Application Assessment

for the

Florida Airports Stormwater Study

Florida Department of Transportation

Clean Water – Safe Airports

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FEDERAL AVIATION ADMINISTRATION:

W. Dean Stringer, P.E.
Lindy McDowell

SPONSOR:

Florida Department of Transportation

William Ashbaker, P.E.
Aaron Smith
Abdul Hatim, Ph.D.

CONSULTANT TEAM:

Wilbur Smith & Associates, Inc.

Harry Downing
Ahmed Noman, P.E.
Paul Snead, P.E.

MEA Group, Inc. (part of Hanson Professional Services Inc.)

Scott Brady, P.E.
Kent Bontrager, P.E.
Ron Ridenour
Larry Bordner
David Sands

Ed Barber & Associates, Inc.

Ed Barber
Dean Mades, P.E.
Rachel Bolyard

University of Florida
Department of Environmental Engineering Sciences

James Heaney, Ph.D, P.E.
John Sansalone, Ph.D., P.E.
Reuben Kertesz
Robert Rooney
Saurabh Raje

REVIEW and GUIDANCE COMMITTEE:

Florida Department of Environmental Protection
Eric Livingston

St. John's River Water Management District
Mike Register, P.E.
Cammie Dewey

South Florida Water Management District
Tony Waterhouse, P.E.
Ed Yaun, P.E.
Allan Levinson

Southwest Florida Water Management District
Robin McGill, P.E.
Bill Copeland

FLORIDA STATEWIDE AIRPORT STORMWATER STUDY
APPLICATION REPORT

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1. PURPOSE and APPROACH

The purpose of this *Application Assessment for the Florida Statewide Airport Stormwater Study* is to provide reasonable assurance that overland flow management recommendations will meet state water management objectives. Additionally, this *Application Assessment* identifies limiting conditions for overland flow, outlines an approach to evaluate projects that have such limiting conditions, and provides guidance for design of a Federal Aviation Administration (FAA) wet pond system. It is intended primarily for the jurisdictional regulatory agencies as a component to formally adopting the *Florida Airports Stormwater Best Management Practices Manual*. These agencies are the Florida Department of Environmental Protection (FDEP), Northwest Florida Water Management District (NWFWMD), St. John's River Water Management District (SJRWMD), South Florida Water Management District (SFWMD), Southwest Florida Water Management District (SWFWMD), Suwannee River Water Management District (SRWMD). It also provides information that will be used to modify the *Best Management Practices Manual* before or during regulatory adoption.

Prior phases of the Florida Statewide Airport Stormwater Study collected field information describing water quality and quantity from runways, taxiways, and aircraft parking aprons. Also collected were data for select best management practices emphasizing overland flow with infiltration. This was presented in the *Technical Report for the Florida Statewide Airport Stormwater Study* in June 2005. Methods to apply the data collected, and other data assembled from literature search, were recommended in a draft *Florida Airports Stormwater Best Management Practices Manual* originally issued in June 2005.

This *Application Assessment* is confined strictly to the airport airside. Although the simulation models and methods may be used for many other project types, the data supporting this effort are specific to runways, taxiways, aprons and airside infields. The FDEP and Water Management Districts outlined general requirements for the Application Assessment. Summarized, they are:

1. Examples shall focus on airside application of the Best Management Practices Manual only. This is consistent with the data collection program, Technical Report and original program objectives.
2. Examples shall consider airside development on a dry site, an intermediate site and a wet site.
3. Examples shall compare runoff quality using the BMP Manual proposed criteria with the runoff quality for a pristine site. Event Mean Concentrations for a pristine site will be those furnished by FDEP.
4. FAA wet pond performance will be evaluated for two possible geometries using computational fluid dynamics (CFD). CFD will also be used to theoretically assess two presumptive wet pond designs. These will serve as a baseline to compare with the CFD predicted performance of FAA wet ponds.
5. Examples shall include a conversion of landside "dirty" site such as industrial/commercial use to an airside use.

1.A Selection of Simulation Models

The Application Assessment required the use of two different simulation models. These examine the impacts of using the recommended, overland flow best management practice and define a probable configuration for an FAA pond, respectively.

The utilization and simulation of overland flow as a best management practice (BMP) for the airside pavement required modeling water quality and quantity on a continuous and event basis. The United States Environmental Protection Agency (EPA) Storm Water Management Model (SWMM) was selected for overland flow evaluation based on these requirements. The SWMM model is described further in Section 1.A.1 Site Model.

Determining a configuration for an FAA pond that had the potential to provide equal or better performance compared to standard presumptive ponds was examined on an event-basis . Hydrologic loadings were based on both a typical flow and a design storm peak flow applied to an aircraft parking apron. The selected approach uses computational fluid dynamics (CFD) to model the transport of a pollutant with specified particle size distribution and specific gravity through selected pond geometries. The CFD model is discussed further in Section 1.A.2 Wet Pond Model.

1.A.1 Site Model

SWMM 5 provides an integrated environment for editing study area input data, running hydrologic, hydraulic and water quality simulations, and viewing the results in a variety of formats. These include color-coded drainage area and conveyance system maps, time series graphs and tables, profile plots, and statistical frequency analyses. Followings are major key SWMM features:

- ☐ Time-varying rainfall including continuous historical simulations
- ☐ Evaporation of standing surface water
- ☐ Rainfall interception from depression storage
- ☐ Infiltration of rainfall into unsaturated soil layers
- ☐ Percolation of infiltrated water into groundwater layers
- ☐ Interflow between groundwater and the drainage system
- ☐ Water quality constituents infiltrate with runoff water
- ☐ Nonlinear routing of overland flow.
- ☐ Flood plain mapping of natural channel systems (SWMM 5 is a FEMA-approved model for NFPI studies)
- ☐ Can handle backwater effects
- ☐ Can incorporate more accurate process dynamics
- ☐ Components of water quality can be explicitly modeled
- ☐ Internal parameter calibration to observed conditions

SWMM operating condition and parameter sensitivities were selected and checked before modeling the airside systems. A parametric evaluation of the simulation time step during wet weather indicated that the choice of time-steps could result in significant differences in rainfall distribution, infiltration and runoff values. Longer time steps distributed the hydrologic volumes over a longer period of time, resulting in reduced intensity of rainfall, greater infiltration and less runoff. Therefore wet weather simulations and routing of historical simulations were conducted at a maximum time step of 5 minute intervals, corresponding to the 5 minute rainfall data used in the simulations. The design event simulation had an even shorter computational time interval; a wet weather time step of 1 minute along with a routing time step of 1 minute. For inter-event hydrologic phenomena such as evapotranspiration (ET), a dry weather time step of 1 hour was used. Surface water routing used the full dynamic wave solving the St.Venant hydrodynamic equations. The dynamic wave algorithm considers mass and momentum simultaneously. In SWMM, default dynamic wave conduit lengthening factors were used and a timestep lengthening of 30 seconds was specified.

The above factors are common to the “dry”, “intermediate” and “wet” site simulations of this study.

1.A.2 Wet Pond Model

Computational fluid dynamics (CFD) has been in use by many industries, including chemical and aerospace engineering, as an effective technique to numerically simulate fluid and particle motion over and through three-dimensional systems. Behavior of jet engines, water turbines, pumps, rocket motors, and most recently water quality best management practices, (BMPs) has been predicted in this manner. In the fields of civil and environmental engineering, CFD is becoming the most powerful predictive and design tool available for water management systems. Essentially a finite volume method (FVM) technique, CFD is capable of handling flows that range from pure laminar, to turbulent, to supersonic. Simulations include all three spatial dimensions (length, width, depth) and time for the continuous and discrete (particles) phases. The benefit of using CFD is as an aid to direct experimental research that limits the cost of pilot and prototype studies. The results of the simulations can be used to select design options with the highest potential for treatment effectiveness. CFD modeling must be coupled with calibration and validation processes. This must, of course, be carried out by physical testing of a prototype system. For this study CFD results for two possible “FAA” ponds were compared with CFD evaluations of selected presumptive ponds authorized by Water Management District rules. This provides a baseline that FDEP and Water Management Districts are familiar with to compare the proposed FAA pond results.

The CFD models discretatization schemes for the wet ponds resulted in computational cell configurations between 2 and 5 million computational cells per wet pond. During a simulation, the transport equations are solved across the entire spatial domain of the wet pond in three dimensions.

1.B Infiltration Approach

A method to quantify infiltration was selected based on review of four options. Options considered were Phi Index, Horton Equation, Natural Resource Conservation Service (NRCS formerly SCS) Curve Number (CN) and Green-Ampt. These are briefly summarized below.

The Phi Index (ϕ) is the difference between total rainfall and observed surface runoff expressed as an infiltration rate (i.e. inches/hour). It is site specific, does not account for antecedent changes in rainfall, soil moisture or groundwater. It must be based on rainfall and runoff instead of more easily measured soil and groundwater properties, and does not correlate site to site.

Horton's equation has a simple mathematical foundation, but fundamental soil parameters and phenomena are not explicitly resolvable. In particular, infiltration capacity is a function of time, ignoring the volume of water infiltrated or whether rainfall rate is greater or lower than infiltration rate based on time. Empirical parameters are included in the equation.

While the NRCS Curve Number has a mathematical foundation, the physical basis with respect to movement of water in a soil with respect to time and physical phenomena is rarely calibrated or measured in practice. (Mishra, et al 2003). As typically used in practice, the CN is selected from a table of typical values associated with broad categories of land use and Hydrologic Soil Groups.

The Green-Ampt equation has a defensible physical basis and parameters are physical soil and groundwater properties. These have been measured and verified in literature, and can be field measured on a site specific basis. (Li, et al 1999). Infiltration rate in the Green-Ampt method varies with rainfall intensity and cumulative infiltration volume. Infiltration ceases whenever the rainfall intensity exceeds the infiltration rate on when the wetting front intersects the groundwater table, completely saturating the soil. Data collected in prior components of the Statewide Airport Stormwater Study was used to compare Green-Ampt predictions with actual measured runoff at the Sarasota Bradenton International Airport. The rainfalls and runoffs recorded in 5-minute intervals for 60 separate events during the 2002 calendar year were compared. The comparison is shown in Figure 1.B-1 following and supports the predictive value of the approach.

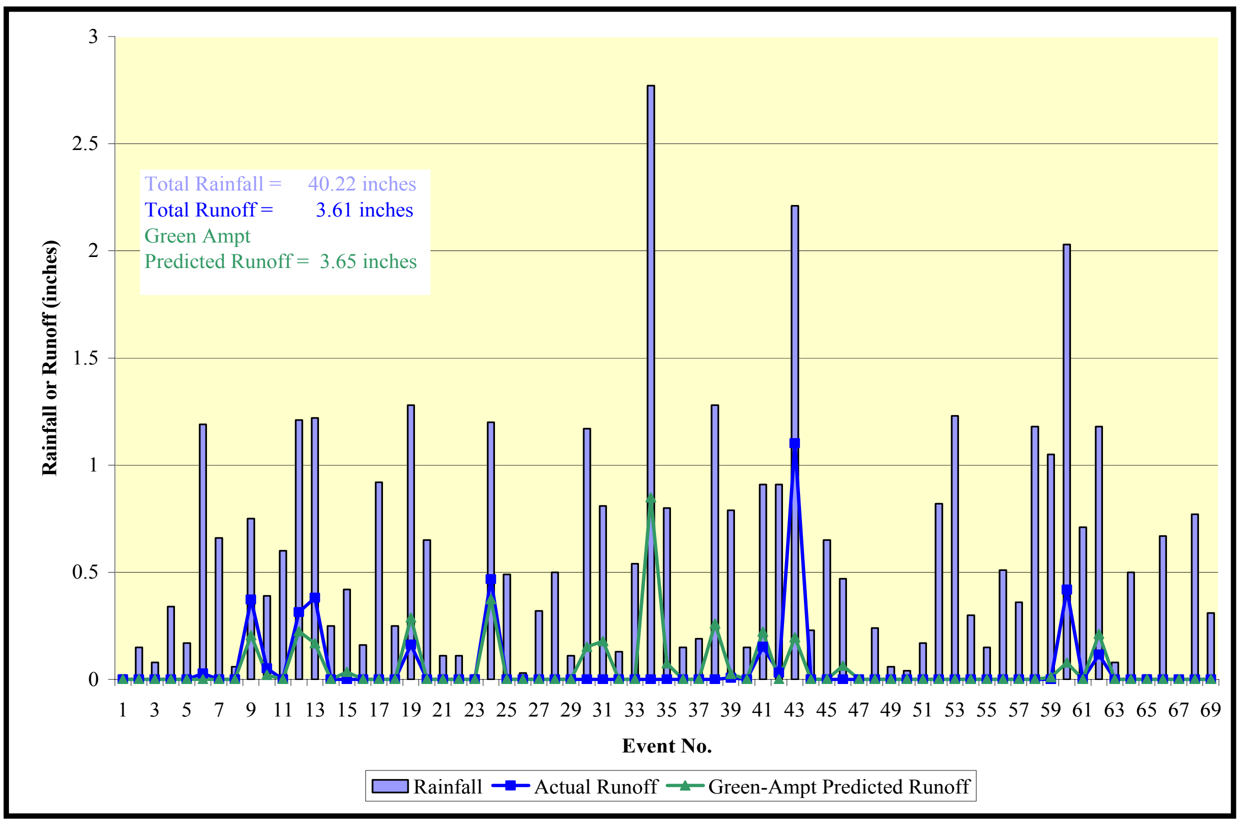

Figure 1.B-1 Comparison of Green-Ampt Predicted and Actual Runoff at Sarasota Bradenton International Airport

Figure 1.B-2 Comparison Chart for Infiltration Methods summarizes the methods considered and the general characteristics of each.

