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Low Impact Development (LID) Structures For Groundwater Management And Watershed Protection In The Amrc10 Watershed, El Paso Texas

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By

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THESIS

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Abstract

The AMRC10 watershed was modeled in HEC-HMS and in Green Values. Theoretical storm water conveyance and capture models were tested in these programs along with several Low environmental Impact Development features to determine their applicability and performance at this site. Lots should all be designed with all roof downspouts draining into raingardens, at least half of all lawns should be natural landscaping using local vegetation, porous pavement should be used for all driveways, sidewalks and non-street pavement and drainage to the stormwater conveyance structures should make use of drainage swales instead of storm water pipes. To manage runoff three detention ponds should be constructed at the hydrologic top of the watershed placed to intercept runoff from above the watershed and manage its passage through the watershed. To convey runoff from the upper detention ponds through the watershed to the lower detention ponds there should be two unlined channels of widths 40ft and 70ft and each with side slopes of 25^{o} and depth of 5ft, spanned by a number of slotted check dams along regular lengths, 2ft tall. Beneath these channels should be a fourth detention pond that feeds into a final pond via an overflow pipe. Sub-watersheds will drain either into one of the two channels, the forth detention pond, or the final pond. Flow rates in the channels will be below 1.5 $^{ft}/_{sec}$, for up to and passing a 10 year storm, but will be exceeded by a 100 year storm. The watershed will infiltrate 65.1 AC-FT annually into the lots and swales above what can be expected of a traditional design. The expected first year savings of this design are $4,200,000. The channel and detention pond designs can be expected to infiltrate at least 87 AC-FT annually.

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Introduction

Overview of Document

The discussion of the study begins with a detailed description of the background of the project, the problem to be solved and objectives to be met. The study site and the geographical, hydrological and metrological characteristics of the area are covered to give further context to the methodology. Methodology is discussed in depth and covers all the testing conducted and the reasons for the tests. The computer models and simulation results are discussed in detail. The details of the storm water system design are made based on the results of these tests and modeling. Following that is a discussion of the results obtained through testing, modeling and performance testing of the resulting design. The last portion of the document covers all of the recommendations made regarding future study and the optimal design. The afterward is the references and an appendix of figures and tables.

Background

The AMRC10 watershed is located on the bench or mesa above the Rio Grande Valley in El Paso, Texas. The sandy soils make the area subject to erosion and this natural tendency is exacerbated by increases in peak runoff discharge caused by upstream development. Overall El Paso has a desert climate making water supply a constant challenge. Water conservation and groundwater mining have also become important issues in the southwest, as recent studies have shown that groundwater is a tenuous resource that is highly susceptible to overuse (Sheng Devere, 2005). The city of El Paso in particular relies heavily on its groundwater sources for municipal distribution, and has only recently sought for ways to address the problem of

groundwater mining (Hutchison, 2006; Sheng and Devere 2005). Since 2006, after an unexpected major storm hit and flooded the city, El Paso has been addressing the concepts of artificial recharge through the management of storm water effluent, thus potentially solving two problems at once (Hutchison, 2008). However, the development of virgin land is gradually causing what was once open soil to be covered with impermeable surfaces that send storm water effluents along lined impermeable conduits to detention ponds. This necessitates very large detention ponds in most new developments, and poses the potential effect on the underlying aquifers, which will alter the cities' overall water budget and has caused salt water infiltration from underlying saline aquifers causing deterioration in the quality. The resulting increase in stormwater runoff needs to be managed, and it is preferred that it be directed into areas of artificial groundwater recharge, so as to preserve that water resource. The Ivey property which sits within the lower portion of the AMRC10 watershed is one such area, which is intended for future development.

Statement of problem

The upper portion of the AMRC10 water shed is mostly developed without adequate controls on the changes in runoff caused by urbanization. The result of this is that most of the storm water effluent generated by the upper portion is gathered and channeled into two areas on the lower watershed. These are the Center arroyo and AMRC10 arroyo. This then leads to very high peak flows through these channels during heavy rainstorm events. Almost the entirety of the lower watershed is composed of fine sand with very light brush cover. The ease of erosion of the fine sand means that high speed of the peak flows could cause a great deal of erosion and

subsequent deposition in the existing detention pond. Fine sand will be readily eroded when being exposed to the flow exceeding 1.5 $^{ft}/_{sec}$ (Fortier, et al., 1926). At face value this implies that conventional concrete lined channels will be the only way to safely manage storm water effluent through this development. However, this comes into conflict with the desired low impact approach for the development. The problem then becomes the creation of a storm water management system that will both satisfy the needs of safe storm water effluent management while simultaneously allowing for low environmental impact, and keeping cost and land use within reasonable levels. The lower portion of the watershed is to be developed and an increase in runoff is expected as a result. The increased runoffs will need to be quantified and captured.

Objectives

The overall goal of this investigation is to determine if a Low environmental Impact Development (LID) can be implemented for the lower watershed without prohibitive expenses or limitations. The specific objectives of this study cover assessment of runoff and preliminary design of a storm water conveyance system with the new LID concept.

- Determine a reasonable storm intensity and duration for the design specifications.
    - Determining what is "reasonable" is expected to be one of the more difficult tasks, related to the expected intensities for the region and the performance expected of the resulting storm water system.

- Design a storm water conveyance system that will handle the flows resulting from the determined reasonable storm intensity and duration, while verifying that flow rate will not exceed 1.5 $^{ft}/_{sec}$. (Fortier, et al., 1926)
    - This will keep necessary maintenance of the system at or below the current standards by minimizing erosion in the unlined channels.
- Design the storm water conveyance system such that it will meet with low environmental impact standards, and quantify the benefits gained from such a system.
- Determine the expected increase in storm water runoff from developing the lower watershed in a low environmental impact manner and quantify the benefits of doing so.
    - Ensure that the design of the storm water conveyance system will be able to manage the runoff resulting from this development.

The anticipated benefits of such a design will be mostly in the realm of water conservation. Conjunctive management will manifest itself in the structures designed to retain and infiltrate storm water and in the surface ponds and channels that will slow down runoffs, mitigate peak runoffs and pond runoff for infiltration. The design of these structures will enhance the friendly environment of the area, creating park space and lots of natural landscape. The specifications for landscaping and the management of runoff will help to sustain future water supplies.

Study site

Geographical locations

The study area is approximately 442 acres of the Ivey property on the far east side of El Paso, Texas. The area is bounded by the I-10 freeway to the northeast, Texas highway 375 to the northwest, the Mesa drain to the southwest and the Socorro Grant to the southeast. The contributing drainage basin of the study is the area of suburban and commercial development to the north of the site. It extends to the Socorro sports center, and includes all the commercial and residential development directly east and west of the sports center. A total of over 1,900 acres of land feed runoff out of the drainage basin. After the proposed development, the site will be comprised mostly of residential and some commercial areas (Moreno Cardenas Inc. 2006).

Climate and weather

The climate of El Paso can easily be summed up as “dry and hot.” The annual evaporation measured in the lower El Paso Valley amounts at 80 to 100 in. However, the individual details of “dry and hot” paint an interesting picture of weather patterns not experienced in most other urban areas throughout the nation. Being part of the Chihuahuan desert, El Paso sees very little yearly precipitation, on average no more than 10 inches. Nearly half of this precipitation happens in the months of July, August, and September. This creates a kind of monsoon season for the city and is the result of warm moist air moving inland from the Gulf of Mexico. The wettest month is August with an average of 1.75 inches of precipitation. July and September are also wet months with an average of 1.49 inches and 1.61 inches of precipitation,

respectively. January through May and November are very dry months, all with an average of under 0.5 inches of precipitation. December October and June are all mildly wet months with averages of around 0.8 inches of precipitation (HAMweather, 2003-2007).

The average daily high temperature is the highest in June at around 95^{o}F. The average daily low temperature is the lowest in January at around 33^{o}F. El Paso is not known as the Sun City without reason. Ten months out of the year the average available hours of sunshine are above 80%. Only in December and January the average available hours of sunshine is just below 80%, while May and June is 90% and April 89% (HAMweather, 2003-2007).

Hydrology

The majority of the area is bluepoint classified soil and almost all of the area has been previously graded, though there are some larger tracts that remain mildly rolling. Natural arroyos provide a great deal of drainage to the area, though there are existing concrete lined channels for drainage through the previously developed areas. The current hydrology of the study site is dominated by two arroyos that enter across I-10 and intersect at the AMRC10 reservoir. All flows from the upper watershed are funneled into these arroyos. They are ephemeral and remain dry throughout the year and only see flows during heavy or prolonged rain storms (Moreno, 2006). Deposition of fine sediment has decreased permeability in the reservoir enough that it sees standing water for an extended time after large or prolonged rain storms. The entire area of the study site is composed mostly of fine sand with some clays and silts, with light brush cover. The area is capable of rapid infiltration that decreases and even

eliminates runoff for smaller rain events. However, in larger or longer storms runoff increases dramatically once the soil's infiltration rate is exceeded (US Army Corps of Engineers, 2007).

Methodology

Testing

A number of field and laboratory tests were performed to determine the characteristics of the study site. A dual ring constant head infiltration test was performed on seven sites across the entire drainage basin. Soil samples were taken at each site and dried. Constant head permeability tests and sieve tests were performed on each of the samples. A specific yield test was conducted on the sample from site 2. The purpose of the testing was to determine characteristics and behavior of soil all around the site during a rain storm.

Specific Yield Test

The test to determine specific yield was held using sand from a test site compacted into a sealed cylinder, saturated and allowed to drain over night. The resulting values were compared to the averages for fine, medium and coarse sand (Fetter, 2001).

Physical Scale Model

A river flow simulation machine was used to run a scale test of the channel designs intended for the development. Detention structures were constructed to scale within the machine and test flows matching the expected results for 10 year and 100 year storm were run through the system. The purpose of this test was to determine some of the real performance capabilities and issues of the system.

Computer Models

A HEC-HMS model was constructed using survey information compiled by Moreno Cardenas engineering firm (Moreno Cardenas Inc. 2006), and survey information previously compiled by the Hydrological team at the Texas AgriLife Research Center. HEC-HMS is a finite element modeling software that simulates overland flows from theoretical rainstorm data (US Army Corps of Engineers, 2010). Culverts, channels and other waterways were measured and delineated. Sub-water sheds were measured for area and impermeable cover, and their soil characteristics were specified based on test results.

Green Values program was used to simulate runoff from developed lots that utilize Low Environmental Impact design features to capture and store storm water. Calibration was based on the assumption that roofs will drain to raingardens at downspouts, half of all lawns will be covered by native landscaping, drainage swales will be used in place of storm water pipes and porous pavement will be used on driveways sidewalks and other non-street pavement.

The application of these properties to the project was done within the context of the differences between El Paso and the reference city for Green Values. In El Paso a raingarden will not be a lush green water sink covered in plants. Rather it will be a permeable area with several large deep root trees such as Desert Willow or Mesquite and covered with gravel. Similar structures will be used to capture the water expected to flow into swales. In El Paso any application of grass will require lots of irrigation which would defeat the purpose of such a structure, so green swales will be replaced with gutter swales using permeable pavers that lead to small catchment areas on street corners that infiltrate the water and have deep rooted trees

to stabilize the soil and pull up water during drought. It has been seen in El Paso that such trees easily survive the dry months of the year by pulling up water that has been stored in the ground beneath it during the raining season.

The output data from Green Values was used for comparison against the output data from HEC-HMS of its runoff determination for lots in the new development area. This allowed for a greater degree of certainty with regard to the HEC-HMS output simulating the development's reaction to rainstorms if developed with LID criteria. Green Values output also contained detailed cost projections and anticipated water savings due to infiltration. It anticipated the decrease in needed storage, but overall overland flow values calculated in the HEC-HMS model were used for detention pond sizing. Green Values used the equivalent of a 5 year El Paso storm to make its calculations and provided detailed documentation of how modeling of the plots and runoff were determined and quantified (CNT, 2010).

Assessment of Rainstorms

Several design storms were used to assess performance of storm water structures in the new development. An Intensity Duration Frequency (IDF) chart that had been constructed for the El Paso area by the Army Corps of Engineers was used to determine the appropriate intensities, durations, and frequencies. This curve was an input for the function of HEC-HMS. Intensity and duration calculations in HEC-HMS were based on a Snyder Hydrograph with a standard lag of 0.21 hours. A 100 year 24 hour storm was used to determine maximum conveyance capabilities while smaller 10 year 24 hour, 5 year 1 hour, and 1 year 1 hour storms were used to quantify performance over a range of loadings (US Army Corps of Engineers, 2007).

Design of Strom Water Capture and Conveyance Systems

Design of the storm water capture and conveyance systems in the new development was based around two key features. First that the newly developed lots would implement LID features to help manage storm water before it becomes runoff. The second is that the major conveyance systems will be mostly unlined to allow for infiltration and provide green spaces for the development that will require no irrigation. This requires the limitations of slope intensity, flow velocities and adherence to the existing natural flow paths. The limiting flow velocity for no erosion on a fine sand surface is 1.5 $^{ft}/_{s}$ (Fortier, et al., 1926).

Structure for Capture of Rainwater and Runoff

Fig. 1 shows a topographical map with watershed delineations for the development that was taken from a document developed by Moreno Cardenas Inc. All drainage in the development will be directed towards the channels or the AMRC10 Pond and Final Pond, marked on the figure. The Capture Pond marked on the figure is intended to catch runoff from two sub watersheds that drain directly into that spot and not allow it to travel into the channels or other ponds. A drainage pipe connecting the AMRC10 Pond to the Final Pond will handle spill over conditions in the AMRC10 Pond during very large storms. The Final Pond can be designed to be the final fate of runoff from the development, or to drain legally allowable amounts into the lower watersheds which will eventually reach the nearby Mesa Drain.

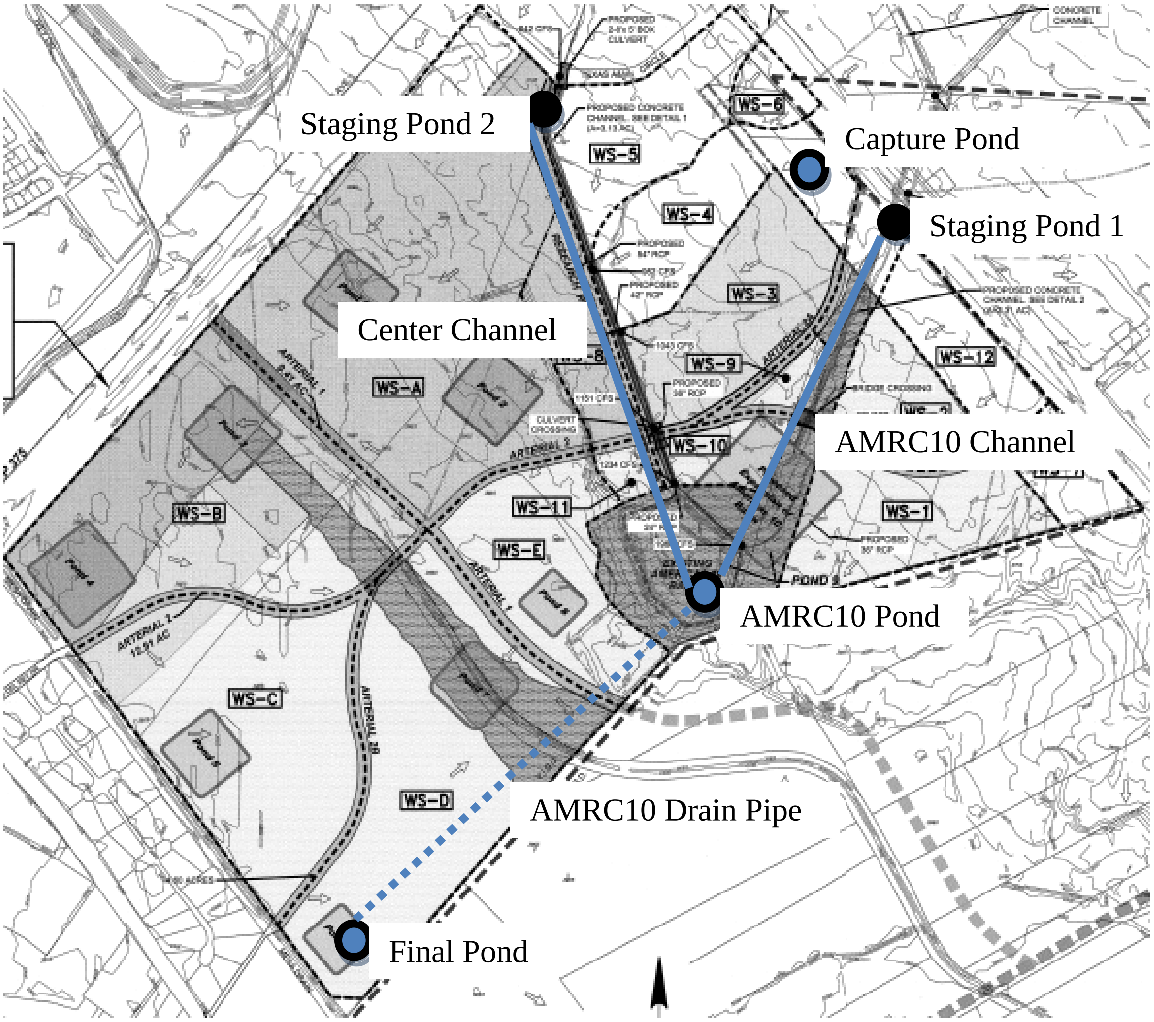

Fig. 1: Development Watershed Delineation by Moreno Cardenas Inc. and Storm Water Conveyance Design for this Study

An important feature of the design will be the staging ponds located at the top of the development intended to manage the flow from the upper watershed. Without these structures in place, the flows passing through the system would be large enough on a yearly basis as to make the use of unlined channels all but impossible. Fig. 4 shows the locations of the staging ponds just below I-10. The design presented includes large structures that will capture

flows from upstream and provide controlled predictable releases to downstream. Shallow long slopes will allow the areas to be used for park land or sports fields, if so desired but will not have storage capacities as large as might be found in standard detention pond designs. Under the two large detention ponds that feed into the channels will be a layer of fine sand over impervious Geomembrane. The two ponds in question are marked with solid black dots on Fig. 1 and Fig. 3. The sand/membrane layers will act as an artificial perched aquifer. The artificial perched aquifer will be constructed using sand available at the site. The characteristics of this available sand are such that, at a size of 5ft deep and an expanse of 10 acres, roughly the size of the staging pond, 10 AC-FT/yr of water can be expected to be available for irrigation from the pond above the AMRC10 arroyo. At 5 ft depth, and covering 2 acres, the artificial aquifer constructed under the staging pond above the Center arroyo will have 1.8 AC-FT/yr of water available for irrigation. Taking the rain distribution of El Paso into consideration, the aquifers can be expected to be saturated at the end of September and at the middle or end of December. From January until June, it should be expected that no recharge will occur and that all irrigation uses will cause draw down. This should be matched with any intended irrigation uses, such as a sports field, or landscaping. A free flowing pipe with a screened opening inside the bottom of the perched aquifer will feed the desert climate trees used to stabilize the slopes of the channels below the ponds. A valve, accessible at the surface just above the first irrigation outlet, will allow for stopping flow when irrigation is not needed. Alternatively, a shallow well can be drilled into the perched aquifer and used to extract water for turf fields in the pond. Either application will allow avoidance of municipal services for irrigation; however the first option, the screened pipe feeding threes for slope stabilization, is much more likely to require

no municipal services for irrigation in the long term as irrigating a sports field will likely quickly deplete the available water available in the saturated perched aquifer.

