areas and the PPS drainage areas.

NO_3 median EMCs were the highest at the Mitchell Block sites, both around 2 mg/l. Figure 30 shows that both Mitchell Block NO_3 datasets are statistically greater than both of the outfall sites. The CTL sites are not statistically different from the outfall sites. The nitrogen species data suggests that the Mitchell Block sites are converting NH_3 in the influent to NO_3 in the effluent. This is a common result in PPS sites (Bean *et al., 2007)*. NO_3 and the sub-base media particles are negatively charged, which allows nitrate to be extremely soluble. NH_3 is positively charged and is easily removed from solution by the sub–base media. The NH_3 is likely nitrified to NO_2 which is oxidized to NO_3 in an aerobic environment. This speciation is undesirable from a practical treatment standpoint, as NO_3 is more biologically available for plant uptake and promotes eutrophication.

TON was calculated by subtracting NH_3 from TKN. The four PPS sample site median EMCs were less than the outfall sites. The Mitchell Block sites were statistically less as shown on Figure 31. All of the PPS sample sites had median EMCs close to 1 mg/l except Walnut, which had a median EMC of 0.42 mg/l. Figure 32 shows the boxplot for TIN data. There are no statistical differences indicated between any of the datasets. From the NO_3 and NH_3 datasets, it is clear that virtually all of the TIN is in the form of NO_3 at the four PPS sample sites.

TN represents the sum of all nitrogen containing species (TKN, NO_2 and NO_3). The data shows median EMCs of 2.77, 2.17, 2.97 and 2.37 mg/l at CTL Pl, CTL DW, Mountain and Walnut respectively. The outfalls had higher EMCs of 3.5mg/l at Howes and 3.21 mg/l at Udall. Figure 27 shows that none of the PPS sample sites are statistically different from each other for TN. The short boxes for Mountain and Walnut indicate that the data had little variation, while the longer boxes at the CTL sites indicate slightly more variable data.

5.3.4 Total Recoverable Metals

Initially, five TR metal parameters were included in the analysis, but due to the high levels of non-detects in the Lead (Pb) and Cadmium (Cd) datasets, they were eliminated from detailed analysis. Table 13 below shows the percent non-detect values in each dataset for all of the sample sites.

Table 13, Percentage Non-detected Values for Lead and Cadmium

	Percent Non-Detect					
	CTL PL	CTL DW	Mountain	Walnut	Howes	Udall
Pb (TR)	83%	100%	100%	100%	33%	25%
Cd (TR)	83%	100%	100%	50%	67%	75%

Pb was not detected for any samples at Mountain, Walnut and CTL DW and was only detected in 17% of the samples at CTL PL. Cd was not detected in 83%, 100%, 100% and 50% of the samples at CTL PL, CTL DW, Mountain and Walnut, respectively. This data indicates that these two parameters are not a concern at any of the PPS sample sites.

All of the sites, except CTL PL and CTL DW, had median EMCs for TR Cr near or below the MDL due to high percentages of non-detects. Median EMCs for CTL PL and CTL DW were 24 and 17.57 μg/l, respectively. Figure 34 indicates that the TR Cr EMCs at the CTL sites are significantly larger than the outfall EMCs. The Mitchell Block sites were not included in the boxplots because of the high percentage of censored data points. The high values at CTL are likely related to the PC material. Data from various PPS sites in the Database showed low chromium concentrations similar to the Mitchell Block sites, but none of the studies looked at PC sites. A study on traditional concrete runoff found elevated dissolved Cr concentrations and identified the Portland cement mixture as the source of Cr (Kayhanian et. al., 2009). It is reasonable that dissolved Cr is being leached into the runoff passing through the PC layer in the CTL lot. Hexavalent chromium (VI) is especially toxic to humans. The makeup of the Cr found at the sites was not investigated, but should be addressed in future monitoring efforts, particularly because this site recharges the groundwater system and may be a potential source for contamination. For reference, California has a public health goal in drinking water of 2 μg/l, about ten times less than the concentrations found at the CTL sites. This trend should be investigated further.

The Mitchell Block sites had median TR Zn EMCs of 18.4 μg/l and 17.3 μg/l approximately equal to the Database median of 18 μg/l. The CTL sites had median EMCs of 14.2 μg/l and 21.7 μg/l for CTL DW and CTL PL, respectively. The outfalls had higher median

EMCs of 62.8 μg/l and 126.5 μg/l at Howes and Udall, respectively, about 70% to 85% higher than the PPS sample sites. All four PPS sample sites in this study are effectively removing TR Zn, based on the outfall data as a surrogate for influent. Figure 35 provides evidence that all of the PPS sample sites, except Walnut, are statistically less than the outfalls. The boxplot for Walnut is especially long, indicating a wide variability in data.

TR Cu median EMCs were 17.65, 14.61, 19.39 and 10.37 μg/l at CTL PL, CTL DW, Mountain and Walnut, respectively. These were much higher than the Database median value reported at 6 μg/l. Median EMCs at Howes and Udall were 8.6 and 21.7 μg/l, respectively. Figure 36 shows that Mountain is statistically larger than Walnut, CTL PL and Howes outfall. As with TR Zn, the Mitchell Block sites only consist of five data points and at Mountain one of those points is an outlier, indicated by the red plus mark above the box. A few studies have shown that PPS are effective at removing TR Cu from stormwater (Brattebo and Booth 2003; Rushton 2001). The data from this study does not indicate substantial removal and in some cases the PPS median EMCs are larger than the surrogate influent values at the outfalls, suggesting that these systems are not adequate for removal of TR Cu, or that the outfall sites are not a representative surrogate for influent to the PPS. Future monitoring should investigate Fort Collins street runoff concentrations to give better context to these results..

5.3.5 Bacteria

Under the Clean Water Act, the EPA's ambient water quality criterion is a geometric mean of 126 E. coli per 100 mL. E. coli is used as an indicator for total fecal coliforms and is particularly important because of associated health risks. The geometric means at CTL DW and CTL PL were 29 and 28 E. coli per 100 mL, respectively and ranged between 10 and 100 E. coli per 100 mL at both sites. At CTL DW, all 10 samples were non-detects and at CTL PL, 12 of 15 samples were non-detects. Mountain ranged between 10 and 6,800 E. coli per 100 mL with a geometric mean of 125 E. coli per 100 mL, with 10 of the 16 samples being non-detects. Walnut ranged from 10 to 560 E. coli per 100 mL with a geometric mean of 62 E. coli per 100 mL, with 10 of the 15 samples being non-detects. Howes and Udall outfalls had geometric means of 3,150 and 3,661 per 100 mL, respectively. The Database does not have E. coli results for PPS due to insufficient datasets.

All of the PPS sample sites had over 60% non-detect values in their datasets. As discussed in the methods section, the non-detect values were adjusted based on sample dilution. From the data it is clear that Mountain had the highest values from the four sample sites, and the lowest percentage non-detects (62.5%). Walnut had a geometric mean half of Mountain's, indicating that Walnut is providing a higher level of bacteria removal. Because the main treatment mechanism is filtration of the particulate portion of bacteria, this result could be related to the sand filter layer in the Walnut sub-base. The CTL data shows that E. Coli does not pose any issues at this site. This may be a function of both the treatment provided and low concentrations in the influent.

5.4 Conclusions

The data presented in this section indicate that the PPS sample sites evaluated appear to be providing treatment for most of the parameters investigated. Several parameters were identified that were not being treated to sufficient levels namely; TR Cr at the CTL sites, TR Cu at all four PPS sample sites and NO3 at the Mitchell Block sites. The TDS EMCs were also elevated at all four PPS sample sites. The bullets below discuss the notable trends and results from the analysis presented above:

- TSS removal ranged between 75% and 93% at the four PPS sample sites based on the outfall data as an influent surrogate and EMCs for all four PPS sample sites were significantly less than the outfall site EMCs based on box plots
- TP removal appears to be occurring at all four PPS sample sites, with removal ranging between 36% and 78% based on the Udall outfall data as an influent surrogate
- Mountain and CTL DW had the lowest TP median EMCs and CTL PL had the highest
- There does not appear to be significant levels of TN removal, although all four of the median EMCs for PPS sample sites were lower than the outfalls
- The Mitchell Block sites had elevated NO_3 median EMCs relative to the outfall sites due to nitrification of NH_3, which is not desirable as the NO_3 species is more readily available for biological uptake than TON and NH_3
- TR Zn datasets for all four PPS sample sites were significantly less than the datasets obtained from the outfalls, based on boxplots, indicating high removal rates by all of the PPS sites
- High TR Cu concentrations were an issue at all of the PPS sample sites in this study, with all four PPS sample sites having larger median EMCs than the Udall outfall site
- The high TR Cr values at CTL appear to be a consequence of the PC material leaching Cr into the exfiltrate, which is a potential health risk due to the toxicity of hexavalent Cr VI, future PC designs should consider that the exfiltrate may contain elevated TR Cr concentrations

- All four PPS sample sites had geometric average *E. coli* values less than the EPA criterion of 126 *E. coli* per 100 mL, and between 96% and 99% less than the outfall sites, indicating that E. coli does not appear to be a cause for concern at any of the sites

6.0 CONCLUSIONS AND RECOMENDATIONS

6.1 Surface Infiltration Tests

Surface infiltration tests were conducted at all three PPS sites in this study. Evidence of clogging was apparent at all of the sites. The average infiltration rates were 2.64, 5.62 and 1.04 in/hr at Walnut, Mountain and CTL, respectively.

The test data indicate that maintenance is necessary at all three sites to improve surface infiltration rates and extend the life of these systems. CTL showed the greatest maintenance need with two of the three tests indicating a completely clogged surface. Also, during numerous events flooding of the southeastern section of the parking lot at CTL was observed, likely due to the clogged sections of the PC parking lot. It is recommended that a wet vacuum street sweeper (non-brush) is used for maintenance based on discussions with UDFCD.

The data at Mountain indicates that sections of the pavers are relatively unclogged, with the largest infiltration rate exceeding 13 in/hr. The other two tests at Mountain yielded infiltration rates less than 2 in/hr. This indicates the need for maintenance in certain sections of the pavers. Walnut had similar results, with the largest infiltration rate greater than 4 in/hr and the other two at or below 2 in/hr, also indicating the need for maintenance. It is recommended that a dry vacuum street sweeper (non-brush) is used at the Mitchell Block sites based on discussions with UDFCD.

All three sites should be maintained on a regular basis (at least one time annually) to extend the life of the systems and ensure proper functionality. These infiltration tests should also be carried out on a regular basis, particularly after maintenance activities to document any improvement in the infiltration rate.

6.2 Runoff Reduction Performance

The flow data collected at the Mitchell Block sites showed that the "no-infiltration" permeable paver designs are capable of reducing runoff by nearly 50% per storm on average. Walnut reduced runoff in 94% of the monitored storms (17 of 18) with a median of 0.17 inches of runoff reduction per storm. Mountain reduced runoff in 73% of monitored storms (11 of 15) with a median of 0.24 inches of runoff reduction per storm. The maximum runoff reduction at Mountain was 0.98 inches and the maximum runoff reduction at Walnut was 0.66 inches. The aggregate total runoff reduction for all of the monitored events was 4.13 inches and 4.09 inches at Mountain and Walnut, respectively. It is clear from these data that both of the designs are capable of reducing storm runoff volumes for individual storms and over long periods of time. Planners and developers should be confident that the "no-infiltration" PPS designs utilized in this study will achieve runoff reduction. Monitoring should continue and future analysis should focus on optimizing the sub-base design to increase evaporation potential and field capacity to allow for greater runoff volume reduction.

Several storms during this study appeared to produce slightly negative runoff reduction at Mountain. It was noted that there was a section of pavers that flooded during some of these storms from upstream runoff. This flooding may have caused the appearance of negative runoff reduction in the results. It is unknown how much this affected the results presented in this document, but future monitoring efforts at this site should address this issue to eliminate this variable.

6.3 Water Quality Performance

No influent data was collected at any of the sample sites in this study. To assess pollutant removal, data from two Fort Collins outfall sites were used as an influent surrogate. Statistical significance was determined with box plots. Effluent data for PPS from the Database were used as a comparison metric for the sites in this study.

All four PPS sample sites had statistically lower concentrations of TSS than the two outfall sites. Median EMC TSS concentrations at the four PPS sample sites in this study indicated between 76% and 92% removal based on the median EMCs at the outfall sites. The Mitchell Block sites had the lowest median TSS EMCs, with Walnut at 9.5 mg/l and Mountain at 15 mg/l. The median EMC reported by the Database was 14 mg/l. CTL DW had the highest median TSS EMC at 26 mg/l, still 75% less than the lowest outfall EMC. These data indicate that all four sample sites are removing significant levels of solids. Three of the four sites achieved greater than 80% TSS removal, a criteria specified by many states for standalone BMPs. This result alone indicates that these systems are a viable option for urban stormwater applications.

TN median EMCs were lower than the outfall sites at all four PPS sample sites. CTL DW had the lowest median EMC at 2.17 mg/l and was statistically less than the Howes outfall. None of the other sites indicated any statistically significant differences. Mountain had the largest median value at 2.97 mg/l. Removal rates, based on the outfall data, ranged from 7% to 38%. No results were reported for PPS from the Database. In areas where TN removal is a priority, these systems should either be designed specifically to improve TN removal or be used in sequence with another BMP as part of a treatment train.

TP median EMCs at the four PPS sample sites were lower than the outfalls. CTL DW and Mountain had median EMCs of 0.089 mg/l and 0.079 mg/l, respectively. These are both below the reported median from the Database data at 0.1 mg/l. Walnut had about twice the median EMC of Mountain at 0.151 mg/l. Removal rates ranged between 36% and 78% based on the Udall outfall data. Mountain and CTL DW were both above 70% removal while Walnut was above 50% removal and CTL PL was the lowest at 36%. Despite the high removal provided by CTL DW, the low removal at CTL PL indicates that this system does not consistently provide high removal rates. In the full infiltration application of this system the exfiltrate is not directly discharged downstream and therefore these TP removal rates are sufficient. Both of the Mitchell Block sites are removing large amounts of TP (over 50% based on outfall data) and should be considered viable options for urban BMP applications with regard to TP removal.

The four PPS sample sites had TR Zn median EMCs between 60% and 90% less than the two outfall sites, ranging between 14.2 μg/l and 21.7 μg/l. The Database reports a TR Zn median EMC of 18 μg/l, indicating the systems in this study performed comparable to the results from the Database. The high removal rates indicated from the outfall data suggest that all three PPS in this study could be sufficient for standalone BMP applications with regard to TR Zn removal.

Median TR Cu EMCs were high at all four of the sample sites, ranging from 73% to 225% greater than the Database median of 6 μg/l. In addition, all of the sites had median TR Cu EMCs greater than Howes outfall and only between 10% and 50% less than Udall outfall. These data indicate that none of the sites in this study are sufficiently removing TR Cu from stormwater. It is important to note that the datasets for TR CU are small, and may not be completely representative of site performances. It is recommended that monitoring continue at these sites to gather a sufficiently large dataset to evaluate in addition to monitoring Fort Collins street runoff to more accurately evaluate the influent to the PPS as the outfall sites may not be representative. If this result holds true, future applications should compensate for low TR Cu removal with design modifications.

Average bacteria concentrations were all below the EPA's ambient water quality criterion of 126 E. coli per 100mL. Walnut, CTL DW and CTL PL had geometric average concentrations of 62, 29 and 28 E. coli per 100 mL, respectively, and all had greater than 65% non-detect samples. Mountain had a geometric average of 125 E. coli per 100 mL, just below the EPA standard. Even so, 63% of the samples were non-detects at Mountain. In addition, the outfalls had values over 3000 E. coli per 100 mL. If these values correspond to the influent of the pavers, then removal ranges from 96% to 99%.

6.4 Alternative Mitchell Block Designs

The Mitchell Block sites used two different designs for the sub-base layers so they could be compared using the water quality and water quantity data obtained. Walnut employs the UDFCD recommended design, which utilizes a sand filter layer below two gravel layers. Mountain uses the Advanced Pavement Technology recommended design which consists of three gravel layers coarsening downward through the sub-base.

The water quantity analysis indicated that Mountain provided 23% more runoff reduction per storm on average, for storms monitored at both sites. Mountain reduced the aggregate total runoff by 25% more than Walnut for storms monitored at both sites. Walnut, however, provided runoff reduction in 94% of the monitored storms and Mountain only reduced runoff in 73% of monitored storms. These results indicate that Mountain provides greater runoff volume reduction. In applications of "no-infiltration" PPS designs where runoff reduction is seen as a priority, it is recommended that the Mountain design be utilized.

Only three of the water quality parameters analyzed showed significant differences between the two sites: TKN, TON and TR Cu. All of which indicated that the Walnut datasets were statistically less than those at Mountain. In addition, Walnut had lower median EMCs for

TSS, TN, TR Zn and E. coli. Mountain had lower median EMCs for TDS, TP and TR Cr. Walnut provided better water quality for the majority of the parameters analyzed.

Both UDFCD and Advanced Pavement Technology recommended designs are viable stormwater BMPs options in urbanized applications, with regard to water quality and water quantity performance. The runoff volume reduction performance exceled at Mountain, but the water quality results suggest that Walnut provides higher removal rates for a greater number of pollutants. UDFCD recommends this design because data have previously shown that systems with a sand filter layer are able to provide better water quality results. This study reinforces those findings for most of the pollutants tested. Future research should implement variations on both designs to optimize performance. Future PPS applications comparing these designs should consider the specific performance needs for the site and proceed accordingly.