Method	Calculation	Site Applicability	Model Parameters
Phi	Rudimentary	High w/ measurement	None
Horton	Spreadsheet	High w/ measurement	Empirical
NRCS –CN*	Rudimentary	Low (rarely measured)	Theoretical basis
Green-Ampt	Spreadsheet	High	Physical basis

*** Reference ___**

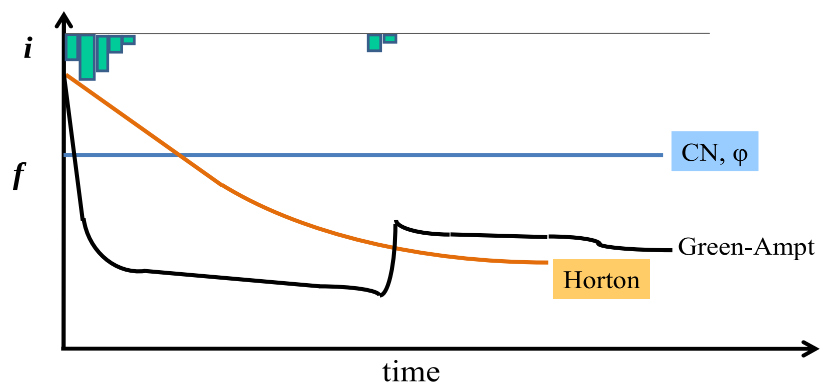

Figure 1.B-2 Comparison Chart for Infiltration Methods

Based on the ability of the Green-Ampt algorithm to model incremental and cumulative infiltration founded upon physically-based soil parameters, in lieu of more empirical or rudimentory infiltration models, it was selected to quantify infiltration in the upper, unsaturated soil zone. The basic form of the equation is:

$$f = K_s \, (1 - M_d \psi / L M_d)$$

Where: f = infiltration rate
K_s = saturated vertical hydraulic conductivity
M_d = initial moisture deficit further defined as the saturated moisture content minus the initial moisture content
Ψ = soil suction
and, L = depth to the infiltrating wetting front which varies with infiltration volume

The equation is solved iteratively. The parameters for initial moisture deficit, soil suction and saturated hydraulic conductivity are the inputs to the SWMM program.

1.C Ground Water Modeling Modifications

The groundwater flow equation used in SWMM has a general form shown in Equation 1.C.1. SWMM utilizes a simplified Dupuit-Forcheimer approximation for flow into a channel.

$GW = A1(H_{gw} - E)^{B1} - A2(H_{sw} - E)^{B2} + A3(H_{gw}H_{sw})$ …………………………. Eq. 1.C.1

In this expression:

Q_{gw} = groundwater flow (cfs per acre),
H_{gw} = elevation of groundwater table (ft),
H_{sw} = elevation of surface water at receiving node (ft), and
E = elevation of node invert (ft), and
B1 and B2 describe the relationship as linear or a power function.

A modification was made to the SWMM 5 computer code by the investigators to allow for a more accurate representation of Dupuit Forcheimer seepage. The code is shown below in C++ and applies to the "gwater.c" module.

```
// Special case for D-F seepage
   if (GW->b1 == 2.0 && GW->a2 == 0.0 && GW->a3 == GW->a1)
   {   lowerDepth = lowerDepth * UCF(LENGTH);
       t1 = GW->a1 * lowerDepth * lowerDepth;
       t3 = (NodeInvert + NodeDepth - A.bottomElev)
            * UCF(LENGTH);
       // ^ This is h2
       t3 = GW->a3 * lowerDepth * t3;
       return (t1 - t3) / UCF(GWFLOW)    }
```

This modification changes the groundwater flow equation to Equation 1.C.2.

$GW = A1(H_{gw})^2 - A3(H_{gw}H_{sw})$ ……………………………………….…. Eq. 1.C.2

The groundwater and surface water flow coefficients require determination of the distance between the farthest point of a given catchment and the edge of the closest trench. These lengths were calculated in feet but converted to L^2 in acres.

Table 1.C-1 Modified Groundwater Parameters Entered into SWMM Simulation

Property	Value
Groundwater Flow Coefficient (fps/acre)	$4K/(L^2)$
Groundwater Flow Exponent	2
Surface Water Flow Coefficient	0
Surface Water Flow Exponent	0
Surface-GW Interaction Coefficient	$4K/(L^2)$
Fixed Surface Water Depth	Computed by flow routing
Threshold Groundwater Elevation.	Equal to node invert elevation

2. PARAMETERS and DESIGN FEATURES

2.A RAINFALL and EVAPORATION/EVAPOTRANSPIRATION DATA

2.A.1 Site Model Rainfall

Rainfall data in 5-minute intervals and 0.01-inch depth increments were available from the earlier components of the Statewide Airport Stormwater Study. The data have an average record period of 21 months and are from 13 different Florida airports. Figure 2.A.1-1 shows the collection locations. From these records, the Orlando International Airport (MCO) rainfall data was selected for pristine and developed airport SWMM simulation.

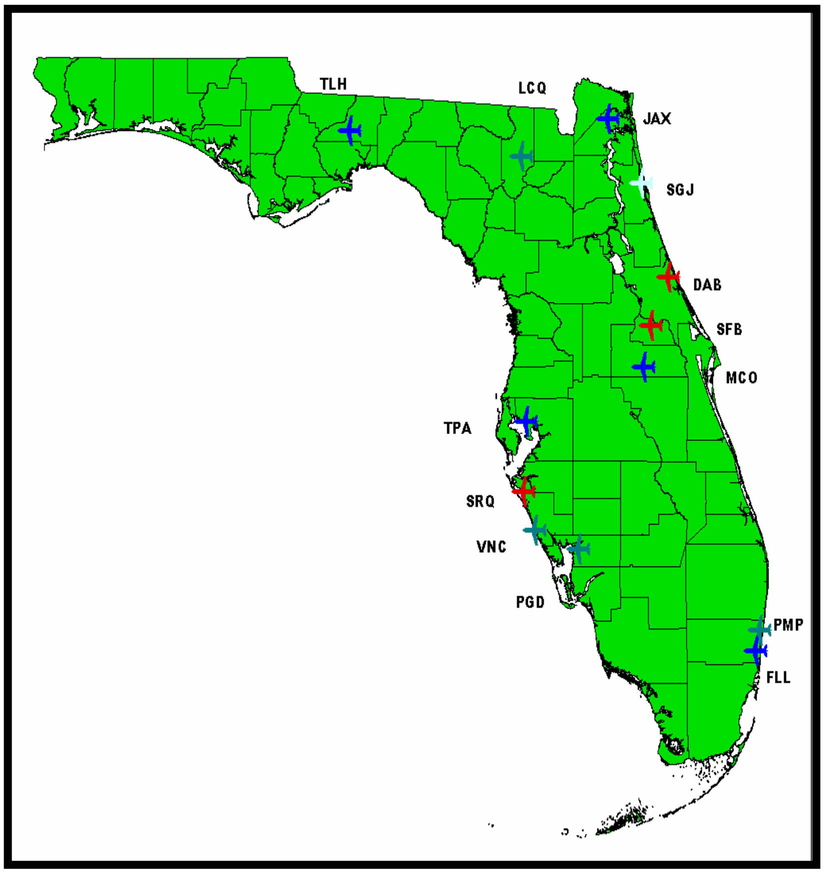

Figure 2.A.1-1 Statewide Airport Stormwater Study Airports

The five-minute, MCO rainfall data began July 1, 2002 and ends February 1, 2004. Figure 2.A.1-2 illustrates the rainfall record.

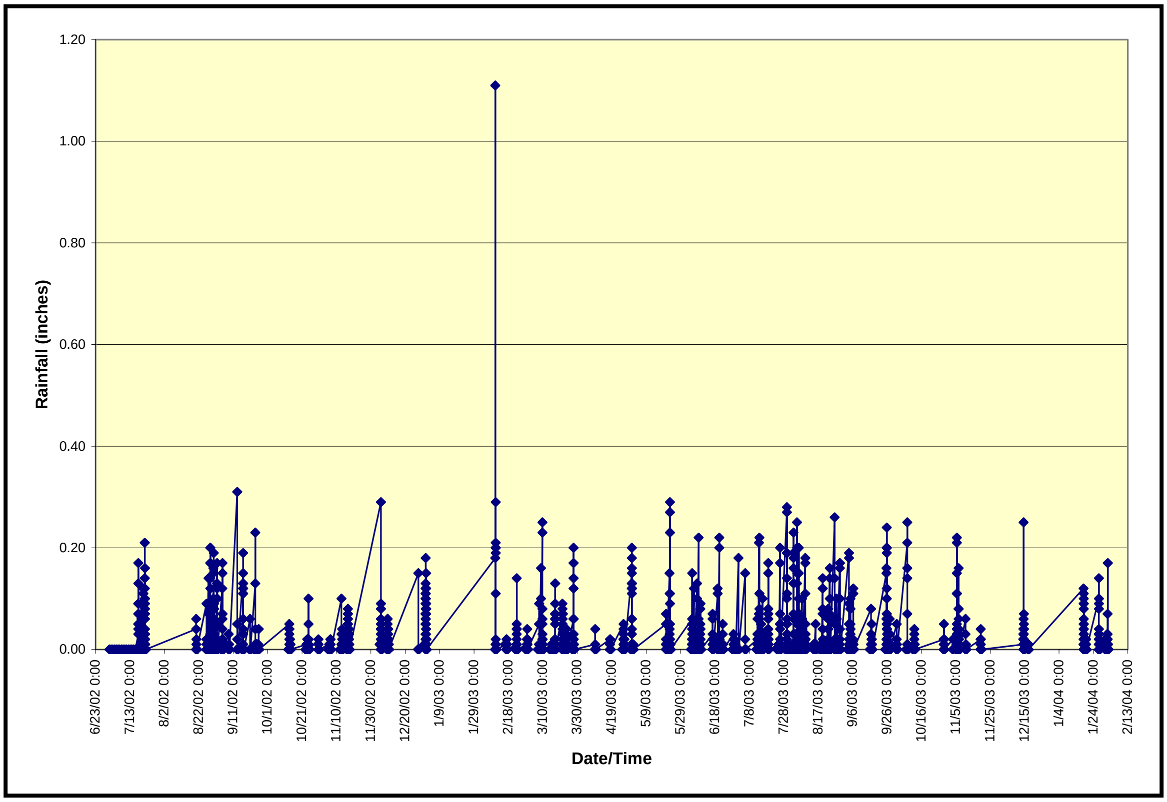

Figure 2.A.1-2 Recorded 5-minute Rainfall Record for Orlando International Airport

A synthetic, 24-hour, NRCS Type-II design storm was inserted into the historical time-series in the wet season on August 8, 2003 at 00:00 time. Actual recorded rainfall precedent to the synthetic event ended at 21:25 hours on August 7, 2003. This provided 2 hours 35 minutes between the end of the real event and the synthetic design storm. This "worst case" scenario overlayed the synthetic event on a saturated airside system. Figure 2.A.1-3 illustrates this rainfall record with the synthetic storm inserted.

The same synthetic design storm was also applied independently of the historical time-series. The independent storm event simulation was run for 48 hours total, providing a 24-hour, post-event period. The post-event period allowed final runoff, infiltration, evapotranspiration and drainage through the airside system.

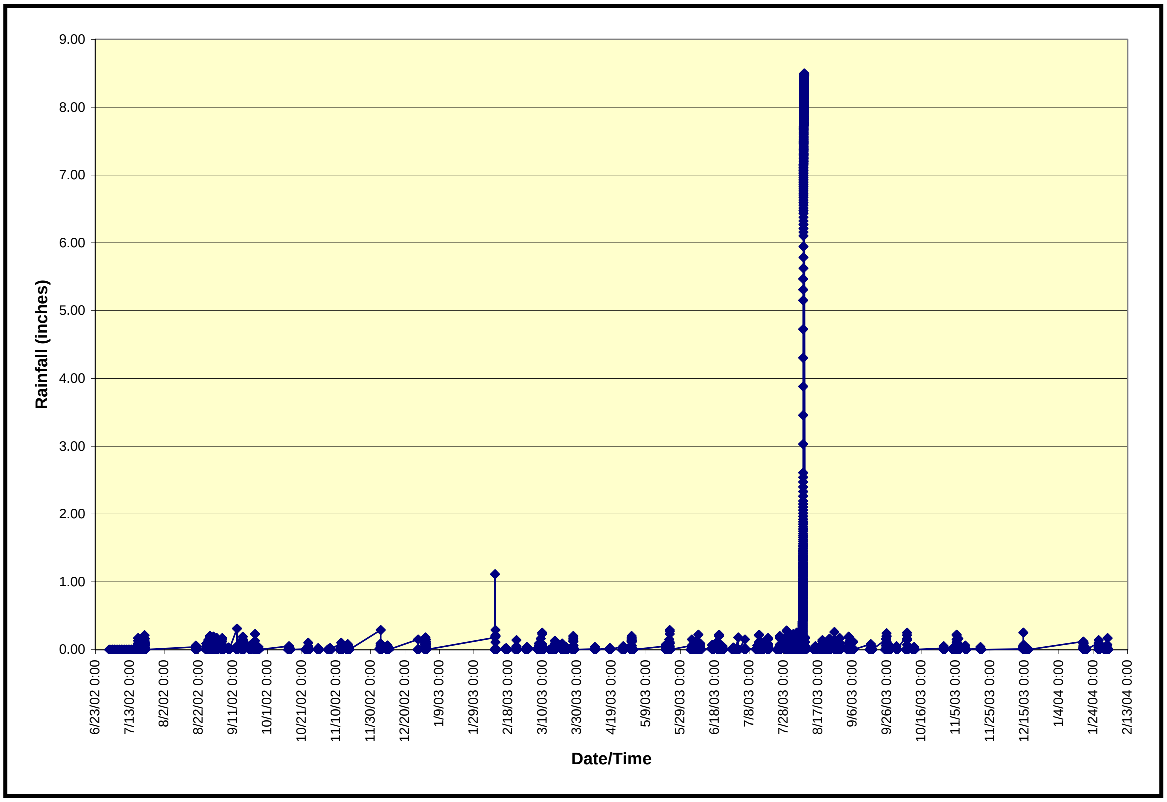

Figure 2.A.1-3 Actual 5-minute Data with Synthetic 24-hour, 25-year Design Storm Added

<u>2.A.2 Site Model Evaporation/Evapotranspiration</u>

The airside land surface is comprised of approximately 15% impervious area and 85% short grass cover. Evapotranspiration data for a determination of soil moisture fluxes during dry periods in a system that combined transpiration by grass cover as well as evaporation were incorporated into the simulation. The values of daily evapotranspiration (ET) were obtained from the University of Florida IFAS Florida Automated Weather Network for a period of record from 01 June 2002 through 01 March 2004. The information is presented visually in Figure 2.A.2-1 following.