Fig. 2 illustrates the composition of the pond, sand layer and Geomembrane to construct the artificial perched aquifer. It also shows the general pathway for the well screened pipe to transport the stored water for gravity fed irrigation.

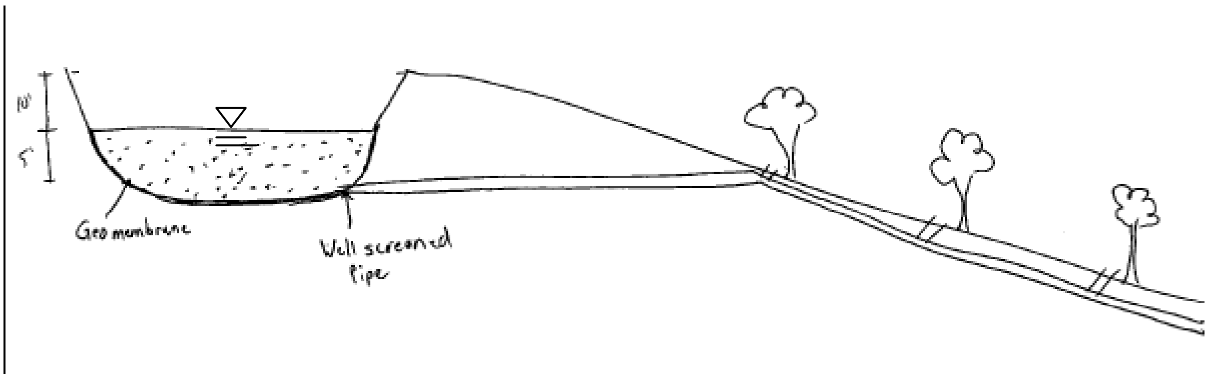

Fig. 2 Pond and Artificial Perched Aquifer

Fig. 3 illustrates the spillways that will allow water from inside the staging ponds to flow into the channels. The long notch through the center of the spillway will be 0.5ft wide at the top and taper to a point. This will allow sediment to flow through the system without getting trapped in the staging ponds. The result is higher flows and the need for greater storage at the end of the system. The benefit is that there will be a much reduced need for annual maintenance due to sedimentation. It is also recommended that a wire mesh be installed over the notch to prevent serious clogging within the opening. The wire mesh can be cleared of debris as needed with much less difficulty than flushing out the whole notch.

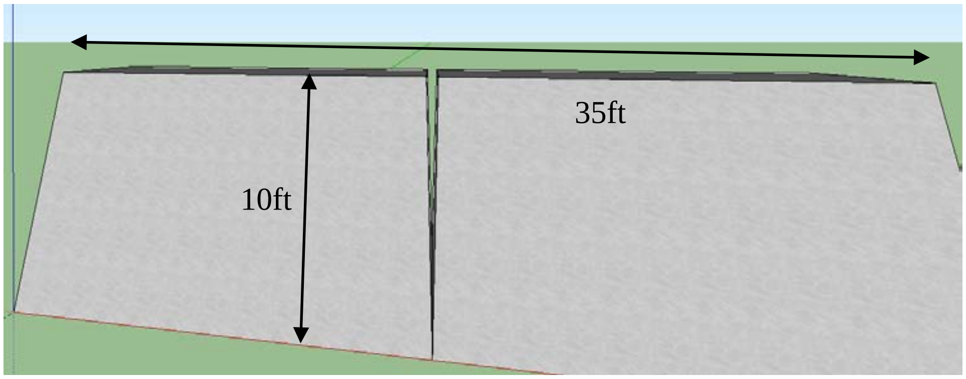

Fig. 3: Staging Pond Spillway

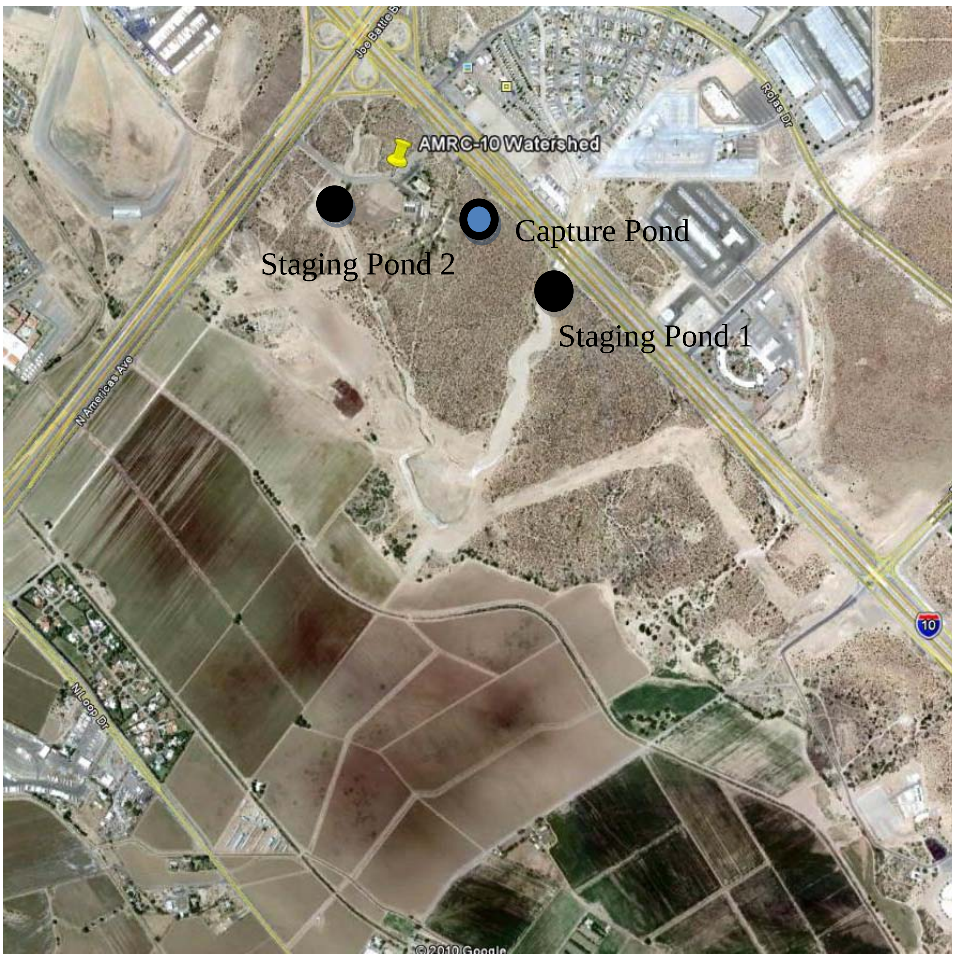

Fig. 4: Staging Pond Locations

Tables 1 through 4 show the elevation, area and discharge relationships calculated for the various ponds to assess storage and release of surface flow through the ponds and, where applicable into the corresponding channels.

Table 1: Staging Pond 1 Elevation/Area/Discharge

Staging Pond 1			
Elevation (ft)	Area (AC)	Discharge (CFS)	Storage (AC-FT)
0	0.01	0.0	0.0
1	4.00	0.7	5.8
2	4.25	4.5	11.7
3	4.50	13.2	17.4
4	4.75	28.4	23.2
5	5.00	51.5	29.0
6	5.25	83.8	34.8
7	5.50	126.4	40.6
8	5.75	180.4	46.4
9	6.00	247.0	52.2
10	6.50	327.1	58.0
11	7.00	404.8	63.8
12	8.00	730.8	69.6
13	9.00	1195.8	75.4
14	10.00	1766.1	81.2

Table 2: Staging Pond 2 Elevation/Area/Discharge

Staging Pond 2			
Elevation (ft)	Area (AC)	Discharge(CFS)	Storage (AC-FT)
0	0.25	0.0	0.0
1	0.50	0.7	1.1
2	0.75	4.5	2.3
3	1.00	13.2	3.4
4	1.25	28.4	4.6
5	1.50	51.5	5.7
6	1.75	129.2	6.8
8	2.25	455.2	8.0
10	2.75	920.2	9.1

Table 3: AMRC10 Pond Elevation/Area/Discharge

AMRC10 Pond			
Elevation (ft)	Area (AC)	Discharge (CFS)	Storage (AC-FT)
0	0.0	0.0	0.0
2	2.0	0.0	8.7
4	3.0	0.0	17.5
6	4.0	0.0	26.2
8	5.0	0.0	34.9
10	6.0	77.7	43.6
12	7.0	403.7	52.4
14	8.0	868.7	61.1
15	8.5	1054.9	65.5

Table 4: Final Pond Elevation/Area

Final Pond		
Elevation (ft)	Area (AC)	Storage (AC-FT)
0	0.0	0.0
1	13.0	11.7
2	13.5	23.3
3	14.0	35.0
4	14.5	46.6
5	15.0	58.3
6	15.5	69.9
7	16.0	81.6
8	16.5	93.2
9	17.0	104.9
10	17.5	116.5
11	18.0	128.2
12	18.5	139.8

Design of Conveyance Channels

The arroyos conveying the runoff are designed unlined and with very shallow slopes and wide channels to keep flows shallow. Slotted check dams are included along the arroyos to increase storage, slow flows and reduce erosion. The performance of these channels and check dams were tested extensively in HEC-HMS and in scale models. Stabilization of the side slopes of these channels is achieved by the use of native desert climate trees such as Desert Willow and Mesquite, which provide significant shade and flower very nicely in the spring. Another potential for slope stabilization is the use of cellular confinement mats that drastically increase soil shear strength and allow for intricate deep root structures in the plants used for landscaping. Further testing of the local soil shear strength would be needed to determine the expected reliability of slope stability. For purposes of design, the angle of repose and typical shear strength of dense compacted sand was used. This allowed for assessment of the effects of flows for various storms in the designed unlined channels. The location of the channels is shown in Fig. 5.

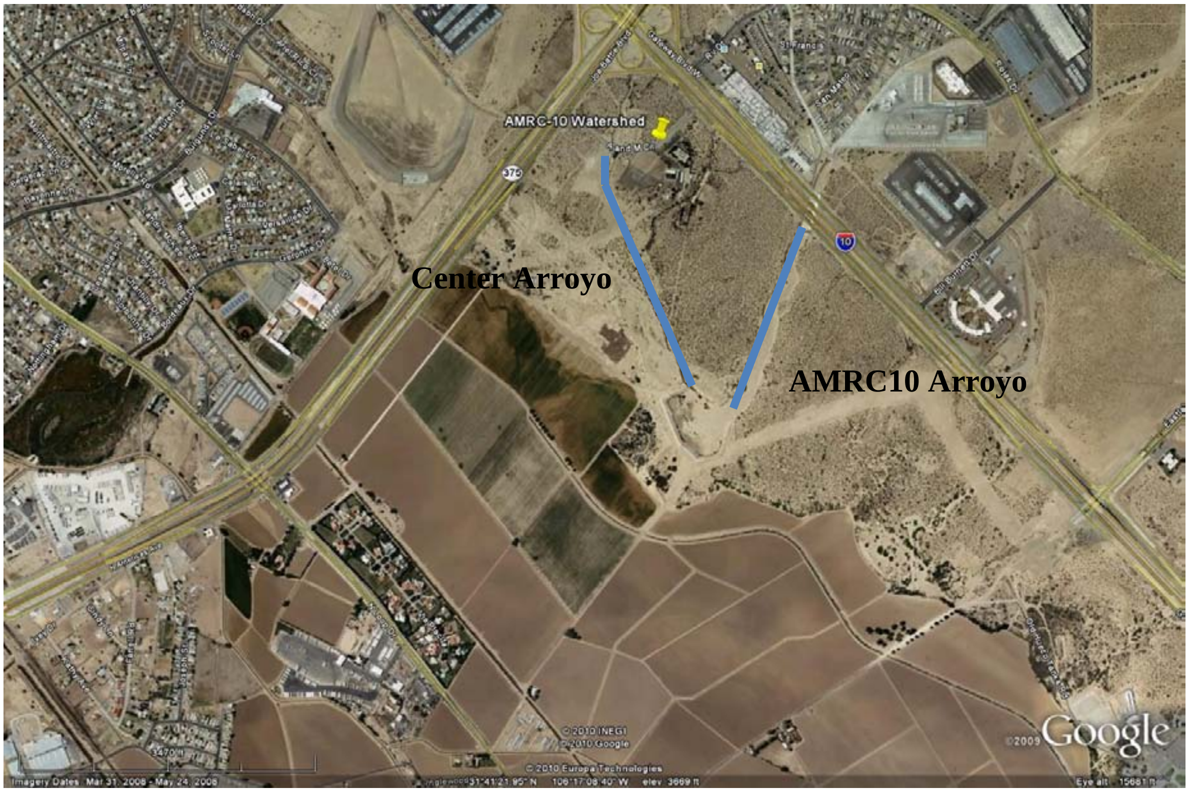

Fig. 5: Location of Center and AMRC10 Arroyos

Fig. 6 through Fig. 10 show a three dimensional scale model of the AMRC10 channel and its check dams. Fig.6 includes a person for perspective. Important to notice in the figure is curve of the check dam and the slots evenly spaced along its length. This design is intended to direct flows through the middle of the channel when runoff is not enough to create deeper flows. Of note is also the stone and mortar construction of the check dam. It is thought that this type of construction will be more aesthetically pleasing for the land owner.

Fig. 7 shows the check dam and channel in cross section. The channel width is 70 ft, with slopes that are 5 ft high covering 11 ft length. This creates a slope angle of roughly 25^{o}. The Center channel will be constructed identically, except that width will be 40 ft instead of 70 ft, and the slots in the check dams will be spaced accordingly.

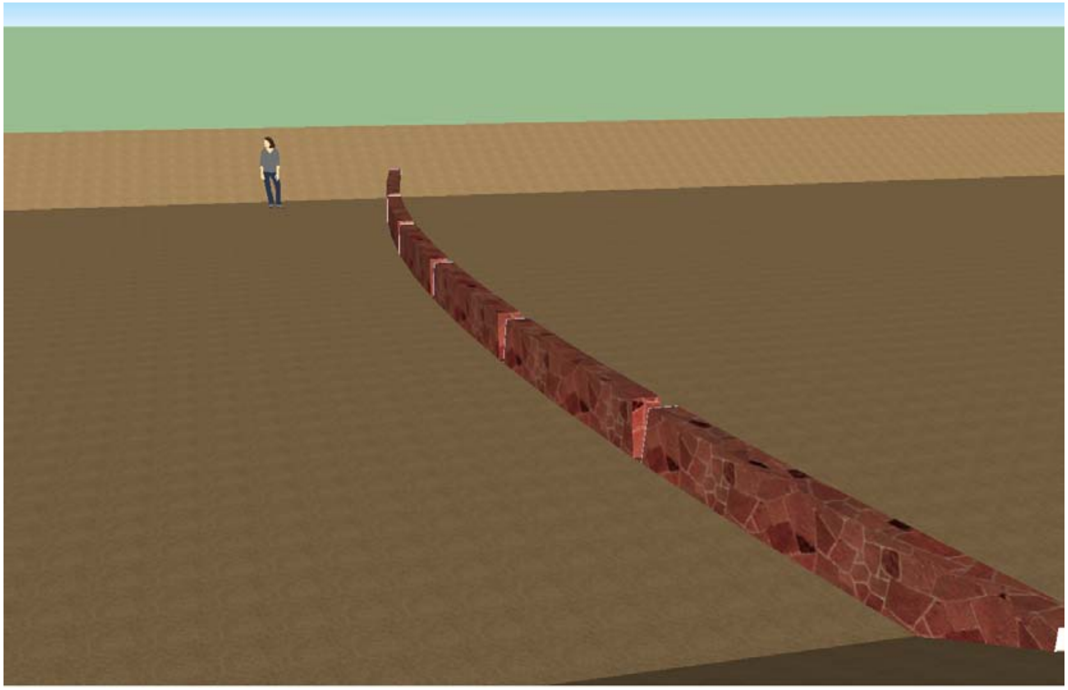

Fig. 6: Scale Model, Check Dam

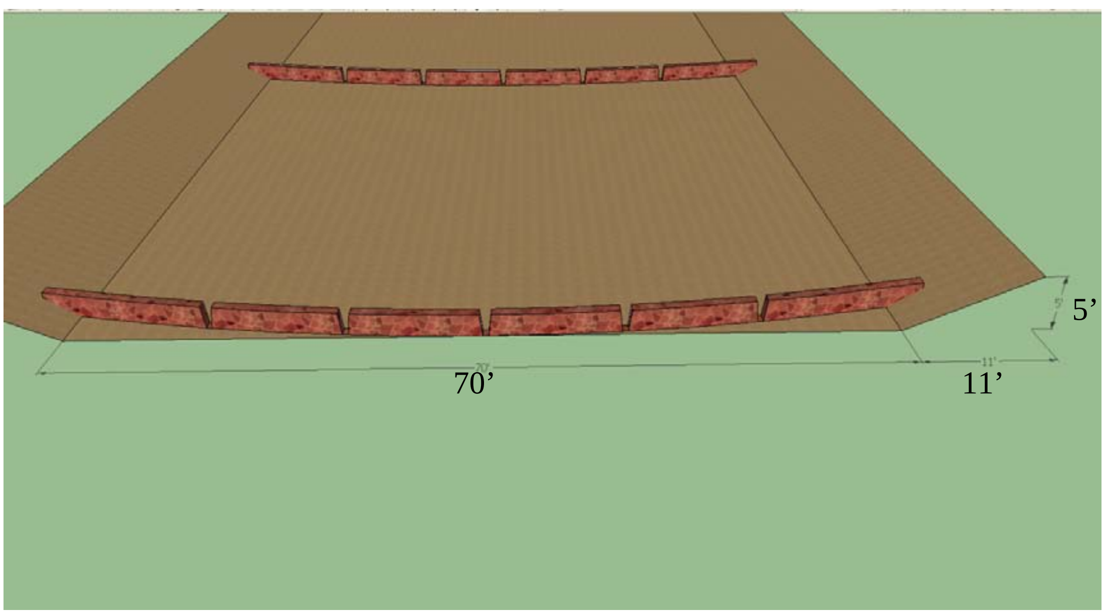

Fig. 7: Scale Model, Check Dam, Channel, and Channel Dimensions

Fig. 8 through Fig. 10 show closer views of the slots in the check dams. Fig. 8 shows the spacing of the slots along the width of the channel. Fig. 9 is a close up of a single slot. The dimensions of the slot are marked on the figure. It is triangular with a 1 ft width at the top and 6 in width at

the base. Fig. 10 shows the dimensions of the spacing of the slots and height of the dam. They are 11 ft 5 in apart and the dam is 2 ft tall. The dams will extend into the side slopes at least half of the 11’ width to prevent flows eating around the edges. This is important to the success of the design.