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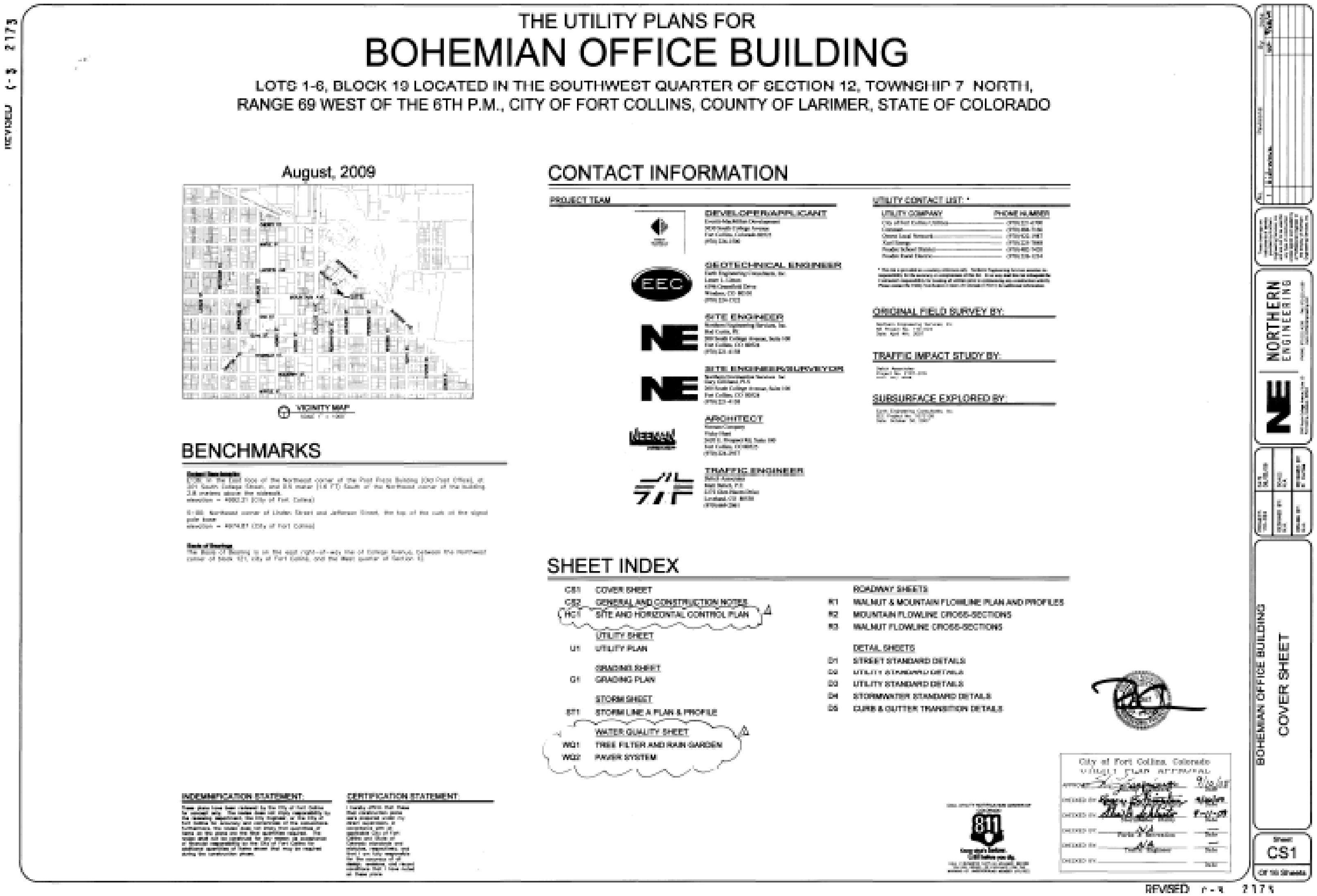
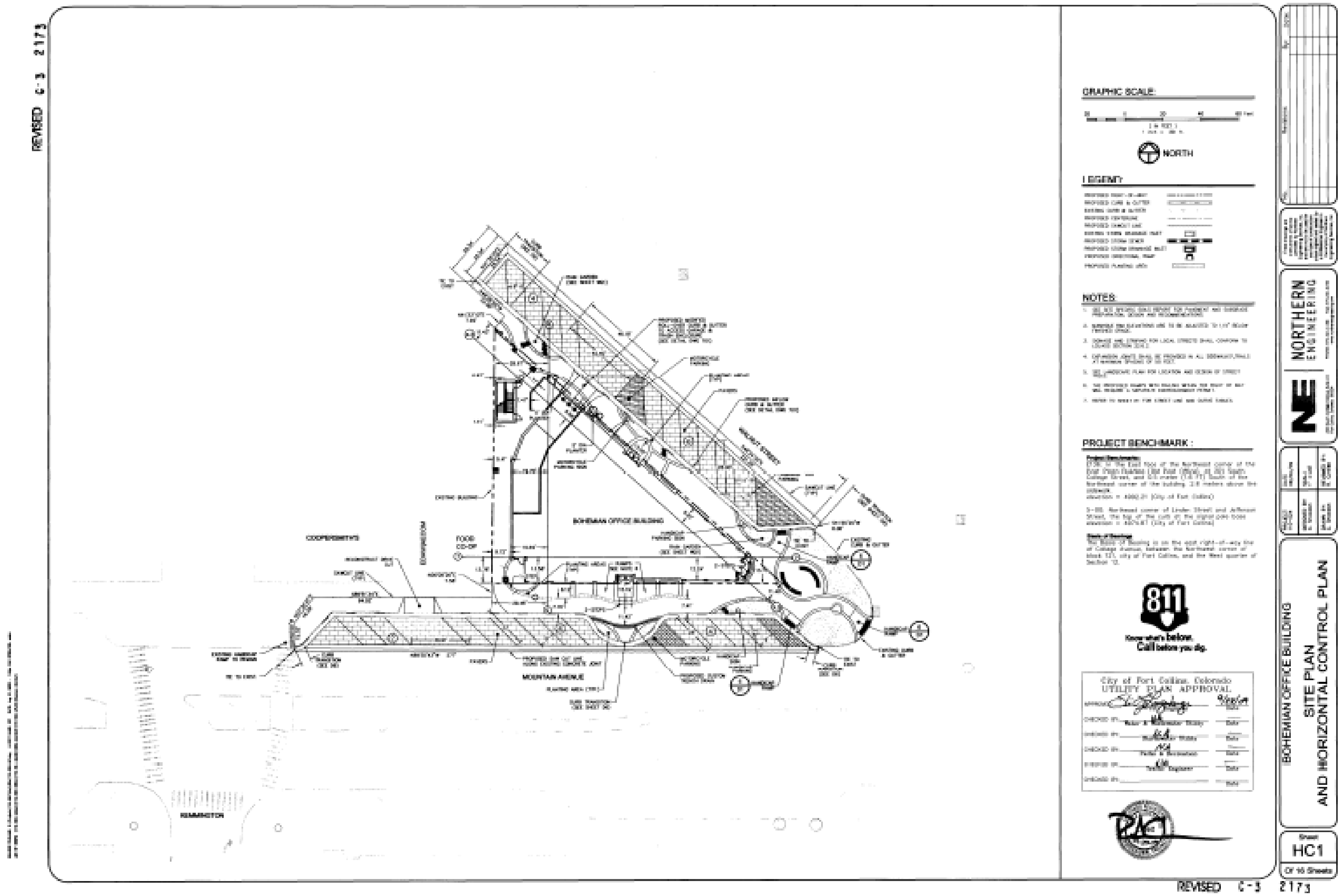
Appendix A: Plan Drawings

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LEGEND:
NOTES:
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BOHEMIAN OFFICE BUILDING
COOPERSMITH'S
FOOD CO-OP
MOUNTAIN AVENUE
REMINGTON
City of Fort Collins, Colorado
UTILITY PLAN APPROVAL
Know what's below. Call before you dig.
811
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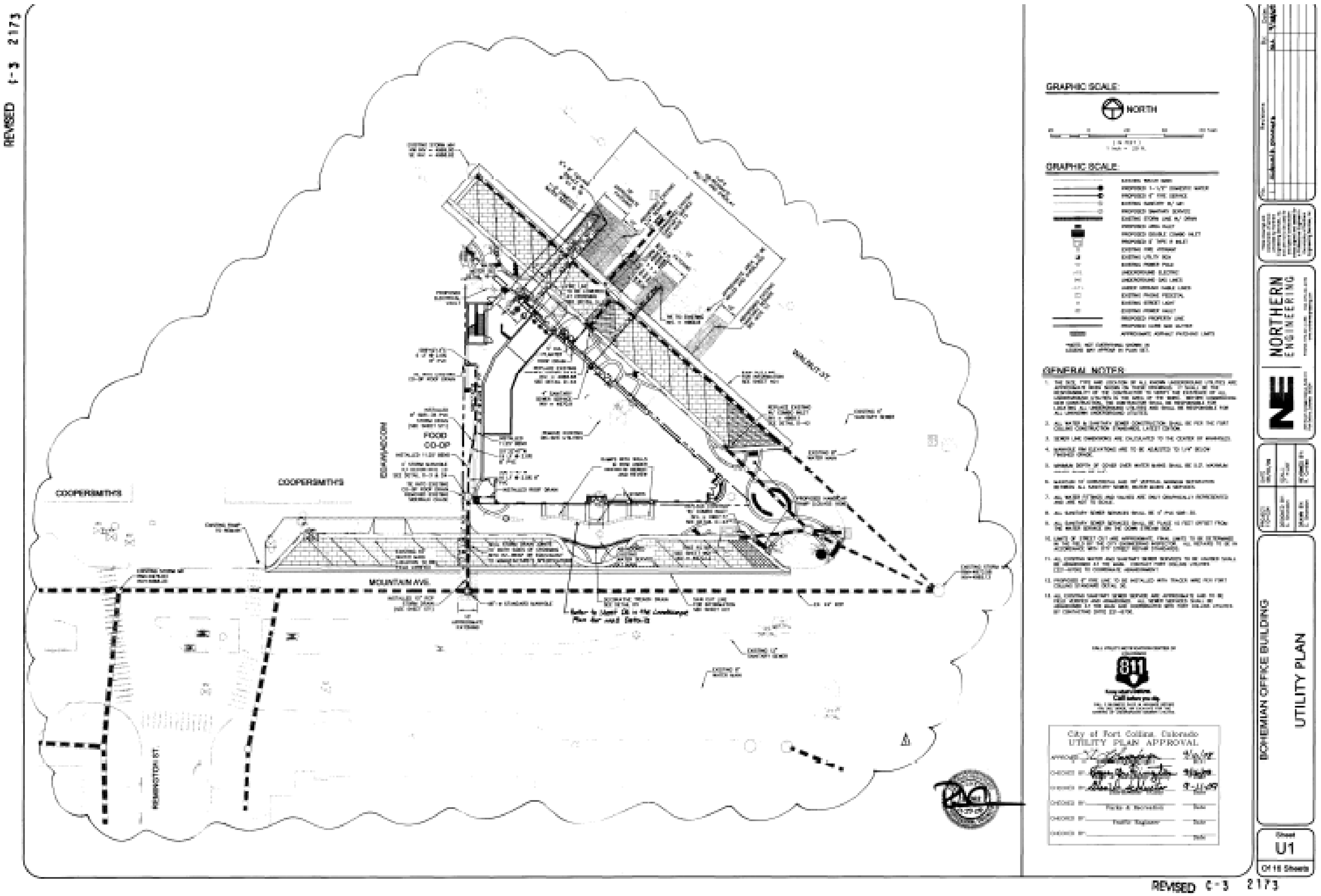
Sheet
HC1
Of 16 Sheets
REVISED C-3 2173



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NORTH
GRAPHIC SCALE:
GENERAL NOTES:
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COOPERSMITH'S
COOPERSMITH'S
FOOD CO-OP
MOUNTAIN AVE.
WALNUT ST.
REMINGTON ST.
City of Fort Collins, Colorado
UTILITY PLAN APPROVAL
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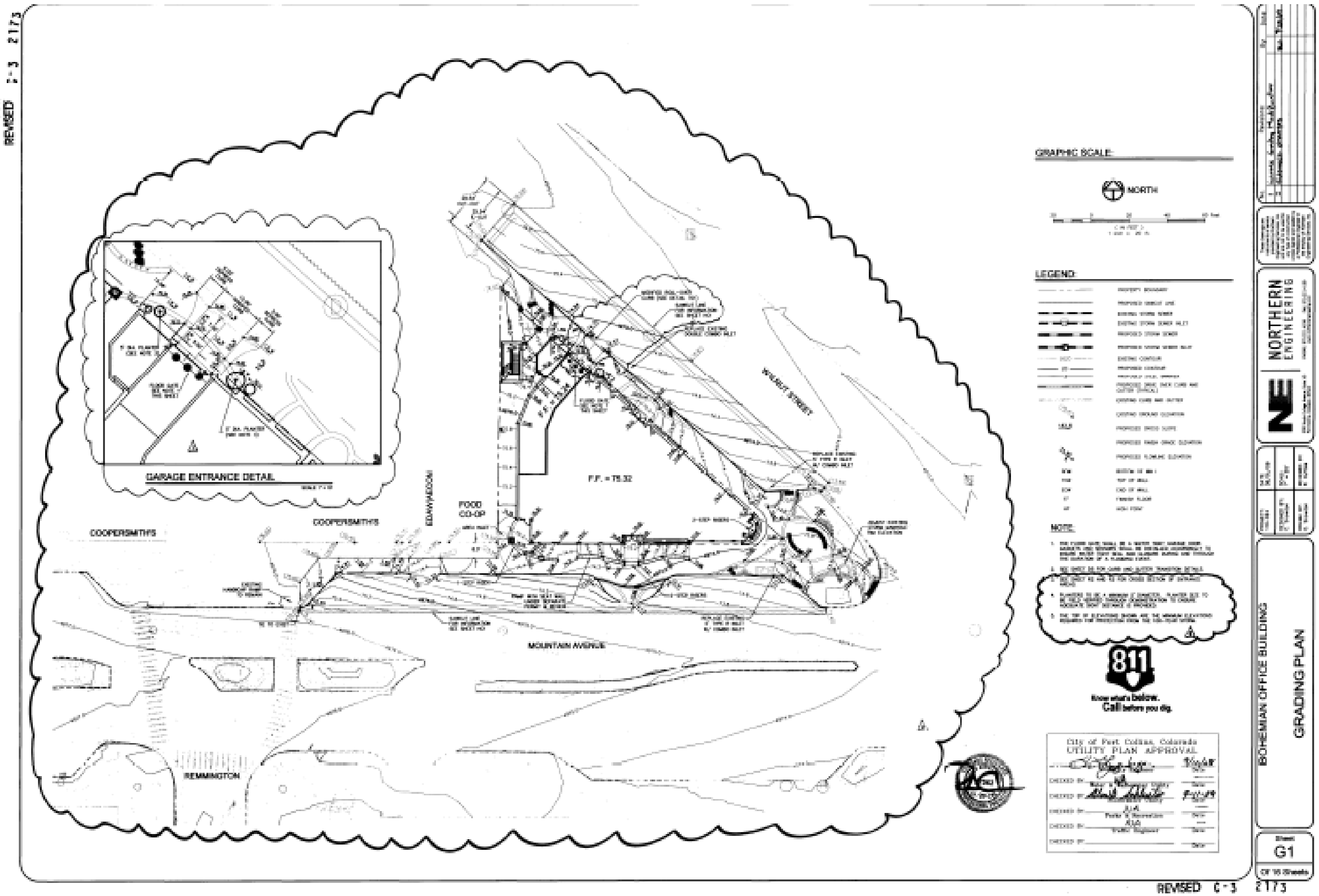
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F.F. = 75.32
MOUNTAIN AVENUE
REMMINGTON
City of Fort Collins, Colorado
UTILITY PLAN APPROVAL
Parks & Recreation
Traffic Engineer
811
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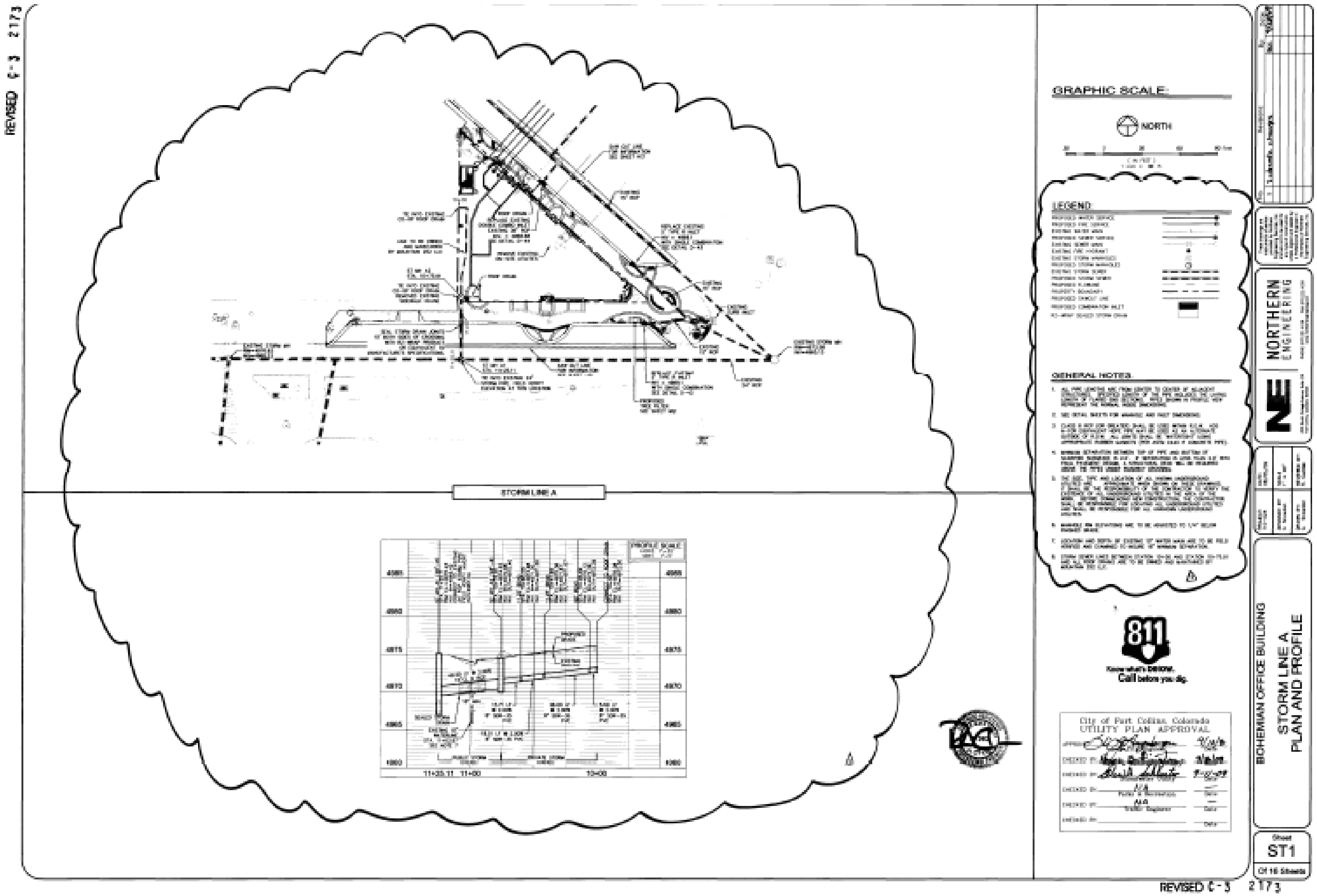