Evapotranspiration and evaporation decline with depth in a soil profile. This relationship is illustrated in Figure 2.A.2-2 following, and is used in the SWMM simulations.

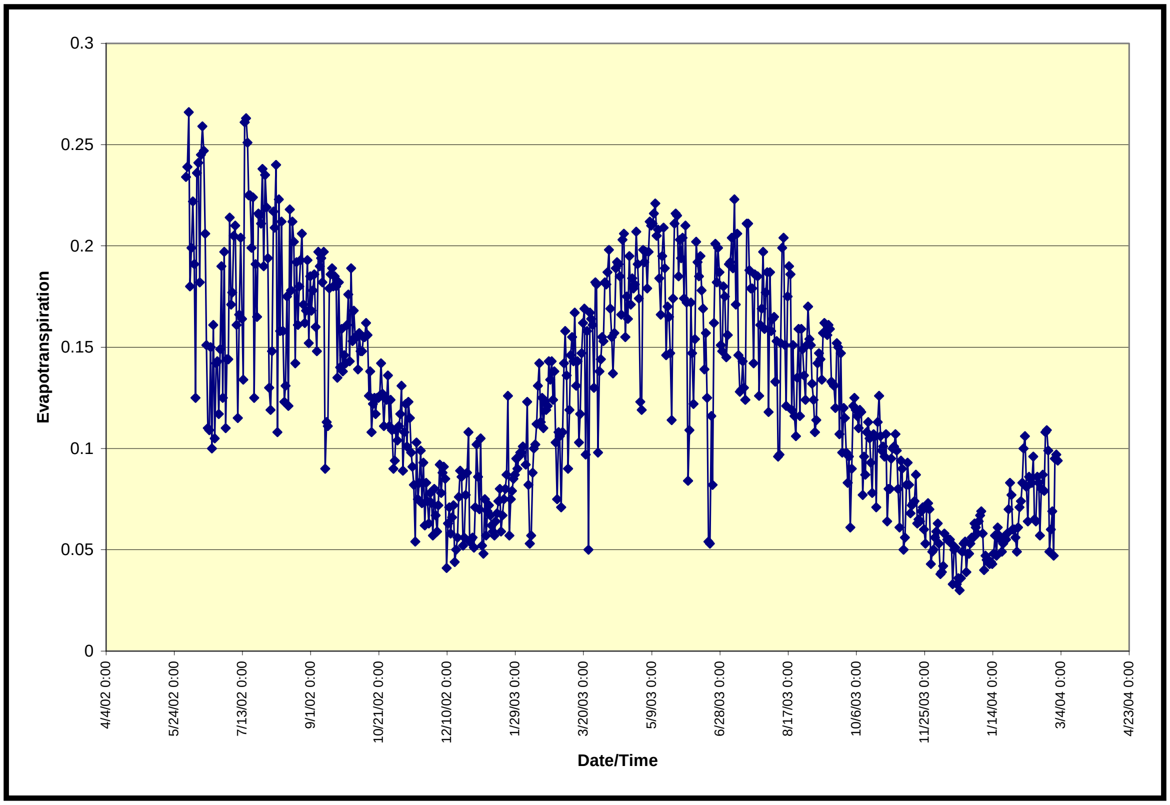

Figure 2.A.2-1 Daily Evapotranspiration (Inches/Day)Measured in Central Florida

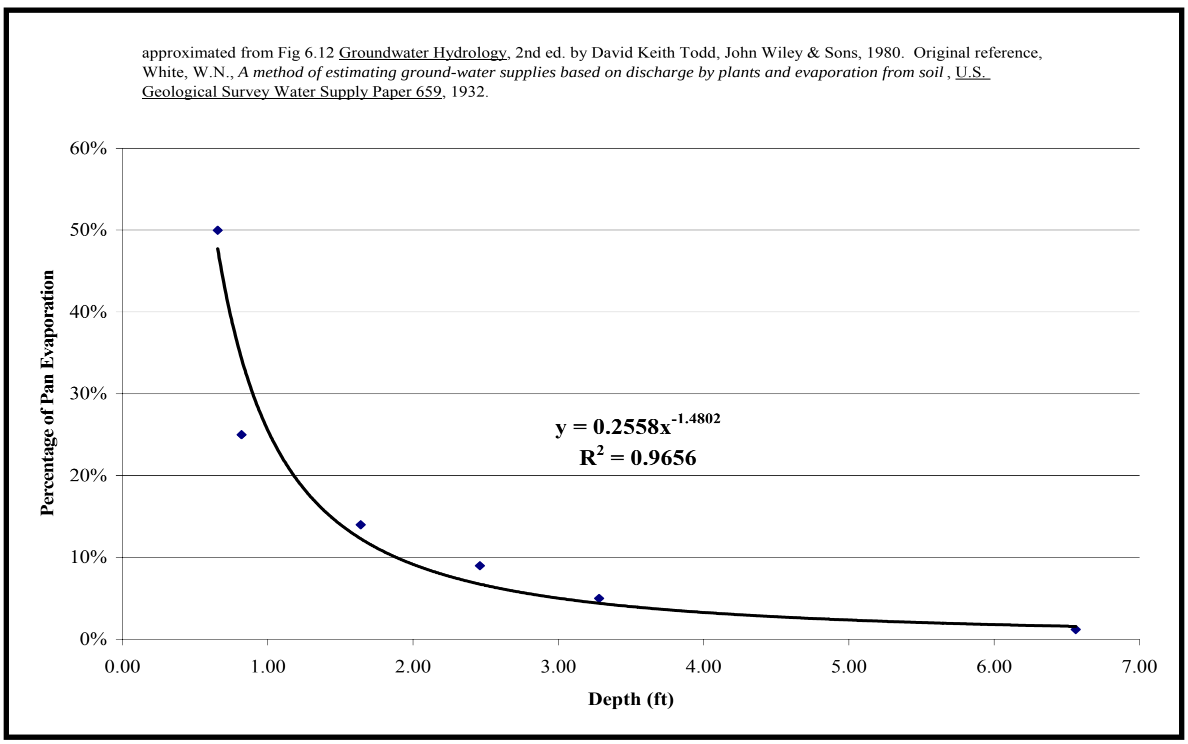

Figure 2.A.2-2 Evaporation as a Function of Depth Below Ground Surface

2.B EVENT MEAN CONCENTRATIONS (EMC)

Constituent EMC data were measured at 13 airports throughout Florida for various airside pavement features. These data are presented and described in the companion *Technical Report for the Florida Statewide Airport Stormwater Study*. Six constituents from that study have corresponding EMC data available for either a "pristine" site or for forest/rangeland that can be used to characterize a pristine site. The six constituents are Total Suspended Solids (TSS), Total Nitrogen (TN), Total Phosphorus (TP), Copper (Cu), Lead (Pb) and Zinc (Zn). Since a pristine site number was not available from FDEP for metals, the data from Harper and Baker (2003) were used for these. Rangeland/forest data from that reference were used for Lead and Zinc. Wetland data from the reference were used for Copper since no rangeland/forest value was reported. The EMC's used in the simulations are given in Table 2.B-1 following:

Table 2.B-1 Event Mean Concentrations in [mg/L] by Landuse

Airside Type	TSS	Total Nitrogen	Total Phosphorus	Copper	Lead	Zinc
Apron (composite type)	7.2	0.398	0.057	0.020	0.010	0.039
Runway, Air Carrier	9.7	0.401	0.049	0.024	0.003	0.065
Taxiway, Air Carrier	24.4	0.569	0.115	0.014	0.005	0.022
Pristine or Undeveloped Site	7.8	1.15	0.074	0.001	0.005	0.006

Mean EMC values from measurements were entered into the SWMM simulations for the different hydrologic functional units. The maximum load deposition was also specified for each pollutant for each landuse. These were established by running the historical simulation, calculating the total wash-off load per event, and specifying the mean load as the maximum load for a given event.

Runoff quality improvement is handled solely as a function of infiltration volume. Interflow from shallow surficial groundwater that exfiltrated back into the channel was assumed to carry no pollutant loads.

2.C POLLUTOGRAPHS

In the original airport stormwater study a series of apron stations used discrete sampling to develop pollutagraphs. Normalizing the data and using the apron results as a surrogate distribution for the other airside pavements yields discrete concentrations as a function of effective rainfall (rainfall producing runoff from pavement). These data are provided in Table 2.C-1 following. These distributions were considered implicitly in the site simulations with the buildup and washoff functions SWMM.

Table 2.C-1 Pollutograph Values for Modeled Constituents

		Effective Rainfall (inches)						
		0.1	0.2	0.3	0.4	0.5	0.6	0.7
Runway Constituent	EMC (mg/l)	Incremental Concentration (mg/l)						
TSS	9.7	24.9	9.0	4.9	3.2	2.3	1.8	1.4
TN	0.401	0.607	0.366	0.273	0.221	0.188	0.164	0.147
TP	0.049	0.085	0.046	0.032	0.025	0.020	0.017	0.015
Cu	0.024	0.058	0.023	0.013	0.009	0.006	0.005	0.004
Pb	0.003	0.009	0.003	0.002	0.001	0.001	0.000	0.000
Zn	0.065	0.123	0.061	0.041	0.031	0.025	0.020	0.018
Taxiway Constituent	EMC (mg/l)	Incremental Concentration (mg/l)						
TSS	24.4	62.5	22.6	12.4	8.1	5.8	4.5	3.6
TN	0.569	0.861	0.520	0.387	0.313	0.266	0.233	0.208
TP	0.115	0.198	0.108	0.075	0.059	0.048	0.041	0.035
Cu	0.014	0.035	0.014	0.007	0.005	0.003	0.002	0.002
Pb	0.005	0.013	0.005	0.002	0.002	0.001	0.001	0.001
Zn	0.022	0.043	0.021	0.014	0.010	0.008	0.006	0.005
Apron Constituent	EMC (mg/l)	Incremental Concentration (mg/l)						
TSS	7.2	18.5	6.7	3.7	2.4	1.7	1.3	1.0
TN	0.398	0.601	0.364	0.270	0.219	0.187	0.163	0.146
TP	0.057	0.099	0.053	0.037	0.029	0.024	0.020	0.018
Cu	0.02	0.049	0.019	0.011	0.007	0.005	0.004	0.003
Pb	0.01	0.026	0.009	0.005	0.003	0.002	0.002	0.002
Zn	0.039	0.074	0.037	0.025	0.018	0.014	0.012	0.011

2.D SIMULATION GEOMETRY

2.D.1 Pristine Site Geometry

The pristine site is an area of approximately 307 acres, exactly consistent with the developed site area. The site is rectangular and has a ground surface slope of 0.1%.

2.D.2 Airport Geometry

The developed condition simulations all use the same airport airside geometry. This consists of a single runway, one full-length parallel taxiway and one apron area as shown in Figure 2.D.2-1.