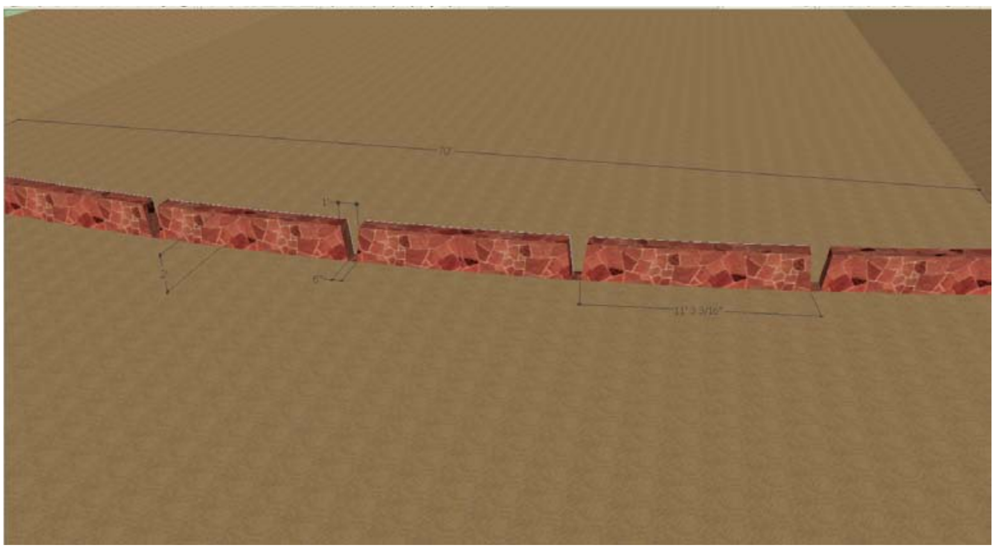

Fig. 8: Check Dam and Slot Dimensions

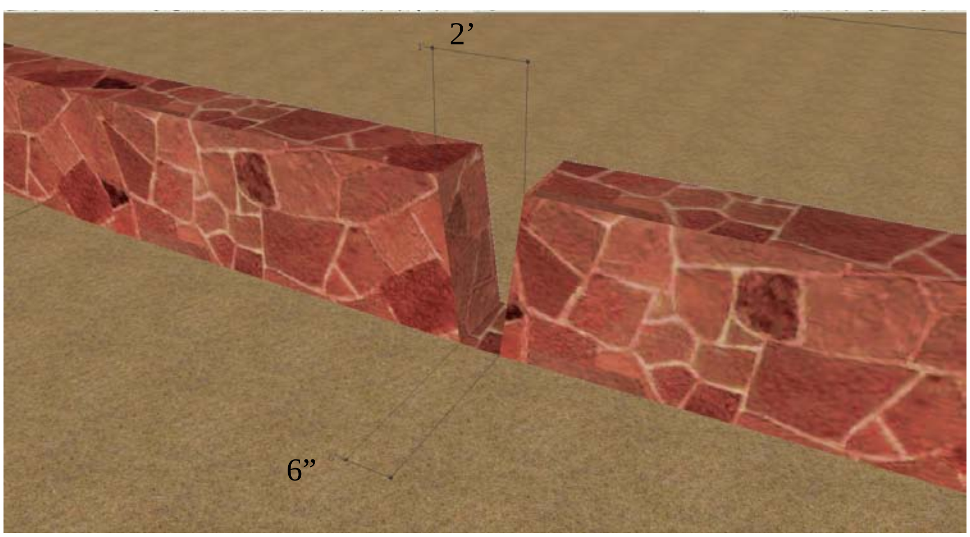

Fig. 9: Slot Dimensions: Detail

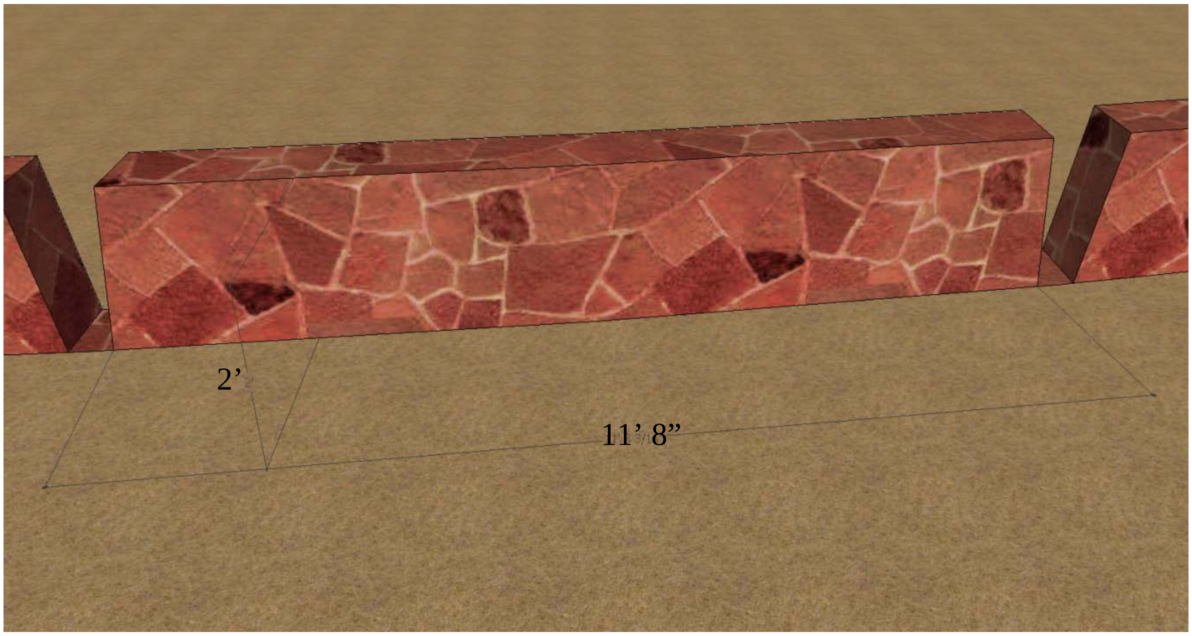

Fig. 10: Check Dam Dimensions: Detail

Results and Discussion

Testing Results

Nine sites within the AMRC-10 watershed were used to run Constant Head Dual Ring Infiltration Tests and to collect soil samples for conducting Constant Head Permeability Tests in the lab and Sieve Tests. Detailed tables and figures on the results of these tests are included in the appendix section. Fig. 11 shows the locations of each of the test sites.

Fig. 11: Test site map

Table 5 shows the coordinates of each of the testing sites as determined by GPS locator.

Table 5: Site Coordinates

	Lat N	Long W
Site 2	31.69369265	106.2783523
Site 3	31.706965	106.278589
Site 4	31.697903	106.2835336
Site 5	31.71005	106.2737819
Site 7	31.702279	106.27858
Site 8	31.696822	106.2831023

Site 1

Site 1 has thus far been unavailable for testing. Past rain storms and heavy sedimentation have left the pond saturated and flooded, so there is no means by which an infiltration test can be conducted. Furthermore, as the “Pond” element in HEC-HMS does not allow for infiltration in compiling, such testing proved to be unnecessary.

Site 2

Site 2 is located within the upper area of the AMRC10 arroyo. The soil is very sandy, and composed entirely of sediment traveling from upstream during rainstorm events. Conducting the test on this site was difficult because the infiltration rate was so high throughout the test. This difficulty caused some rather erratic and not fully reliable results. For the final calculation and data presentation a large number of outliers were removed from the results in order to create a more reasonable estimation of the infiltration rates.

A second test was conducted on the same spot, immediately after the first test. While the first test was a constant head infiltration test, the second was a direct measurement test with dynamic head. After the data for both tests was analyzed and outliers were removed from the set, the results of both the tests were reasonably similar.

For both tests, data was plotted and a trend line was established using the power method, as this most closely resembles the behavior of infiltrating water. R^2 values were determined and outliers were altered until suitable curve fitting was established. The resulting conclusions were fairly consistent between both tests. Stable infiltration is expected to be in the area of 9 inches

per hour (in/hr). Average permeability was found to be 49.1 $^{in}/_{hr}$ by the constant head permeability test in the lab. It is believed that testing errors created the huge discrepancy in the test results. Sieve testing showed the soil to have an AASHTO classification of A-3, fine sand.

Site 3

The soil appeared to be composed of sandy clay. The entire area has been graded and compacted and much of the soil is heavily consolidated. This made the test slightly difficult to set up for but fairly free of difficulties to conduct.

The recorded data for site 3 was very stable and easily plotted. There were no serious outliers. A trend line was established using the power method and the R^2 value was determined. The resulting values were found to be very reliable. Saturated infiltration rates for site 3 are expected to be in the area of 1.2 in/hr. Average permeability was found to be 0.75 $^{in}/_{hr}$ by the constant head permeability test. Sieve testing showed the soil to have an AASHTO classification of A-3, fine sand.

Site 4

Site 4 is located to the north of the AgriLife center in the undeveloped graded area adjacent to the AgriLife arroyo. The area is mostly sandy with some clay and a little silt. It has been previously graded, but has been heavily influenced by subsequent rainfall events. The soil is easily eroded and has had its slope significantly altered by surface flows during larger storms. The sandy consistency of the soil made the test easy to initiate, but the resulting high rate of

infiltration made the test slightly difficult to properly conduct. The data from the test was similarly somewhat erratic and needed to be adjusted to provide usable results.

Unfortunately there were a large number of outliers which make the results fairly unreliable. After eliminating the outliers a reasonable trend line was established using the power method. The line had a suitable R^2 value. The resulting saturated infiltration rate was found to be in the area of 11 $^{in}/_{hr}$. Average permeability was found to be 0.33 $^{in}/_{hr}$ by the constant head permeability test. Sieve testing showed the soil to have an AASHTO classification of A-2, loamy sand.

Site 5

Site 5 is located a little to the east of Loop 375 and a little south of the Socorro stadium. The soil in the area was mostly sandy clay. There were scattered occurrences of caliches and some larger bits of gravel spread all throughout the site. The soil was stiffly consolidated, and that coupled with the instances of caliches made preparing the test site somewhat difficult. The test itself, however, proved simple and easy to conduct, as the slower infiltration time allowed for readings to be more easily recorded.

The data collected for site 5 was fairly consistent with only a few outliers that occurred only briefly after the test was reset and restarted. All the rest of the data for site 5 follows similar changes. The data was plotted and a trend line was established using the power method. The R^2 value was within acceptable ranges once the few outliers were removed from consideration. Infiltration rate when saturated was found to be in the area of 4 in/hr. Average permeability

was found to be 2.55 $^{in}/_{hr}$ by the constant head permeability test. Sieve testing showed the soil to have an AASHTO classification of A-2, loamy sand.

Site 6

Site 6 was intended to be the commercial site north of the Socorro stadium. The site was abandoned due to time constraints and existing similarities with site 3.

Site 7

Site 7 is located on the northwest border of the residential area that is just north east of I-10. The soil at this site consisted of a sand layer spread over clayey sand. It was fully graded and compacted but not very consolidated. It is representative of the locations in the residential area that are not covered by impermeable linings nor have vegetative cover, as both types would have fairly high infiltration. Setting up the test in such soft soil was simple, but performing the test required several resets to refill the tubes.

The infiltration rate was fairly quick here, and fairly steadily so, but the resulting data was unfortunately more erratic than would be preferred. Each time the test was reset the rates changed drastically and so there were a number of wide outliers that needed to be removed before the data was really usable. As such, the results are somewhat telling, but not completely reliable. Once the outliers were removed a trend line was able to be established using the power method and the R^2 value was within the acceptable range. The infiltration rate when saturated was found to be in the area of 4.5 in/hr. Average permeability was found to be 2.31

$^{in}/_{hr}$ by the constant head permeability test. Sieve testing showed the soil to have an AASHTO classification of A-3, fine sand.

Site 8

Site 8 is located immediately adjacent to the AgriLife research center. The soil in this area was sandy with some clays and silts. It was fully graded and compacted, and fairly consolidated. There is some light brush cover on the site, but it is mostly bare. The softness of the soil made the test easy to set up. The resulting infiltration rates made the test fairly simple to conduct, and the resulting findings were fairly stable.

The resulting data from this test was very consistent. There were no obvious outliers in any of the recordings, and a trend line was able to be established using the power method without removing any data points. The R^2 value was reasonably high and the curve fit the plotted points fairly well. The resulting saturated infiltration rate was found to be in the area of 3.0 in/hr. Average permeability was found to be 0.19 $^{in}/_{hr}$ by the constant head permeability test. Sieve testing showed the soil to have an AASHTO classification of A-3, fine sand.

Site 9

Site 9 is located mostly to the east of the Socorro stadium. It is a very large area of largely undeveloped land, which is currently under the process of development. Infiltration was found to be in the area of 8 $^{in}/_{hr}$. The R^2 value was, however, fairly low at 0.24 and so the results cannot be taken and clearly indicative. Average permeability was found to be 2.43 $^{in}/_{hr}$ by the

constant head permeability test. Sieve testing showed the soil to have an AASHTO classification of A-3, fine sand.

In summary, the testing was fairly successful with some variation and inconsistencies not altogether outside of expected ranges. The resulting data was enough to properly calibrate the HEC-HMS model for expected infiltration in the sub basins.

Modeling results

Primary concerns were over sizing involved the two major channels, AMRC10 arroyo and Center Channel arroyo. HEC-HMS models clearly showed that flows from the upper watershed are concentrated through these two conduits. As the main runoff control systems, designing them in an LID method was a major goal and a large challenge. Sizing the channels was a heavily iterative process that was dependant on features within the channel and the width of the channel itself. Each of the channels will be crossed by a series of staggered 2 ft tall check dams. The accumulation of sediment behind these dams was anticipated to become a problem and as such they were designed to allow outflow to wash sediment away from behind the check dams.

Center channel width is 40 ft. It holds 14 check dams evenly spaced along its length. The slope of the channel is 0.01 $^{ft}/_{ft}$. The channel is 2958 ft long. AMRC10 channel width is 70 ft. It holds 27 check dams evenly spaced along its length. The slope of the channel is 0.024 $^{ft}/_{ft}$. The channel is 2742 ft long. The existing physical conditions were maintained as much as possible in the design so as to preserve natural slope and flow direction. This will prevent erosion from flow direction changes caused by development. It is a necessary feature of unlined channel design (Temple, D.M., etc all, 2003).

One year, five year, and ten year El Paso design storms were used to determine the performance of the systems at different sizes until an effective system size was found. The limiting factor was keeping flow speeds under 1.5 $^{ft}/_{sec}$ while also keeping the system size small enough to be economical. Flow speeds were calculated from the results of the model estimations of water elevation, volumetric flow rates, and channel widths.

For a 1-year storm the highest estimated flow speed in the AMRC10 channel was in the area of 0.2 $^{ft}/_{sec}$, 6.1CFS at 0.45ft depth. Many portions of the channel did not experience any flow at all. The highest flow in the Center Channel was estimated to be around 0.4 $^{ft}/_{sec}$, 18.5CFS at 1.1ft depth. It should be noted that flow concentrations are expected where individual sub-watersheds are outputting into the channel. Channel lining may be necessary at these junctions.

For a 10-year storm the highest estimated flow speed in the AMRC10 channel is 1.22 $^{ft}/_{sec}$, 230CFS, at 2.7ft depth. Highest estimated flow for the Center channel was 1.5 $^{ft}/_{sec}$, 163.4CFS at 2.75ft depth. Similarly, spiked flow speeds are expected where sub watersheds dump into the channel. The resulting higher flows below the sub watersheds are evidence of the increase.

For a 100-year storm the highest estimated flow speed in the AMRC10 channel is 3.1 $^{ft}/_{sec}$, 728CFS at 3.4ft depth. In the Center Channel the highest estimated flow speed was 2.8 $^{ft}/_{sec}$, 388CFS at 3.5ft depth. For the 100 year storm the two channels should directly infiltrate around 31 Acre-ft of runoff.

Table 6 lists the calculated maxes.

Table 6: HEC-HMS Model Results

Storm Size	Center Channel			AMRC10 Channel		
	Depth (ft)	Speed (ft/sec)	Flow (CFS)	Depth (ft)	Speed (ft/sec)	Flow (CFS)
1	1.10	0.4	18.5	0.45	0.20	6.1
10	2.75	1.5	163.4	2.70	1.22	230.0
100	3.50	2.8	388.0	3.40	3.10	728.0

Results of Scale Model Test

In the scale simulation test a river flow simulation machine was used to construct a scale model of a stretch of the Center Channel to assess the effects of real flowing water as would result from storm runoff on the strength, integrity and performance of the design. The model channel was constructed in sand as a 1:40 scale stretch with similarly scaled runoff detention structures. The check dams were constructed of sheet aluminum cut to exactly simulate the designed check dams at a 1:40 scale. Also tested were check dams constructed of what would be large boulders cemented into the channel bad, at scale. The concept is that such an approach with natural building materials would be much more aesthetically pleasing and preferable from a developer and buyer point of view. It was desired to see how such structures would perform under the same loading as the aluminum scale check dam models.

The machine holding the constructed scale model channel and check damns is shown in Fig. 12. Fig. 13 is a close-up of the check dam structures within the model. In Fig. 12 one can see clearly the scale height and open flow spaces of the boulder check dam and the scale height and slotted openings in the standard check dam.

Fig. 12: Scale Model of Center Channel for Testing

Fig. 13: Scale Model Close Up

Fig. 14 and Fig. 15 show water flowing through the model channel to a depth concurrent with computer modeled 10 year storm. You can see clearly the retention and flow velocity reduction

caused by the check dams. It can also be seen in Fig. 15 that the flow is directed more toward the center of the channel by the curvature of the check dams. During the test it was seen that the slots in the check dams will create significant erosion just beneath them and that this was be guarded against. Gabions placed just beneath the slots are suggested. In the bolder check dams it was seen that flow in between the gaps will cause significant erosion as well. It will be necessary to cement them in place with adequate footing and to place gabions directly beneath the boulders to prevent erosion there. Flowing dye through the system showed localized flow speed increases through the slots, between the boulders and in flow over the check dams. This localized increased flow speed is what caused increased erosion around the check dams and cannot easily be mitigated outside of reinforcing against the potential erosion.

Fig. 14: Scale Model Check Dam Performance 10 Year

Fig. 15: Check Dam Flow Direction

Fig. 16 shows the system at full flow during 10 year depth simulation. Little significant erosion was observed except for within the boulder check dams and just below the standard check dams. It can be reasonably surmised that the system will hold up well to this size of a storm.

Fig. 16: Scale Model 10 Year Steady Flow

Fig. 17 shows full flow at depth for a simulated 100 year storm. The green dye used to indentify localized flow speeds is also seen in the figure. The dye demonstrated areas where flow speeds were significantly decreased and the small areas directly around the checks where speeds were significantly increased. In the figure the slope erosions caused by such a high flow are seen.