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LEGEND
GENERAL NOTES
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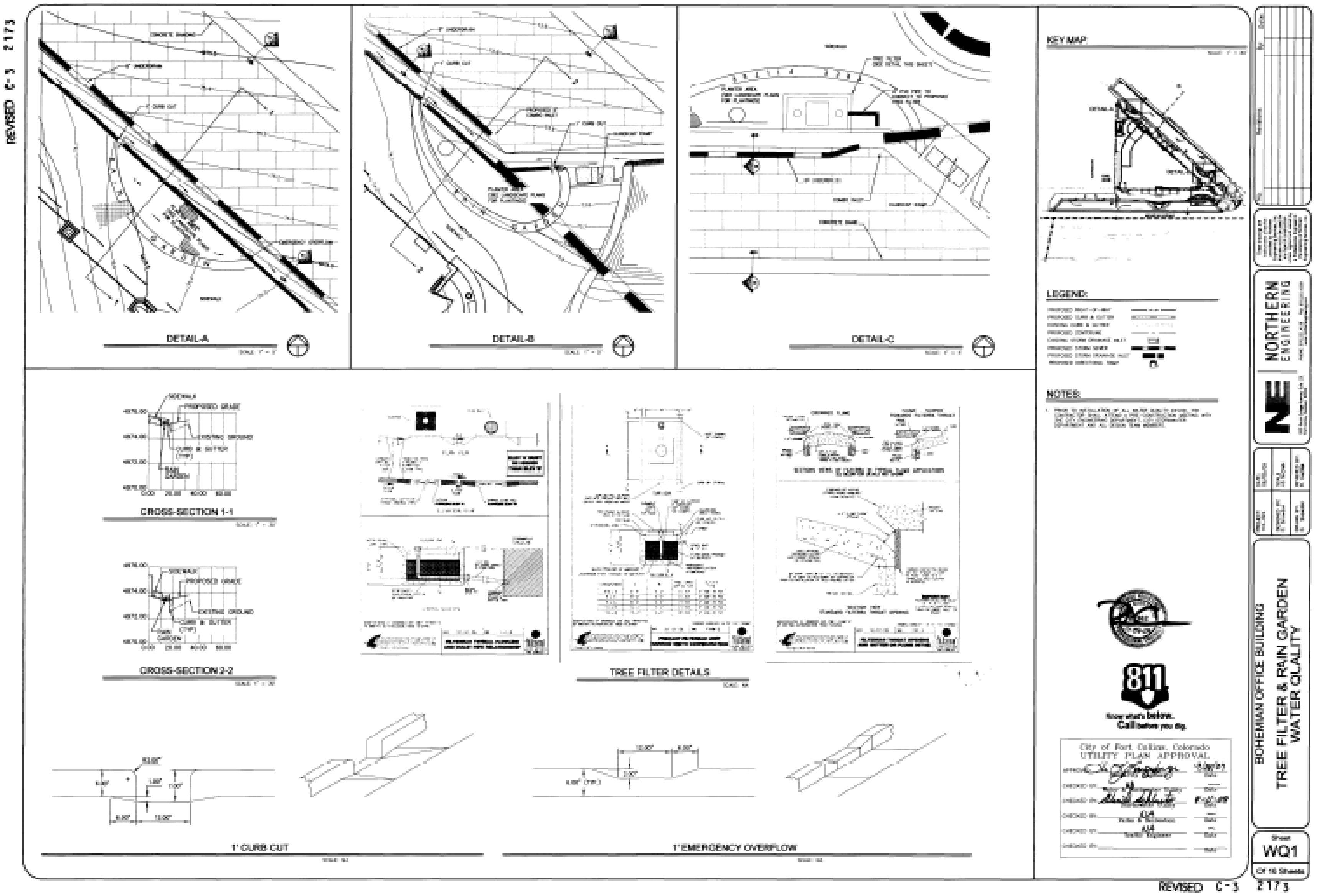
ST1
Of 16 Sheets
REVISED C-3 2173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DETAIL-B
DETAIL-C
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CROSS-SECTION 2-2
TREE FILTER DETAILS
1' CURB CUT
1' EMERGENCY OVERFLOW
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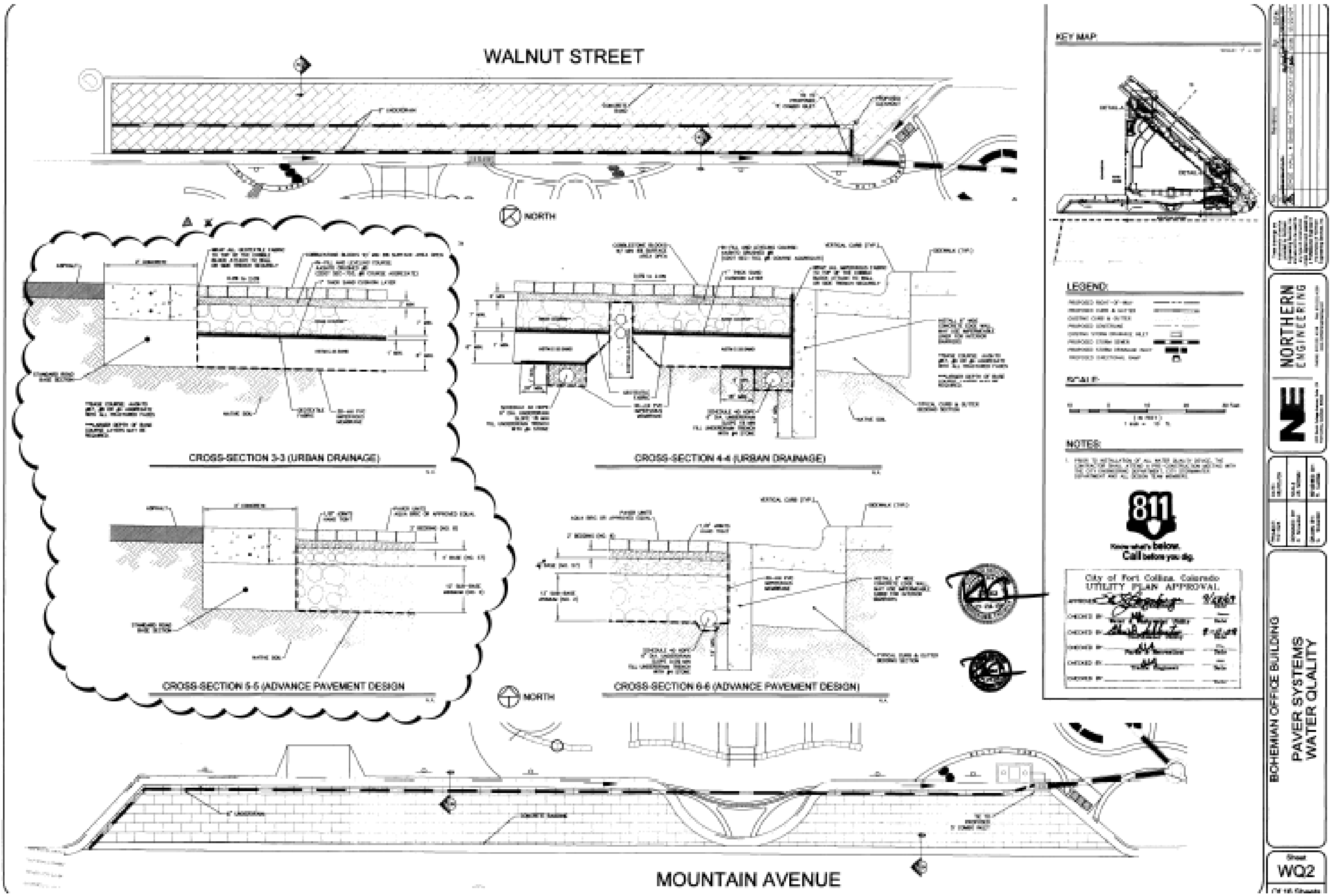
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CROSS-SECTION 5-5 (ADVANCE PAVEMENT DESIGN)
CROSS-SECTION 6-6 (ADVANCE PAVEMENT DESIGN)
NORTH
MOUNTAIN AVENUE
KEY MAP
LEGEND:
SCALE:
NOTES:
811
Know what's below.
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WATER QUALITY
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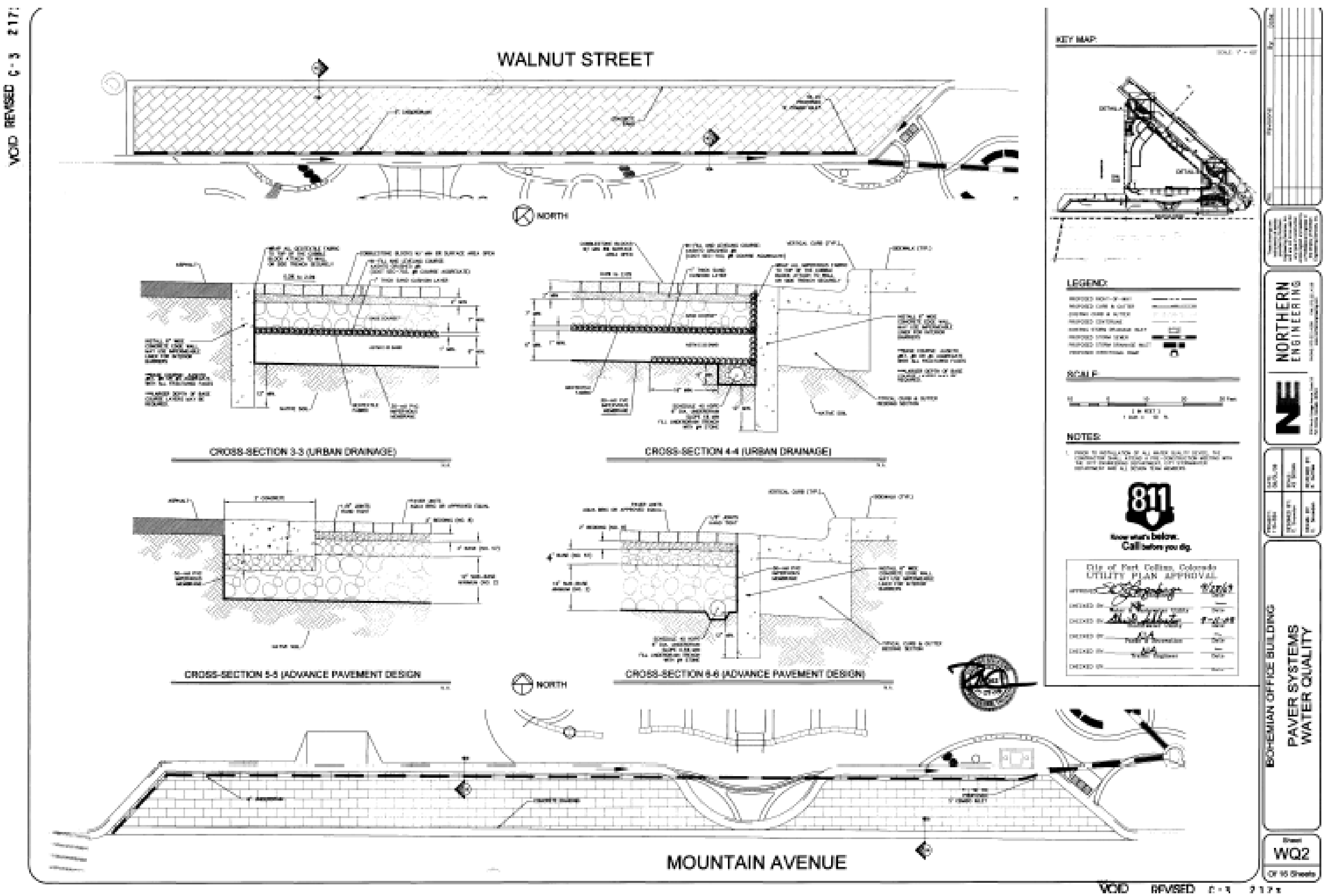

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CROSS-SECTION 5-5 (ADVANCE PAVEMENT DESIGN
CROSS-SECTION 6-6 (ADVANCE PAVEMENT DESIGN)
NORTH
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KEY MAP
LEGEND
SCALE
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Know what's below.
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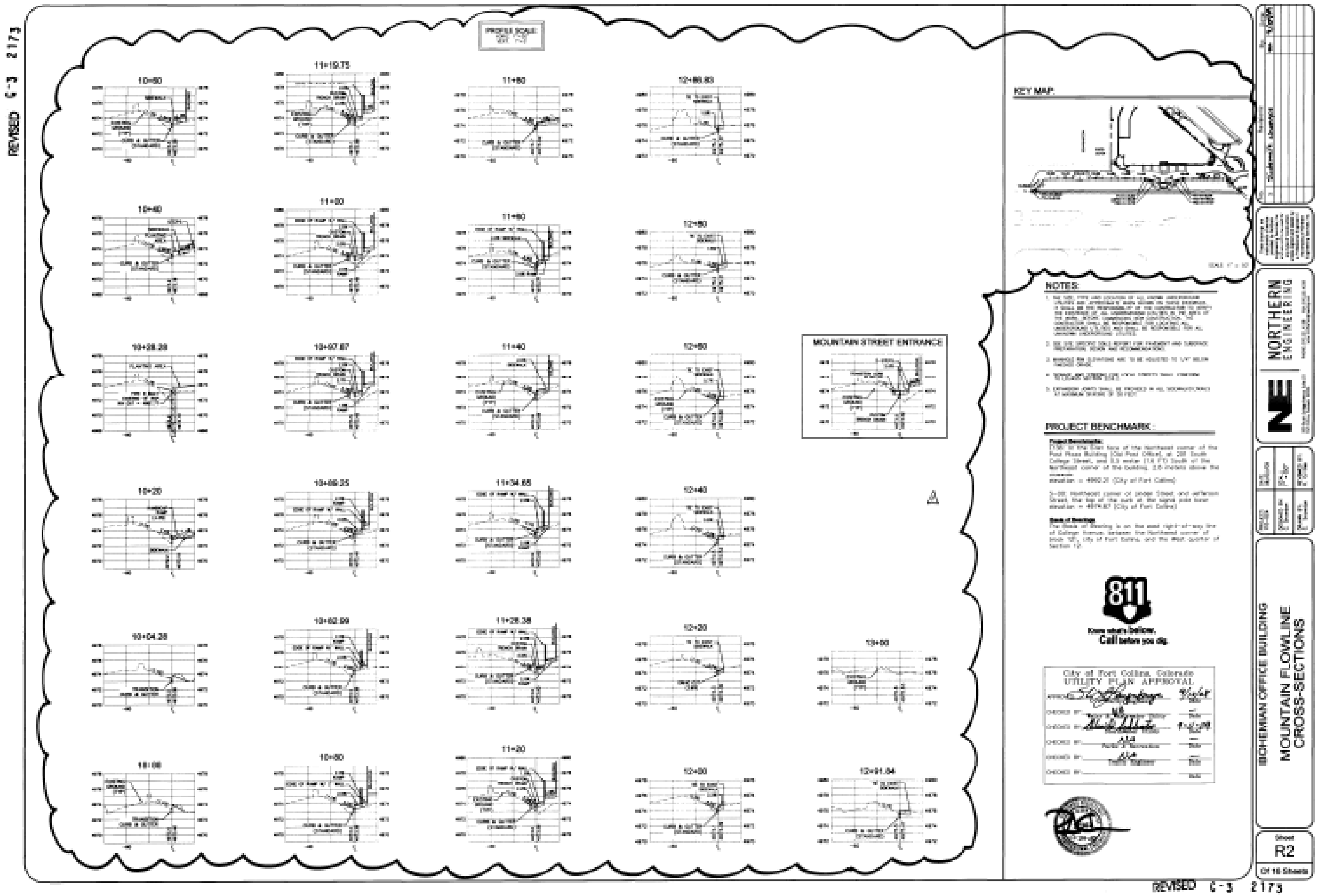
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11+60
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11+60
12+60
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10+97.67
11+40
12+60
MOUNTAIN STREET ENTRANCE
10+20
10+69.25
11+34.65
12+40
10+04.28
10+62.99
11+28.38
12+20
13+00
10+00
10+60
11+20
12+00
12+91.84
KEY MAP
NOTES
PROJECT BENCHMARK
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Know what's below. Call before you dig.
City of Fort Collins, Colorado
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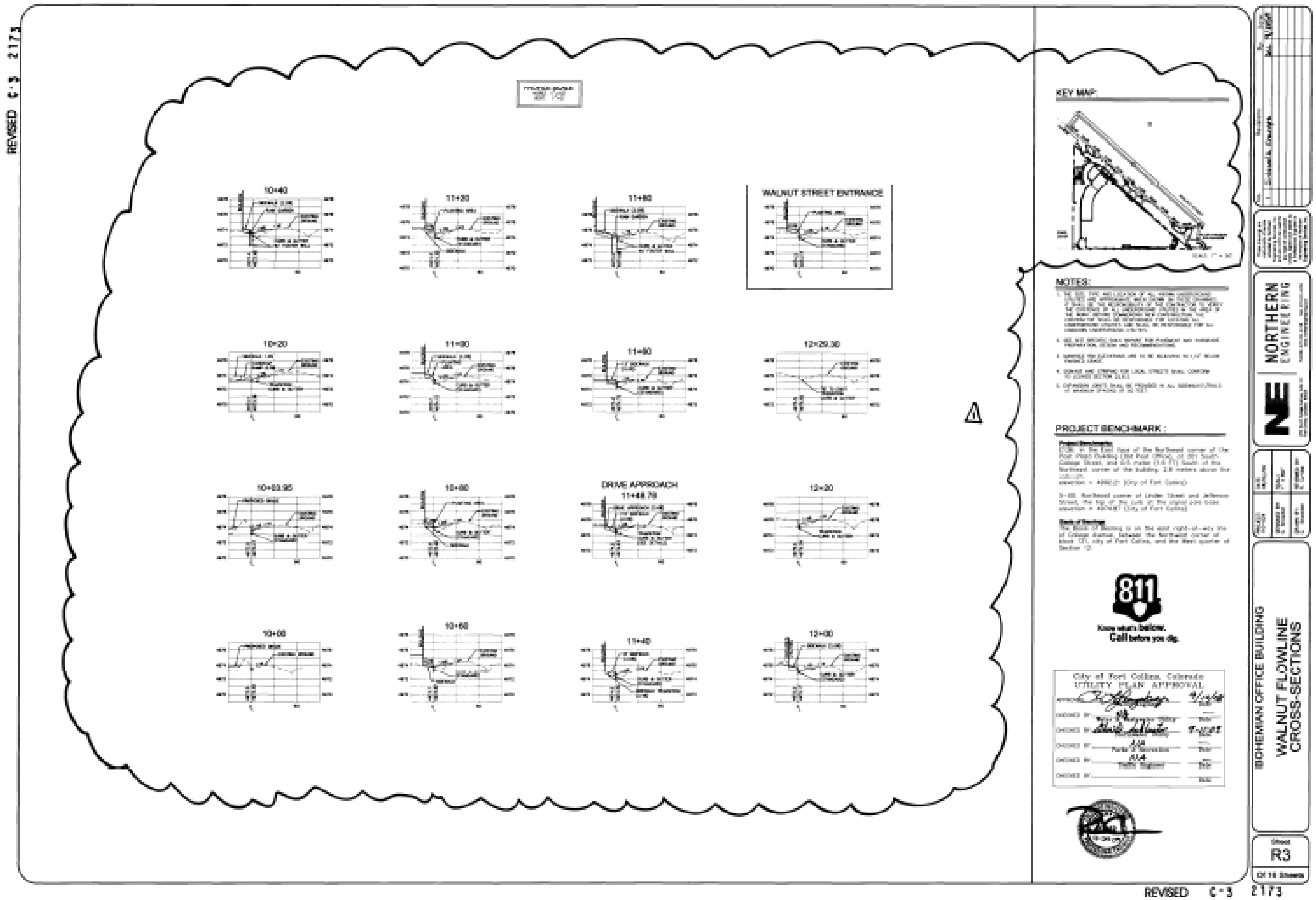