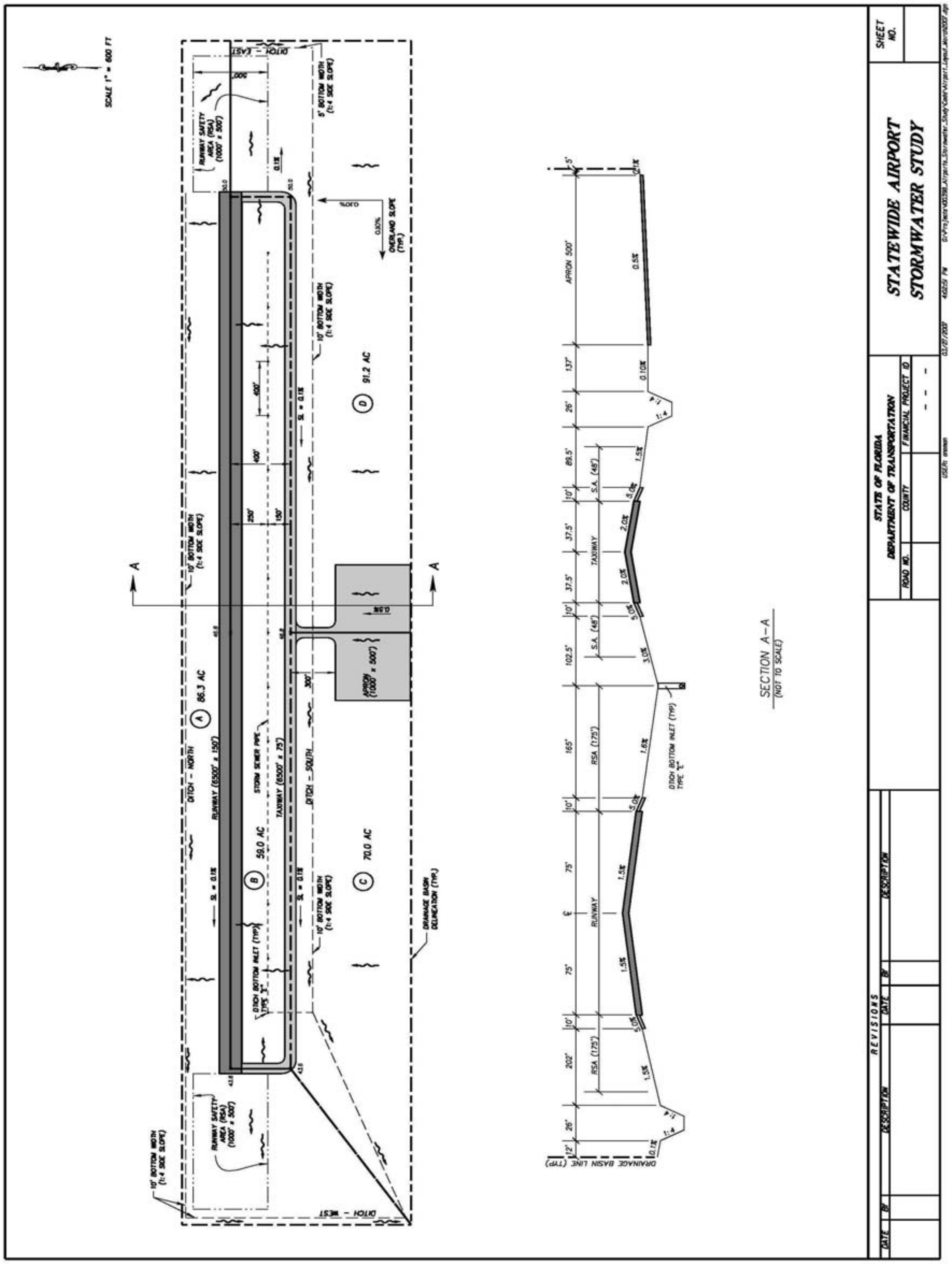

Figure 2.D.2-1 Developed Airport Airside Simulation Geometry

The runway is oriented east-west and is 6,500 feet long and 150 feet wide. The taxiway is parallel to the runway and is 75 feet wide. The taxiway provides access to the 1,000 foot x 500 foot rectangular apron. The geometry is generally consistent with FAA design requirements for Airplane Design Group IV, which includes large jets such as the Boeing 757. It was selected to represent the typical maximum pavement area per unit length for the runway and taxiway system. That is, it would be applicable to most commercial runway/taxiway systems and would be more severe than most general aviation (GA) airports with respect to the impervious/pervious area ratio.

The cross sections used for the model airport are consistent with FAA criteria and are also illustrated in Figure 2.D.2-1. The runway is designed to have a 1.5% transverse slope and 0.1% longitudinal slope in the westerly direction. The taxiway is designed to the same 0.1% longitudinal slope, but has a 2.0% transverse slope. The apron slopes down in the northerly direction at 0.5% to facilitate drainage. All pavements for this model GA airport are assumed to be constructed with asphalt and the roughness coefficient for asphalt is used in the SWMM simulations. FAA guidelines also require a minimum 10-foot wide stabilized shoulder with a 1.5% to 5.0% transverse slope for runways and taxiways. This can be either paved or unpaved. It is common for commercial airports to have paved shoulders and general aviation airports to have unpaved shoulders. The study airport has unpaved shoulders with the maximum 5.0% transverse slope.

FAA requires runway and taxiway safety areas adjacent to the pavement that are suitable for reducing the risk of damage to airplanes in the event of an undershoot, overshoot or excursion from the runway or taxiway. The safety areas must be object free (excepting those needed for navigation such as signs and lights), regularly maintained, and are typically covered with turf. Drainage structures cannot be located within these designated safety areas. The runway safety area for the study airport is 500 feet wide centered on the runway centerline (175 feet from the runway pavement edge) and extends 1,000 feet beyond each end of the runway. The taxiway safety area is 171 feet wide centered on the taxiway centerline (48 feet from the taxiway pavement edge). A cross slope of 1.5% to 3.0% is required within the safety areas; 1.5% is used for the study airport.

The portions of the airport not included in the safety areas are assumed to be predominantly flat and a 0.10% overland slope similar to the pristine site is used for them. Overall, the airport site slopes mildly down from east to west. Four trapezoidal ditches with 15 feet bottom width, 8 feet deep and 4H:1V [horizontal:vertical] side slopes, are positioned along the perimeter of the airport and outside the safety area to convey runoff. Runoff from the exterior paved surfaces is routed to shoulders, across grass safety areas and ultimately to the perimeter ditches. A 0.10% slope is used on the ditch system coinciding with the overall overland slope of the airport. Infield areas between the taxiway and runway drain across the shoulders, over the safety areas and are collected into a pipe system through grate inlets. The grate inlets are designed and modeled level with the ground surface. Ultimately, all airport stormwater discharges to the southwest corner of the site.

The percent imperviousness varies from basin to basin. It ranges from a low of 12% impervious area to a high of 52% impervious area for this model airport. As a total site, the airside land surface is comprised of approximately 15% impervious area and 85% grass cover. This range of percent imperviousness is consistent with a typical airport airside due to FAA requirements for the minimum safety areas and obstruction free zones. The internal routing method used in the SWMM simulations is impervious to pervious. This is equivalent to non-directly connected impervious areas (non-DCIA) in other runoff models.

2.D.3 Simulation Model Elements

Proper site discretization provides a more realistic simulation of flow length and travel time when compared to aggregated models (Lee and Heaney, 2003). Highly aggregated models will underestimate surface contact time, infiltration, and pollutant routing. Also, they will often produce an incorrect time of concentration for a site. The hypothetical airport site modeled in this study has an overall slope of 0.1% to the west with significant flows north and south to the trapezoidal ditches shown with dark blue channel centers in Figure 2.D.3-1. These channels flow in a westerly direction at a 0.1 % slope.

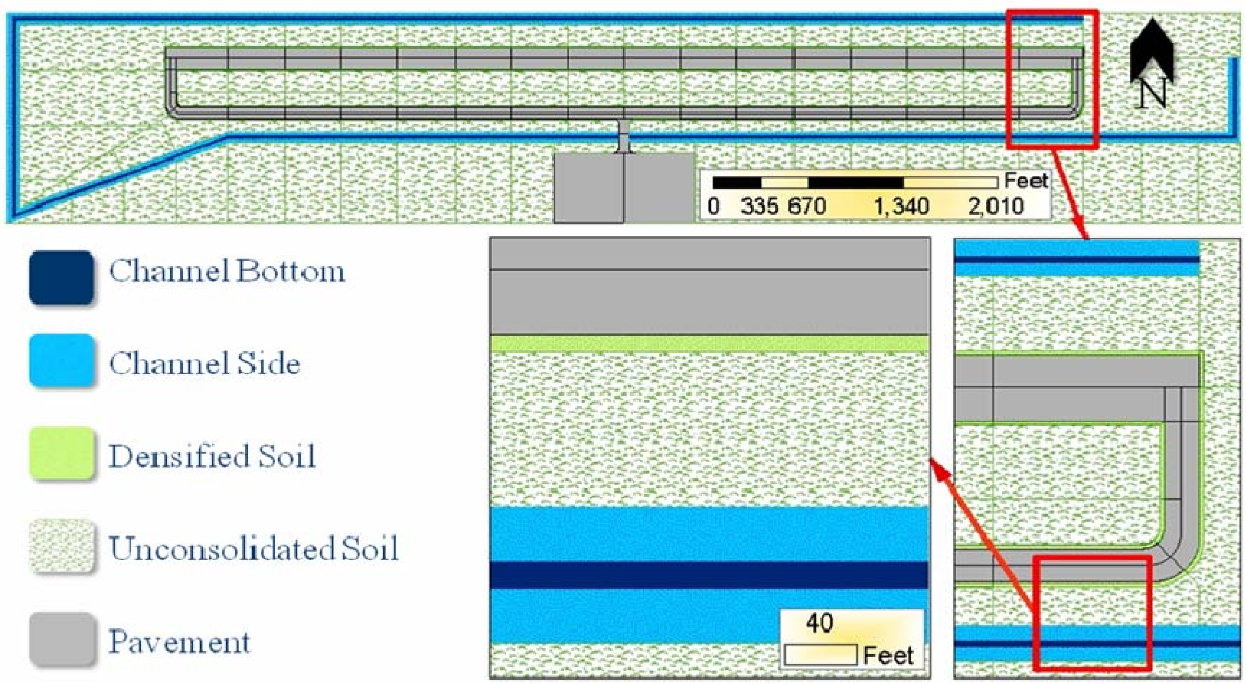

Figure 2.D.3 -1 Hydrologic Functional Units (HFU) for the developed airside of the airport.

By discretizing the channel into 400 foot segments, with each segment receiving run-on from subcatchments to the north and south, and connecting the 400 foot segments in series, the model simulated the diagonal overland flow paths for the subcatchments while still providing westerly flow via the trenches. The subcatchments north of the crowned runway centerline drain to the north trench while those catchments south of the crowned taxiway centerline drain to the south trench. Exceptions to these common drainage patterns are the far west subcatchments which drain into the west portion of the trench system, the far east subcatchment which drains into the east trench, and the overland flow areas which drain into a below grade pipe system (not shown) between the taxiway and runway. This pipe system drains to the west, eventually discharging into the south trench. The individual catchments, trench segments, pavement segments and pipe segments are configured in SWMM as separate Hydrologic Functional Units (HFU).

The airport airside configuration used in the SWMM simulations is as shown in Figure 2.D.3-2 where the outfall at the bottom left of the schematic is the lowest drainage point of the model. The schematic of subcatchments, junctions and outfalls is the SWMM equivalent of the more commonly reviewed basin and nodal diagrams.

The pristine condition simulation layout is shown in Figure 2.D.3-3. The area is represented as one subcatchment which flows west. Surface flow and infiltration parameters for both the pristine and developed conditions are discussed in Section 2.E following.

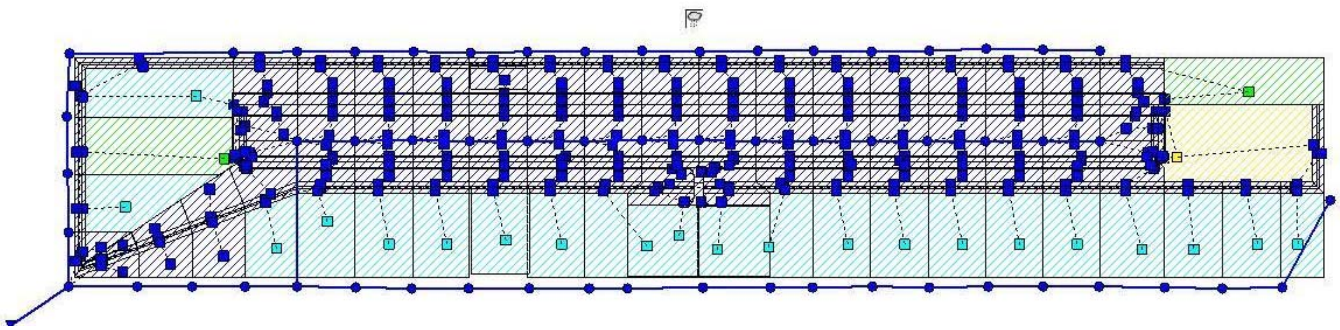

Figure 2.D.3-2 SWMM Schematic of Developed Airside Subcatchment Connectivity

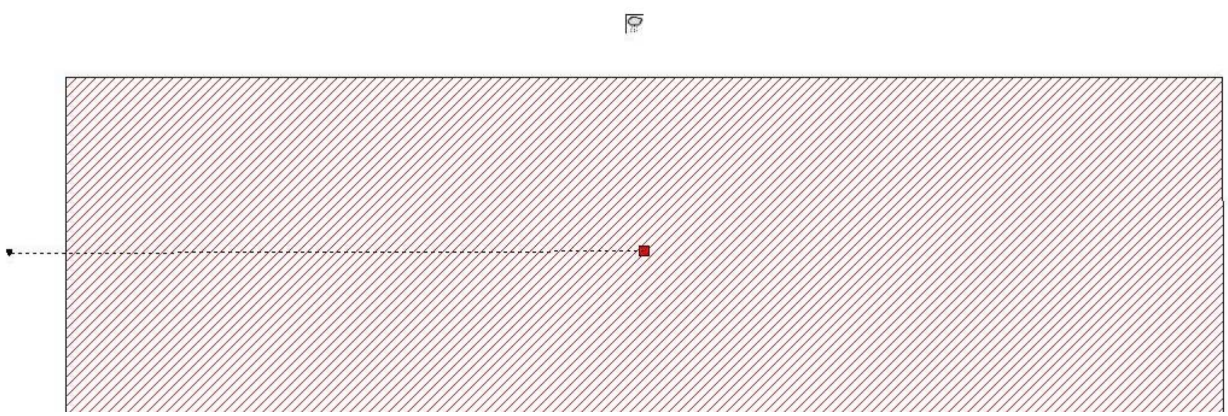

Figure 2.D.3-3 SWMM Schematic for Pristine Conditions

2.E OVERLAND FLOW PARAMETERS

Overland flow parameters have been widely characterized for use in many models and simulation tools including SWMM (Emmet 1968, Rouhipour et. al 1999, Prosser 2000). These parameters are briefly described. Manning's n is a roughness coefficient that characterizes overland flow hydraulic resistance. Depression storage functions provide initial abstraction for an event by storing a fraction of the rainfall on the impervious or pervious surfaces. This initial abstraction is renewed between events by evaporation parameters. Table 2.E-1 illustrates the overland flow and surface storage parameters used for pervious and impervious HFU in all simulations. These values fall within the range of values presented in the literature.