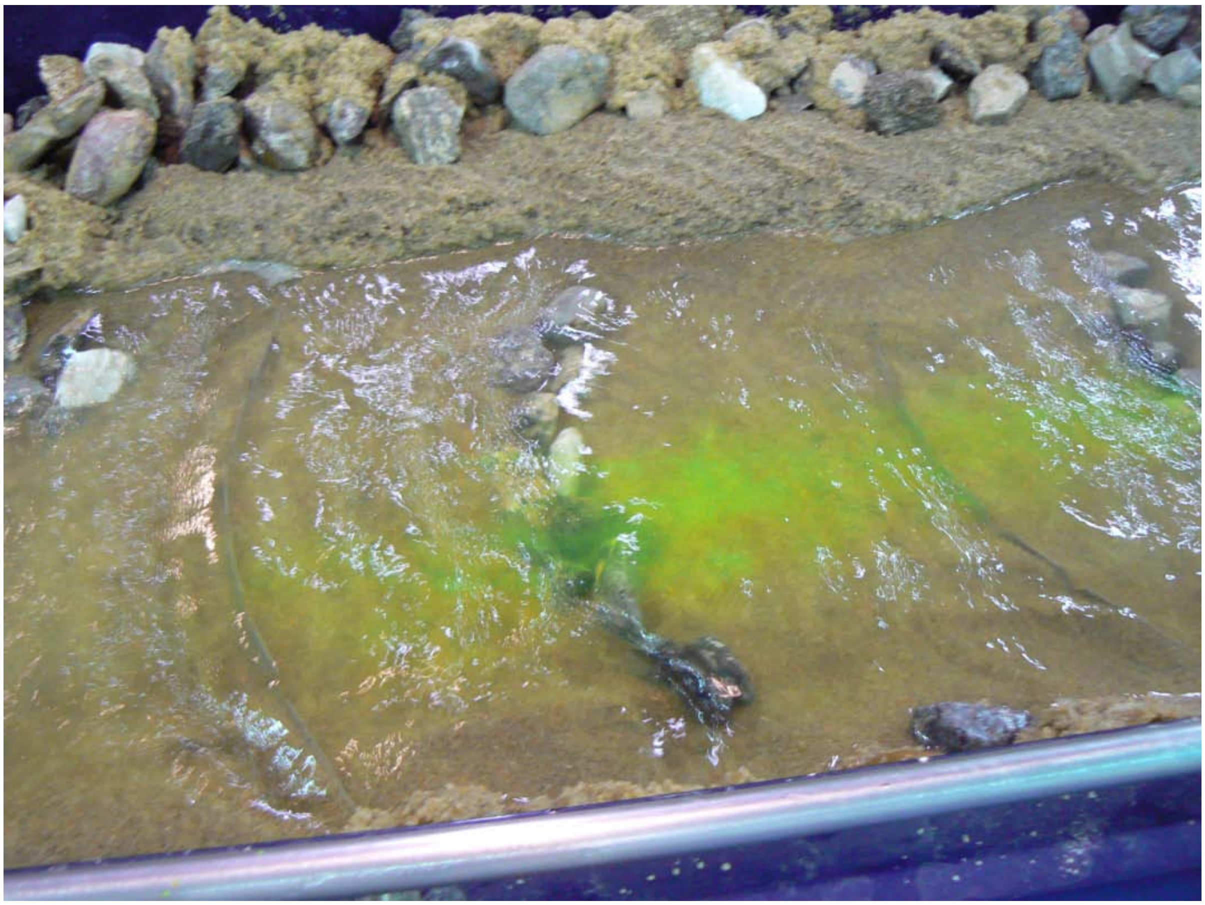

Fig. 17: Scale Model 100 Year Steady Flow

Fig. 18 shows the aftermath of 100 year flow simulation. In particular, what is shown in the figure is the most damaged portion of the model. During a 100 year storm significant erosion was seen along all channel sides and below each check dam, especially the final dam. It was demonstrated that each dam significantly backed up flows behind it, decreasing flow speeds for the checks behind it and decreasing the erosion they experienced. As the final check, the last

check dam experienced the highest velocity flows and the most erosion. It can be expected that where the channels expels into a detention pond, or where flows are increased by an inlet structure there will need to be some extra reinforcement against erosion.

Slope degradation was also witnessed at higher flows. As the sand used in the machine is relatively comparable to the sand that will be available on site it becomes apparent that some form of slope stabilization will be needed if it is intended that such large scale erosion and destruction in the system is to be avoided. Fortunately, the design is specified such that a different, stronger soil composition is to be used on the side slopes and other reinforcement methods are to be applied as well.

The overall results of the scale model test have to do with the expected maintenance of the channels. As one would expect needed maintenance increases with the size of the storm. Since most upkeep will revolve around managing sedimentation and looking at the damaged caused in the testing one can make the assumption that the typical annual storm will require little to no maintenance. However the larger storms will cause a small degree of damage or sediment deposition. For 100 year storms or higher it can be expected that some slope reconstruction may be necessary. As it is impossible to prevent all cases of erosion, even under very low flows, some sediment management will be needed on an annual basis, but this value can be expected to be fairly low and perhaps even negligible.

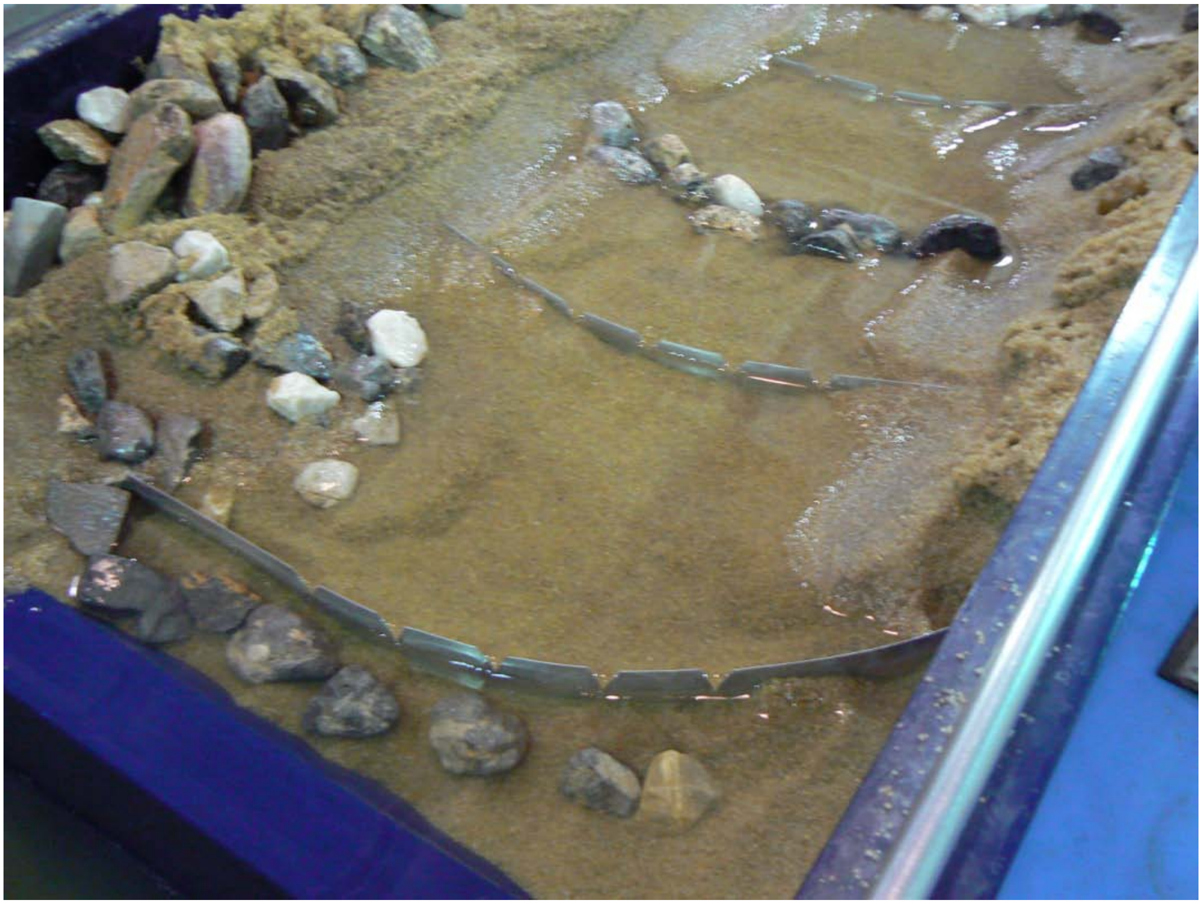

Fig. 18: Scale Model After 100 Year

By the specific yield test it was determined that the specific yield for the soil at site 2 is 18%. It should be considered that the specific yield for different grain sizes in the event of soil engineering or placement in different locations will see some fluctuation and further calculations should be made in these cases. Fig. 19 shows the test during draining and weighing.

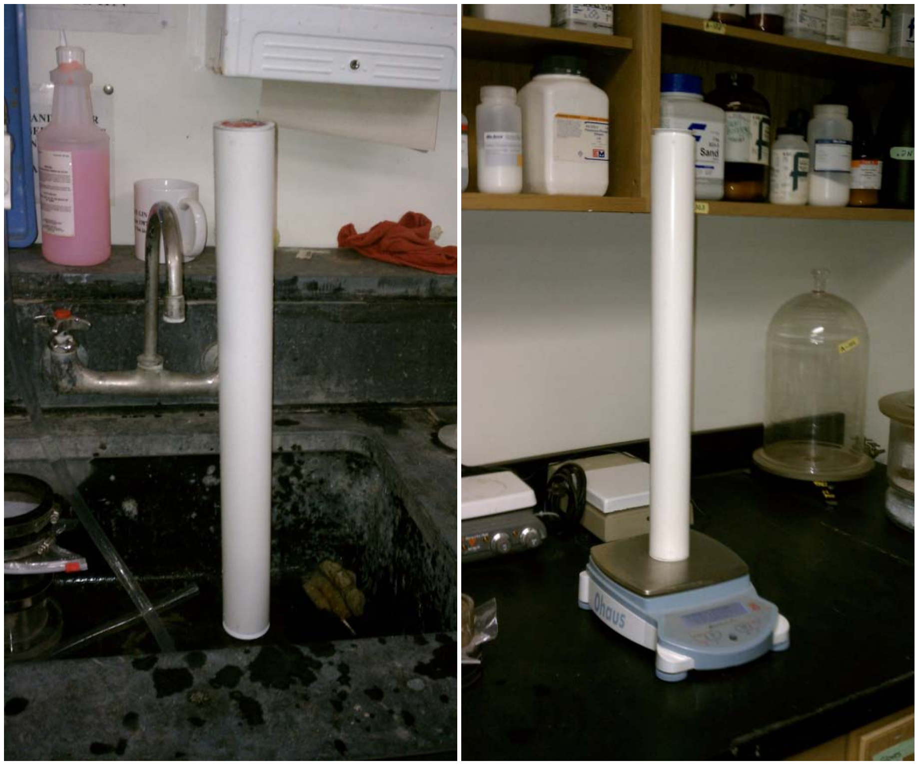

Fig. 19: Specific Yield Test

Storage, Retention and Water Savings – HEC-HMS and Green Values Comparison (anticipated annual)

Outflows calculated on each lot were compared between HEC-HMS and Green Values for an El Paso 5 year storm to calibrate the choices within the models and verify similitude.

Green Values calculated a first year savings of $4,200,000 on the development if certain Low Impact development features were implemented on each lot. In this particular design roofs will drain to raingardens at all downspouts. Half of all lawns will be covered by gardens with natural native landscaping. Porous pavements will be used on driveways sidewalks and other non-street pavements. Drainage swales will be used in place of stormwater pipes. A 50 year life cycle was assumed for all cost and benefit calculations. In calculating dollar values, Green

Values used Low Cost, Mid Cost and High Cost estimates for Construction, Maintenance and component Lifespan. These estimates were applied individually to concrete sidewalk and driveway, curbs and gutters, detention basins, green roofs, native plants, porous pavement, rain garden, sewer pipes, standard roof, street, trees, turn, vegetated swale averaged, vegetated swale planting, vegetated swale, and planter box. Each item was estimated as applicable for the specifics of this development and those that were not applicable were not included in the estimation. The Green Values website cited in this document provides a list of textbook citations as justification for its estimation procedures. Per lot life cycle costs Reduction can be expected to be $57,000 and total life cycle costs reductions can be expected to be $13,400,000. First year savings per lot can be expected to be in the range of $18,000. Per lot benefits over the 50 year life cycle can be expected to be increased by $650 and total life cycle benefits to be increased by $157,900. Benefits are calculated based on an assessment of reduced air pollutants, carbon dioxide sequestration, tree value, energy use and urban heat island effect reduction, groundwater recharge, reduced energy use, total suspended solids and total phosphorus removal, reduced treatment benefits, aesthetic, erosion prevention, flood prevention, habitat, mobility, property value, public health, raingardens, recreation, salt use reduction, shelter and sound absorption. As with cost estimates, a detailed list of citations and methods can be found on the Green Values website cited in this document. Table 7 summarizes the results of this analysis.

Table 7: Costs and Benefits Green Value Analysis

Costs			
Present Value Over 50 Year Life Cycle	Conventional	Green	Reduction
Per Lot Life Cycle Costs	$ 260,903.00	$ 203,825.00	$ 57,079.00
Total Life Cycle Costs	$ 61,051.00	$ 47,695,007.00	$ 13,356,377.00
First Year Site Construction and Maintenance Costs	Conventional	Green	Reduction
Per Lot Costs	$ 60,630.00	$ 42,567.00	$ 18,063.00
Total Costs	$ 14,187,514.00	$ 9,960,695.00	$ 4,226,819.00
Benefits			
Present Value Over 50 Year Life Cycle	Conventional	Green	Reduction
Per Lot Life Benefits	$ -	$ 675.00	$ 675.00
Total Life Benefits	$ -	$ 157,874.00	$ 157,874.00

Costs and benefits were further subjected to a breakout analysis. The findings of this analysis are summarized on Table 8. It presents a number of the same values as Table 7 but includes the present worth of the 50 year lifecycle savings for public costs and homeowner costs based on a per lot and total basis. Taking the total first year costs and maintenance saving and converting them to an equivalent annual worth across the 50 year life cycle, it is found that the equivalent annual savings is $164,000. This means that the first year savings experienced are the same as spending $164,000 less per year on maintenance and upkeep.

Table 8: Cost and Benefit Breakout Green Values Analysis

Cost Breakout			
Developer's Construction and Maintenance Costs	Conventional	Green	Reduction
Per Lot Costs	$ 60,630.00	$ 42,567.00	$ 18,063.00
Total Costs	$ 14,187,514.00	$ 9,960,695.00	$ 4,226,819.00
Present Value Over 50 Year Life Cycle Public Costs	Conventional	Green	Reduction
Per Lot Life Cycle Cost	$ 9,717.00	$ 7,252.00	$ 2,465.00
Total Life Cycle Cost	$ 2,273,720.00	$ 1,696,858.00	$ 576,862.00
Present Value Over 50 Year Life Cycle Homeowner costs	Conventional	Green	Reduction
Per Lot Life Cycle Cost	$ 190,556.00	$ 154,006.00	$ 36,550.00
Total Life Cycle Cost	$ 44,590,150.00	$ 3,603,745.00	$ 8,552,696.00
Benefit Breakout			
Present Value Over 50 Year Life Cycle Public Benefits	Conventional	Green	Increase
Per Lot Life Cycle Benefits	$ -	$ 675.00	$ 675.00
Total Life Cycle Benefits	$ -	$ 157,874.00	$ 157,874.00
Present Value Over 50 Year Life Cycle Homeowner Benefits	Conventional	Green	Increase
Per Lot Life Cycle Benefits	$ -	$ -	$ -
Total Life Cycle Benefits	$ -	$ -	$ -

Green Values also calculated an annual increase in recharge from the developed lots of 65.1 AC-FT/yr, over what would be expected from a conventional development. The channel and detention pond designs can be expected to infiltrate at least 90.5 AC-FT annually. Of this, the 11.8 AC-FT/yr that is stored under the staging ponds can be counted against required annual

irrigation requirements for parkland and counted as a public savings calculated against water costs. This is in addition to those public savings calculated by Green Values.

Alternative Designs

Steep Sided Detention/Staging Ponds

It is to be noted that significant changes in performance can be obtained by altering certain design choices. The staging damns at the top of the developments are very strong controlling factors dictating the flow rates and water depths passing through the main channels. The presented design contains ponds that have wide shallow, non-reinforced slopes that behave more like shallow pools than like detention ponds. This allows for their use as fields or park space during off seasons and flood control during rainy seasons. If altered to have steep, reinforced slopes and significantly higher storage, while covering the same area, they will no longer be able to serve such purposes but can easily be designed such that even a 100 year storm will not cause significant damage in any of the channels. Such a design would see flow rates in the channels reduced by more than 50% in some cases.

Concrete Lined Channels

If the unlined channels were to be redesigned as concrete lined channels significantly less land space would be needed. However, they would no longer be available for designation as park land and would significantly increase flow speeds during large storms. Because of this they would need to be fenced as to avoid injury or death in the channels during large storms. Storage in the detention ponds in the lower portion of the development would need to be

significantly increased to account for the elimination of any infiltration in the channels during rainstorms. Even 1 year storms would see the need for some detention in the lower ponds where the unlined design sees almost no flow in the channels for a 1 year storm and no use of storage in the lower ponds.

Cemented Boulders as Check Dams

In the scale test, placing rocks that cover the same space as the check dams, in terms of height and slots for flow, were tested alongside the scale check dams. It was found that these performed easily as well as the check dams themselves as a means of slowing flow speeds while looking significantly more aesthetically pleasing. If such materials could be found at full scale, cementing them into the locations of the check dams and placing gabions beneath them would allow for flow control structures that look far more natural to the surrounding landscape and still maintain the same level of performance that is expected of the designed check dams. This also carries the potential for some decrease in cost as large stone tends to be readily available in the El Paso area and is a common building material for stone walls. For such a design, more reinforcement against erosion would be necessary. A sizable area just beneath the boulder check dams would need to be covered with gabions.

Conclusions and recommendations

The most important conclusion reached through experimentation and modeling is that an environmentally friendly design can be implemented in this development without being prohibitively expensive. The greatest hurdle to achieving this has been identified to be the volume of flow coming into the development from the upper portion of the watershed and this

exploration has demonstrated that the problem can be mitigated and the flows passing through the development can be controlled to a great degree, allowing for designs that increase infiltration and large amounts of natural landscaping.

The required flow speed is below 1.5 $^{ft}/_{sec}$ in order to prevent erosion in the unlined channels and avoid an excessive need for annual maintenance. In order to achieve this it is recommended that staging ponds be installed, hydraulically, at the top of the development in order to capture and manage the runoff that originates in the upper portion of the watershed. These ponds can be designed to allow controlled flows through the development that will be much simpler to manage and control. The viability of this approach has been shown through modeling and experimentation. Depending on the slopes used in the two main staging ponds the flows passing through the development can be controlled to varying degrees. With standard steep slopes the ponds will have enough storage to reduce runoff flows through the development to a degree that even a 100 year storm will not cause flow speeds higher than 1.5 $^{ft}/_{sec}$. However, the benefits of shallow slopes are recommended as there will be less need for unsightly fencing and the ponds will be able to be used for park land or sports fields. The result is that most storms, up to and beyond a 10 year storm, will not cause flow speeds high enough to damage any of the storm water structures. However, it is noted that for this design storms that are much larger than a 10 year storm will require maintenance of the structures to prevent serious damage.