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DRIVE APPROACH
11+48.78
12+20
10+00
10+60
11+40
12+00
KEY MAP:
NOTES:
PROJECT BENCHMARK:
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Know what's below.
Call before you dig.
City of Fort Collins, Colorado
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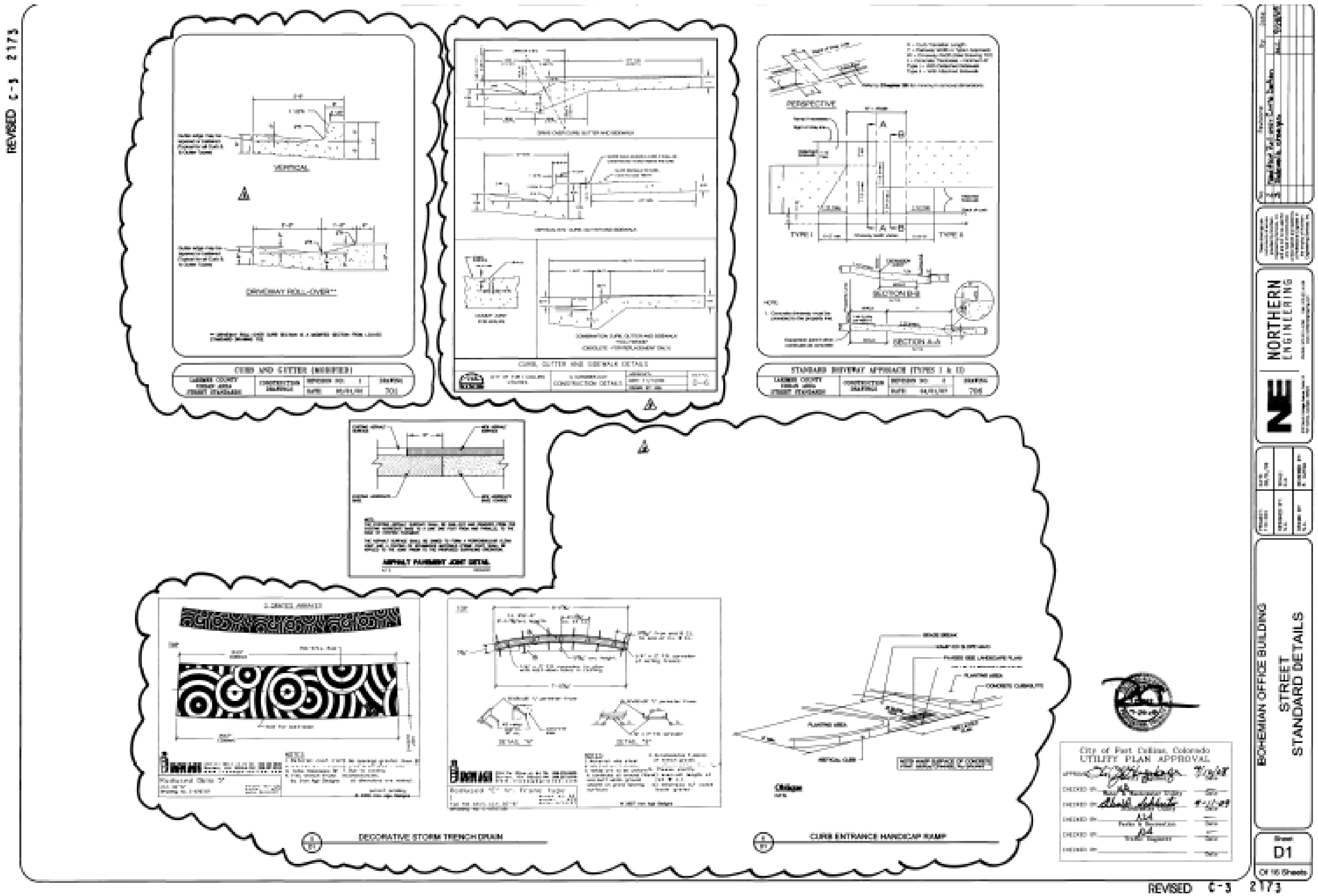
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VERTICAL
DRIVEWAY ROLL-OVER**
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CURB, GUTTER AND SIDEWALK DETAILS
PERSPECTIVE
TYPE I
TYPE II
SECTION B-B
SECTION A-A
STANDARD DRIVEWAY APPROACH (TYPES I & II)
ASPHALT PAVEMENT JOINT DETAIL
DETAIL "A"
DETAIL "B"
Oblique
DECORATIVE STORM TRENCH DRAIN
CURB ENTRANCE HANDICAP RAMP
City of Fort Collins, Colorado
UTILITY PLAN APPROVAL
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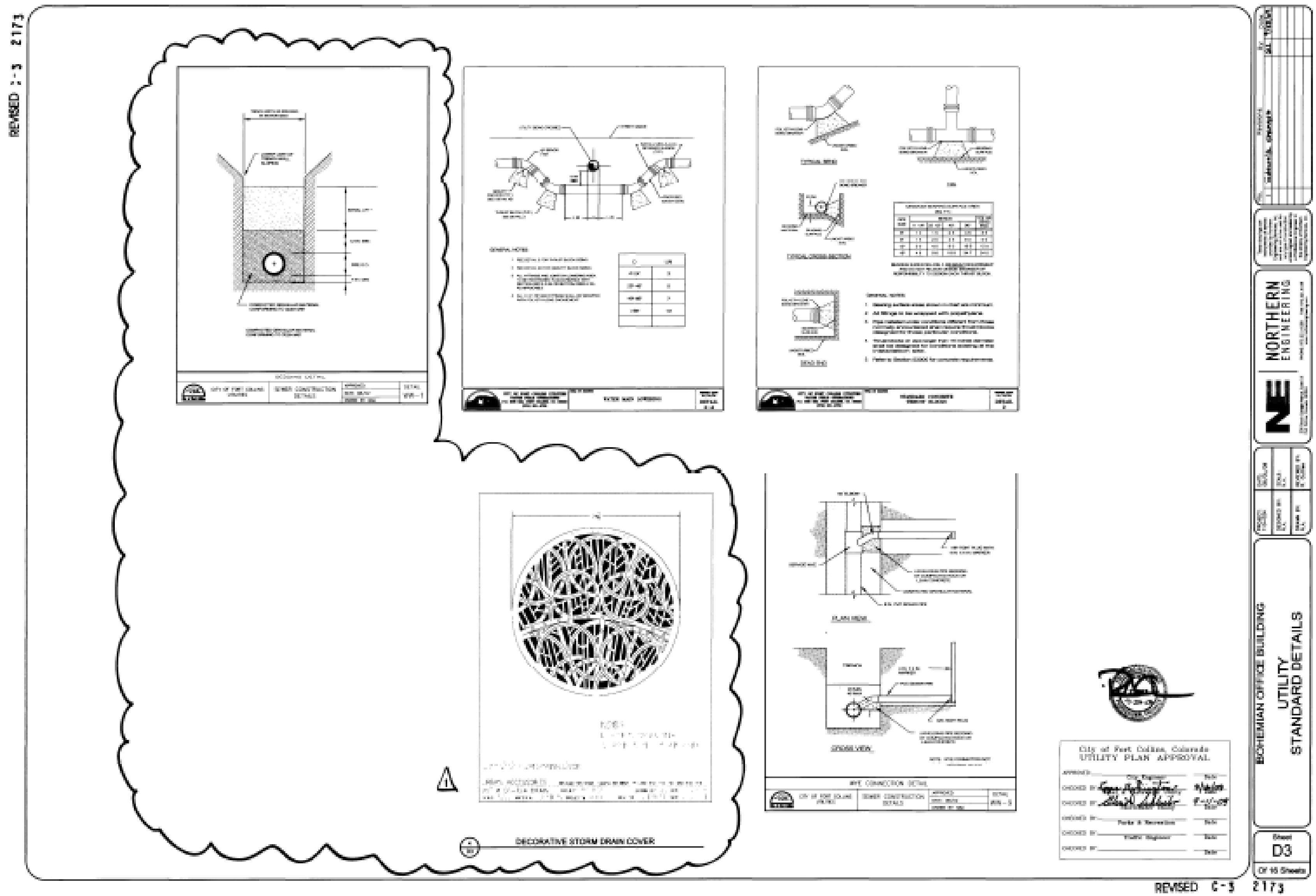
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DECORATIVE STORM DRAIN COVER
WYE CONNECTION DETAIL
PLAN VIEW
CROSS VIEW
City of Fort Collins, Colorado
UTILITY PLAN APPROVAL
City Engineer
Parks & Recreation
Traffic Engineer
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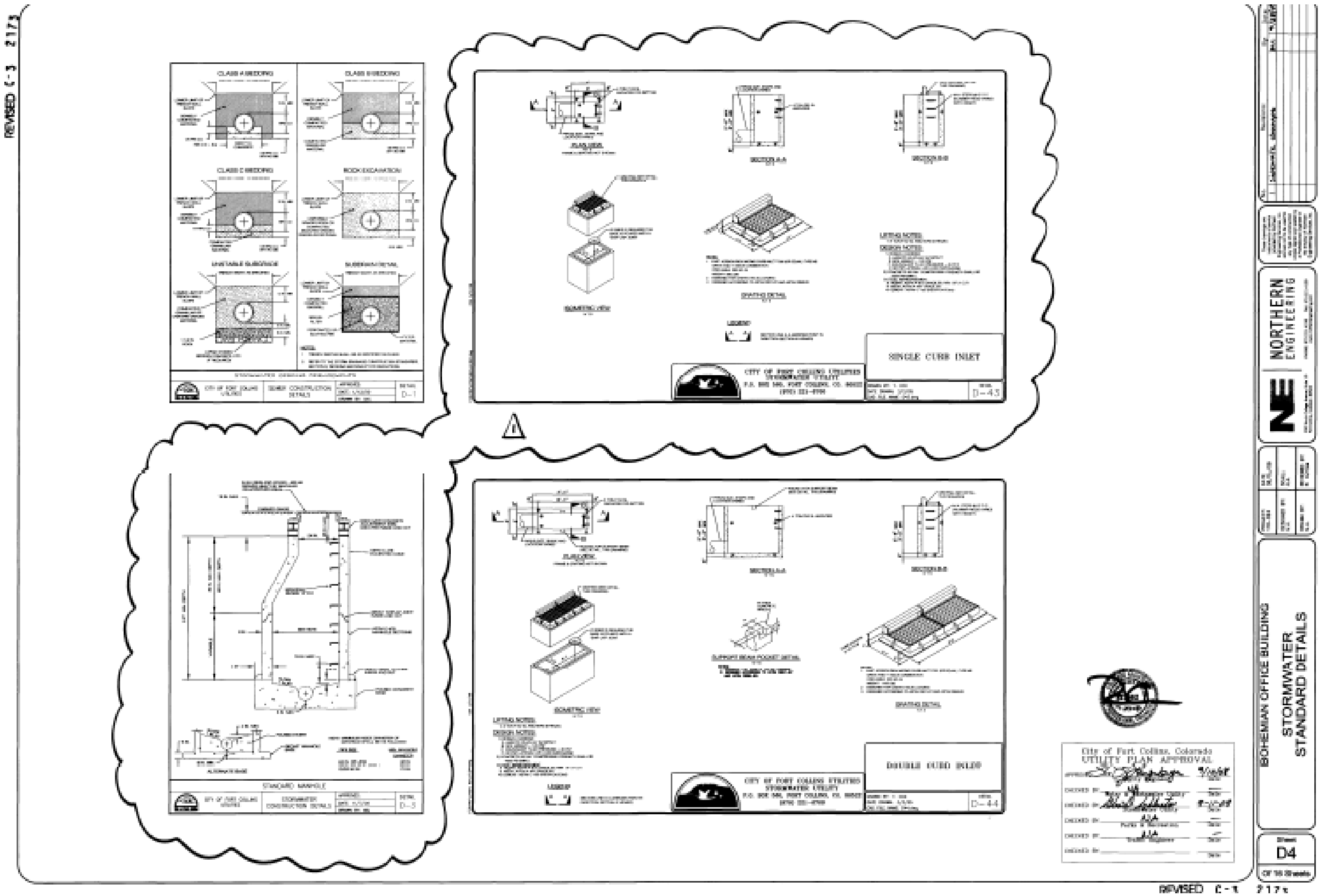
Of 16 Sheets
REVISED C-3 2173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CLASS B BEDDING
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UNSTABLE SUBGRADE
SUBDRAIN DETAIL
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STORMWATER UTILITY
D-43
STANDARD MANHOLE
D-3
DOUBLE CURB INLET
D-44
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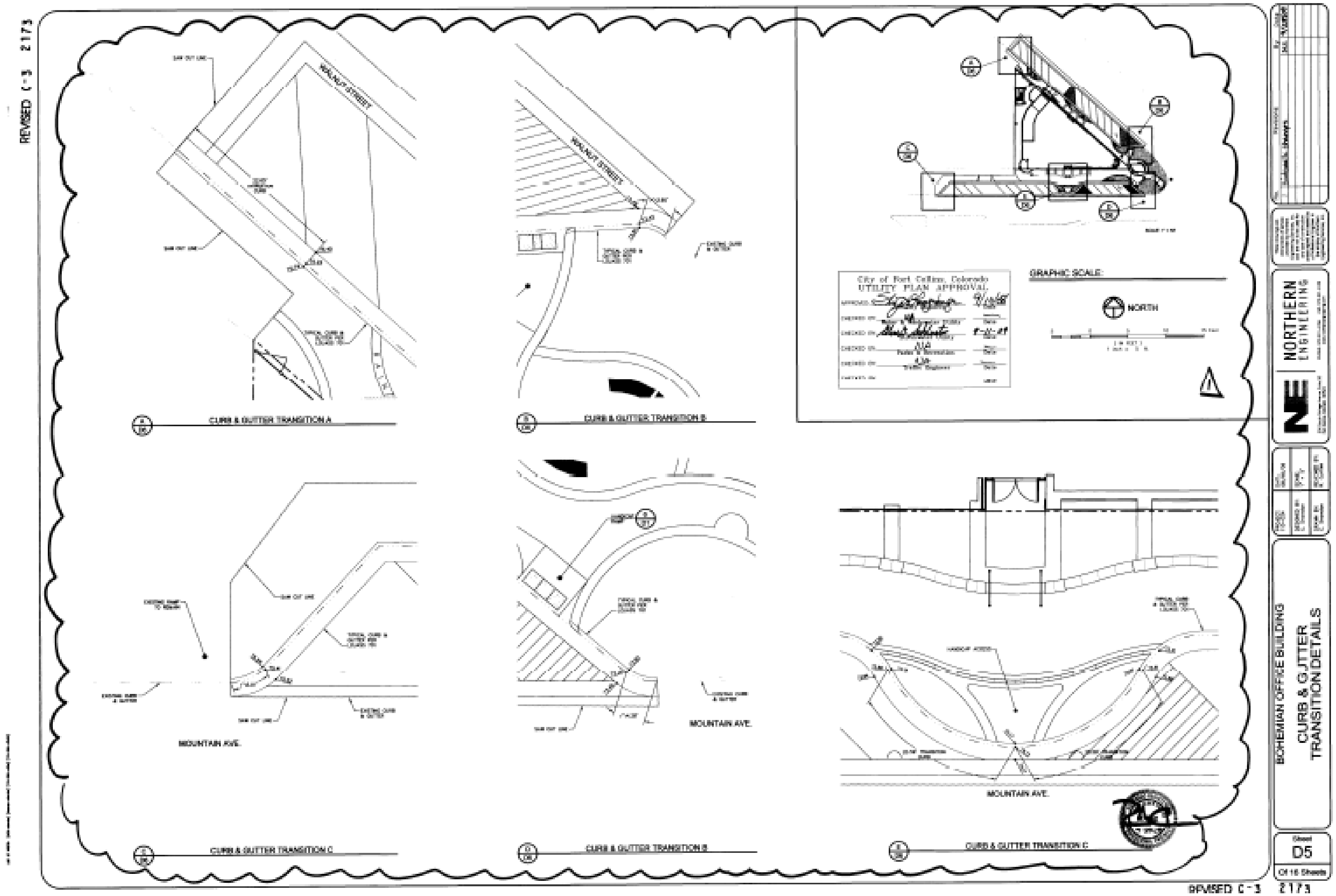
D4
REVISED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CURB & GUTTER TRANSITION B
MOUNTAIN AVE.
CURB & GUTTER TRANSITION C
CURB & GUTTER TRANSITION B
MOUNTAIN AVE.
CURB & GUTTER TRANSITION C
City of Fort Collins, Colorado
UTILITY PLAN APPROVAL
GRAPHIC SCALE:
NORTH
NORTHERN ENGINEERING
BOHEMIAN OFFICE BUILDING
CURB & GUTTER TRANSITION DETAILS
Sheet
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REVISED C-3 2173

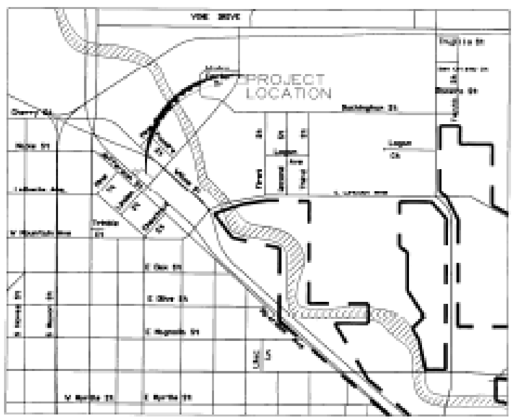
ADDITIONAL C-3 2079

PUBLIC IMPROVEMENTS CONSTRUCTION PLANS FOR

351 LINDEN STREET

PROPERTY IS LOCATED IN THE NORTHWEST QUARTER OF SECTION 12,
TOWNSHIP 7 NORTH, RANGE 69 WEST OF THE 6TH P.M.,
CITY OF FORT COLLINS, LARIMER COUNTY, COLORADO

SEPTEMBER, 2008



VICINITY MAP

SCALE: 1"=1000'

SHEET INDEX

REMODEL SHEET INDEX
(BY WOHNRADE CIVIL ENGINEERS)
PREVIOUS SUBMITTAL

- C1 TITLE SHEET
- C2 GENERAL NOTES
- C3 EXISTING CONDITIONS PLAN
- C4 OVERALL UTILITY PLAN
- C3 GRADING PLAN
- C4 GENERAL DETAILS

PARKING LOT SHEET INDEX
(BY CTL THOMPSON)

- C1 TITLE SHEET
- C2 EXISTING CONDITIONS
- C3 GRADING PLAN
- C4 GENERAL NOTES

PROJECT BENCHMARK

Designation --- Z 401 Elev.= 4958.37
In Fort Collins, at the junction of East Vine Drive and Linden Street, in the south edge of a highline tower footing, 14.2 meters (46.6 feet) north of the centerline of East Vine Drive, 10.3 meters (33.8 feet) west of the extended centerline of Linden Street, and 0.3 meter (1.0 feet) north of the south edge of the footing. Note-- The footing is 66 inches in diameter, 33 feet deep and set into 11 feet of bedrock. 0.6 meters above the drive.

Designation --- E 136 Elev.= 4992.21
In the town of Fort Collins, in the east face of the northwest corner of the U.S. Post Office Building, bronze tablet set vertically, 3 feet above the ground. Recovered in good condition. A new description follows. In Fort Collins, set vertically in the east face of the northeast corner of the Post Plaza Building (Old Post Office), at 201 South College Street, and 0.5 meter (1.6 feet) south of the northeast corner of the building. 0.8 meters above the sidewalk.

Benchmarks provided by Rick Lewis of R.A. Surveys, Fort Collins.

IMPORTANT NOTE:
ALL GRADES SHOWN WERE BASED ON AN OLD BONNET BOLT BENCHMARK. WHEN USING THE 2 BENCHMARKS ON THE COVER PAGE, THE GRADES SHOWN ON THE CTL THOMPSON AND WOHNRADE CIVIL PLANS MUST BE RAISED BY 2.06'.

CALL UTILITY NOTIFICATION CENTER OF COLORADO
1-800-922-1987
OR 1-303-232-1991 IN METRO DENVER
CALL 3 BUSINESS DAYS IN ADVANCE BEFORE YOU DIG, GRADE, OR EXCAVATE FOR THE MARKING OF UNDERGROUND MEMBER UTILITIES.