Table 2.E-1: Overland Flow and Surface Storage Parameters

FUNCTIONAL LAND UNIT	MANNING'S N	DEPRESSION STORAGE (IN)	IMPERVIOUS AREA (%)
Channel Bottom	0.04	0.1	0
Channel Side	0.025	0.075	0
Densified Soil	0.035	0.075	0
Unconsolidated Soil	0.035	0.15	0
Pavement	0.012	0.05	100

2.F SOIL and GROUNDWATER CONDITIONS

Soil and groundwater conditions are explicitly modeled in the SWMM simulations. This differs from the common practice of estimating runoff using NRCS Curve Numbers based on land use and Hydrologic Soil Groups. Hydrologic Soil Groups incorporate two conditions, soil type/texture and depth to water table, into a single classification. For example, a Group D mapping unit may contain highly permeable soils in an area of high groundwater, impervious soils in an area of low groundwater, moderately permeable soils in an area of high groundwater, and so forth. The Green-Ampt parameters used in the SWMM model define the soil properties, and the groundwater depths and recharge/discharge define the groundwater conditions as separate, interacting elements. However, for clarity with common practice, the case studies are related to Hydrologic Soil Groups A for the dry, B and C for the intermediate, and D for the wet site conditions.

Aquifer parameters used in the model are summarized in Table 2.F-1. The porosity, hydraulic conductivity, and unsaturated zone moisture are familiar values. Conductivity slope and tension slope reflect the documented variation of hydraulic conductivity (gravitational) and matrix potential (capillary suction) with soil saturation levels. Under conditions of more conductive soils such as clean sands, a higher conductivity slope is typical. This reflects the predominant role of gravitational drainage in them. In contrast, a higher tension slope was established for finer-grained soils such as silt or loam that exhibit higher capillary suction and relatively lower gravitational drainage.

Potential evapotranspiration is limited by wilting point. Wilting point is the point at which saturation of the soil is high enough to kill plants, limiting transpiration from the unsaturated zone.

The lower groundwater loss rate for deep percolation was determined using a USGS database for the Floridian aquifer (footnote). The value is reduced for the wet site conditions more typical of an aquiclude as opposed to an aquitard.

Table 2.F-1 Aquifer Parameters Entered into SWMM Simulation

Property	A-Soil Value	B-Soil Value	C-Soil Value	D-Soil Value
Porosity (in/in)	0.40	0.39	0.34	0.39
Wilting Point (in/in)	0.078	0.03	0.04	0.04
Field Capacity (in/in)	0.3	0.078	0.1	0.1
Conductivity (in/hr)	0.41	0.394	0.118	0.1
Conductivity Slope (in/hr)	12	2.247	.394	.394
Tension Slope (in)	14	209	2,268	2,268
Upper Evaporation Fraction	0.35	0.3	0.35	0.3
Lower Evaporation Depth (ft)	10	8	8	8
Lower GW Loss Rate (in/hr)	0.006	0.0004	0.0004	0.00000004
Bottom Elevation (ft)	0	0	0	0
Unsaturated Zone Moisture (in/in)	0.35	0.078	0.1	0.14

2.F.1 Dry Site Conditions

Hydrologic Soil Group A is represented by the Green-Ampt parameters and groundwater conditions in the Dry Site simulations. NRCS characterizes Hydrologic Soil Group A as having a low runoff potential with a high infiltration rate, even if thoroughly wetted. For the dry case simulation, the input data used closely follows soil and groundwater properties of Candler Sand (CaB) from Marion County, Florida which is considered nearly level, excessively drained sandy soil with a depth of 60 to 80 inches. This type of soil usually occurs on sandy ridges in uplands. Groundwater is assumed to be at a depth of more than 72 inches in SWMM's input coinciding the soil survey's reported value.

For the historic pristine conditions and future airport buildout conditions, the soil and groundwater properties are essentially equal. The following Green-Ampt infiltration parameters for Type A soils are used in the Dry Conditions model:

Textural Classification = sand
Soil Saturated Hydraulic Conductivity [K_s] = 12.0 (in/hr)
Soil Capillary Suction head [ψ] = 5.0 (in)
Initial Soil Moisture Deficit [M_d] = 0.35 (fraction)

For continuous simulation, it is important to recover the field capacity of the unsaturated soil layer which becomes saturated following a storm event. SWMM uses one-dimensional groundwater equation for groundwater flow which is defined by the groundwater and aquifer parameters used in the model. Values used for these parameters were determined from a literature review for all soil types simulated in the model. Table 2.F-1 lists these.

Although a fixed groundwater elevation is specified in the model at the beginning of the simulation, groundwater elevations are varied based on the parameters defined in the groundwater and aquifer component of SWMM. For the dry case, groundwater levels remain more than 6 feet beneath the ground surface.

2.F.2 Intermediate Sites Conditions

The Intermediate Sites Conditions consider two different cases correlated to Hydrologic Soil Groups B and C. NRCS characterizes Hydrologic Soil Group B as having moderate infiltration rates when thoroughly wetted. Hydrologic Soil Group C is characterized as having low infiltration rates when thoroughly wetted. For these cases, the Green Ampt parameters vary in the highly compacted shoulder areas.

Table 2.F.2-1 summarizes parameters for the safety areas and channel slopes and bottoms for both B and C soils. This table also applies to the pristine condition. Parameters in Table 2.F.2-2 apply to the highly compacted 10 foot shoulder area along the edge of the runway and taxiway. Saturated conductivity and moisture deficit parameters were established for these generic conditions using relations published by Carsel and Parish (1988). Suction head (Ψ) was estimated for the generic conditions using Li, Buchberger and Sansalone (1999) and Rawls et. al (1983). Additionally, physical measurements were taken in-situ at selected airports, and laboratory tests were run on returned samples to verify parameter reasonableness.

Table 2.F.2-1 Unconsolidated Soil Parameters for Intermediate Cases

Hydrologic Soil Group	Textural Classification	K_s (in/hr)	Ψ (inches)	Md (fraction)
B	Loam	1.0	2.5	0.35
C	Sandy Clay Loam	0.5	8.2	0.29

Table 2.F.2-2 Fully Compacted Soil Parameters for Intermediate Cases

Hydrologic Soil Group	Textural Classification	K_s (in/hr)	Ψ (inches)	Md (fraction)
B	Loam	0.4	3.7	0.31
C	Sandy Clay Loam	0.1	10.3	0.24

Although a fixed groundwater elevation is specified in the model at the beginning of the simulation, groundwater elevations are varied based on the parameters defined in the groundwater and aquifer component of SWMM. Generally, groundwater for these simulations is from 3 to 6 feet beneath ground surface.

2.F.3 Wet Site Conditions

Hydrologic Soil Group D is represented by the Green-Ampt parameters and groundwater conditions in the Wet Site simulations. NRCS characterizes Hydrologic Soil Group D as having a very low infiltration rate when thoroughly wetted. For both the pristine and developed airside conditions, the soil and groundwater properties are essentially equal. The limiting case includes ground water within 12 to 18 inches of the ground surface as the simulation progresses, and very low hydraulic conductivity. That is, the model approximates a worst case Type D condition for Florida. The following Green-Ampt infiltration parameters for Type D soils are used in the Wet Conditions model:

Textural Classification	= sandy clay loam
Soil Saturated Hydraulic Conductivity [K_s]	= 0.1 (in/hr)
Soil Capillary Suction head [ψ]	= 3.5 (in)
Initial Soil Moisture Deficit [M_d]	= 0.2 (fraction)

2.G. WET POND GEOMETRIES and INFLOWS

2.G.1 Southwest Florida Water Management District (SWFWMD) 14-Day Wet Pond

The SWFWMD 14 day pond was designed to specifications consistent with guidelines set forth in SWFWMD Basis of Review Section 6.4.1 dated March 2004 and in SWFWMD Technical Procedure TP/SWP-022**.** The pond was designed with the following considerations:

- ☐ 2:1 aspect ratio (length: width) at the control elevation,
- ☐ a littoral shelf at the effluent end,
- ☐ a horizontal weir with a drawdown orifice effluent control structure,
- ☐ a maximum permanent pool depth of 8 feet
- ☐ a 24 hour treatment volume drawdown rate, and
- ☐ a "14-day" permanent pool volume consistent with SWFWMD guidelines.

The permanent pool volume slopes were set at 4H:1V [horizontal:vertical], while the littoral shelf and embankments above the control elevation were set at 100:1 and 12:1 respectively. The shelf slope was calculated in order to maintain the SWFWMD littoral shelf depth criteria of 1 foot to 3 feet below the control elevation, as well as the area criteria of one third of the permanent pool volume. The embankment slopes were calculated in accordance with the SWFWMD treatment volume height criteria of 18 inches, while the treatment volume was calculated as 1 inch of rainfall over the watershed area (11.5 acre paved apron). The influent and effluent pipes were designed with a 42 inch diameter and located along the centerline of the basin and above the control elevation. The effluent control structure consisted of a horizontal weir approximately 10 feet long with a drawdown orifice at the control elevation with a diameter of slightly less than 1½ inches. Design features are illustrated in Figures 2.G.1-1 and 2.G.1-2 and summarized in Tables 2.G.5-1 through 2.G.5-3.

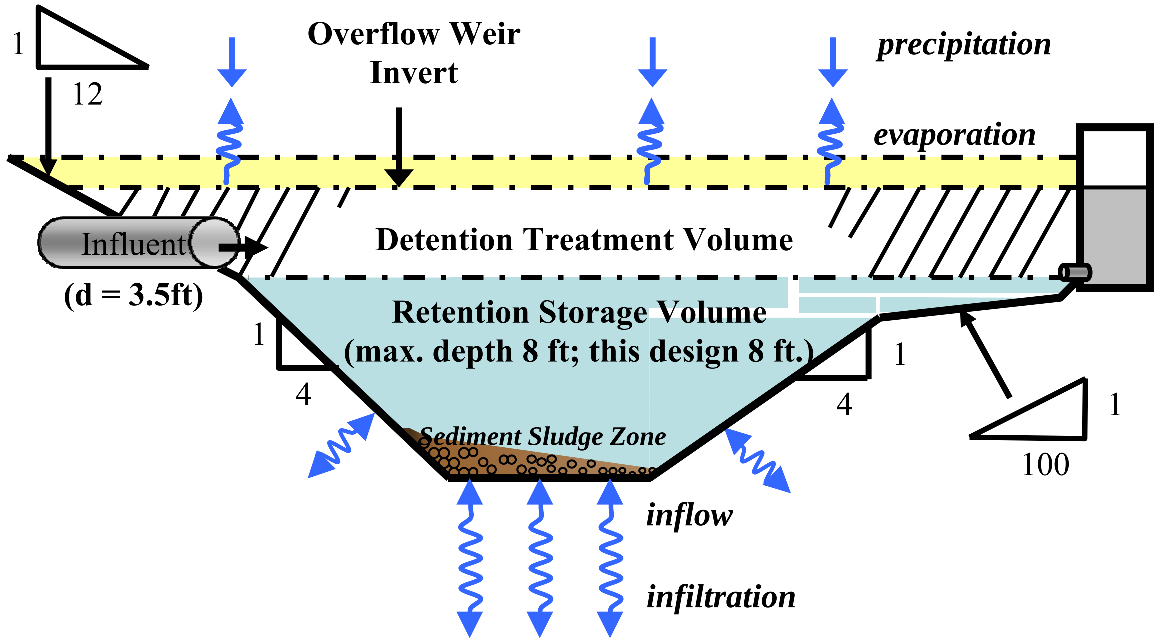

Figure 2.G.1-1 Longitudinal Section of SWFWMD Pond. *(NOT TO SCALE)*

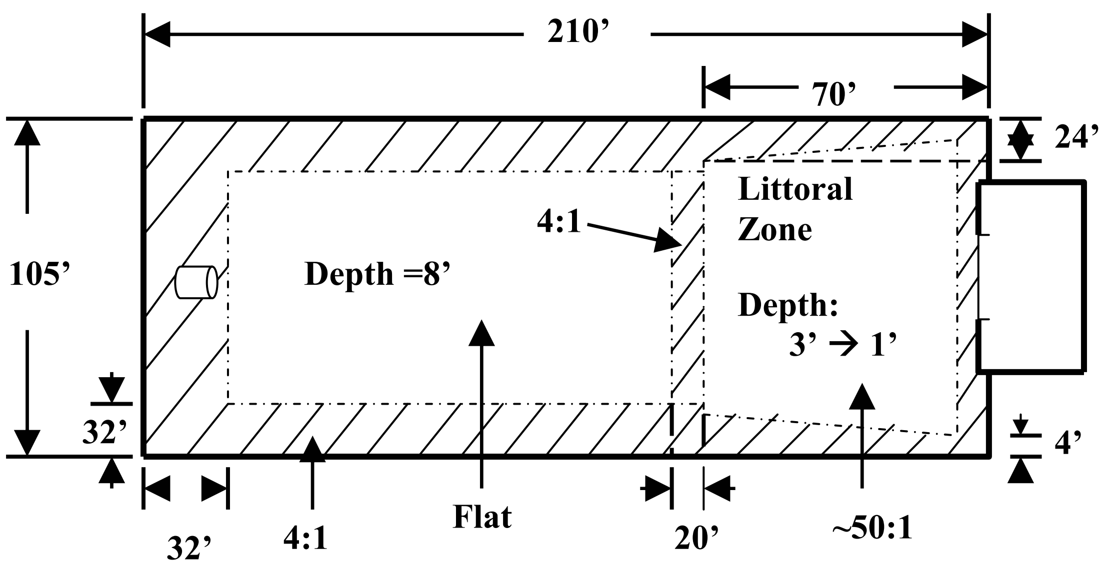

Figure 2.G.1-2 Plan View of SWFWMD Pond at the Top of the Permanent Pool Volume.
(NOT TO SCALE)

2.G.2 St. Johns River Water Management District (SJRWMD) 21-Day Wet Pond

The SJRWMD 21 day pond was designed to specifications consistent with guidelines set forth in SJRWMD Rule 40C-42.026 (4), F.A.C., using subparagraph (d) 2 option . The pond was designed with the following considerations:

- ☐ 2:1 aspect ratio (length:width) at the control elevation,
- ☐ no littoral shelf,
- ☐ a horizontal weir with a drawdown orifice effluent control structure,
- ☐ a 48 hour treatment volume drawdown rate,
- ☐ a maximum permanent pool depth of 12 feet, and
- ☐ a "21-day" permanent pool volume consistent with SJRWMD guidelines.