It is recommended that unlined channels be designed to convey the runoff to its final destination using curved check dams as a means of further controlling the speed of flows. The

curve of the checks will direct flow toward the middle of the channel so that typical flows will not erode the banks. This has also been verified through experimentation. The checks should either be a masonry wall with five evenly placed “V” slots that allow for immediate flow, or should be constructed of cemented large boulders or boulder-like masonry structures. The “V” slots or the gaps between boulders will allow flow to locally speed up and wash unwanted sediment from behind the checks so that they are not buried. The staging ponds above the channels will prevent an inordinate amount of sediment to wash into the channels to begin with, and in this way the conveyance system will maintain its functionality through the annual storm cycle. Slope stability in the banks of the channels will be maintained by the planting of Desert Willow and Mesquite trees all along the banks. Or by the installation of cellular confinement mats that drastically increase soil shear strength. Performance of such as system has been verified through scale simulation. It is recommended that the use of the boulder check dam design and the planting of Desert Willow and Mesquite be used as this will create the maximum aesthetic value of the system while maintaining performance.

For the development lots it is recommended that designs be implemented that have all roof downspouts draining into raingardens, at least half of all lawns should be natural landscaping using local vegetation, porous pavement should be used for all driveways, sidewalks and non-street pavement, and drainage to the stormwater conveyance structures should make use of drainage swales instead of storm water pipes. The property owner will need to confer with an appropriate design firm to properly designate slopes, sizing, and other appropriate design specifics to implement these criteria.

Before actual design and construction it is recommended that further testing be done, specifically that pilot tests be run for the intended channel and detention pond designs. This will make certain that they can be expected to perform as has been shown in computer modeling. A pilot scale of each channel, and the staging ponds including their underlying artificial perched aquifers are highly recommended.

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Appendix

Table A1: Intensity Frequency Duration

ARI* (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day
1	0.22	0.33	0.41	0.55	0.68	0.79	0.83	0.94	1.03	1.19	1.28	1.45	1.64	1.81	2.28	2.69	3.21	3.66
2	0.28	0.43	0.53	0.71	0.88	1.02	1.07	1.20	1.31	1.52	1.64	1.86	2.10	2.33	2.93	3.45	4.10	4.68
5	0.38	0.57	0.71	0.96	1.19	1.38	1.43	1.58	1.71	2.00	2.15	2.45	2.78	3.10	3.85	4.50	5.29	6.05
10	0.45	0.69	0.86	1.15	1.43	1.67	1.72	1.87	2.01	2.37	2.56	2.91	3.32	3.71	4.56	5.29	6.17	7.03
25	0.56	0.85	1.05	1.41	1.75	2.06	2.11	2.27	2.41	2.88	3.15	3.56	4.07	4.57	5.52	6.34	7.31	8.29
50	0.64	0.97	1.20	1.62	2.01	2.36	2.42	2.58	2.72	3.29	3.62	4.07	4.67	5.25	6.25	7.13	8.15	9.21
100	0.72	1.10	1.37	1.84	2.28	2.69	2.75	2.91	3.04	3.71	4.13	4.60	5.32	5.98	7.01	7.93	9.00	10.13
200	0.81	1.24	1.53	2.07	2.56	3.03	3.10	3.25	3.36	4.16	4.68	5.17	5.99	6.76	7.79	8.74	9.83	11.03
500	0.94	1.42	1.76	2.38	2.94	3.50	3.58	3.71	3.79	4.79	5.46	5.99	6.96	7.84	8.91	9.82	10.91	12.19
1000	1.03	1.57	1.95	2.63	3.25	3.88	3.97	4.08	4.14	5.29	6.12	6.66	7.73	8.73	9.80	10.69	11.73	13.04

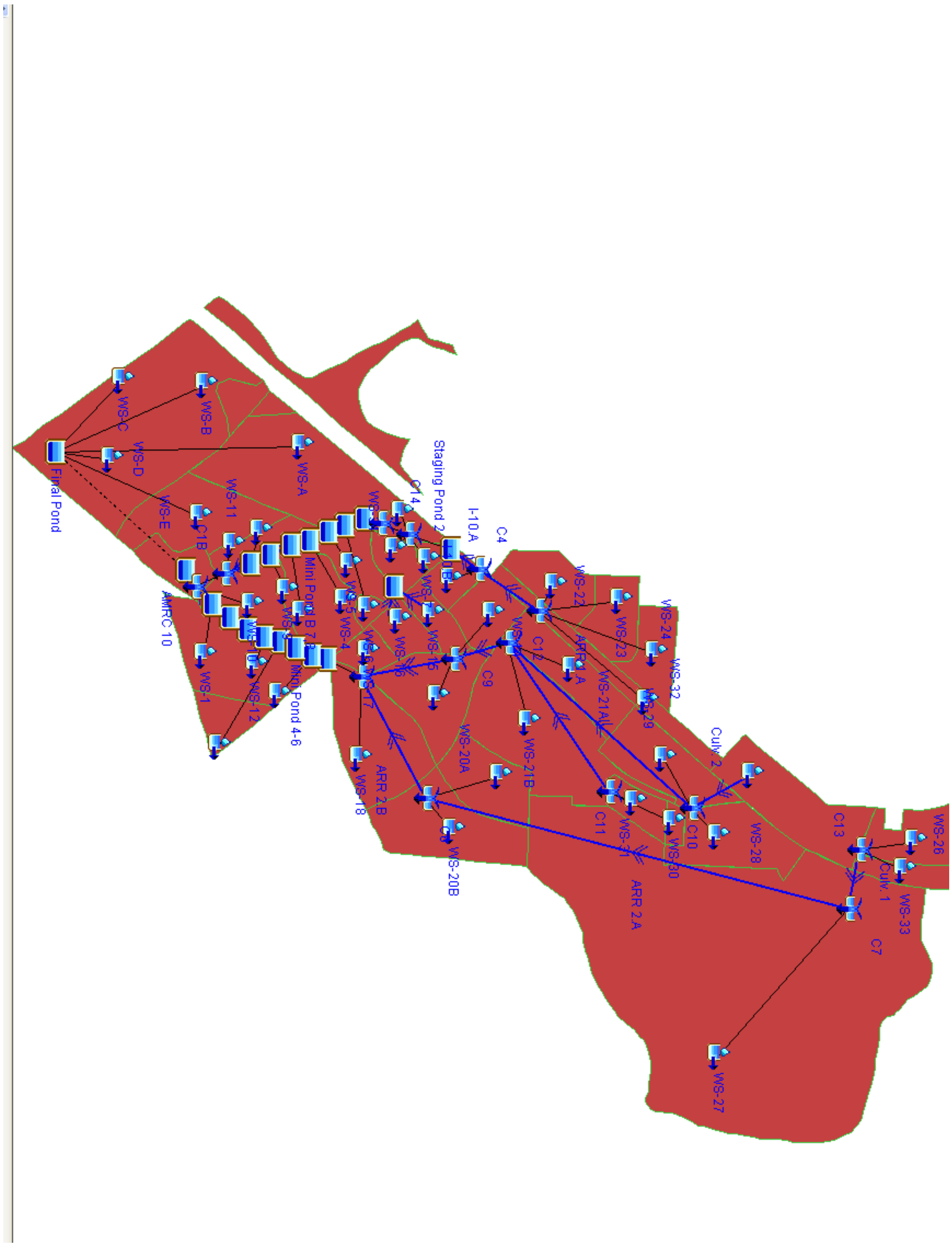

Fig. A1: HEC-HMS UI Model

Table A2: Sub basin Parameters

WS	Area (Mi^2)	Initial Loss (in)	Constant Rate (in/hr)	Impervious (%)	Standard Lag (hr)	Peaking Coefficient
24	0.039	0.8	0.4	0	0.21	0.67
32	0.048	0.5	0.25	100	0.21	0.67
23	0.042	0.1	0.05	85	0.21	0.67
22	0.044	0.1	0.05	50	0.21	0.67
13	0.022	0.1	0.05	20	0.108	0.67
7	0.023	0.5	0.25	30	0.138	0.67
35	0.038	0.5	0.25	25	0.126	0.67
34	0.018	0.1	0.25	10	0.105	0.67
15	0.023	0.1	0.05	80	0.21	0.67
16	0.023	0.1	0.05	83	0.21	0.67
6	0.007	0.5	0.25	40	0.122	0.67
5	0.066	0.1	0.05	20	0.21	0.67
4	0.066	0.5	0.25	5	0.21	0.67
3	0.058	0.5	0.25	0	0.21	0.67
8	0.103	0.5	0.25	0	0.21	0.67
25	0.091	0.1	0.05	90	0.21	0.67
29	0.069	0.8	0.4	4	0.21	0.67
28	0.07	0.1	0.05	80	0.21	0.67
31	0.023	0.8	0.4	0	0.124	0.67
30	0.007	0.1	0.05	50	0.113	0.67
21B	0.155	0.1	0.05	90	0.4	0.67
21A	0.06	0.1	0.05	80	0.21	0.67
19	0.075	0.5	0.25	65	0.21	0.67
14	0.035	0.1	0.05	10	0.21	0.67
26	0.044	0.1	0.05	60	0.21	0.67
33	0.015	0.1	0.05	100	0.1	0.67
27	0.921	0.8	0.4	5	1.22	0.67
20B	0.094	0.1	0.05	90	0.21	0.67
20A	0.05	0.1	0.05	90	0.21	0.67
18	0.097	0.8	0.4	60	0.25	0.67
17	0.018	0.1	0.05	50	0.21	0.67
2	0.099	0.5	0.25	5	0.21	0.67

Table A3: Arroyo Surveys

T	Station	Elevation
	0	3697.525
	16	3697.525
	17	3688.525
	87	3689.17
	120	3689
	157	3690
	158	3702.57
	179	3702.57
s	Sation	Eleveation
	0	3697.525
	16	3697.525
	17	3688.525
	50	3689
	87	3689.17
	157	3689.57
	158	3702.57
	179	3702.57
AR 3.C(O)	Station	Elevation
	0	3701.545
	3.5	3695.85
	12	3692.93
	20	3692.93
	40	3692.93
	55	3693
	75	3693.2
	78.65	3698.3
AR 3.C(Q)	Station	Elevation
	0	3603.75
	4.45	3603.49
	5	3599.72
	8	3599.7
	12	3599.69
	18.5	3599.74
	19	3603.36
	33.93	3603.635

ARR 3.B(K)	Station	Elevation
	0	3630.29
	14	3630.46
	23.48	3633.3
	31.8	3626.44
	59	3626.43
	66.1	3630.43
	94	3628.38
	105	3627.9
ARR .3B(M)	Station	Elevation
	0	3706.49
	4	3703.68
	6	3703.55
	8	3705.99
	11	3704.54
	13	3705.21
	15	3703.2
	20	3707.41
ARR 3.C(N)	Station	Elevation
	4.6	3699.915
	9.6	3695.3
	23.5	3695.41
	27.3	3695.18
	36.8	3695.95
	45.2	3695.53
	60.5	3699.21
	73.7	3699.56

ARR 3C(P)	Station	Elevation
	0	3699.25
	7.8	3698.97
	10	3690.25
	13.5	3688.74
	44	3688.95
	81	3689.04
	104.7	3697.09
	112	3697.66
ARROYO-T1	Station	Elevation
	5.8	3624.77
	7	3617.73
	13	3613.73
	95	3614.83
	221	3614.62
	225	3617.3
	228.2	3623.11
	235.5	3623.91
ARROYO-TT	Station	Elevation
	0	3623.58
	8.7	3622.67
	10	3615.58
	18	3611.07
	102	3611.74
	196.5	3612.39
	199	3621.51
	203.5	3621.46
ARROYO-U	Station	Elevation
	0	3718.34
	57	3714.2
	69.1	3706.91
	137	3707.56
	199	3707.2
	205	3708.12
	208	3720.5
	254	3721.01

ARROYO-U1	Station	Elevation
	0	3718.34
	57	3714.2
	69.1	3706.91
	137	3707.56
	199	3707.2
	205	3708.12
	208	3720.5
	254	3721.01
ARROYO-V	Station	Elevation
	0	3734.62
	4	3734.37
	24	3725.05
	55	3722.3
	64	3716.18
	116	3715.41
	147	3718.22
	162.6	3736.09
ARROYO-V1	Station	Elevation
	11.2	3657.505
	21	3632.53
	27	3627.9
	65	3626.894
	114	3626.8
	124	3632.845
	126.2	3655.02
	130.4	3655.035

Table A4: Channel Routing

Culv. 1	
Length (ft)	390
Slope (ft/ft)	0.03
Manning's n	0.013
Shape	Rectangle
Width (ft)	4
Material	Concrete Box
ARR 2.A	
Length (ft)	6000
Slope (ft/ft)	0.022
Manning's n	0.03
Shape	Rectangle
Width (ft)	40
Material	Unlined
ARR 2.B	
Length (ft)	5000
Slope (ft/ft)	0.017
Manning's n	0.013
Shape	Rectangle
Width (ft)	50
Material	Concrete Lined
Culv. 2	
Length (ft)	390
Slope (ft/ft)	0.05
Manning's n	0.013
Shape	Rectangle
Width (ft)	18
Material	Concrete Box
ARR 1.A	
Length (ft)	2600
Slope (ft/ft)	0.019
Manning's n	0.03
Shape	Rectangle
Width (ft)	50
Material	Unlined

ARR 1.B	
Length (ft)	2500
Slope (ft/ft)	0.025
Manning's n	0.03
Shape	Rectangle
Width (ft)	60
Material	Concrete Lined
ARR 1.C	
Length (ft)	2400
Slope (ft/ft)	0.014
Manning's n	0.012
Shape	Rectangle
Width (ft)	60
Material	Concrete Lined
ARROYO-V	
Length (ft)	350
Slope (ft/ft)	0.006
Manning's n	0.03
Shape	Eight Point
Left Manning's n	0.03
Right Manning's n	0.03
Cross Section	ARROYO-V
Material	Unlined
AYYOU-V1	
Length (ft)	140
Slope (ft/ft)	0.014
Manning's n	0.03
Shape	Eight Point
Left Manning's n	0.03
Right Manning's n	0.03
Cross Section	ARROYO-V1
Material	Unlined

ARROYO-U	
Length (ft)	535
Slope (ft/ft)	0.021
Manning's n	0.03
Shape	Eight Point
Left Manning's n	0.03
Right Manning's n	0.03
Cross Section	ARROYO-U
Material	Unlined
ARROYO-U1	
Length (ft)	235
Slope (ft/ft)	0.017
Manning's n	0.03
Shape	Eight Point
Left Manning's n	0.03
Right Manning's n	0.03
Cross Section	ARROYO-U1
Material	Unlined
ARROYO-T1	
Length (ft)	300
Slope (ft/ft)	0.023
Manning's n	0.03
Shape	Eight Point
Left Manning's n	0.03
Right Manning's n	0.03
Cross Section	ARROYO-T1
Material	Unlined

ARROYO-TT	
Length (ft)	140
Slope (ft/ft)	0.029
Manning's n	0.03
Shape	Eight Point
Left Manning's n	0.03
Right Manning's n	0.03
Cross Section	ARROYO-TT
Material	Unlined
ARROYO-T	
Length (ft)	380
Slope (ft/ft)	0.026
Manning's n	0.03
Shape	Eight Point
Left Manning's n	0.03
Right Manning's n	0.03
Cross Section	ARROYO-T
Material	Unlined
ARROYO-S	
Length (ft)	330
Slope (ft/ft)	0.024
Manning's n	0.03
Shape	Eigth Point
Left Manning's n	0.03
Right Manning's n	0.03
Cross Section	ARROYO-S
Material	Unlined
Culv. 3	
Length (ft)	1680
Slope (ft/ft)	0.03
Manning's n	0.013
Shape	Rectangle
Width (ft)	16
Material	Concrete Lined

I-10.A	
Length (ft)	400
Slope (ft/ft)	0.03
Manning's n	0.013
Shape	Rectangle
Width (ft)	20
Material	Concrete Box
I-10.B	
Length (ft)	400
Slope (ft/ft)	0.03
Manning's n	0.013
Shape	Circle
Diameter (ft)	4
Material	Concrete Cylinder
I-10.C	
Length (ft)	400
Slope (ft/ft)	0.03
Manning's n	0.013
Shape	Circle
Diameter (ft)	4
Material	Concrete Cylinder
ARR 3.B (K)	
Length (ft)	250
Slope (ft/ft)	0.012
Manning's n	0.03
Shape	Eight Point
Left Manning's n	0.03
Rigth Manning's n	0.03
Cross Section	ARR 3.B(K)
Material	Unlined

ARR 3.B(L)	
Length (ft)	380
Slope (ft/ft)	0.003
Manning's n	0.03
Shape	Eight Point
Left Manning's n	0.03
Right Manning's n	0.03
Cross Section	ARR 3.B(L)
Material	Unlined
ARR 3.B(M)	
Length (ft)	650
Slope (ft/ft)	0.014
Manning's n	0.03
Shape	Eight Point
Left Manning's n	0.03
Right Manning's n	0.03
Cross Section	ARR 3.B(M)
Material	Unlined
ARR 3.C(N)	
Length (ft)	775
Slope (ft/ft)	0.006
Manning's n	0.03
Shape	Eight Point
Left Manning's n	0.03
Right Manning's n	0.03
Cross Section	ARR 3.C(N)
Material	Unlined

ARR 3.C(O)	
Length (ft)	220
Slope (ft/ft)	0.014
Manning's n	0.03
Shape	Eight Point
Left Manning's n	0.03
Right Manning's n	0.03
Cross Section	ARR 3. C(O)
Material	Unlined
ARR 3.C(P)	
Length (ft)	300
Slope (ft/ft)	0.007
Manning's n	0.03
Shape	Eight Point
Left Manning's n	0.03
Right Manning's n	0.03
Cross Section	ARR 3.C(P)
Material	Unlined
ARR 3.C(Q)	
Length (ft)	380
Slope (ft/ft)	0.026
Manning's n	0.03
Shape	Eight Point
Left Manning's n	0.03
Right Manning's n	0.03
Cross Section	ARR 3.C(Q)
Material	Unlined