City of Fort Collins, Colorado
UTILITY PLAN APPROVAL
APPROVED:
CHECKED BY:
CHECKED BY:
CHECKED BY:
CHECKED BY:
CHECKED BY:

CTL | THOMPSON

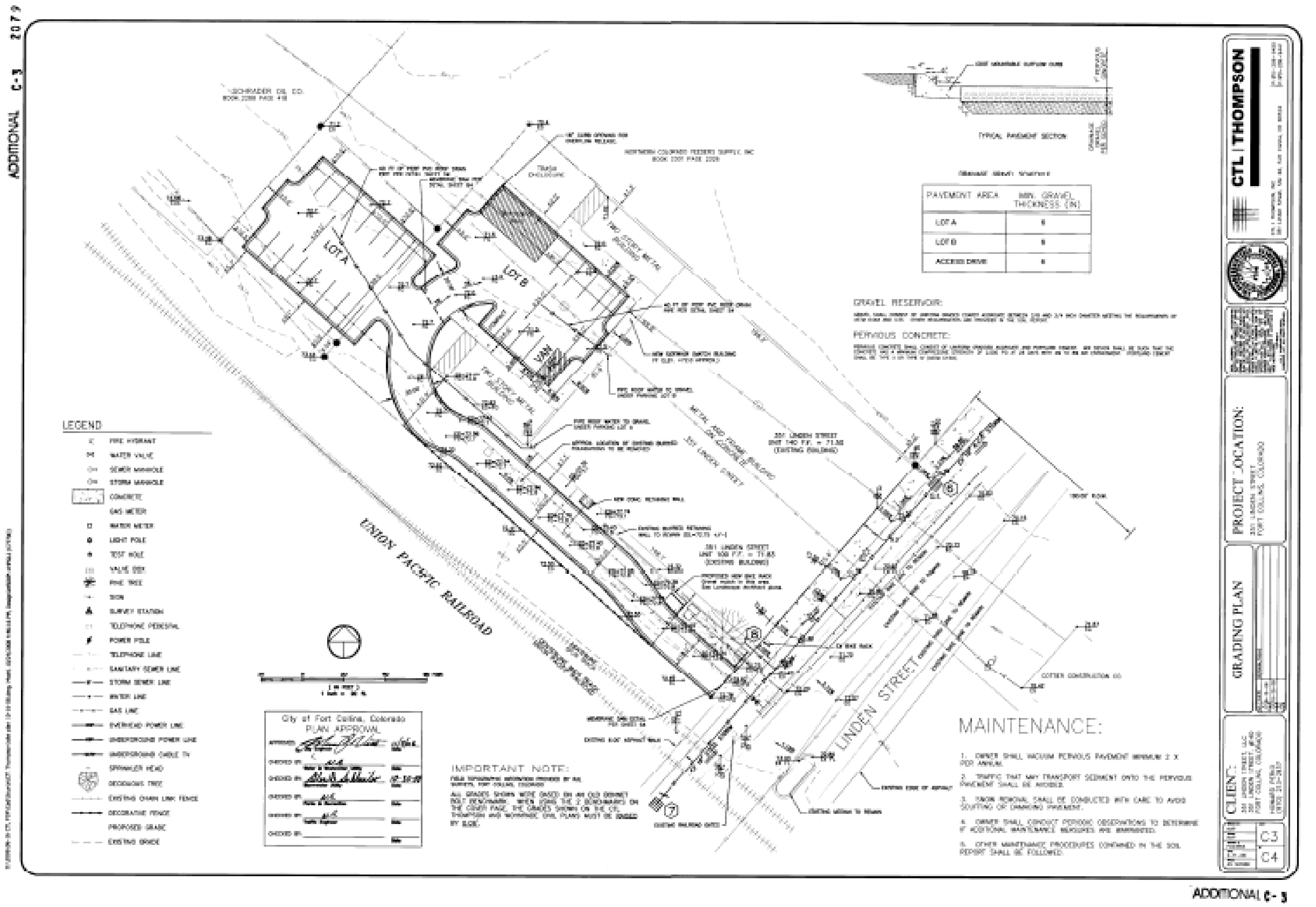
PROJECT LOCATION:
351 LINDEN STREET
FORT COLLINS, COLORADO

MASTER TITLE SHEET

CLIENT:

COVER

ADDITIONAL C-3 20i



ADDITIONAL C-3
CTL | THOMPSON
GRADING PLAN
PROJECT LOCATION:
CLIENT:
C3
C4
LOT A
LOT B
UNION PACIFIC RAILROAD
LINDEN STREET
TYPICAL PAVEMENT SECTION
PAVEMENT AREA
MIN. GRAVEL THICKNESS (IN)
LOT A
LOT B
ACCESS DRIVE
GRAVEL RESERVOIR:
PERVIOUS CONCRETE:
LEGEND
FIRE HYDRANT
WATER VALVE
SEWER MANHOLE
STORM MANHOLE
CONCRETE
GAS METER
WATER METER
LIGHT POLE
TEST HOLE
VALVE BOX
PINE TREE
SIGN
SURVEY STATION
TELEPHONE PEDESTAL
POWER POLE
TELEPHONE LINE
SANITARY SEWER LINE
STORM SEWER LINE
WATER LINE
GAS LINE
OVERHEAD POWER LINE
UNDERGROUND POWER LINE
UNDERGROUND CABLE TV
SPRINKLER HEAD
DECIDUOUS TREE
EXISTING CHAIN LINK FENCE
DECORATIVE FENCE
PROPOSED GRADE
EXISTING GRADE
City of Fort Collins, Colorado
PLAN APPROVAL
IMPORTANT NOTE:
MAINTENANCE:
1. OWNER SHALL VACUUM PERVIOUS PAVEMENT MINIMUM 2 X PER ANNUM.
2. TRAFFIC THAT MAY TRANSPORT SEDIMENT ONTO THE PERVIOUS PAVEMENT SHALL BE AVOIDED.
3. SNOW REMOVAL SHALL BE CONDUCTED WITH CARE TO AVOID SCUFFING OR DAMAGING PAVEMENT.
4. OWNER SHALL CONDUCT PERIODIC OBSERVATIONS TO DETERMINE IF ADDITIONAL MAINTENANCE MEASURES ARE WARRANTED.
5. OTHER MAINTENANCE PROCEDURES CONTAINED IN THE SOIL REPORT SHALL BE FOLLOWED.
COTTER CONSTRUCTION CO

GENERAL NOTES:

MAINTENANCE:

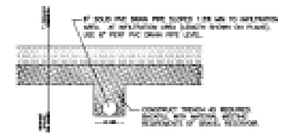
CONSTRUCTION:

City of Fort Collins, Colorado
UTILITY PLAN APPROVAL

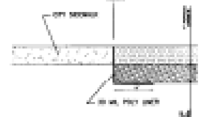
CHECKED BY:

CHECKED BY:

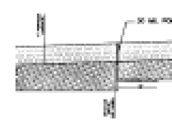
CHECKED BY:



ROOF DRAIN DETAIL



MEMBRANE DAM DETAIL
(SIDEWALK)



MEMBRANE DAM DETAIL
(GRADE CHANGE)

CTL | THOMPSON

PROJECT LOCATION:

GENERAL NOTES

CLIENT:

C4

C4

Appendix B: Program Code at Mitchell Block

Mountain Program Code

```
'CR200 Series Datalogger

'Program to run ISCO automatic sampler based on water level measurements

'obtained using Campbell Scientific 4500 Pressure Transducer.

'date: 3/21/2010

'program author: Chris Olson, Colorado State University


'**** This program for Mountain Avenue sampling site *******

'Site characteristics: Paver area = 4000 ft2 , run-on area = 7200 ft2

'Expected runoff generated by 0.5 inch storm = 350 ft3

'Sample trip at 22 ft3 will collect 16 bottles during 0.5 inch storm, 8 bottles at 0.3 inch storm



'Important Notes:

'Pressure transducer sensor is 0.125 inches above bottom when lying flat

'Program uses 2 rating curves, one for bottom orifice only, one for bottom orifice + v-notch weir (6.5
inches above bottom)

'Orifice equation: Cd*oarea*(2*g*depth/12)^0.5  --> computes discharge in cfs

'90 degree V-notch weir equation: Cdweir * (2g)^.5 * ((depth-6.5)/12)^2.5  --> computes discharge in cfs

'Cdweir for weir eqn was calibrated in June 2010 with field data by CSU



'Declare Public Variables

Public batt_volt
```

```
Public CS450(2)
Public Flag(1)  'Flag used to reset/stop sampling
Public orificeQ  'flowrate through orifice (cfs)
Public vnotchQ    'flowrate through v-notch weir
Public Q          'total flowrate (cfs)
Public Depth       'water depth (inches)
Public intVolume    'discharge volume for scan interval (cf)
Public totalVolume  'cumulative discharge volume for sample (cf)
Public samplenumber 'bottle/sample number
Public pressureadj  'adjusted pressure (psi)

'Rename variable names
Alias CS450(1) = Pressure
Alias CS450(2) = Temp_C

'Declare Other Variables

'Declare Constants
Const tripVolume = 22      'volume to trip ISCO (cf)
Const Cd = 0.62           'orifice discharge coefficient
Const Cdweir = 0.540       'weir discharge coefficient
Const g = 32.2            'gravitational constant
Const oarea = 0.000340885  'orifice area (ft2)
Const offset = 0.125       'pressure transducer sensor is 0.125 inches above bottom

'Define Data Tables
```

```
DataTable (MtnAve,1,-1)
        DataInterval (0,1,min)
        Minimum (1,batt_volt,0,0)
  Sample (1,Pressure)
  Sample (1,pressureadj)
  Sample (1,Depth)
  Sample (1,Temp_C)
  Sample (1,Q)
  Sample (1,intVolume)
  Sample (1,totalVolume)
  Sample (1,samplenumber)
  Sample (1,orificeQ)
  Sample (1,vnotchQ)
EndTable

Sub calcflow
 'compute flow rate from water level (cfs)
 'need to add "background pressure" back, found to cause significant underestimation of flow
  Depth = Depth + 0.005 * 2.31*12
  orificeQ = Cd*oarea*(2*g*Depth/12)^0.5
  If Depth > 6.5 Then vnotchQ = Cdweir * ((2*g)^.5) * ((Depth-6.5)/12)^2.5
  If Depth <= 6.5 Then vnotchQ = 0
  Q = orificeQ + vnotchQ
  'Compute discharge volume (cf) since last scan (1 minute interval)
  intVolume = Q*60
  'Compute cumulative discharge volume since last sample
```

```
  totalVolume = totalVolume + intVolume
EndSub

Sub sampling
  Flag(1) = false  'start counting samples
   'Quit subroutine if 24 samples have been collected
  If samplenumber > 24 Then ExitSub
  'If cumulative discharge volume >= sampling interval
  If totalVolume >=  tripVolume Then
    'Set Control Port 2 to high to trip ISCO, delay, set back to low
    PortSet (C2,1)
    Delay (250,msec)
    PortSet(C2,0)
    'Reset cumulative discharge volume = 0
    totalVolume = 0
    'Record sample number
    samplenumber = samplenumber + 1
  EndIf
EndSub

'Main Program
BeginProg
'Scan every 1 minute
 Scan (1,min)
   'Reset samplenumber by setting flag 1 to -1
   If Flag(1) = true Then samplenumber = 0
```

```
'Measure battery voltange
        Battery (batt_volt)

'Measure water level with CS450 pressure transducer
SDI12Recorder (CS450,0M!,1.0,0)

 'Adjust pressure for "background pressure" ~ 0.005 psi (change in calcflow subroutine if altered)
pressureadj = Pressure -.005
If pressureadj < 0 Then pressureadj = 0

'Convert pressure (psig) to water depth (inches)
Depth = pressureadj * 2.31*12 + offset

'If depth > 0.225 inches than compute flowrate
If Depth > offset + 0.1 Then Call calcflow

'If Level > 1.49 then run sampling sub
'Must have 1.5 inches of water to collect a sample
If Depth > 1.49 Then Call Sampling

'Call Output Tables if measurements are occuring.
        If Depth > offset + 0.1 Then CallTable MtnAve
        'CallTable MtnAve
       NextScan
EndProg
```

Walnut Program Code

```
'CR200 Series Datalogger

'Program to run ISCO automatic sampler based on water level measurements

'obtained using Campbell Scientific 4500 Pressure Transducer.

'date: 3/21/2010

'program author: Chris Olson, Colorado State University



'****** This program for Walnut Avenue sampling site ***********

'Site characteristics: Paver area = 3500 ft^2, run on area = 3500 ft^2


'Important Notes:

'Pressure transducer sensor is 0.125 inches above bottom when lying flat

'Program uses 2 rating curves, one for bottom orifice only, one for bottom orifice + v-notch weir (6.5 inches above bottom)

'Orifice equation: Cd*oarea*(2*g*depth/12)^0.5  --> computes discharge in cfs

'90 degree V-notch weir equation: Cdweir * (2g)^.5 * ((depth-6.5)/12)^2.5  --> computes discharge in cfs

'Cdweir for weir eqn was calibrated in June 2010 with field data by CSU



'Declare Public Variables

Public batt_volt

Public CS450(2)

Public Flag(1)  'Flag used to reset/stop sampling

Public orificeQ  'flowrate through orifice (cfs)

Public vnotchQ    'flowrate through v-notch weir

Public Q          'total flowrate (cfs)
```

```
Public Depth        'water depth (inches)

Public intVolume    'discharge volume for scan interval (cf)

Public totalVolume  'cumulative discharge volume for sample (cf)

Public samplenumber 'bottle/sample number

Public pressureadj  'adjusted pressure (psi)


'Rename variable names

Alias CS450(1) = Pressure

Alias CS450(2) = Temp_C


'Declare Other Variables


'Declare Constants

Const tripVolume = 12      'volume to trip ISCO (cf)

Const Cd = 0.62            'orifice discharge coefficient

Const Cdweir = 0.595       'weir discharge coefficient

Const g = 32.2             'gravitational constant

Const oarea = 0.000340885  'orifice area (ft2)

Const offset = 0.125       'offset for pressure transducer (inches)


'Define Data Tables

DataTable (WalnutAve,1,-1)

        DataInterval (0,1,min)

        Minimum (1,batt_volt,0,0)

  Sample (1,Pressure)

  Sample (1,pressureadj)
```

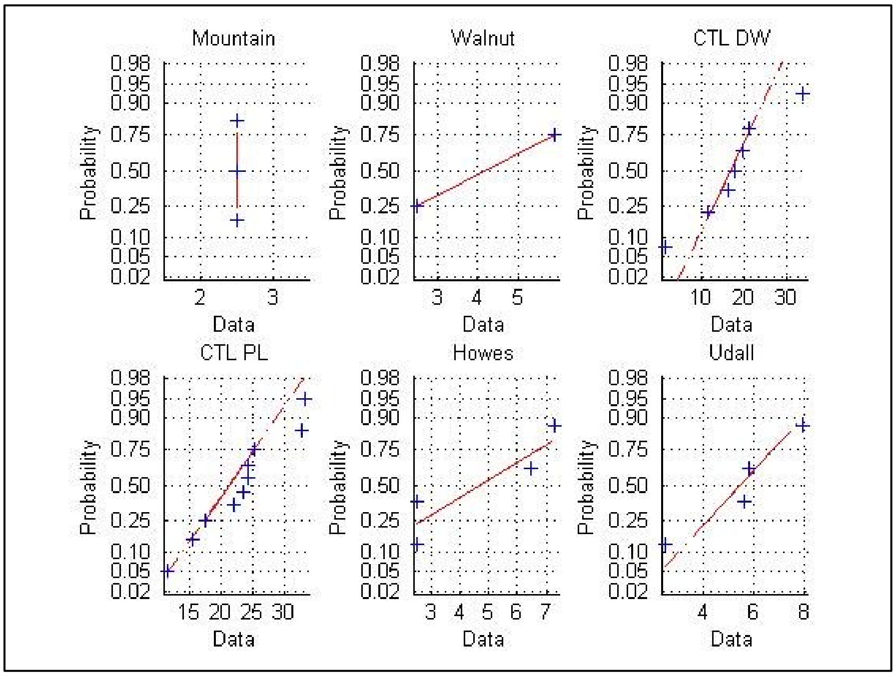
```
  Sample (1,Depth)
  Sample (1,Temp_C)
  Sample (1,Q)
  Sample (1,intVolume)
  Sample (1,totalVolume)
  Sample (1,samplenumber)
  Sample (1,orificeQ)
  Sample (1,vnotchQ)
EndTable

Sub calcflow
 'compute flow rate from water level (cfs)
 'need to add "background pressure" back, found to cause significant underestimation of flow
  Depth = Depth + 0.01 * 2.31*12
  orificeQ = Cd*oarea*(2*g*Depth/12)^0.5
  If Depth > 6.5 Then vnotchQ = Cdweir * ((2*g)^.5) * ((Depth-6.5)/12)^2.5
  If Depth <= 6.5 Then vnotchQ = 0
  Q = orificeQ + vnotchQ
  'Compute discharge volume (cf) since last scan (1 minute interval)
  intVolume = Q*60
  'Compute cumulative discharge volume since last sample
  totalVolume = totalVolume + intVolume
EndSub

Sub sampling
  Flag(1) = false  'start counting samples
```

```
  'Quit subroutine if 22 samples have been collected (two bottles short)
  If samplenumber > 22 Then ExitSub
  'If cumulative discharge volume >= sampling interval
  If totalVolume >=  tripVolume Then
    'Set Control Port 2 to high to trip ISCO, delay, set back to low
    PortSet (C2,1)
    Delay (250,msec)
    PortSet(C2,0)
    'Reset cumulative discharge volume = 0
    totalVolume = 0
    'Record sample number
    samplenumber = samplenumber + 1
  EndIf
EndSub


'Main Program
BeginProg
'Scan every 60 seconds
 Scan (1,min)
   'Reset samplenumber by setting flag 1 to -1
   If Flag(1) = true Then samplenumber = 0


   'Measure battery voltange
           Battery (batt_volt)


   'Measure water level with CS450 pressure transducer
```

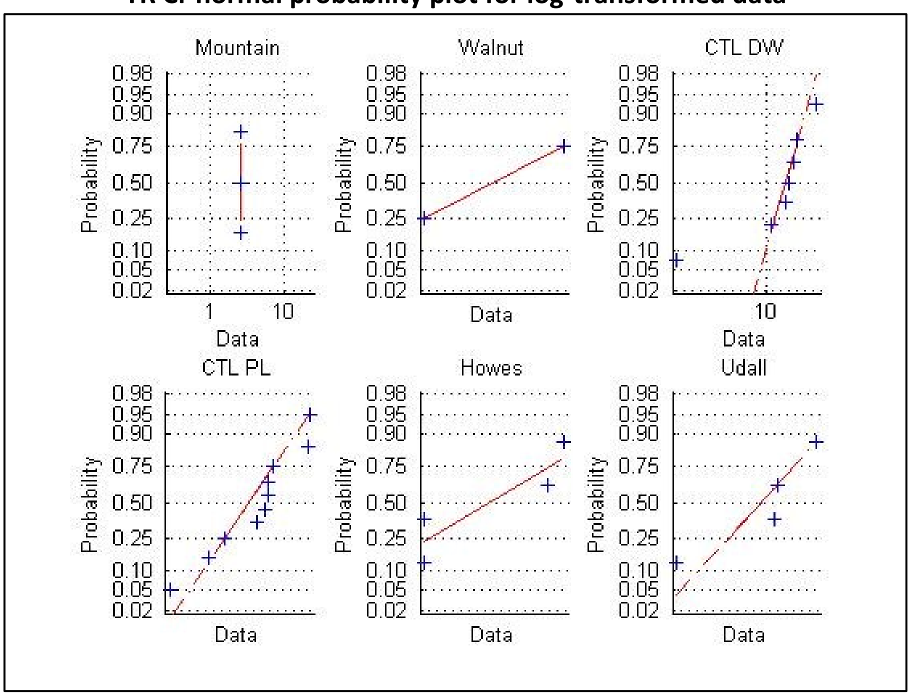
```
    SDI12Recorder (CS450,0M!,1.0,0)

    'Correct for "background pressure" ~ 0.01 psi (change in calcflow subroutine if altered)
    pressureadj = Pressure - 0.01
    If pressureadj < 0 Then pressureadj = 0

    'Convert pressure (psig) to water depth (inches)
    Depth = pressureadj * 2.31*12 + offset

    'If water is present compute flowrate
    If Depth > offset + 0.1 Then Call calcflow

    'If Level > 1.49 then run sampling sub
    'Must have 1.5 inches of water to collect a sample
    If Depth > 1.49 Then Call Sampling

    'Call Output Tables if measurements are occurring
    If Depth > offset + 0.1 Then Call Table WalnutAve
        NextScan
EndProg
```

Appendix C: Probability Plots and ROS Plots

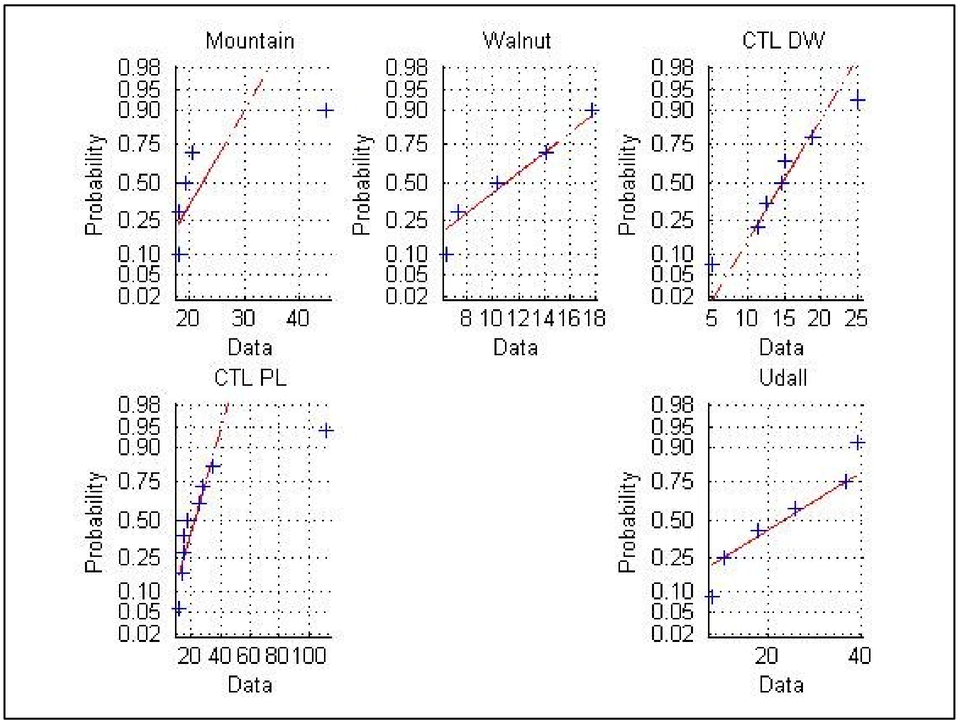
TR Cr normal probability plot for original data