SJRWMD guidelines specify that the treatment volume can be calculated as 1 in. multiplied by the area of the watershed, or 2.5 inches times the area of the watershed that is impervious. In our case the watershed area is the paved apron, therefore the whole watershed is impervious. Given this consideration, the treatment volume was defined as 2.5 in. times the area of the watershed. The permanent pool volume and embankment slopes were set at 4H:1V [horizontal:vertical], and the depth of the permanent pool volume was set to the maximum allowed 12 feet below the control elevation. The influent and effluent pipes were designed as 42 inch diameter and located along the centerline of the basin and above the control elevation. The effluent control structure consisted of an approximately 10 foot horizontal weir with a drawdown orifice at the control elevation with a diameter of slightly less than 1½ inches. Design features are illustrated in Figures 2.G.2-1 and 2.G.2-2 and summarized in Tables 2.G.5-1 through 2.G.5-3.

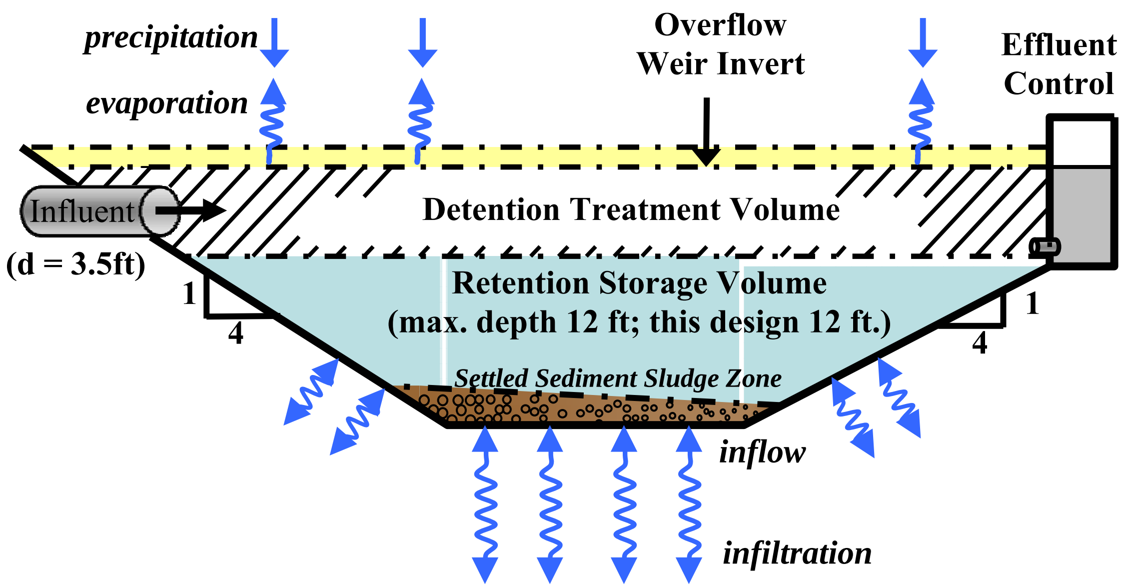

Figure 2.G.2-1 Longitudinal Section of SJRWMD Pond. *(NOT TO SCALE)*

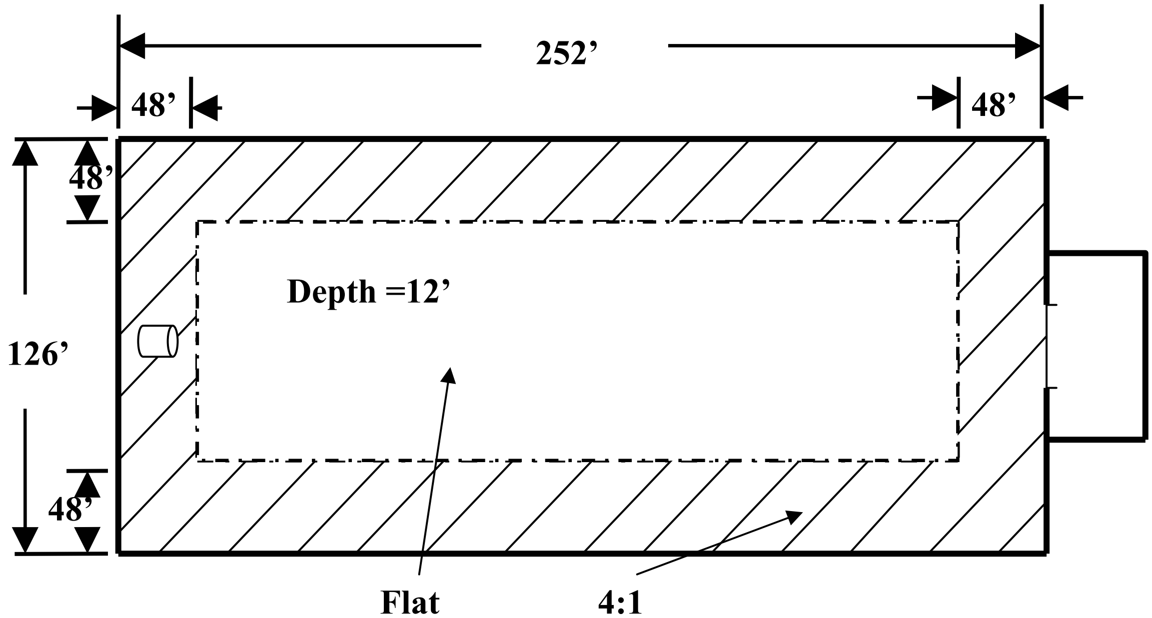

Figure 2.G.2-2 Plan View of SJRWMD Pond at the Top of the Permanent Pool Volume
(NOT TO SCALE)

2.G.3 Federal Aviation Administration (FAA) Square Pond Configuration

The FAA square pond was designed to specifications consistent with guidelines set forth in FAA letters to Florida airports and the Florida Department of Community Affairs dated November 17, 1997. A single deviation from FAA guidance was included in the design. This is in the plan dimensions, where the pond is square instead of linear. It is introduced to reflect the most severe geometries of existing water management ponds that were in existence before the circular was issued. The FAA guidance does not address water management features such as maximum depth or permanent pool volume. The pond was designed with the following features:

- ☐ 1:1 aspect ratio (length: width) at the control elevation,
- ☐ a horizontal weir with a drawdown orifice effluent control structure,
- ☐ no littoral shelf, a non-vegetated shelf 6 feet below normal water level,
- ☐ a 48 hour treatment volume drawdown rate,
- ☐ concrete lined embankments,
- ☐ a maximum depth of 12 feet, and
- ☐ a 5 acre-foot permanent pool volume.

The treatment volume was calculated using SJWMD criteria of 2.5 inches of rain multiplied by the impervious watershed area. The permanent pool volume side slopes were set at 2H:1V [horizontal:vertical] below the water level. Pond depth was set to the SJWMD maximum allowed of 12 feet below the control elevation. The embankment slopes above the water level were designed with a slope of 4H:1V and a concrete liner, with a Manning’s N of 0.056, was set from 2 feet above to 4 feet below the control elevation. The influent and effluent pipes were designed at 42-inch diameter and located along the centerline of the basin and above the control elevation. The effluent control structure consisted of an approximately 10 foot horizontal weir with a drawdown orifice at the control elevation with a diameter of slightly less than 1½ inches.

Design features are illustrated in Figures 2.G.3-1, which illustrates the longitudinal features for both FAA ponds, and 2.G.3-2. They are also summarized in Tables 2.G.5-1 through 2.G.5-3.

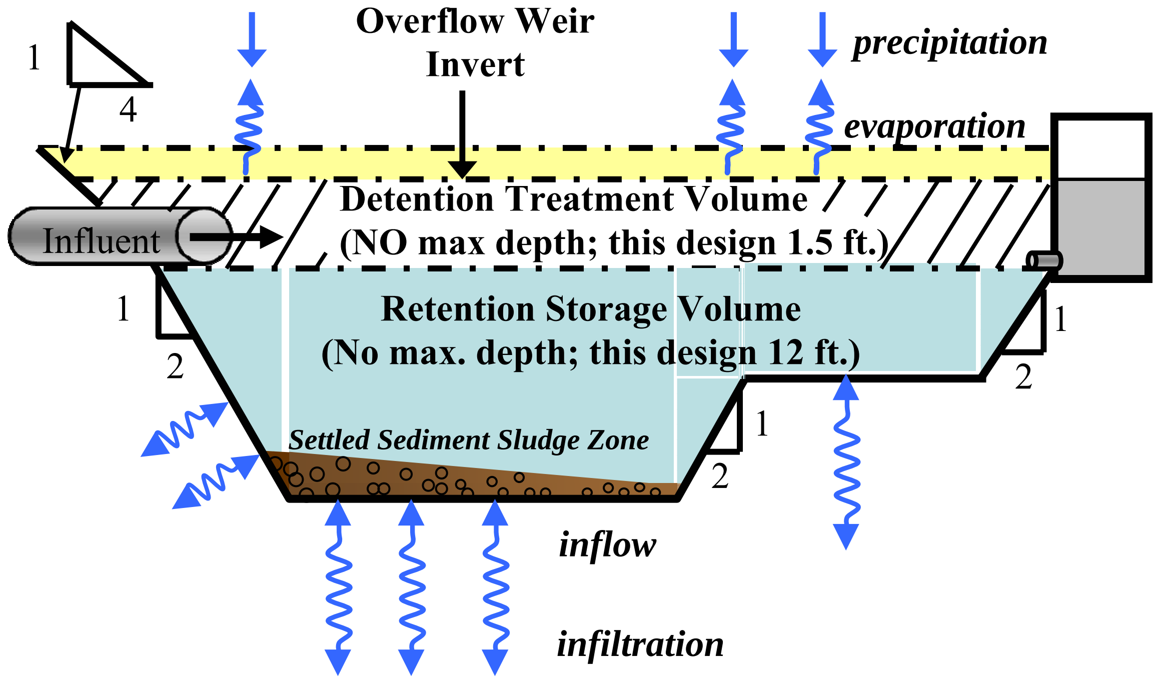

Figure 2.G.3-1 Longitudinal Section of FAA 1:1 and 10:1 Ponds *(NOT TO SCALE)*

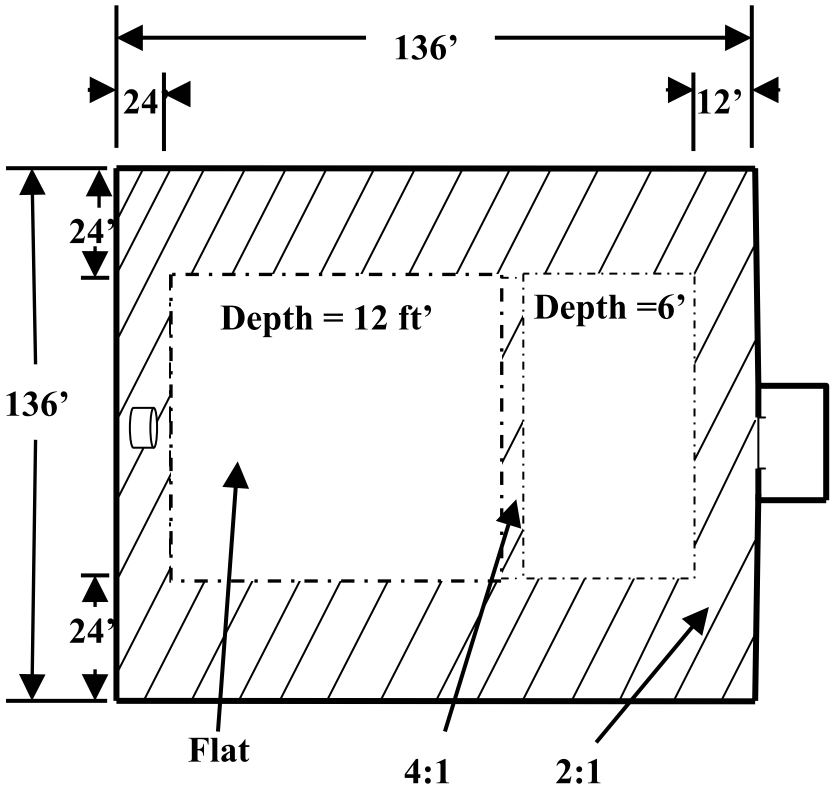

Figure 2.G.3-2 Plan View of FAA 1:1 Pond at the Top of the Permanent Pool Volume.
(NOT TO SCALE)

2.G.4 Federal Aviation Administration (FAA) Linear Pond Configuration

The FAA rectangular pond was designed to specifications consistent with guidelines set forth in FAA Advisory Circular 150/5200-33A. The pond was designed with the following considerations:

- ☐ 10:1 aspect ratio (length: width) at the control elevation,
- ☐ a horizontal weir with a drawdown orifice effluent control structure,
- ☐ no littoral shelf, a non-vegetated shelf 6 feet below normal water level,
- ☐ a 48 hour treatment volume drawdown rate, and
- ☐ a 7 acre-foot permanent pool volume.