Table A5: Site 2 Infiltration Tests

Time (min)	Tube Depth (in)	Change in Tube Depth (in)	Change in Tube Volume (in^3)	Change in Inner Ring Depth (in)	Rate of Infiltration (in/min)	Rate of Infiltration (in/hr)	Infiltration (CFS/Acre)
0.0	18.4						
1.0	17.9	0.4	4.2	0.0	0.0	2.2	2.3
2.0	16.8	1.1	10.8	0.1	0.1	5.7	5.8
3.0	15.4	1.4	13.2	0.1	0.1	7.0	7.1
4.0	13.9	1.6	15.0	0.1	0.1	8.0	8.0
5.0	12.3	1.6	15.0	0.1	0.1	8.0	8.0
6.0	10.6	1.8	16.8	0.1	0.1	8.9	9.0
7.0	8.8	1.8	17.4	0.2	0.2	9.3	9.3
8.0	6.9	1.9	18.0	0.2	0.2	9.6	9.7
9.0	4.9	2.0	19.2	0.2	0.2	10.2	10.3
10.0	2.8	2.1	19.8	0.2	0.2	10.5	10.6
14.5	21.3						
15.0	20.8	0.5	4.8	0.0	0.1	5.1	5.1
16.0	19.7	1.1	10.2	0.1	0.1	5.4	5.5
17.0	18.4	1.3	12.0	0.1	0.1	6.4	6.4
18.0	16.8	1.6	15.6	0.1	0.1	8.3	8.4
19.0	15.1	1.7	16.2	0.1	0.1	8.6	8.7
20.0	13.4	1.8	16.8	0.1	0.1	8.9	9.0
21.0	11.5	1.9	18.0	0.2	0.2	9.6	9.7
22.0	9.6	1.9	18.0	0.2	0.2	9.6	9.7
23.0	7.8	1.9	18.0	0.2	0.2	9.6	9.7
24.0	5.7	2.1	19.8	0.2	0.2	10.5	10.6
25.0	3.8	1.9	18.6	0.2	0.2	9.9	10.0
26.0	1.6	2.1	20.4	0.2	0.2	10.8	10.9
30.2	21.5						
32.0	19.9	1.6	15.6	0.1	0.1	4.5	4.6
34.0	17.0	2.9	27.6	0.2	0.1	7.3	7.4
36.0	13.5	3.5	33.7	0.3	0.1	8.9	9.0
38.0	9.9	3.6	34.3	0.3	0.2	9.1	9.2
40.0	6.1	3.8	36.7	0.3	0.2	9.7	9.8
42.0	2.4	3.8	36.1	0.3	0.2	9.6	9.7

0.0	2.5	8.9		
5.0	3.4	8.0	0.2	10.5
10.0	4.5	6.9	0.2	
15.0	5.3	6.2	0.2	
20.0	6.1	5.3	0.2	10.5
25.0	7.1	4.3	0.2	
30.0	7.9	3.5	0.2	9
35.0	8.6	2.8	0.2	9
40.0	9.4	2.0	0.2	9

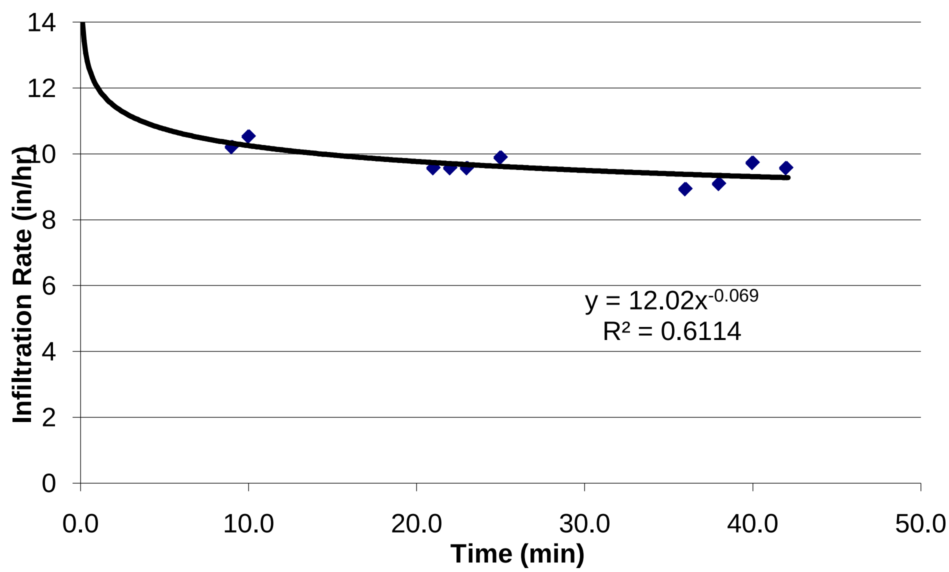

Fig. A2: Site 2 Static Head Infiltration Test

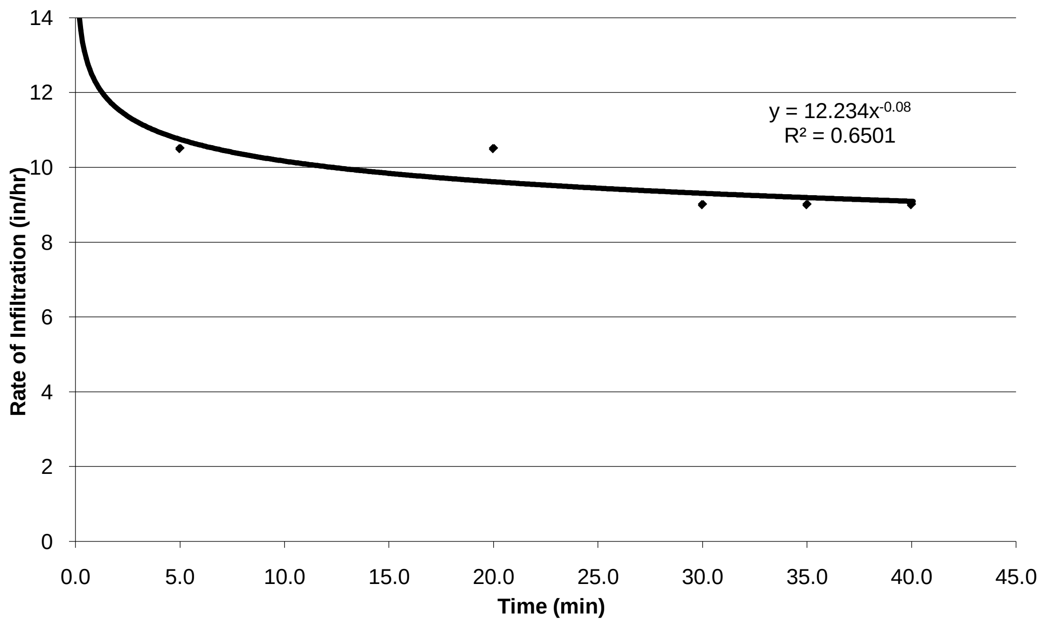

Fig. A3: Site 2 Falling Head Infiltration Test

Table A6: Site 2 Permeability Test

Time (sec)		Time Interval	Height (cm)		Water Level Change (cm)	Permeability (cm/s)	Permeability (in/hr)
Start	End		Start	End			
0	120	120	53.1	49.2	3.9	0.033	46.0
120	240	120	49.2	45.2	4	0.033	47.2
240	470	230	45.2	37.2	8	0.035	49.2
470	821	351	37.2	24.9	12.3	0.035	49.6
821	1148	327	24.9	13.4	11.5	0.035	49.8
1148	1292	144	13.4	8.4	5	0.035	49.2
1292	1365	73	8.4	5.7	2.7	0.037	52.4
					Average	0.035	49.1

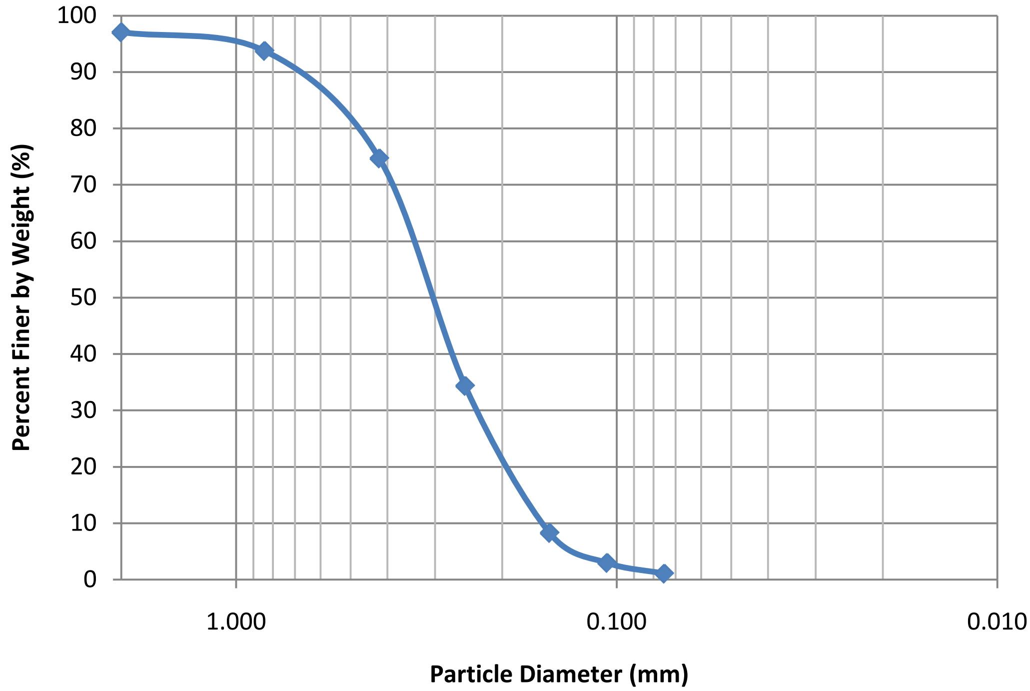

Fig. A4: Site 2 Sieve Test

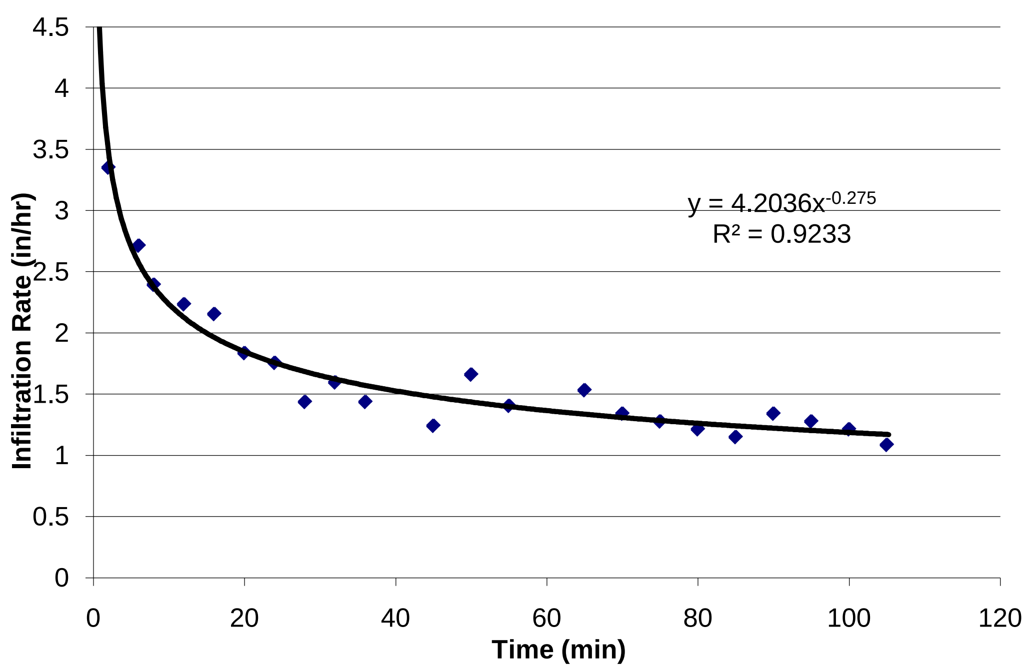

Fig. A5: Site 3 Static Head Infiltration Test

Table A7: Site 3 Infiltration Test

Time (min)	Tube Depth (in)	Change in Tube Depth (in)	Change in Tube Volume (in^3)	Change in Inner Ring Depth (in)	Rate of Infiltration (in/min)	Rate of Infiltration (in/hr)	Infiltration (CFS/Acre)
0	20.8						
2	19.5	1.3	12.6	0.1	0.1	3.3	3.4
6	17.4	2.1	20.4	0.2	0.0	2.7	2.7
8	16.4	0.9	9.0	0.1	0.0	2.4	2.4
12	14.7	1.8	16.8	0.1	0.0	2.2	2.3
16	13.0	1.7	16.2	0.1	0.0	2.2	2.2
20	11.6	1.4	13.8	0.1	0.0	1.8	1.8
24	10.2	1.4	13.2	0.1	0.0	1.8	1.8
28	9.1	1.1	10.8	0.1	0.0	1.4	1.4
32	7.8	1.3	12.0	0.1	0.0	1.6	1.6
36	6.7	1.1	10.8	0.1	0.0	1.4	1.4
45	4.5	2.2	21.0	0.2	0.0	1.2	1.3
50	2.9	1.6	15.6	0.1	0.0	1.7	1.7
55	1.5	1.4	13.2	0.1	0.0	1.4	1.4
62.5	19.8						
65	19.1	0.8	7.2	0.1	0.0	1.5	1.5
70	17.8	1.3	12.6	0.1	0.0	1.3	1.4
75	16.5	1.3	12.0	0.1	0.0	1.3	1.3
80	15.3	1.2	11.4	0.1	0.0	1.2	1.2
85	14.2	1.1	10.8	0.1	0.0	1.1	1.2
90	12.9	1.3	12.6	0.1	0.0	1.3	1.4
95	11.6	1.3	12.0	0.1	0.0	1.3	1.3
100	10.4	1.2	11.4	0.1	0.0	1.2	1.2
105	9.4	1.1	10.2	0.1	0.0	1.1	1.1

Table A8: Site 3 Permeability Test

Time (min)		Time Interval	Height (cm)		Water Level Change	Permeability (cm/s)	Permeability (in/hr)
Start	End		Start	End			
0	33	33	54.9	54.5	0.4	0.00020	0.28
33	210	177	54.5	51.4	3.1	0.00029	0.41
210	241	31	51.4	51	0.4	0.00022	0.30
0	68820	68820	51	35.5	15.5	0.00000	0.01
					Average	0.00018	0.25
Time (min)		Time Interval	Height (cm)		Water Level Change	Permeability (cm/s)	Permeability (in/hr)
Start	End		Start	End			
0	33	33	55.5	52.4	3.1	0.00157	2.21
33	241	208	52.4	52.1	0.3	0.00002	0.03
0	68820	68820	52.1	41.8	10.3	0.00000	0.003
					Average	0.00053	0.75

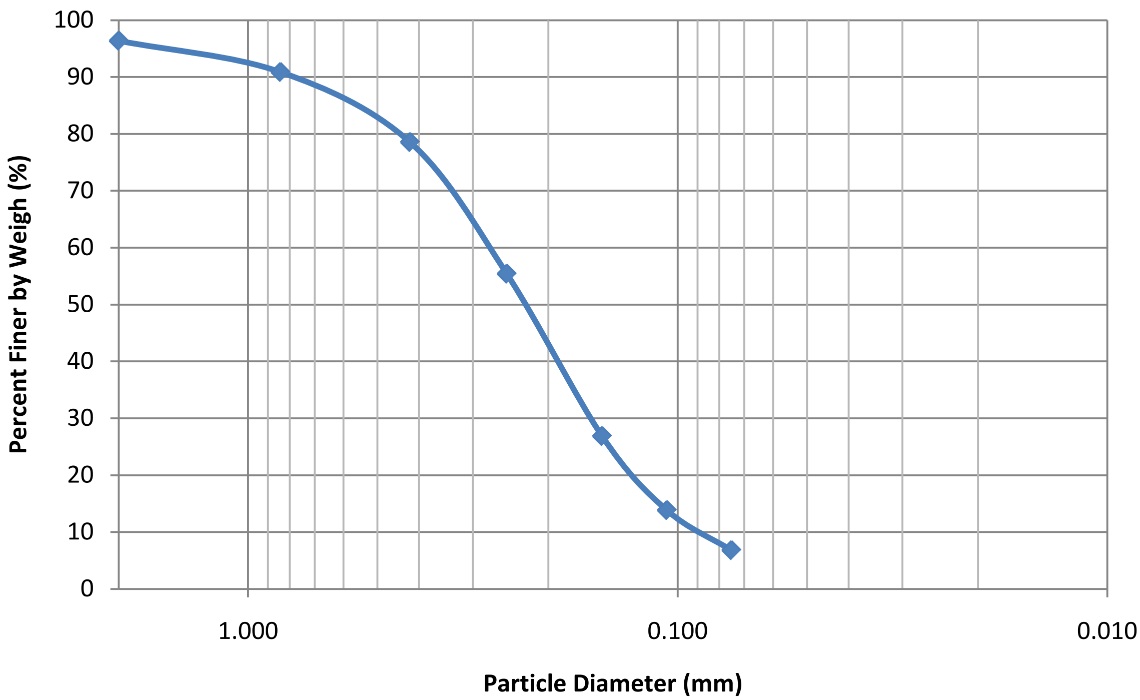

Fig. A6: Site 3 Sieve Test

Table A9: Site 4 Infiltration Test

Time (min)	Tube Depth (in)	Change in Tube Depth (in)	Change in Tube Volume (in^3)	Change in Inner Ring Depth (in)	Rate of Infiltration (in/min)	Rate of Infiltration (in/hr)	Infiltration (CFS/Acre)
0	21.5						
2.5	13.0	8.5	81.7	0.7	0.3	17.4	17.5
3	11.4	1.6	15.0	0.1	0.3	16.0	16.1
4	8.1	3.3	31.9	0.3	0.3	16.9	17.0
5	4.8	3.4	32.5	0.3	0.3	17.2	17.4
6	1.5	3.3	31.3	0.3	0.3	16.6	16.7
0	17.5						
2	14.9	2.6	24.6	0.2	0.1	6.5	6.6
4	11.3	3.7	35.5	0.3	0.2	9.4	9.5
6	6.8	4.4	42.7	0.4	0.2	11.3	11.4
8	2.3	4.6	43.9	0.4	0.2	11.6	11.7
							0.0
0	22.0						0.0
2	19.3	2.8	26.4	0.2	0.1	7.0	7.1
4	15.6	3.6	34.9	0.3	0.2	9.3	9.3
6	11.4	4.2	40.3	0.4	0.2	10.7	10.8
8	6.8	4.7	45.1	0.4	0.2	12.0	12.1
10	2.0	4.8	45.7	0.4	0.2	12.1	12.2
0	21.4						
3	17.0	4.4	42.7	0.4	0.1	7.5	7.6
4	15.1	1.9	18.6	0.2	0.2	9.9	10.0
6	11.0	4.1	39.1	0.3	0.2	10.4	10.5
8	6.5	4.5	43.3	0.4	0.2	11.5	11.6
10	1.8	4.8	45.7	0.4	0.2	12.1	12.2
0	21.8						
2	19.5	2.3	21.6	0.2	0.1	5.7	5.8
4	16.0	3.5	33.7	0.3	0.1	8.9	9.0
6	11.9	4.1	39.7	0.4	0.2	10.5	10.6
8	7.4	4.4	42.7	0.4	0.2	11.3	11.4
10	2.8	4.7	45.1	0.4	0.2	12.0	12.1