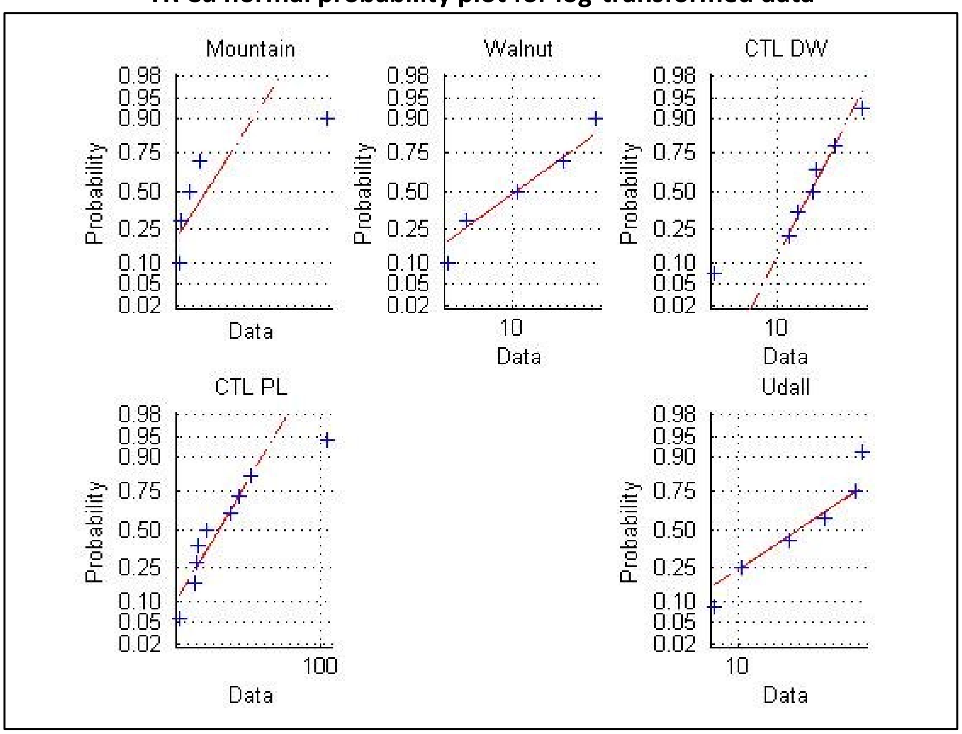
TR Cr normal probability plot for log-transformed data



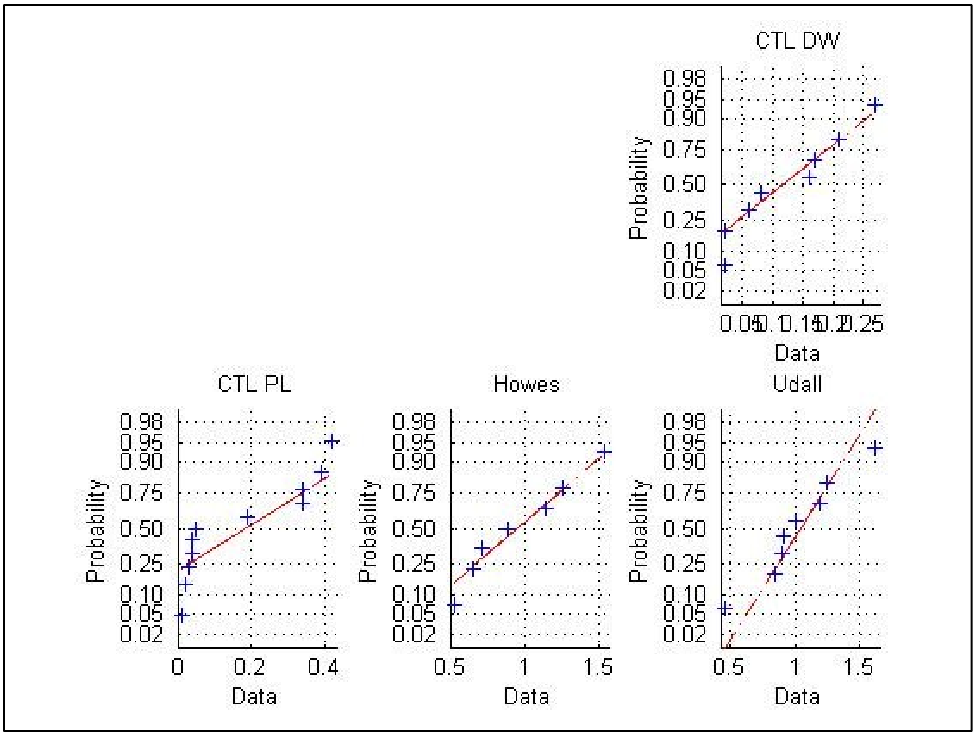
TR Cu normal probability plot for original data



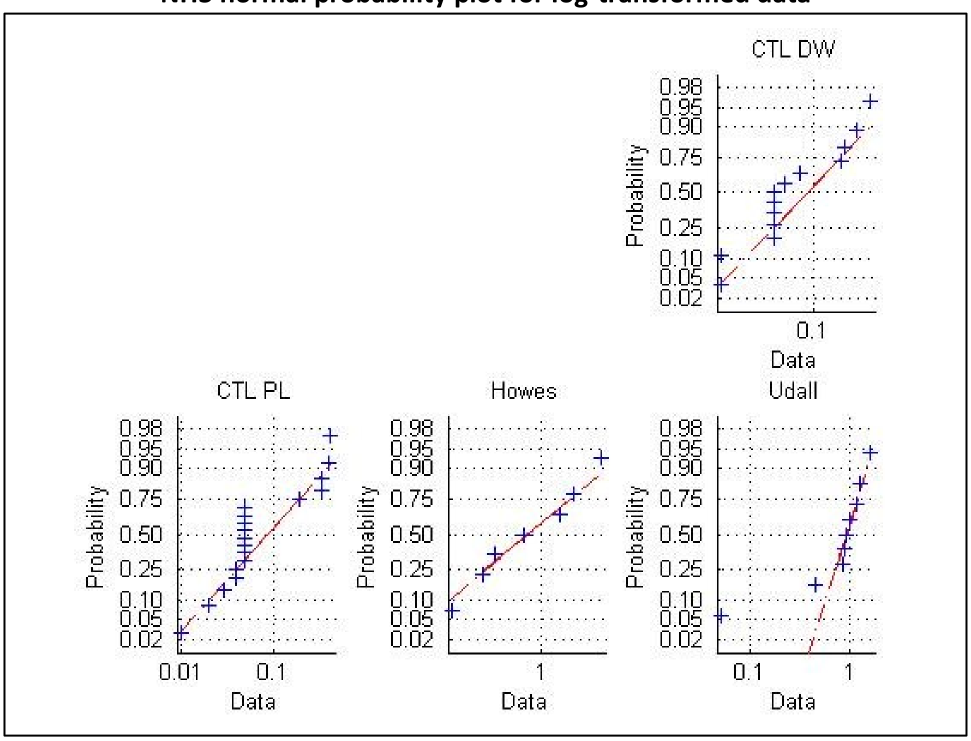
TR Cu normal probability plot for log-transformed data



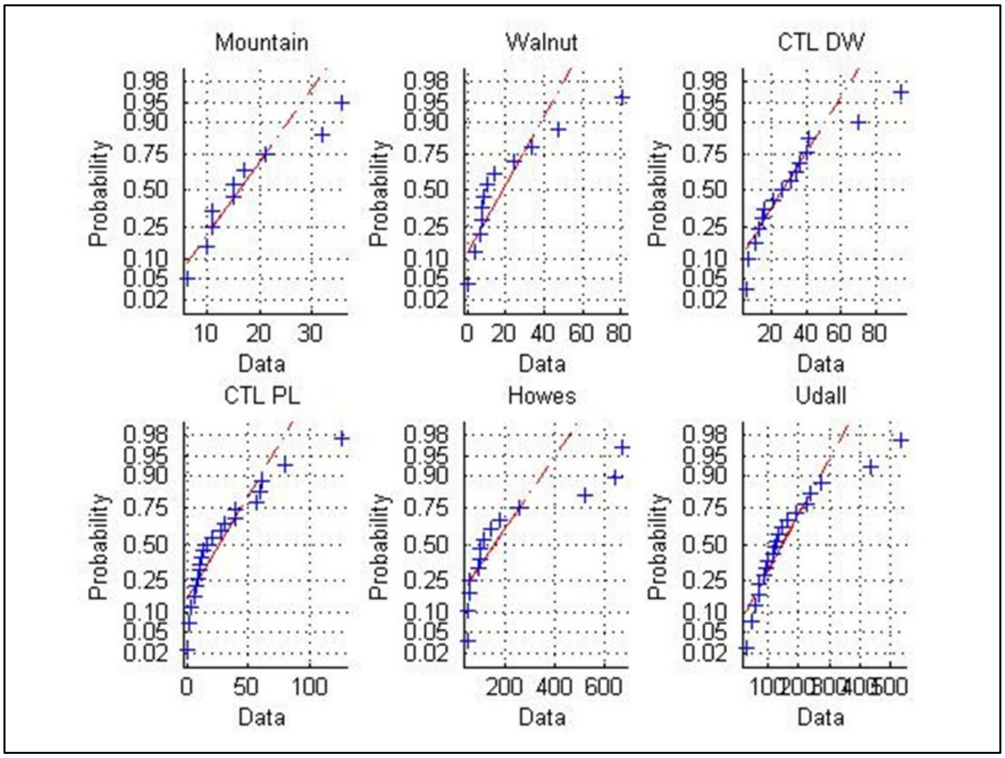
NH3 normal probability plot for original data



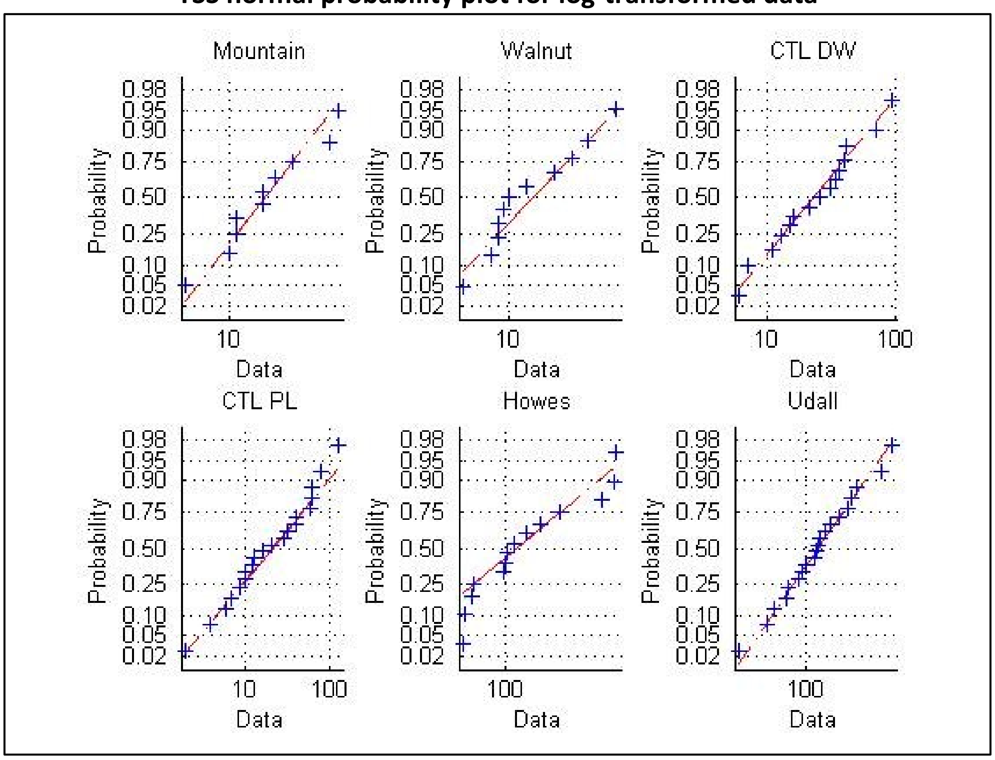
NH3 normal probability plot for log-transformed data



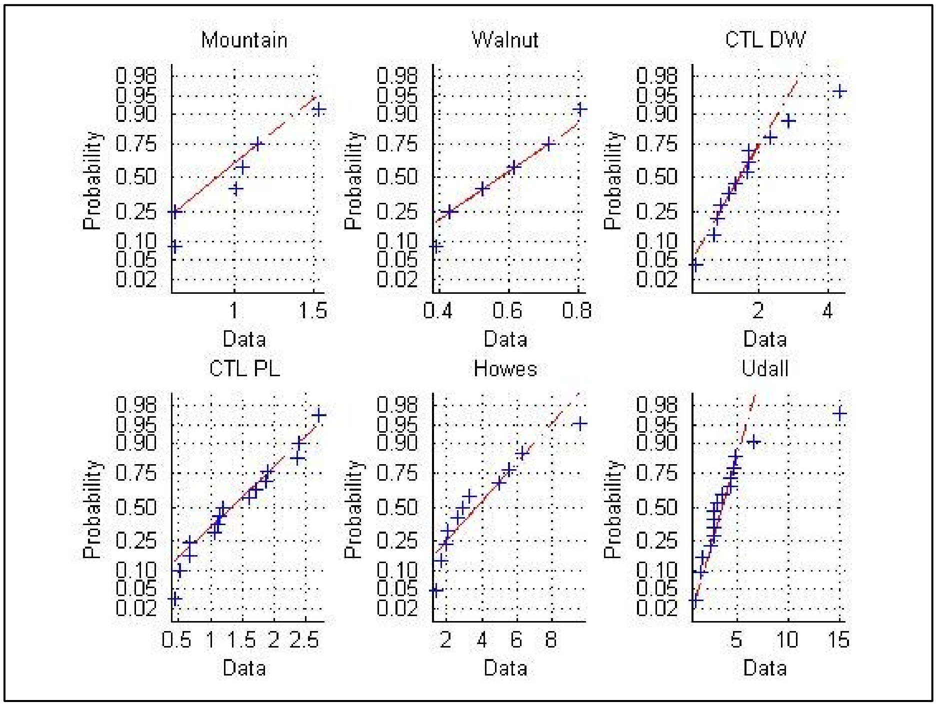
TSS normal probability plot for original data



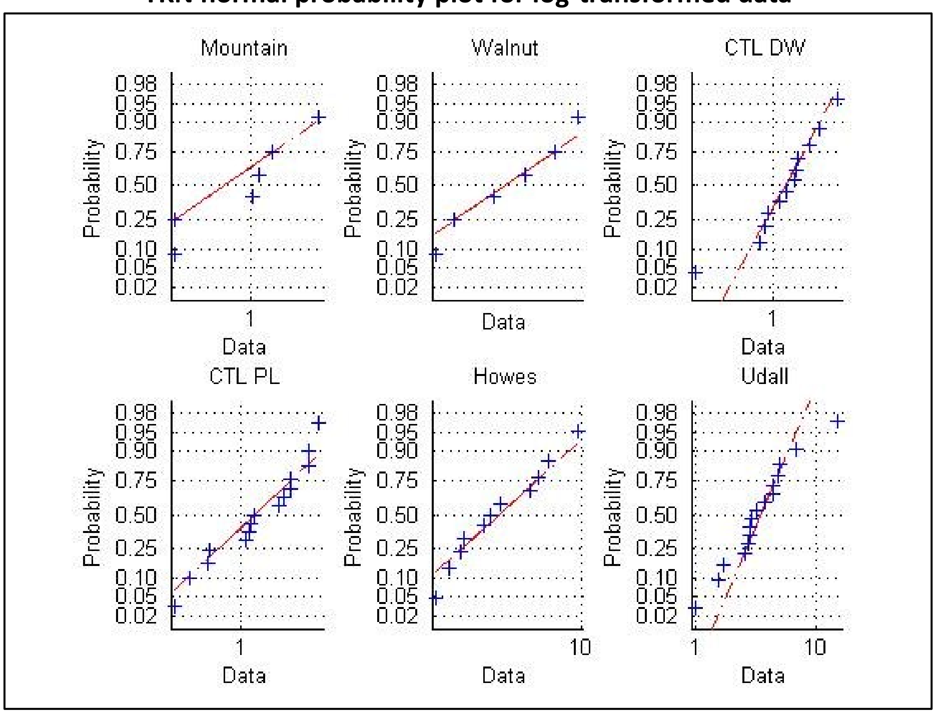
TSS normal probability plot for log-transformed data



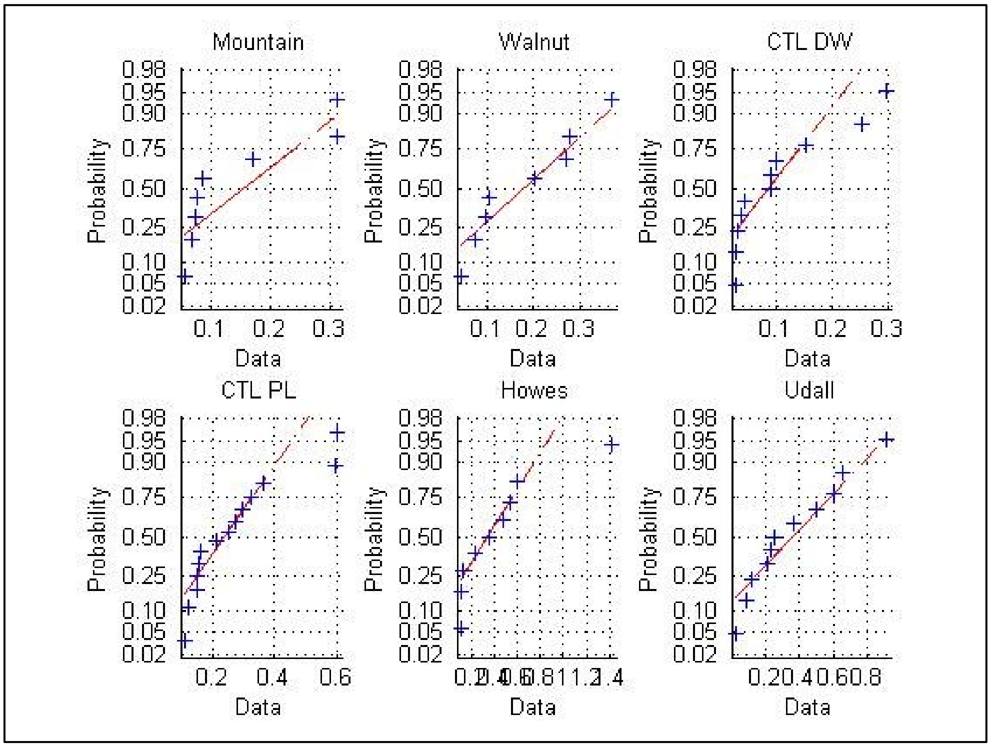
TKN normal probability plot for log-transformed data