The treatment volume was calculated using SJWMD criteria of 2.5 inches of rain multiplied by the impervious watershed area. The permanent pool volume side slopes were set at 2H:1V [horizontal:vertical] below the water level. Pond depth was set to the SJWMD maximum allowed of 12 feet below the control elevation. The embankment slopes above the water level were designed with a slope of 4H:1V and a concrete liner, with a Manning's N of 0.056, was set from 2 feet above to 4 feet below the control elevation. The influent and effluent pipes were designed at 42-inch diameter and located along the centerline of the basin and above the control elevation. The effluent control structure consisted of an approximately 10 foot horizontal weir with a drawdown orifice at the control elevation with a diameter of slightly less than 1½ inches.

Design features are illustrated in Figures 2.G.3-1, which illustrates the longitudinal features for both FAA ponds, and 2.G.4-1. They are also summarized in Tables 2.G.5-1 through 2.G.5-3.

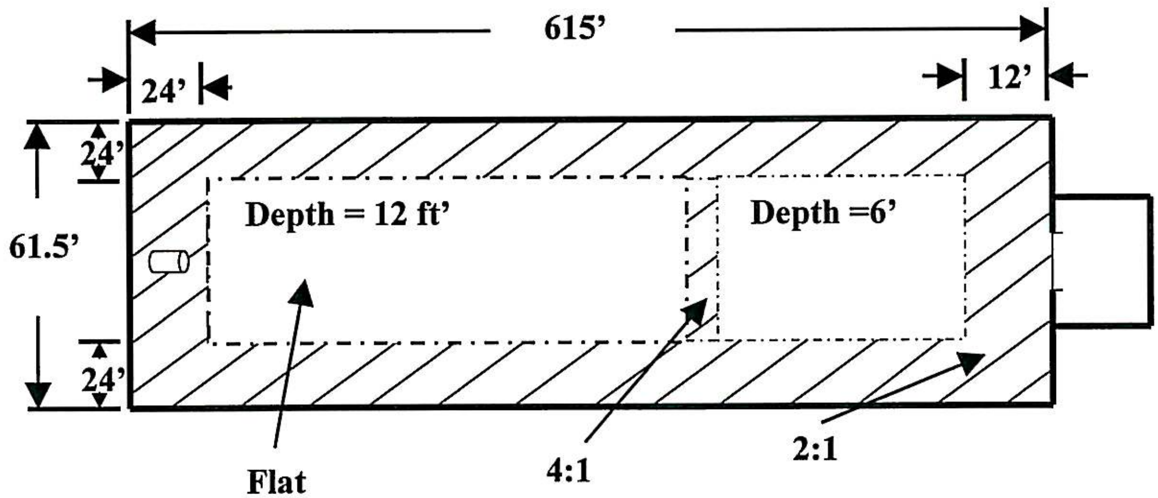

Figure 2.G.4-1 Plan View of FAA 10:1 Pond at the Top of the Permanent Pool Volume. *(NOT TO SCALE)*

2.G.5 Wet Pond Design Parameters Summary Tables

Table 2.G.5-1 Summary Design Parameters for Wet Ponds

DESIGN PARAMETERS: (*@permanent pool elevation*)	BASIN DESIGN CLASSIFICATIONS:			
	FAA	FAA	SJRWMD	SWFWMD
Length: Width Ratio	1:1	10:1	2:1	2:1
Retention storage length (feet)	136	615	252	210
Retention storage width (feet)	136	61.5	126	105
Retention storage depth (feet)	12	12	12	8
Retention storage volume (ft3)	217,800	304,920	217,800	130,680
Basin side slopes (H:V)	2:1	2:1	4:1	4:1
Littoral shelf depth (feet)	6	6	N/A	2◊1
Littoral shelf length (feet)	46	205	N/A	70
Wet season evacuation, t_{90} (days)	21	> 21	21	14

Table 2.G.5-2 Summary Design Parameters for Wet Ponds

DESIGN PARAMETERS: *(@ overflow elevation)*	BASIN DESIGN CLASSIFICATIONS:			
	FAA	FAA	SJRWMD	SWFWMD
Length:Width Ratio @ppv	1:1	10:1	2:1	2:1
Treatment volume length (feet)	184	645	311	312
Treatment volume width (feet)	184	92	185.5	207
Treatment volume depth (feet)	4.3	2.1	2.85	1.5
Detention treatment volume (ft3)	104,544	104,544	104,544	41,818
Treatment volume drawdown (hr)	48	48	48	24
Basin side slopes (H:V)	2:1	2:1	4:1	12:1

Table 2.G.5-3 Summary Design Parameters for Wet Ponds

HYDRAULIC APPURTENANCE DESIGN PARAMETERS:	BASIN DESIGN CLASSIFICATIONS:			
	FAA	FAA	SJRWMD	SWFWMD
Length:Width Ratio @ppv	1:1	10:1	2:1	2:1
Inflow pipe diameter (ft.)	3.5	3.5	3.5	3.5
Detention outflow structure	Weir	Weir	Weir	Weir
Weir (broad-crested) length (ft.)	9.8	9.8	9.8	9.8
Detention drawdown structure	Orifice	Orifice	Orifice	Orifice
Orifice diameter (ft.)	0.12	0.12	0.12	0.12
Outflow pipe diameter (ft.)	3.5	3.5	3.5	3.5

2.G.6 Wet Pond Model Rainfall and Peak Inflow

Wet pond simulations are event based as discussed in Section 1.A.2. Two inflow conditions from the apron were simulated for various wet pond designs. These are based on Intensity-Duration-Frequency (IDF) information for the 25-year event as defined by the Florida Department of Transportation (FDOT), and a median peak flow for real rainfall events calculated based on measured data from nine of the Statewide Airport Stormwater Study airports.

FDOT IDF curves for Zone 10 were selected for defining one wet pond event since they have the highest rainfall depth for a 25-year, 24-hour storm in Florida. The curve is reproduced as Figure 2.G.6-1 following.

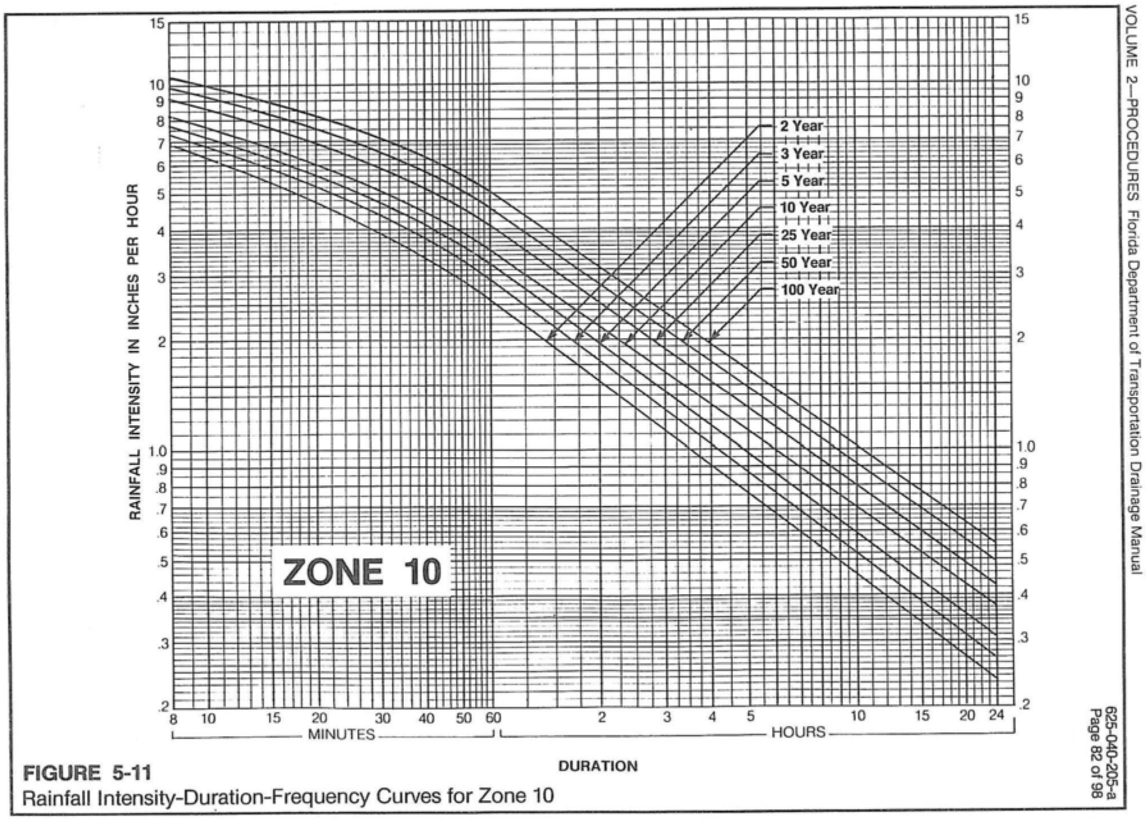

Figure 2.G.6-1 FDOT Intensity Duration Frequency Curve Used for Computational Fluid Dynamics Model

The 25-year, 24-hour intensity from this curve, 0.43 inches per hour was applied to the 11.38 acre apron included in the study. Apron properties for defining the event are:

- impervious concrete surface
- flow length = 1,000 feet
- apron slope = 0.5%
- Manning's n = 0.012
- depression storage = 0.05 inches

The time of concentration (T_C) and peak flow rate (Qp) were determined using an iterative process until the modeled T_C equaled the duration of the design storm for the intensity specified. Forty-five minutes was established as the time of concentration using this method. The correlating peak flow for the Tc of 45 minutes was 60.14 cubic feet per second (cfs). This was specified as the Qp for the design storm to be used in the pond comparison analyses.

Rainfall time-series from nine airports were used, each in one of ten simulations, to determine a median runoff for all the sites combined. The specific records used are listed in Table 2.A.3-1 and may be located using the domestic identifier on Figure 2.A.1-1.

Table 2.G.6-1 Airport Data Used for Wet Pond Median Peak Flow

AIRPORT NAME	DOMESTIC IDENTIFIER
Charlotte County Airport	PGD
Daytona Beach International Airport	DAB
Fort Lauderdale – Hollywood International Airport	FLL
Lake City Municipal Airport	LCQ
Orlando International Airport	MCO
Pompano Beach Airpark	PMP
Tallahassee Regional Airport	TLH
Tampa International Airport	TPA
Venice Municipal Airport	VNC

The same methodology was used to determine the time of concentration as described for the FDOT IDF simulation. This time is also a surrogate for inter event time and was used in the SWMM analysis tool to separate the runoff into discrete events, each at least 45 minutes apart from the nearest event. The peak flows for all events at all sites were tabulated and the median Qp for the entire simulation was then selected as the Qp_{50}. This value was determined to be 3.8 cfs.

2.G.7 Wet Pond Pollutant Inflow Loadings

The influent particulate loading used for all of the CFD based wet-pond simulation runs consisted of a sandy silt particle size gradation. The particle size gradation, ranging from 10 to 100 micrometers (μm), can be seen in Figure 2.G.7-1. The particulate specific gravity was established as 2.56. This is lower than metals, about the same as soil solids and higher than organic debris and represents a reasonable surrogate for the combined pollutant inflow.

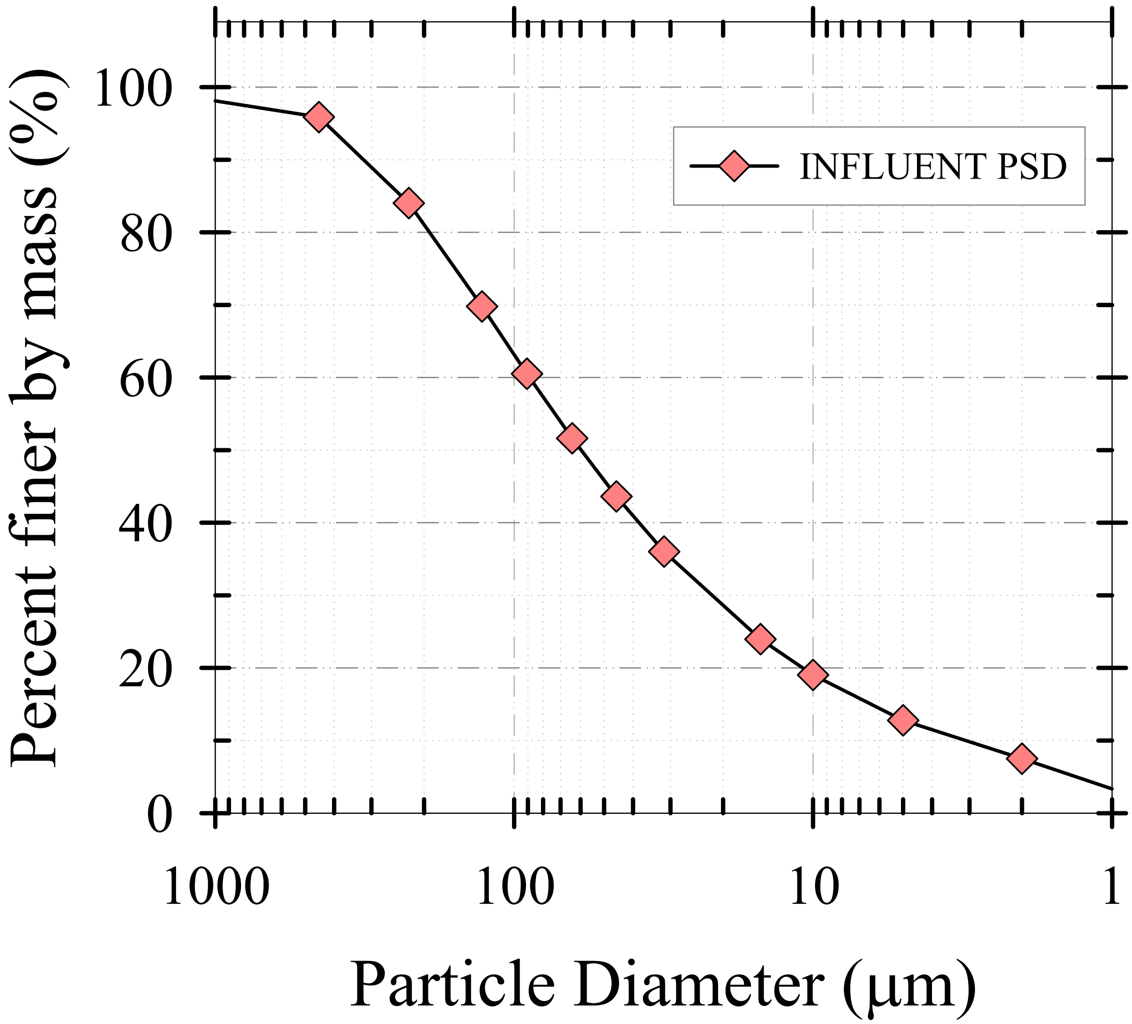

Figure 2.G.7-1 Influent Particulate Loading for All Wet Ponds in the Comparative Study [concentration = 100 (mg/l)]

3. SIMULATION RESULTS

3.A Dry Site Conditions

3.A.1 Pristine Site

3.A.1.a Hydrology

Type A soil has a low runoff potential by definition, even if thoroughly wetted. Results for the continuous simulation of historic 19 months rainfall, a 25-year, 24-hour design storm embedded into the historic 19 months of rainfall data and a single event of 25-year, 24-hour design storm indicate no runoff occurs from the pristine site. Table 3.A.1.a-1 provides a water balance summary.