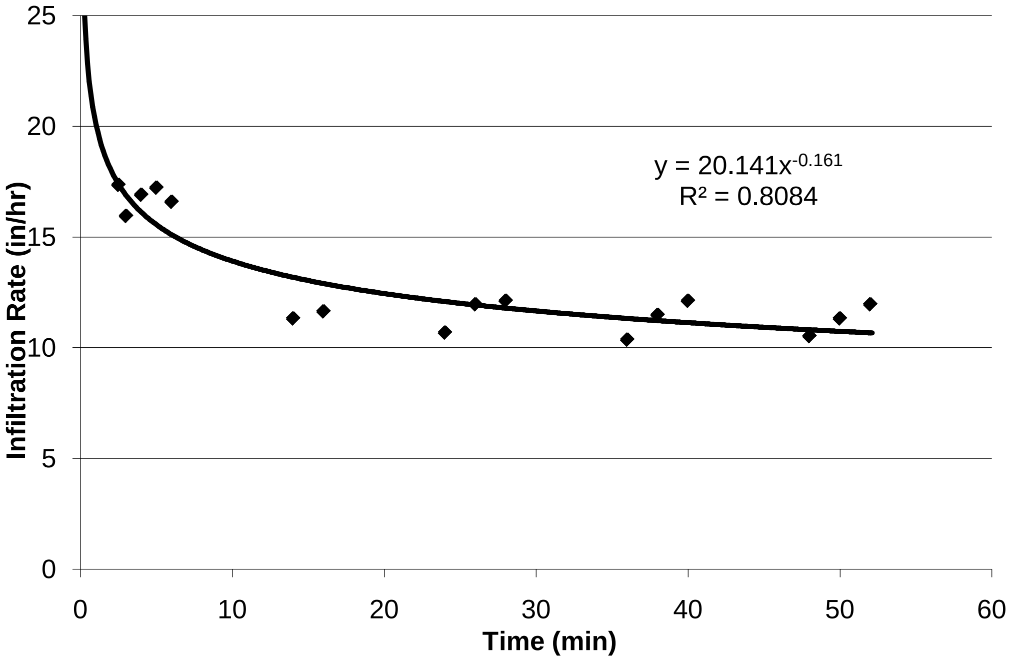

Fig. A7: Site 4 Static Head Infiltration Test

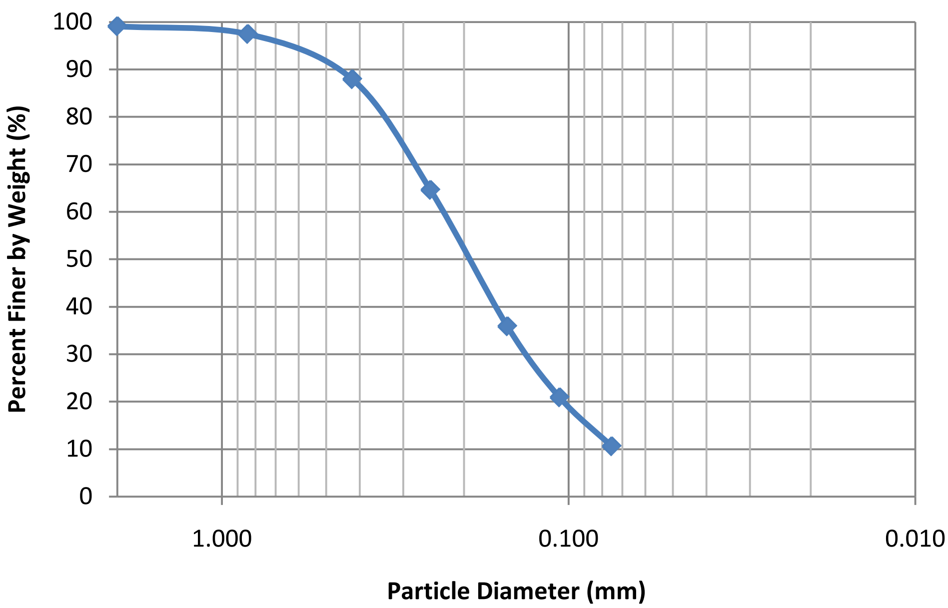

Fig. A8: Site 4 Sieve Test

Table A10: Site 4 Permeability Test

Time		Time Interval	Height (cm)		Water Level Change	Permeability (cm/s)	Permeability (in/hr)
Start	End		Start	End			
0	10260	10260	54.7	53	1.7	0.00016	0.23
10260	15000	4740	53	52.2	0.8	0.00017	0.24
0	240660	240660	54.9	13.2	41.7	0.00017	0.25
					Average	0.00017	0.24
Time		Time Interval	Height (cm)		Water Level Change	Permeability (cm/s)	Permeability (in/hr)
Start	End		Start	End			
0	3420	3420	54.6	53.9	0.7	0.00020	0.29
3420	10440	7020	53.9	51.8	2.1	0.00029	0.42
10440	14040	3600	51.8	50.8	1	0.00027	0.39
14040	17760	3720	50.8	49.8	1	0.00026	0.38
0	68340	68340	49.8	35	14.8	0.00021	0.31
0	10260	10260	35	33	2	0.00019	0.28
10260	15000	4740	33	32.1	0.9	0.00019	0.27
0	240660	240660	54.5	9.2	45.3	0.00019	0.27
					Average	0.00023	0.33

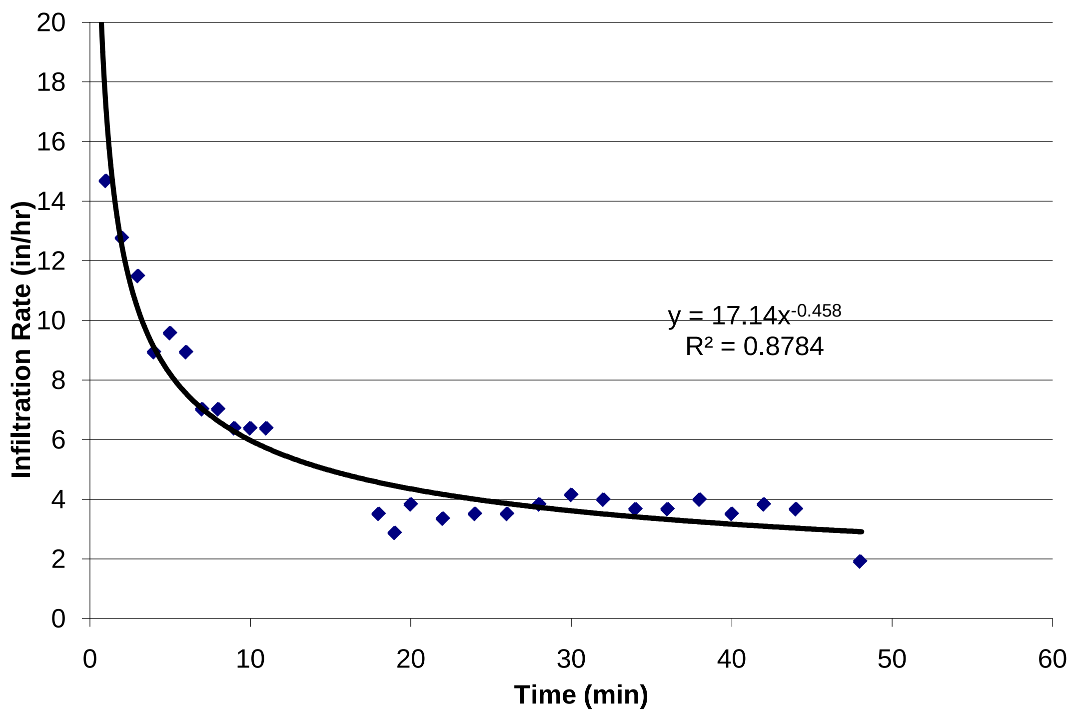

Fig. A9: Site 5 Static Head Infiltration Test

Table A11: Site 5 Infiltration Test

Time (min)	Tube Depth (in)	Change in Tube Depth (in)	Change in Tube Volume (in^3)	Change in Inner Ring Depth (in)	Rate of Infiltration (in/min)	Rate of Infiltration (in/hr)	Infiltration (CFS/Acre)
0	21.5						
1	18.6	2.9	27.6	0.2	0.2	14.7	14.8
2	16.1	2.5	24.0	0.2	0.2	12.8	12.9
3	13.9	2.3	21.6	0.2	0.2	11.5	11.6
4	12.1	1.8	16.8	0.1	0.1	8.9	9.0
5	10.3	1.9	18.0	0.2	0.2	9.6	9.7
6	8.5	1.8	16.8	0.1	0.1	8.9	9.0
7	7.1	1.4	13.2	0.1	0.1	7.0	7.1
8	5.8	1.4	13.2	0.1	0.1	7.0	7.1
9	4.5	1.3	12.0	0.1	0.1	6.4	6.4
10	3.3	1.3	12.0	0.1	0.1	6.4	6.4
11	2.0	1.3	12.0	0.1	0.1	6.4	6.4
14	22.2						
15	22.1	0.1	1.2	0.0	0.0	0.6	0.6
16	21.8	0.3	2.4	0.0	0.0	1.3	1.3
17	21.5	0.3	3.0	0.0	0.0	1.6	1.6
18	20.8	0.7	6.6	0.1	0.1	3.5	3.5
19	20.3	0.6	5.4	0.0	0.0	2.9	2.9
20	19.5	0.8	7.2	0.1	0.1	3.8	3.9
22	18.2	1.3	12.6	0.1	0.1	3.3	3.4
24	16.8	1.4	13.2	0.1	0.1	3.5	3.5
26	15.4	1.4	13.2	0.1	0.1	3.5	3.5
28	13.9	1.5	14.4	0.1	0.1	3.8	3.9
30	12.3	1.6	15.6	0.1	0.1	4.1	4.2
32	10.8	1.6	15.0	0.1	0.1	4.0	4.0
34	9.3	1.4	13.8	0.1	0.1	3.7	3.7
36	7.9	1.4	13.8	0.1	0.1	3.7	3.7
38	6.3	1.6	15.0	0.1	0.1	4.0	4.0
40	4.9	1.4	13.2	0.1	0.1	3.5	3.5
42	3.4	1.5	14.4	0.1	0.1	3.8	3.9
44	2.0	1.4	13.8	0.1	0.1	3.7	3.7
48	0.5	1.5	14.4	0.1	0.0	1.9	1.9

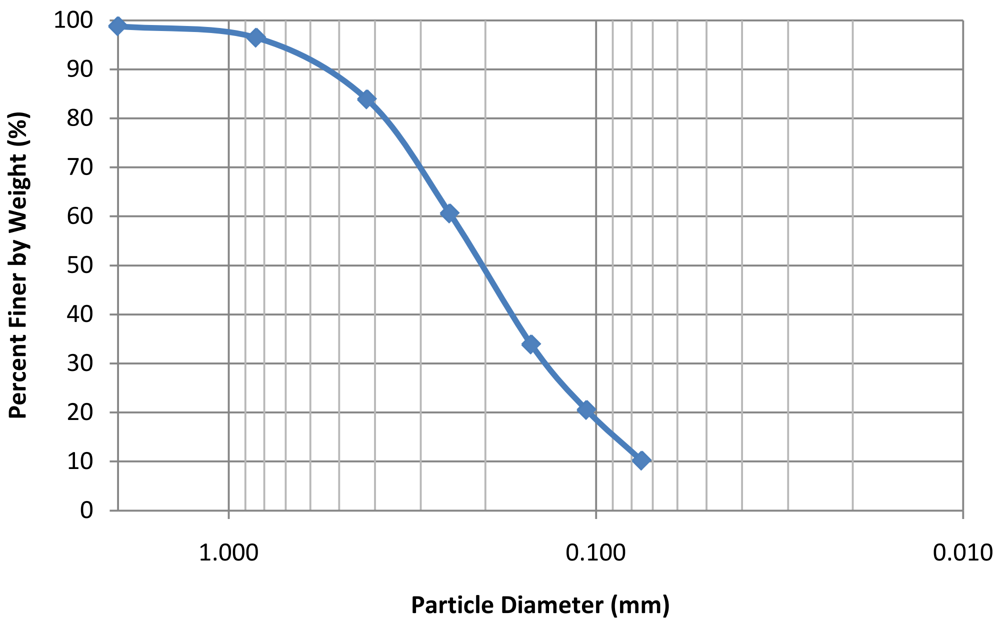

Fig. A10: Site 5 Sieve Test

Table a12: Site 5 Permeability Test

Time		Time Interval	Height (cm)		Water Level Change	Permeability (cm/s)	Permeability (in/hr)
Start	End		Start	End			
0	8160	8160	55	52.1	2.9	0.00036	0.50
8160	11700	3540	52.1	51.5	0.6	0.00017	0.24
11700	14400	2700	51.5	50.6	0.9	0.00033	0.47
					Average	0.00028	0.41
Time		Time Interval	Height (cm)		Water Level Change	Permeability (cm/s)	Permeability (in/hr)
Start	End		Start	End			
0	8160	8160	54.5	45.4	9.1	0.0011	1.58
8160	11700	3540	52.1	41.1	11	0.0031	4.40
11700	14400	2700	41.1	37.9	3.2	0.0012	1.68
					Average	0.0018	2.55

Table A13: Site 7 Infiltration Test

Time (min)	Tube Depth (in)	Change in Tube Depth (in)	Change in Tube Volume (in^3)	Change in Inner Ring Depth (in)	Rate of Infiltration (in/min)	Rate of Infiltration (in/hr)	Infiltration (CFS/Acre)
0.0	21.5						
2.0	20.3	1.2	11.4	0.1	0.1	3.0	3.1
4.0	18.3	2.0	19.2	0.2	0.1	5.1	5.1
6.0	16.2	2.1	20.4	0.2	0.1	5.4	5.5
8.0	13.9	2.3	21.6	0.2	0.1	5.7	5.8
10.0	11.8	2.2	21.0	0.2	0.1	5.6	5.6
12.0	9.6	2.1	20.4	0.2	0.1	5.4	5.5
14.0	7.6	2.1	19.8	0.2	0.1	5.3	5.3
16.0	5.6	1.9	18.6	0.2	0.1	4.9	5.0
18.0	3.4	2.3	21.6	0.2	0.1	5.7	5.8
20.0	1.3	2.1	19.8	0.2	0.1	5.3	5.3
26.0	21.3						
28.0	20.6	0.7	6.6	0.1	0.0	1.8	1.8
30.0	19.2	1.4	13.8	0.1	0.1	3.7	3.7
32.0	17.7	1.5	14.4	0.1	0.1	3.8	3.9
34.0	16.0	1.7	16.2	0.1	0.1	4.3	4.3
36.0	14.2	1.8	17.4	0.2	0.1	4.6	4.7
38.0	12.3	1.9	18.0	0.2	0.1	4.8	4.8
40.0	10.4	1.9	18.0	0.2	0.1	4.8	4.8
42.0	8.7	1.8	16.8	0.1	0.1	4.5	4.5
44.0	6.8	1.9	18.0	0.2	0.1	4.8	4.8
46.0	4.9	1.9	18.6	0.2	0.1	4.9	5.0
48.0	2.9	1.9	18.6	0.2	0.1	4.9	5.0
51.1	21.4						
52.0	20.9	0.5	4.8	0.0	0.0	2.7	2.8
54.0	19.4	1.5	14.4	0.1	0.1	3.8	3.9
56.0	17.8	1.6	15.6	0.1	0.1	4.1	4.2
58.0	16.0	1.8	17.4	0.2	0.1	4.6	4.7
60.0	14.3	1.8	16.8	0.1	0.1	4.5	4.5
62.0	12.5	1.8	16.8	0.1	0.1	4.5	4.5
64.0	10.7	1.8	17.4	0.2	0.1	4.6	4.7
66.0	8.7	2.0	19.2	0.2	0.1	5.1	5.1
68.0	6.9	1.8	16.8	0.1	0.1	4.5	4.5
70.0	4.6	2.3	22.2	0.2	0.1	5.9	6.0
72.0	2.7	1.9	18.6	0.2	0.1	4.9	5.0

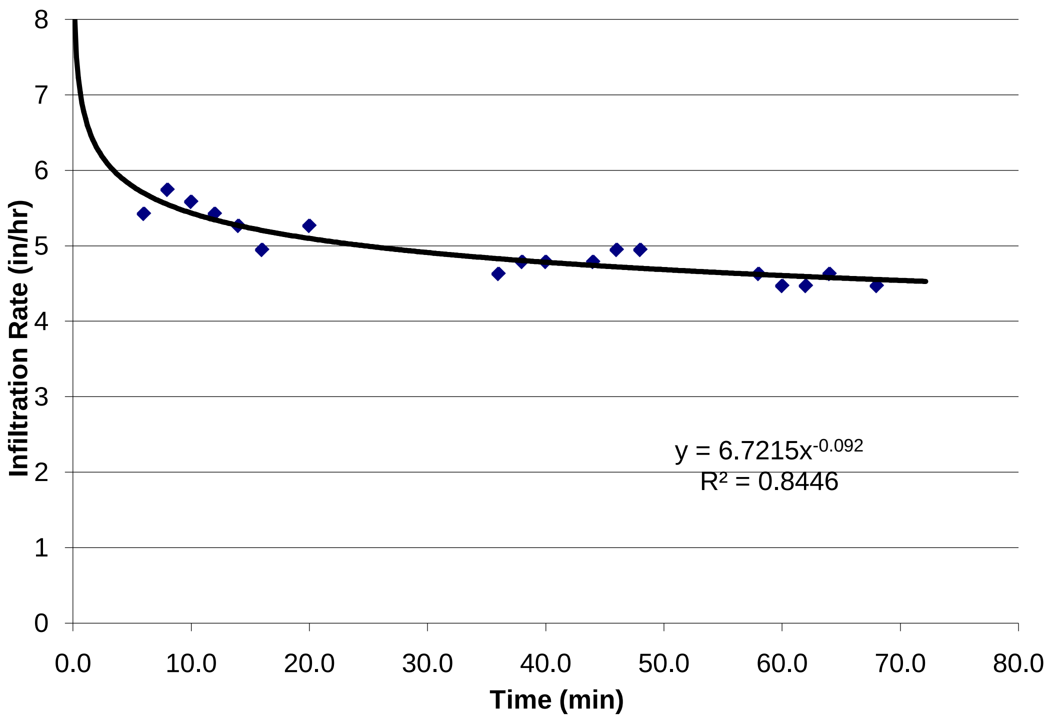

Fig. A11: Site 7 Static Head Infiltration Test

Table A14: Site 7 Permeability Test

Time		Time Interval	Height (cm)		Water Level Change	Permeability (cm/s)	Permeability (in/hr)
Start	End		Start	End			
0	1260	1260	54.1	53.6	0.5	0.00040	0.56
1260	1800	540	53.6	53.2	0.4	0.00074	1.05
1800	2460	660	53.2	52.7	0.5	0.00076	1.07
2460	3120	660	52.7	52.1	0.6	0.00091	1.29
					Average	0.00070	0.99
Time		Time Interval	Height (cm)		Water Level Change	Permeability (cm/s)	Permeability (in/hr)
Start	End		Start	End			
0	1260	1260	54.8	52.9	1.9	0.0015	2.14
1260	1800	540	52.9	52	0.9	0.0017	2.36
1800	2460	660	52	50.9	1.1	0.0017	2.36
2460	3120	660	50.9	49.8	1.1	0.0017	2.36
					Average	0.0016	2.31