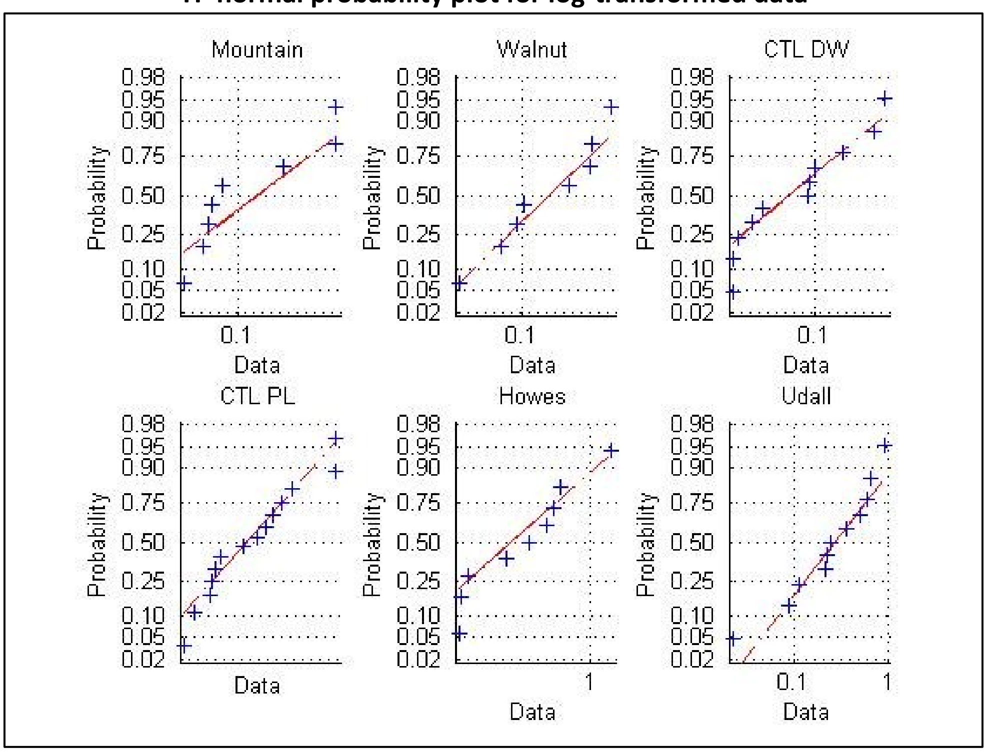
TKN normal probability plot for log-transformed data



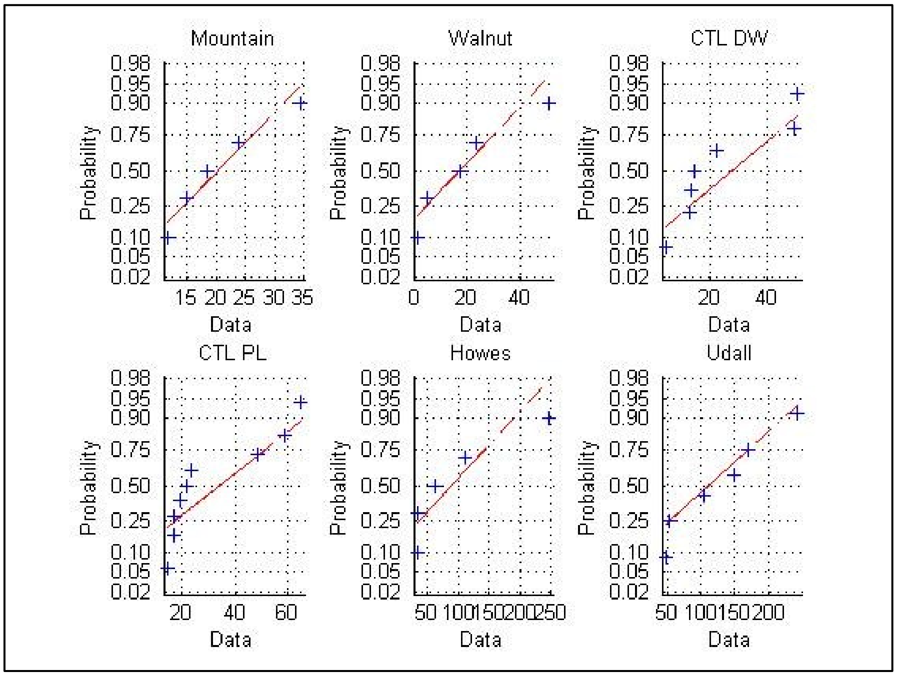
TP normal probability plot for log-transformed data



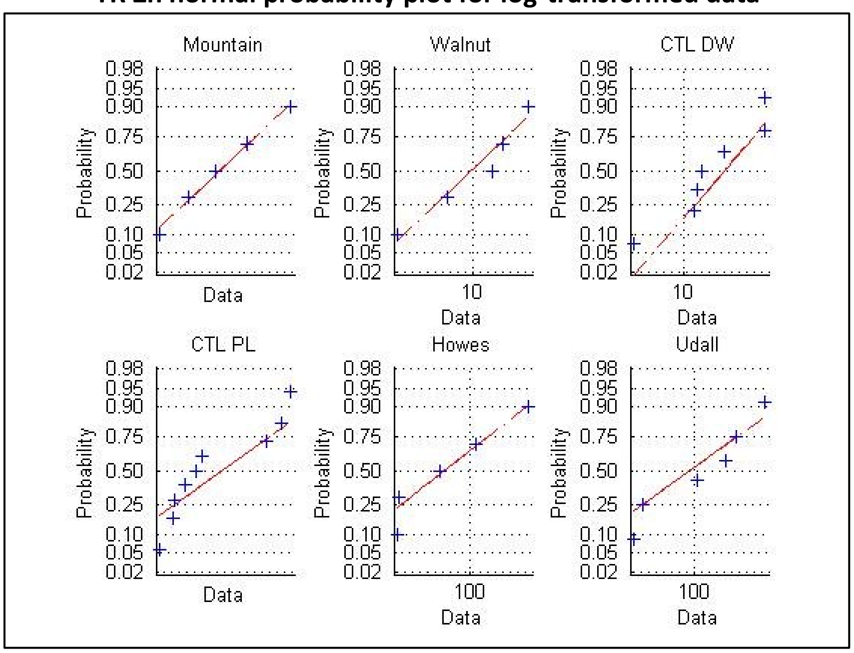
TP normal probability plot for log-transformed data



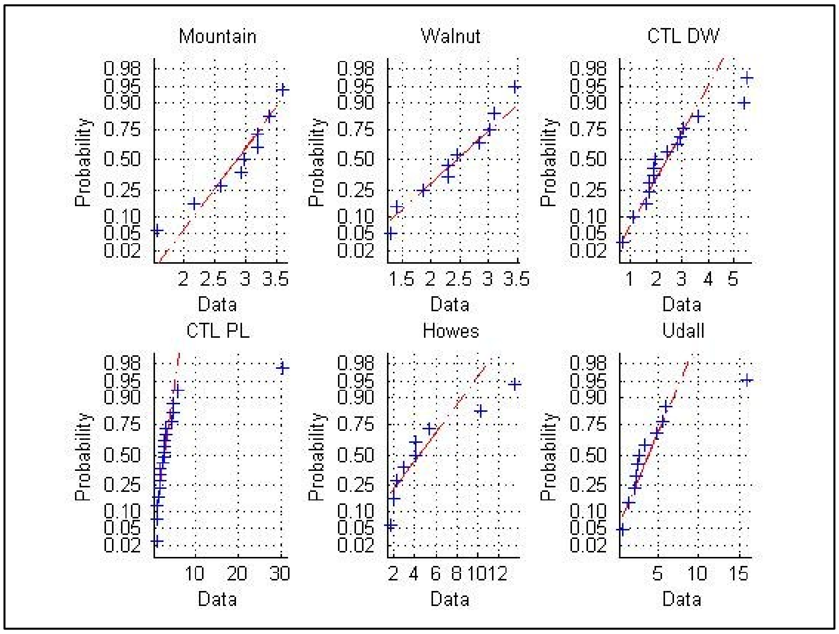
TR Zn normal probability plot for log-transformed data



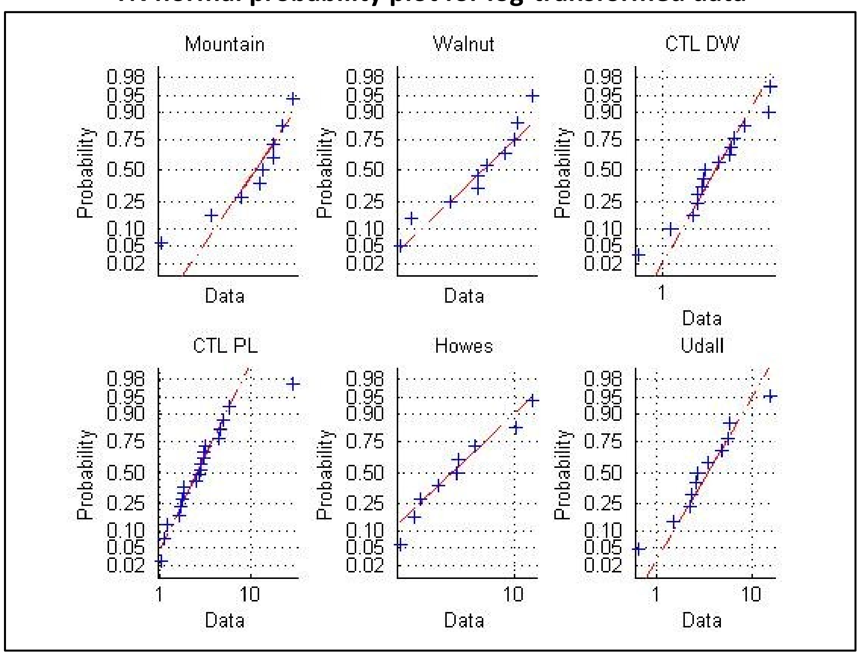
TR Zn normal probability plot for log-transformed data



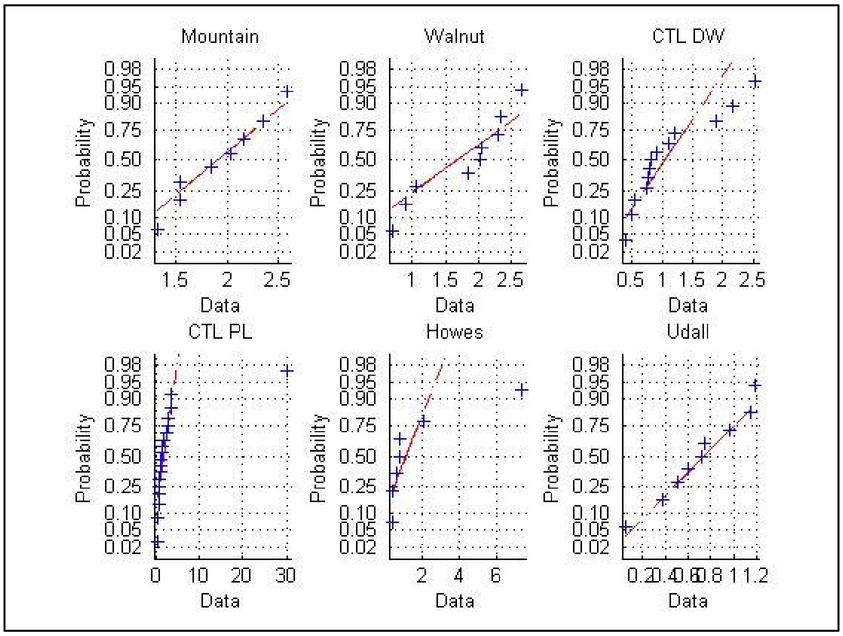
TN normal probability plot for log-transformed data



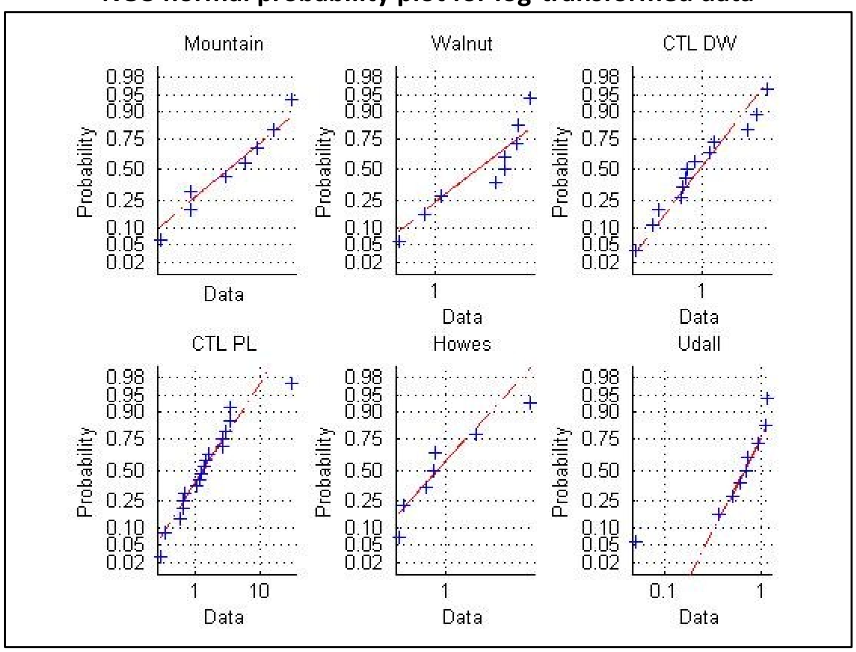
TN normal probability plot for log-transformed data



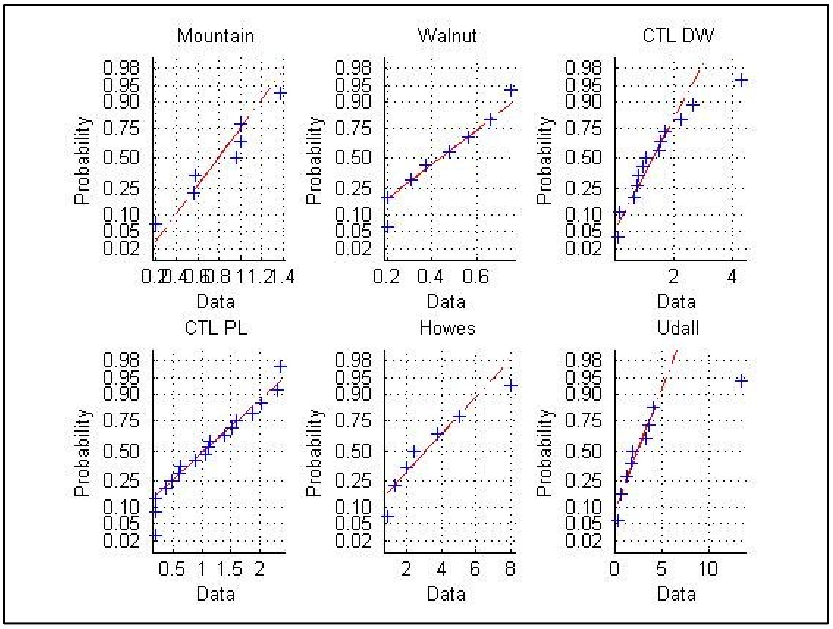
NO3 normal probability plot for log-transformed data



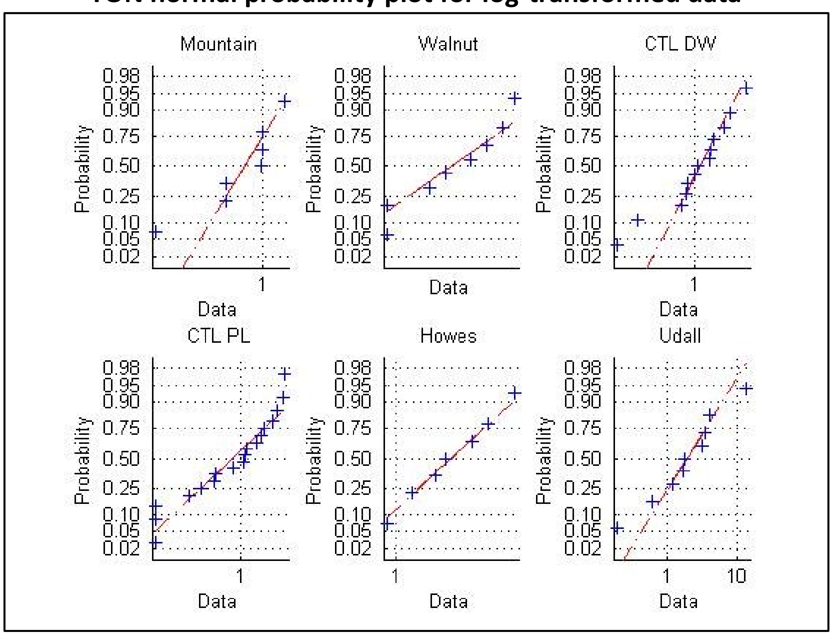
NO3 normal probability plot for log-transformed data



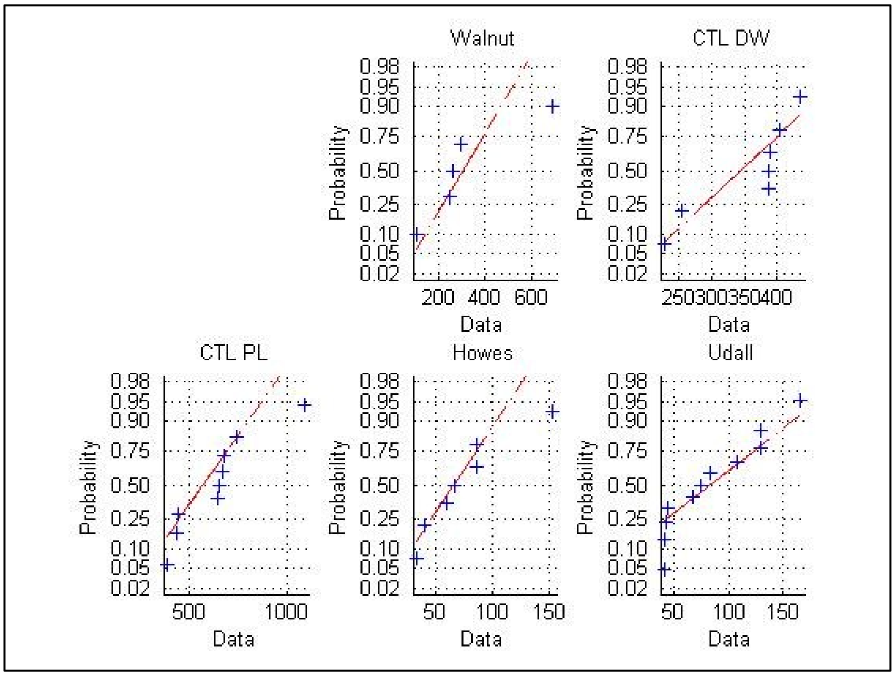
TON normal probability plot for log-transformed data



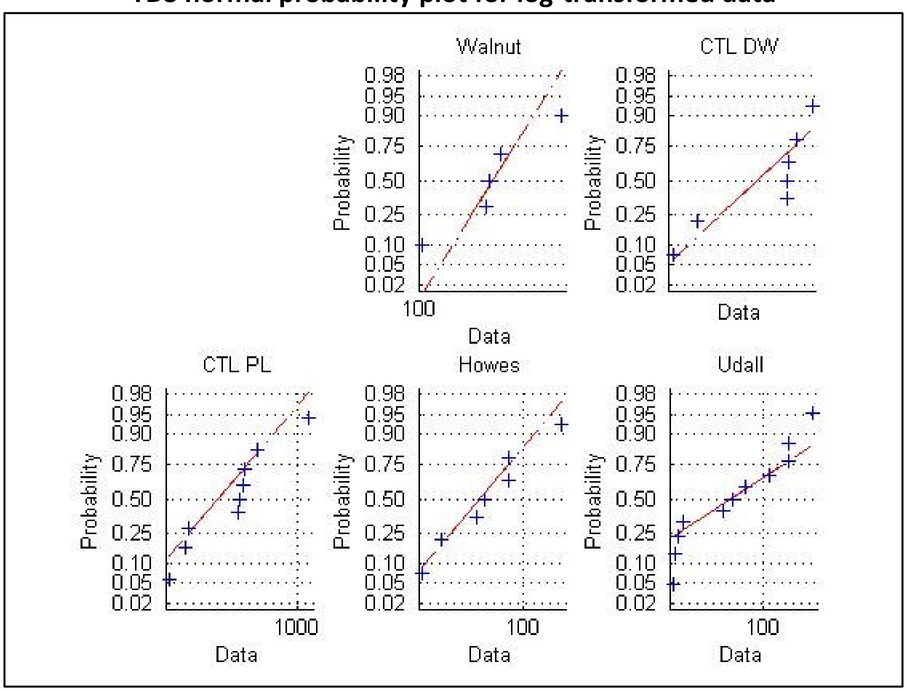
TON normal probability plot for log-transformed data

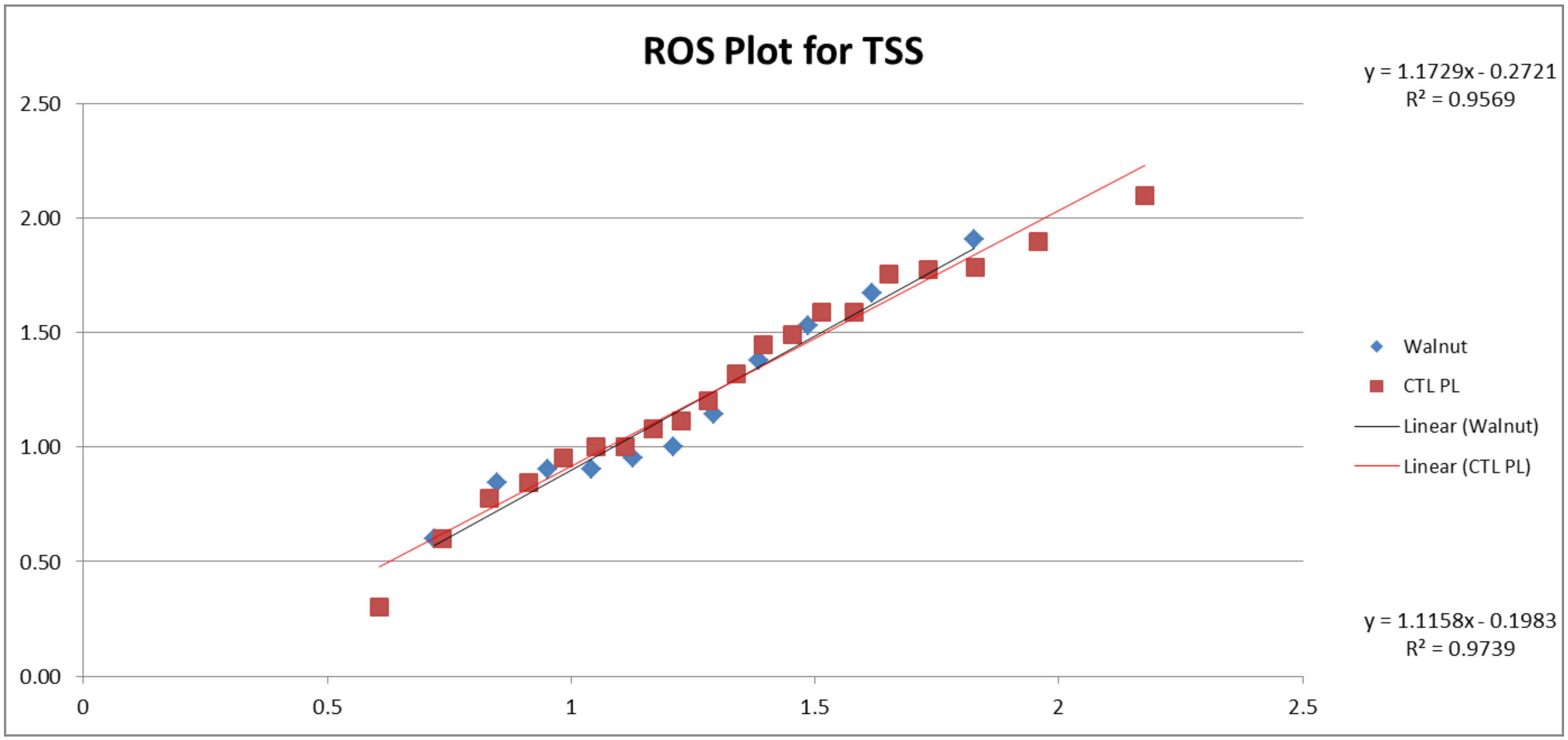


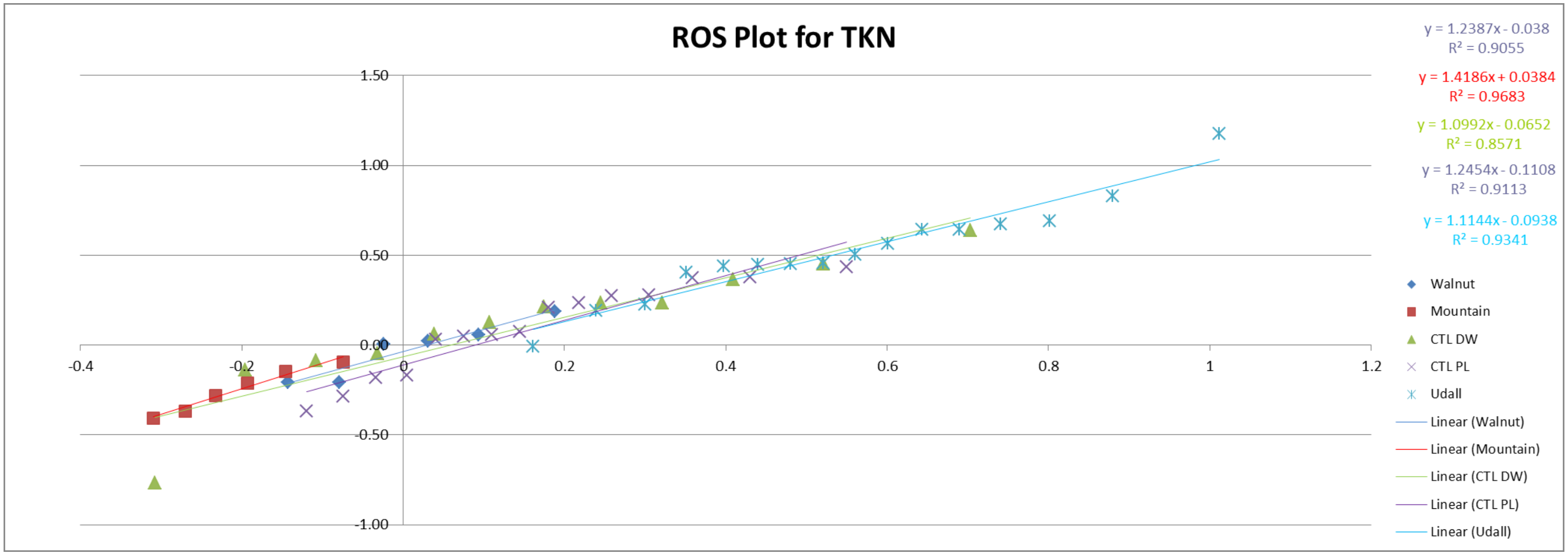
TDS normal probability plot for log-transformed data



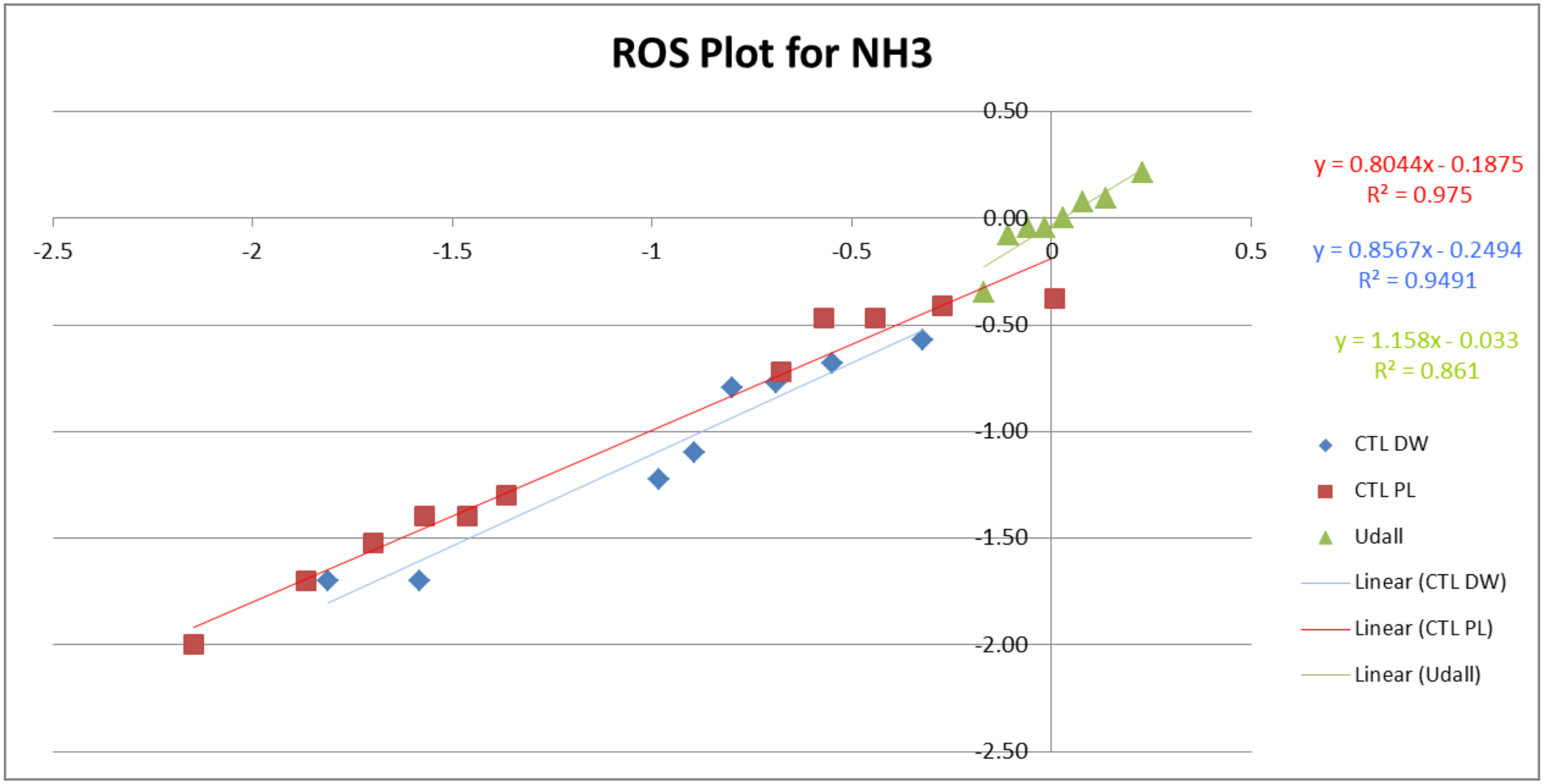
TDS normal probability plot for log-transformed data




ROS Plot for TSS
y = 1.1729x - 0.2721
R² = 0.9569
y = 1.1158x - 0.1983
R² = 0.9739
2.50
2.00
1.50
1.00
0.50
0.00
0
0.5
1
1.5
2
2.5
Walnut
CTL PL
Linear (Walnut)
Linear (CTL PL)



ROS Plot for TKN
y = 1.2387x - 0.038
R² = 0.9055
y = 1.4186x + 0.0384
R² = 0.9683
y = 1.0992x - 0.0652
R² = 0.8571
y = 1.2454x - 0.1108
R² = 0.9113
y = 1.1144x - 0.0938
R² = 0.9341
1.50
1.00
0.50
0.00
-0.50
-1.00
-0.4
-0.2
0
0.2
0.4
0.6
0.8
1
1.2
Walnut
Mountain
CTL DW
CTL PL
Udall
Linear (Walnut)
Linear (Mountain)
Linear (CTL DW)
Linear (CTL PL)
Linear (Udall)


ROS Plot for NH3
0.50
0.00
-0.50
-1.00
-1.50
-2.00
-2.50
-2.5
-2
-1.5
-1
-0.5
0
0.5
y = 0.8044x - 0.1875
R² = 0.975
y = 0.8567x - 0.2494
R² = 0.9491
y = 1.158x - 0.033
R² = 0.861
CTL DW
CTL PL
Udall
Linear (CTL DW)
Linear (CTL PL)
Linear (Udall)

Appendix D: Full Water Quality Summary

	Constituent	Units	MDL	CTL Parking Lot			CTL Driveway			Mountain Ave			Walnut Ave		
				D/N	Avg EMC	Median EMC	D/N	Avg EMC	Median EMC	D/N	Avg EMC	Median EMC	D/N	Avg EMC	Median EMC
Conventional															
	Suspended Solids	mg/L		20/21	30	16	15/15	31	26	10/10	17	15	11/12	21	10
	Alkalinity	mg/L		20/20	501	417	15/15	216	215	10/10	94	90	12/12	132	119
	Hardness	mg/L		19/19	21	19	14/14	77	35	8/8	60	33	10/10	222	57
Organics															
	Chemical Oxygen Demand	mg/L		17/17	65	60	12/12	50	46	8/8	36	42	10/10	39	31
	Total Organic Carbon	mg/L		21/21	11	9	15/15	13	11	10/10	11	11	12/12	8	7
Nutrients															
	Ammonia (NH3)	mg/L	0.10	11/18	0.14	0.09	10/13	0.10	0.06	3/7	0.08	0.05	2/8	0.06	0.05
	Nitrite (NO2)	mg/L	0.05	6/10	0.10	0.07	6/7	0.10	0.07	5/6	0.10	0.11	1/6	0.03	0.02
	Nitrate (NO3)	mg/L	0.05	18/18	3.09	1.31	13/13	1.05	0.79	8/8	1.85	1.84	9/9	1.75	2.02
	Nitrate + Nitrite	mg/L		18/18	3.09	1.31	13/13	1.05	0.79	8/8	1.85	1.84	9/9	1.75	2.02
	Total Kjeldahl Nitrogen	mg/L	0.50	15/18	1.25	1.13	12/13	1.53	1.34	6/7	0.92	1.01	6/8	0.51	0.48
	Organic N	mg/L		18/18	1.11	1.05	13/13	1.43	1.07	7/7	0.85	0.96	8/8	0.46	0.42
	Total Nitrogen	mg/L		19/19	4.28	2.77	14/14	2.57	2.17	9/9	2.80	2.97	10/10	2.39	2.37
	Total Phosphorous	mg/L		14/14	0.27	0.23	11/11	0.10	0.09	8/8	0.14	0.08	8/8	0.18	0.15
	Total Inorganic Nitrogen	mg/L		17/17	3.35	1.54	12/12	1.17	0.915	7/7	1.88	1.83	8/8	1.73	1.94
Metals															
	Cadmium (D)	ug/L	0.50	0/4	0.25	0.25	0/3	0.25	0.25	0/3	0.25	0.25	0/3	0.25	0.25
	Cadmium (TR)	ug/L	0.50	1/6	0.30	0.25	0/4	0.25	0.25	0/3	0.25	0.25	1/2	0.40	0.40
	Chromium (D)	ug/L	5.00	8/8	20.86	20.03	5/6	13.88	14.55	0/4	2.50	2.50	0/2	2.50	2.50
	Chromium (TR)	ug/L	5.00	10/10	22.98	24.00	6/7	17.27	17.57	0/5	2.50	2.50	1/2	4.20	4.20
	Copper (D)	ug/L	5.00	10/10	12.93	12.16	8/8	10.73	11.16	6/6	16.28	13.50	5/7	6.69	6.87
	Copper (TR)	ug/L	5.00	9/9	30.41	17.65	7/7	14.61	14.61	5/5	24.25	19.39	5/5	11.19	10.37
	Lead (D)	ug/L	5.00	0/4	2.50	2.50	0/3	2.50	2.50	0/3	2.50	2.50	0/3	2.50	2.50
	Lead (TR)	ug/L	5.00	1/6	3.16	2.50	0/4	2.50	2.50	0/3	2.50	2.50	0/2	2.50	2.50
	Zinc (D)	ug/L	5.00	7/10	11.7	11.6	2/8	5.52	5.50	2/6	5.2	5.5	1/7	6.22	2.50
	Zinc (TR)	ug/L	5.00	9/9	31.7	21.7	7/7	23.8	14.2	5/5	20.6	18.4	4/5	19.484	17.3
Ions															
	Calcium (D)	mg/L		17/17	1.82	1.70	13/13	23.39	5.91	8/8	13.31	8.17	10/10	50.37	12.20
	Potassium (D)	mg/L		17/17	90.24	82.60	14/14	51.18	49.77	8/8	9.83	9.30	10/10	10.77	5.73
	Magnesium (D)	mg/L	0.100	15/18	0.56	0.49	9/13	0.56	0.44	7/7	5.64	2.12	9/9	23.91	8.65
	Sodium (D)	mg/L		15/15	143.01	123.94	12/12	96.19	87.11	7/7	74.58	57.81	9/9	149.96	72.49
	Chloride	mg/L		15/15	36.31	11.80	12/12	86.93	8.15	8/8	80.20	15.70	10/10	226.76	48.30
	Sulfate	mg/L		10/10	33.94	19.05	9/9	32.02	32.40	5/5	66.60	32.70	7/7	41.37	33.90
CSU Lab Tests															
	Total Dissolved Solids	mg/L		9/9	636.44444	653	7/7	355.42857	387	4/4	250	235	5/5	321	260
	Total Carbon	mg/L		2/2	55.32	55.32	2/2	67.89	67.89	2/2	40.00	40.00	2/4	40.08	40.08
	Inorganic Carbon	mg/L		2/2	36.41	36.41	2/2	45.39	45.39	2/2	20.37	20.37	2/4	26.34	26.34
Field Measured				Range	Avg	Median	Range	Avg	Median	Range	Avg	Median	Range	Avg	Median
	Temp	^{o}C		1.6 - 23.9	11.4	10.5	1.6 - 25.6	11.6	10.9	4.7 - 23.3	13.8	13.9	5.8 - 23.3	13.9	13.3
	DO	mg/L		4.0 - 10.7	7.7	8.3	5.1- 12.7	8.2	8.7	-	-	-	-	-	-
	Conductivity	uS/cm		683 - 4130	1289	999	490 - 917	677	657	203 - 250	230	237	168 - 480	308	295
	pH			9.2 - 12.6	11.1	11.1	10.3 - 12.2	11.6	11.9	8.3 - 9.8	9.1	9.0	8.2- 9.8	8.9	8.8
	Chloride	mg/L		16 - 750	180	55	21 - 560	191	84	7 - 256	111	81	14- 830	276	24
Bacteria				Range	GeoMean	Median	Range	GeoMean	Median	Range	GeoMean	Median	Range	GeoMean	Median
	Ecoli	#/100 mL		10 - 100	28	25	10 - 100	47	30	10 - 6800	124	100	10 - 560	62	100

LIST OF ABBREVIATIONS

PPS ... Permeable Pavement System
EPA ... Environmental Protection Agency
TKN ... Total Kjedahl Nitrogen
TON ... Total Organic Nitrogen
TIN ... Total Inorganic Nitrogen
TN ... Total Nitrogen
TR ... Total Recoverable
SCB ... Sample Collection Box
EIA ... Effective Impervious Area
WQA ... Water Quality Act
NPDES ... National Pollution Discharge Elimination System
BMP ... Best Management Practice
LID ... Low Impact Development
CSU ... Colorado State University
PC ... Porous Concrete
PICP ... Permeable Interlocking Concrete Pavers
MDL ... Minimum Detection Limit
EMC ... Event Mean Concentration
BASS ... Bio-Aquifer Storm System
UDFCD ... Urban Drainage and Flood Control District
PL ... Parking Lot
DW ... Driveway
PT ... Pressure Transducer
PCL ... Pollution Control Lab
MLE ... Maximum Likelihood Estimation
KM ... Kaplan-Meirs
ROS ... Regression Order Statistics
OLS ... Ordinary Least Squares
SWMM ... Stormwater Management Model

TP……………………………………………………………………………Total Phosphorous

TSS………………………………………………………………...Total Suspended Solids

TDS…………………………………………………………………..Total Dissolved Solids

Cr…………………………………………………………………………...…..Chromium

Pb……………………………………………………………………………………..Lead

Cu……………………………………………………………………………………...Copper

Zn……………………………………………………………………………………Zinc

Cd…………………………………………………………………………...Cadmium

NO3..Nitrate

NO_2……………………………………………………………………………..Nitrite

NH_3…………………………………………………………………………Ammonia

NH_4………………………………………………………………………….Ammonium

NURP…………………………………………………….National Urban Runoff Program