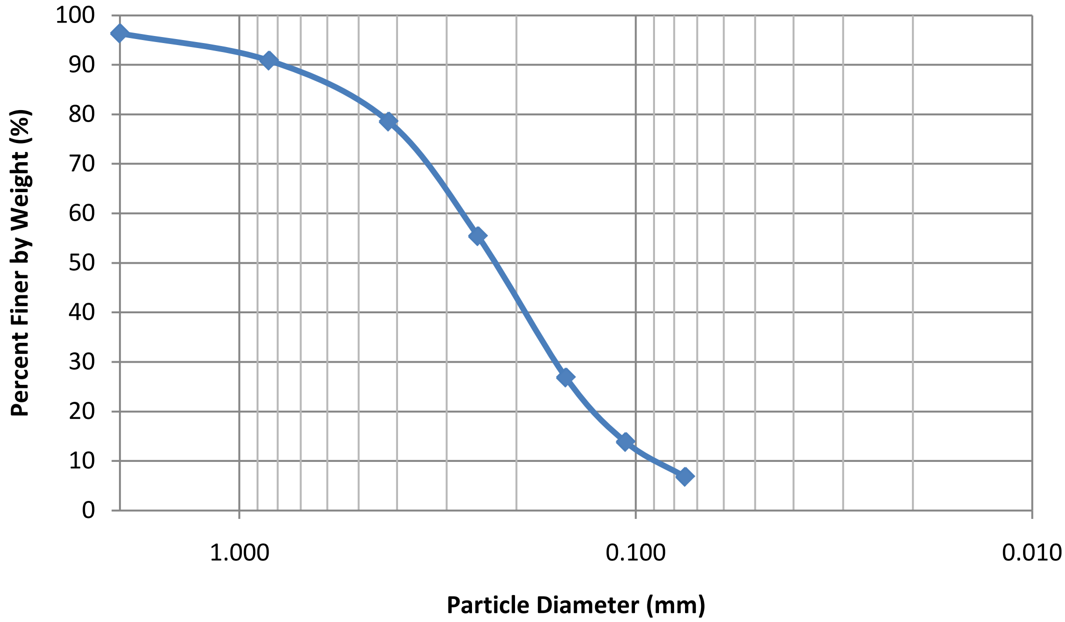

Fig. A12: Site 7 Sieve Test

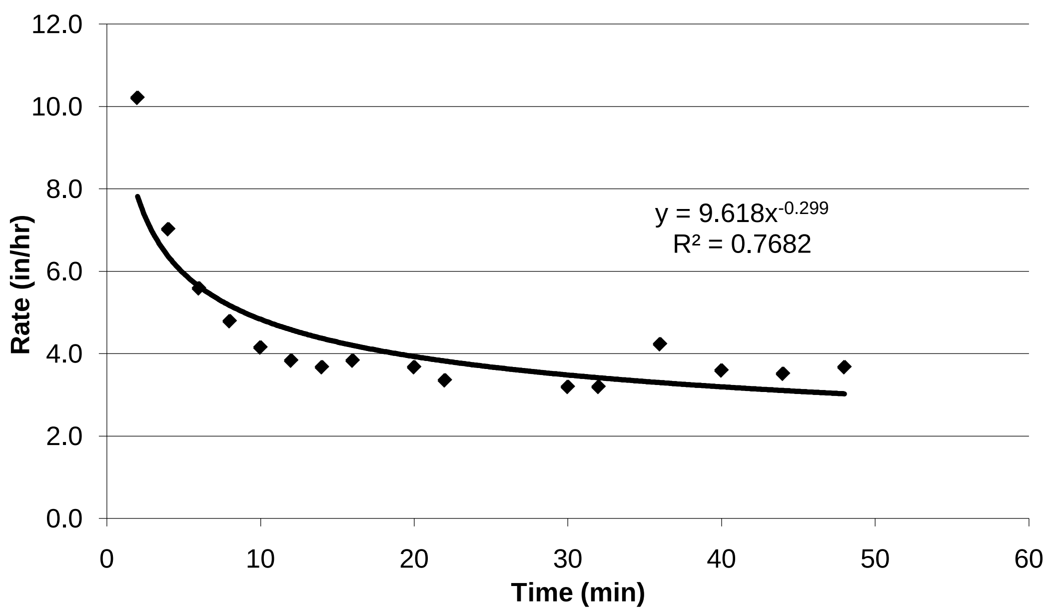

Fig. A13: Site 8 Static Head Infiltration Test

Table A15: Site 8 Infiltration Test

Time (min)	Tube Depth (in)	Tube Depth (in)	Change in Tube Depth (in)	Change in Tube Volume (in^3)	Change in Inner Ring Depth (in)	Rate of Infiltration (in/min)	Rate of Infiltration (in/hr)	Infiltration (CFS/Acre)
0	21.5							
4	16.4		5.1	49.3	0.4	0.1	6.5	6.6
8	11.5		4.9	46.9	0.4	0.1	6.2	6.3
12	6.8	21.4	4.8	45.7	0.4	0.1	6.1	6.1
22		12.8	8.7	83.5	0.7	0.1	4.4	4.5
26		6.8	5.9	57.1	0.5	0.1	7.6	7.6
30		1.9	4.9	46.9	0.4	0.1	6.2	6.3
0	22.0							
2	18.0		4.0	38.5	0.3	0.2	10.2	10.3
4	15.3		2.8	26.4	0.2	0.1	7.0	7.1
6	13.1		2.2	21.0	0.2	0.1	5.6	5.6
8	11.2		1.9	18.0	0.2	0.1	4.8	4.8
10	9.6		1.6	15.6	0.1	0.1	4.1	4.2
12	8.1		1.5	14.4	0.1	0.1	3.8	3.9
14	6.6		1.4	13.8	0.1	0.1	3.7	3.7
16	5.1		1.5	14.4	0.1	0.1	3.8	3.9
18	3.9		1.3	12.0	0.1	0.1	3.2	3.2
20	2.4		1.4	13.8	0.1	0.1	3.7	3.7
22	1.1	21.8	1.3	12.6	0.1	0.1	3.3	3.4
26		19.4	2.4	22.8	0.2	0.1	3.0	3.1
28		17.4	2.1	19.8	0.2	0.1	5.3	5.3
30		16.1	1.3	12.0	0.1	0.1	3.2	3.2
32		14.9	1.3	12.0	0.1	0.1	3.2	3.2
36		11.6	3.3	31.9	0.3	0.1	4.2	4.3
40		8.8	2.8	27.0	0.2	0.1	3.6	3.6
44		6.0	2.8	26.4	0.2	0.1	3.5	3.5
48		3.1	2.9	27.6	0.2	0.1	3.7	3.7

Table A16: Site 8 Permeability Test

Time		Time Interval	Height (cm)		Water Level Change	Permeability (cm/s)	Permeability (in/hr)
Start	End		Start	End			
0	1500	1500	54.7	54.6	0.1	6.7E-05	0.09
1500	3600	2100	54.6	54.5	0.1	4.8E-05	0.07
0	66060	66060	54.5	50	4.5	6.8E-05	0.10
0	1740	1740	50	49.9	0.1	5.7E-05	0.08
0	1380	1380	49.9	49.8	0.1	7.2E-05	0.10
0	1440	1440	49.8	49.7	0.1	6.9E-05	0.10
0	12120	12120	49.7	49	0.7	5.8E-05	0.08
0	900	900	49	48.7	0.3	0.00033	0.47
0	5400	5400	48.7	48.6	0.1	1.9E-05	0.03
0	238140	238140	48.6	36.6	12	5.0E-05	0.07
					Average	8.4E-05	0.12
Time		Time Interval	Height (cm)		Water Level Change	Permeability (cm/s)	Permeability (in/hr)
Start	End		Start	End			
0	3120	3120	54.7	54.4	0.3	9.6E-05	0.14
3120	5460	2340	54.4	54	0.4	0.00017	0.24
0	238140	238140	54	23.5	30.5	0.000136	0.18
					Average	0.00013	0.19

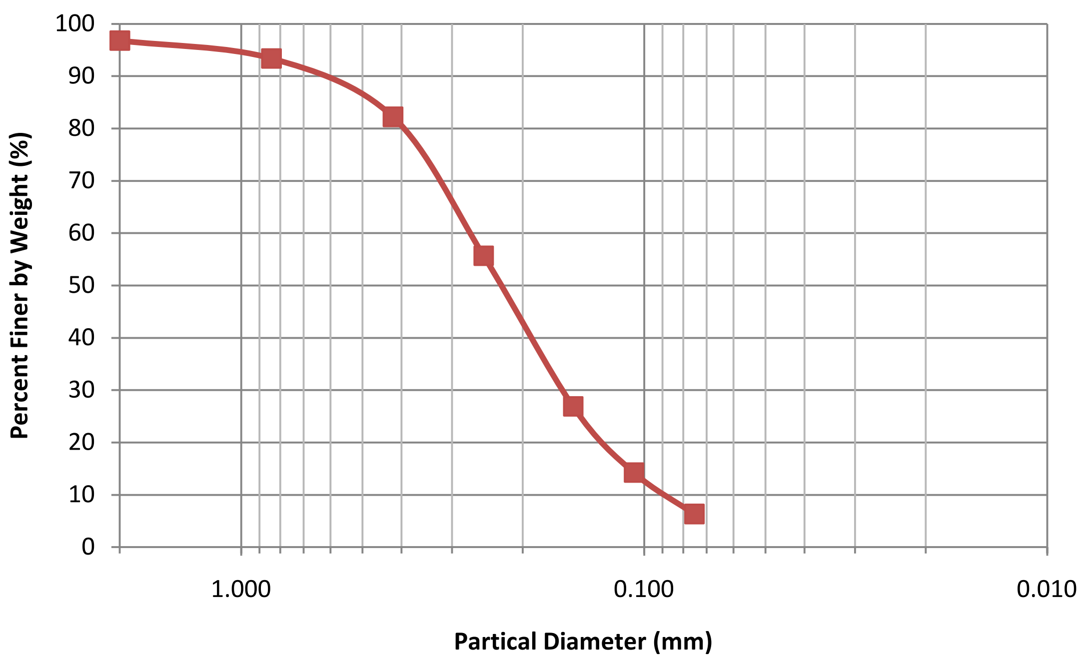

Fig. A14: Site 8 Sieve Test

Table A17: Site 9 Infiltration Test

Time (min)	Tube Depth (in)	Change in Tube Depth (in)	Change in Tube Volume (in^3)	Change in Inner Ring Depth (in)	Rate of Infiltartion (in/min)	Rate of Infiltration (in/hr)	Infiltration (CFS/Acre)
1.00	21.5						
2.00	20.1	1.4	13.5	0.1	0.1	7.1	7.2
3.00	18.9	1.2	11.5	0.1	0.1	6.1	6.2
4.00	17.9	1.0	9.6	0.1	0.1	5.1	5.1
5.00	16.3	1.6	15.4	0.1	0.1	8.2	8.2
6.00	14.5	1.8	17.3	0.2	0.2	9.2	9.3
7.00	13.3	1.2	11.5	0.1	0.1	6.1	6.2
8.00	11.7	1.6	15.4	0.1	0.1	8.2	8.2
9.00	9.9	1.8	17.3	0.2	0.2	9.2	9.3
10.00	8.6	1.3	12.5	0.1	0.1	6.6	6.7
11.00	6.9	1.7	16.3	0.1	0.1	8.7	8.7
13.00	5.4	1.5	14.4	0.1	0.1	3.8	3.9
14.00	3.8	1.6	15.4	0.1	0.1	8.2	8.2
17.17	2.2	1.6	15.4	0.1	0.0	2.6	2.6
18.00	21.4						
19.00	20.6	0.8	7.7	0.1	0.1	4.1	4.1
20.00	19.6	1.0	9.6	0.1	0.1	5.1	5.1
21.00	18.4	1.2	11.5	0.1	0.1	6.1	6.2
23.00	17.1	1.3	12.5	0.1	0.1	3.3	3.3
24.00	14.5	2.6	25.0	0.2	0.2	13.3	13.4
25.00	12.8	1.7	16.3	0.1	0.1	8.7	8.7
27.00	11.3	1.5	14.4	0.1	0.1	3.8	3.9
28.00	8.2	3.1	29.8	0.3	0.3	15.8	16.0
29.00	6.3	1.9	18.3	0.2	0.2	9.7	9.8
30.00	4.9	1.4	13.5	0.1	0.1	7.1	7.2
31.00	3.3	1.6	15.4	0.1	0.1	8.2	8.2
34.25	1.6	1.7	16.3	0.1	0.0	2.7	2.7
35.00	21.4						
36.00	20.2	1.2	11.5	0.1	0.1	6.1	6.2
37.00	19.1	1.1	10.6	0.1	0.1	5.6	5.7
38.00	17.8	1.3	12.5	0.1	0.1	6.6	6.7
39.00	16.6	1.2	11.5	0.1	0.1	6.1	6.2
40.00	15.2	1.4	13.5	0.1	0.1	7.1	7.2
41.00	13.7	1.5	14.4	0.1	0.1	7.7	7.7
42.00	12.3	1.4	13.5	0.1	0.1	7.1	7.2
43.00	10.8	1.5	14.4	0.1	0.1	7.7	7.7
44.00	9.2	1.6	15.4	0.1	0.1	8.2	8.2
45.00	7.6	1.6	15.4	0.1	0.1	8.2	8.2
46.00	6.0	1.6	15.4	0.1	0.1	8.2	8.2
47.00	4.4	1.6	15.4	0.1	0.1	8.2	8.2
48.00	2.8	1.6	15.4	0.1	0.1	8.2	8.2
49.00	1.1	1.7	16.3	0.1	0.1	8.7	8.7

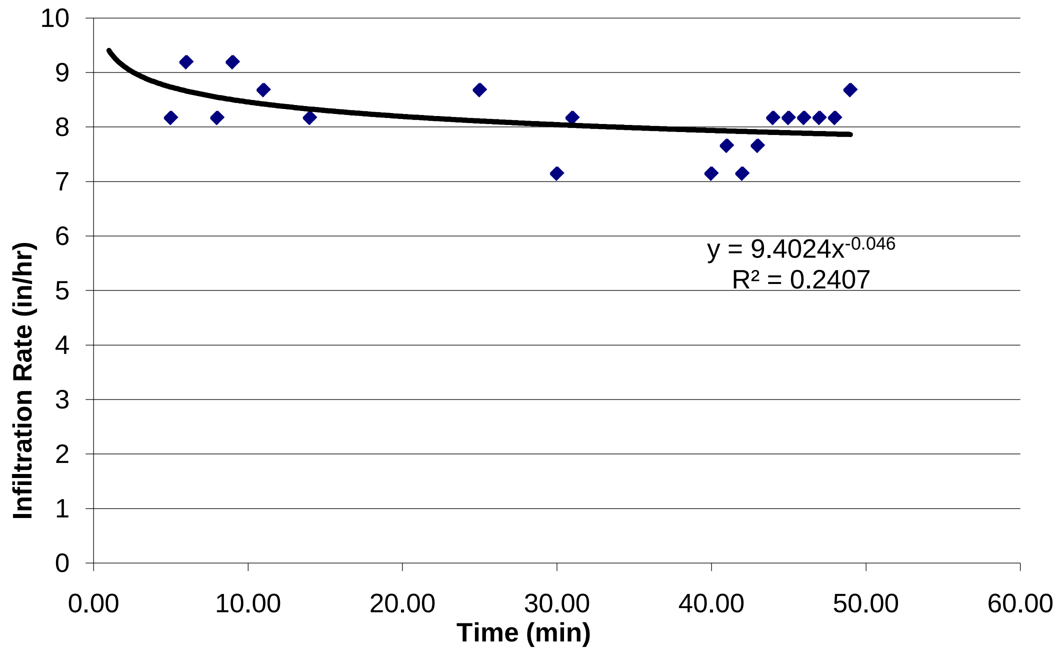

Fig. A15: Site 9 Static Head Infiltration Test

Table A18: Site 9 Permeability Test

Time		Time Interval	Height (cm)		Water Level Change	Permeability (cm/s)	Permeability (in/hr)
Start	End		Start	End			
0	840	840	54.8	53.8	1	0.001	1.7
840	13380	12540	53.8	38.3	15.5	0.001	1.8
13380	14700	1320	38.3	36.5	1.8	0.001	1.9
14700	15300	600	36.5	35.7	0.8	0.001	1.9
					Average	0.001	1.8
Time		Time Interval	Height (cm)		Water Level Change	Permeability (cm/s)	Permeability (in/hr)
Start	End		Start	End			
0	840	840	54.4	53.1	1.3	0.002	2.2
840	13380	12540	53.1	31.2	21.9	0.002	2.5
13380	14700	1320	31.2	28.9	2.3	0.002	2.5
14700	15300	600	28.9	27.8	1.1	0.002	2.6
					Average	0.002	2.4

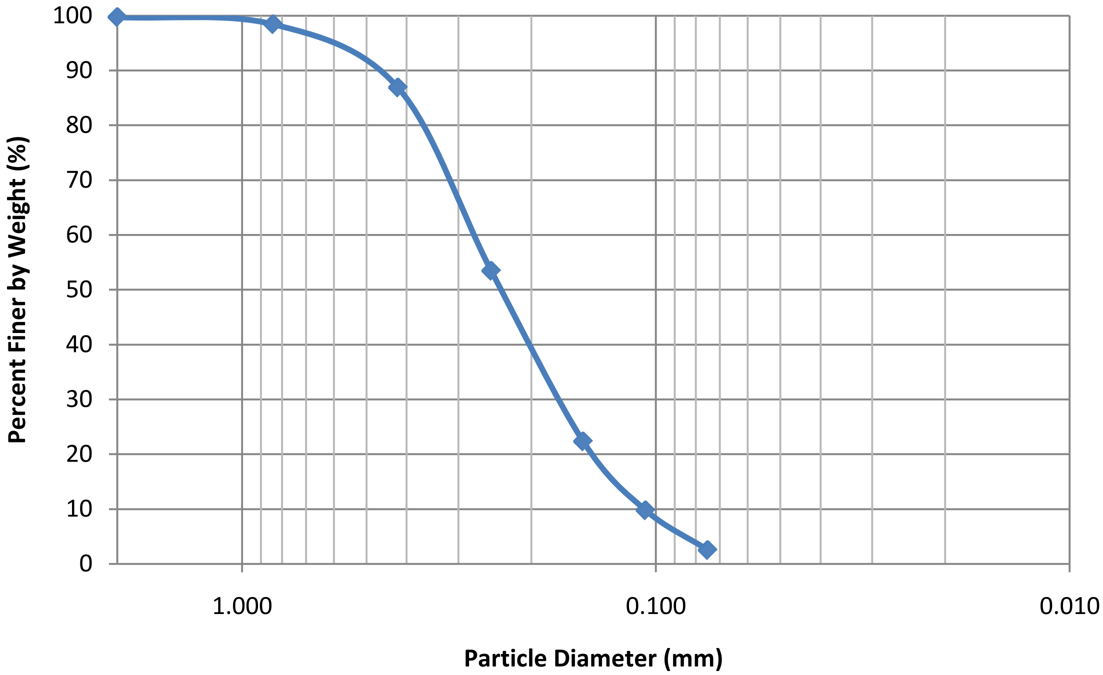

Fig. A16: Site 9 Sieve Test

Curriculum Vita

Ricardo Sabino Marmolejo was born and raised in El Paso, Texas. The first son of Daniel and Monica Dolores Marmolejo, he graduated from Canutillo High School, El Paso, Texas, in the spring of 2002 and entered The University of Texas at El Paso that fall. After graduating with a degree in Civil Engineering in the fall of 2008 he began working as a researcher for the Texas AgriLife Research Center. He was awarded a grant for his research on the AMRC10 watershed by TGRI in early 2010. He presented on behalf of his research group at the Rio Grande Initiatives 2010 Conference in Alpine, Texas.

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