Table 3.A.1.a – 1 Pristine Site Hydrology For Dry Site (Type A Soils) *

	SITE RAINFALL SETS		
	HISTORIC	HISTORIC W/DESIGN	DESIGN
PRECIPITATION (inches)	63.1	71.6	8.5
EVAPORATION (inches)	0.8	0.9	0.1
INFILTRATION (inches)	62.3	70.7	8.4
RUNOFF (inches)	0.0	0.0	0.0

* See Figures in Appendix B for additional water balance details.

3.A.1.b Runoff Water Quality

In the absence of runoff, the Type A soil condition will not result in a pollutant loading into the adjacent waterbody under pristine conditions. Likewise, without runoff there is no associated Event Mean Concentration.

3.A.2 Developed Site

3.A.2.a Hydrology

Utilizing soil data for the Type A soil conditions and water table elevations six feet below ground surfaces, Orlando International Airport's (MCO) historic 19 months of continuous rainfall simulation shows no runoff for the model GA airport's airside constituents. Embedding a 25-year, 24-hour design storm into this historical rainfall data yields a runoff of 0.01 inches from a total of 71.6 inches of rainfall. Simulating the single, 25-year, 24-hour storm also yields a runoff of 0.01 inches from a total of 8.5 inches of rainfall. Table 3.A.2.a-1 summarizes the developed site hydrology.

Table 3.A.1.a – 2 Developed Site Hydrology for Dry Site (Type A Soils) *

	SITE RAINFALL SETS		
	HISTORIC	HISTORIC W/DESIGN	DESIGN
PRECIPITATION (inches)	63.1	71.6	8.5
EVAPORATION (inches)	1.7	1.8	0.1
INFILTRATION (inches)	61.7	70.2	8.4
RUNOFF (inches)	0.0	0.01	0.01

* See Figures in Appendix B for additional water balance details.

3.A.2.b Runoff Water Quality

The simulation results for the water quality constituents of the continuous simulation of historic 19 months rainfall show no pollutant loading into the outfall. EMC values become non-applicable in this case. When the 25-year, 24-hour design storm is considered, the trace runoff yields trace pollutant EMC values consistent with the 0.01 inches runoff. The results are summarized in Table 3.A.2.b-1 following. These are lower concentrations than those furnished for a pristine site.

The explanation for this is contained in the quantity and distribution of pollutants in the airside runoff measured during the Stormwater Study and presented in Section 2.C Pollutagraphs When rainfall is sufficient to produce runoff from the entire site, the majority of pavement water quality pollutants have infiltrated. The runoff from the pavement, representing the highest runoff volume for these site conditions, is at this point considerably cleaner than the pristine site EMC. In effect, it dilutes the EMC in water leaving the airport. Note, however, that Pristine Site EMC values are for those sites generating runoff, and are provided for comparison purposes only.

Table 3.A.2.b-1 Developed Site Runoff Water Quality for Dry Site (Type A Soils)

	EMC					
	TSS [mg/L]	TP [mg/L]	TN [mg/L]	Cu [mg/L]	Pb [mg/L]	Zn [mg/L]
	Historical Rainfall					
Pristine Site	0	0	0	0	0	0
Developed Airside	0	0	0	0	0	0
	Historical Rainfall + 25-year, 24-hour Design Storm					
Pristine Site	7.8	0.074	1.15	0.001	0.005	0.006
Developed Airside	0.141	0.001	0.02	~0	~0	~0

3.A.3 Dry Site Summary

Due to the significant pervious areas required by the FAA for airside operations, there is effectively no stormwater runoff for the developed airside with Type A soil conditions. Overland flow in the airside safety and object free areas to infiltrate stormwater runoff and significantly reduce runoff is the best management practice for dry sites of this type. The model results validate the recommendations made in the *Florida Airports Stormwater Best Management Practices Manual* for runway and taxiway stormwater management for dry site conditions. This BMP recommendation may also apply to aprons depending if the specific site geometry conditions are satisfied.

3.B Intermediate Sites Conditions

3.B.1 Pristine Sites

3.B.1.a Hydrology

Table 3.B.1.a-1 provides a water balance summary for the Pristine Intermediate Site for Type B Soils. Table 3.B.1.a-2 provides a water balance summary for the Pristine Intermediate Site for Type C Soils.

Table 3.B.1.a – 1 Pristine Site Hydrology for Intermediate Site (Type B Soils) *

	SITE RAINFALL SETS		
	HISTORIC	HISTORIC W/DESIGN	DESIGN
PRECIPITATION (inches)	63.1	71.6	8.5
EVAPORATION (inches)	0.8	2.3	0.2
INFILTRATION (inches)	62.3	67.7	7.9
RUNOFF (inches)	0.0	1.6	0.4

* See Figures in Appendix C for additional water balance details

Table 3.B.1.a – 2 Pristine Site Hydrology for Intermediate Site (Type C Soils) *

	SITE RAINFALL SETS		
	HISTORIC	HISTORIC W/DESIGN	DESIGN
PRECIPITATION (inches)	62.5	71.0	8.5
EVAPORATION (inches)	1.1	1.2	0.2
INFILTRATION (inches)	61.4	68.1	6.7
RUNOFF (inches)	~0.0	1.6	1.6

* See Figures in Appendix D for additional water balance details

In the historic rainfall analysis for pristine conditions, C soils demonstrate a higher ET from the surface due to increased ponding on the surface as compared to a B soil. This does not occur for the historical with design event simulations because the C soil has more flow transpiring from the unsaturated zone, creating more potential infiltration and pulling water into the soil matrix from the surface. Deep percolation is higher for the pristine historic time series simulations than the historic event with the design storm because of the relatively short duration of the design storm and subsequent surface water routing resulting in fewer dry time steps. Groundwater infiltration is higher for the C soil historic analysis when compared to the B soil because the horizontal conductivity of the aquifer is much higher in the B soil simulation than in the C soil simulation. In this case, water does not enter from the unsaturated zone to the saturated zone, which is represented here as groundwater infiltration. Note for pristine conditions that precipitation varies slightly between B and C soil condition simulations in-part due to minor variability in simulation time steps.

In both the B and C soil cases, the runoff in the pristine condition is about 1.6 inches when the design storm is applied. This result is considerably less than typically held concepts of runoff for sites with these soils. It is, however, consistent with established flood management practice in several state water control districts. These limit runoff in 24-hour, 25-year storms to 1-inch in 24 hours based on historic precedent. The practice has been found to limit flooding to historic values. The simulation results provide support to the practice, provided runoff is estimated based on physically measured soil parameters and Green-Ampt infiltration analyses.

3.B.1.b Runoff Water Quality

When runoff occurs on the pristine site, the water quality is described by the Event Mean Concentration data given in Table 2.B-1 for the pristine site. The information is also shown in Tables 3.B.2.b-1 and 3.B.2.b-2 for comparison with post-development conditions.

3.B.2 Developed Site

3.B.2.a Hydrology

Table 3.B.2.a-1 provides a water balance summary for the Developed Intermediate Site for Type B Soils. Table 3.B.2.a-2 provides a water balance summary for the Developed Intermediate Site for Type C Soils.

Table 3.B.2.a – 1 Developed Site Hydrology for Intermediate Site (Type B Soils) *

	SITE RAINFALL SETS		
	HISTORIC	HISTORIC W/DESIGN	DESIGN
PRECIPITATION (inches)	63.1	71.6	8.5
EVAPORATION (inches)	2.0	2.1	0.2
INFILTRATION (inches)	61.1	67.1	6.0
RUNOFF (inches)	0.5	2.9	2.3

* See Figures in Appendix C for additional water balance details

Table 3.B.2.a – 2 Developed Site Hydrology for Intermediate Site (Type C Soils)

	SITE RAINFALL SETS		
	HISTORIC	HISTORIC W/DESIGN	DESIGN
PRECIPITATION (inches)	63.1	71.6	8.5
EVAPORATION (inches)	2.3	2.4	0.2
INFILTRATION (inches)	59.4	64.1	4.7
RUNOFF (inches)	2.1	5.8	3.6

* See Figures in Appendix D for additional water balance details

Under developed conditions, the historic simulation surface ET does not differ as much between B and C conditions, but ET is higher than that of pristine conditions. This is because roughly the same volume of ponding occurs in the historical analyses. The same amount of volume will thus evaporate. The unsaturated and saturated zone evapotranspiration rates are higher in the C soil condition because the water table reaches into the lower evaporative depth more often and because the upper evaporative fraction is higher for a C soil than a B soil. The water table is driven higher because less unsaturated flow proceeds to the culverts rather than vertically, raising the aquifer. The higher unsaturated and saturated zone ET values for the C condition simulations may be a result of the water table which began much higher in this case, again allowing for more depth in the zone where ET is acting. Unsaturated and saturated zone ET values are lower when compared to pristine conditions because water is drawn away from the surface to the long trench network in developed conditions, and is a result of positive drainage conditions. The historic simulation exfiltration rate is almost three times higher under type C soil conditions, as compared to B soils, which is also due to the much higher elevation of the initial groundwater table in the C-soil simulation.

Post-development runoff is higher for both the B and C soil conditions. This is more pronounced for the C condition, as expected. The values are still lower than typically held concepts for developed sites. However, they are very consistent with the field measured data for airside areas for the given soil and groundwater conditions. The information presented in Section 1B for Sarasota Bradenton International Airport is for B/C soil condition with similar groundwater elevations.

Note that the impervious area on a typical airside represents less than 15 % of the total airside area. Also, this impervious area is distributed in a linear geometric manner that allows infiltration into the pervious airside vegetated soils. The simulation results are consistent with this condition and physical reality.

3.B.2.b Runoff Water Quality

Developed airside EMC values for TSS, TP, and TN are lower than pristine conditions for B and C soils under the historical rainfall simulations. Total metals (sum of dissolved and particulate fractions for Cu, Pb and Zn) do not change significantly. The outfall EMC's are higher for the historical analysis than the design event analysis under C soil conditions whereas the opposite is case under B soil conditions.

Table 3.B.2.b-1 Comparison of Developed Site Runoff Water Quality for Intermediate Site (Type B Soils) with Pristine Site Quality

	EMC					
	TSS	TP	TN	Cu	Pb	Zn
	[mg/L]	[mg/L]	[mg/L]	[mg/L]	[mg/L]	[mg/L]
	Historical Rainfall					
Pristine Site	7.8	0.074	1.15	0.001	0.005	0.006
Developed Airside	1.1	0.010	0.11	~0.001	~0.001	0002
	25-year, 24-hour Design Storm					
Pristine Site	7.8	0.074	1.15	0.001	0.005	0.006
Developed Airside	2.0	0.025	0.37	0.001	0.002	0.002

Table 3.B.2.b-2 Comparison of Developed Site Runoff Water Quality for Intermediate Site (Type C Soils) with Pristine Site Quality

	EMC					
	TSS	TP	TN	Cu	Pb	Zn
	[mg/L]	[mg/L]	[mg/L]	[mg/L]	[mg/L]	[mg/L]
	Historical Rainfall					
Pristine Site	7.8	0.074	1.15	0.001	0.005	0.006
Developed Airside	1.0	0.009	0.28	~0.001	~0.001	0.002
	25-year, 24-hour Design Storm					
Pristine Site	7.8	0.074	1.15	0.001	0.005	0.006
Developed Airside	1.0	0.008	0.12	~ 0	~0.001	~0.001

Similar to the dry site (Type A Soils) condition, the explanation for this is contained in the quantity and distribution of pollutants in the airside runoff measured during the Stormwater Study and presented in Section 2.C Pollutagraphs. The vast majority of rain events do not exceed either the intensity or volume infiltration characteristics of either the Type B or Type C soils. When rainfall is sufficient or sufficiently intense to produce runoff from the entire site, the majority of pavement water quality pollutants have infiltrated. The runoff from the pavement, representing the highest runoff volume for these site conditions, is at this point cleaner than the pristine site EMC. In effect, it dilutes the EMC in water leaving the airport for these conditions.

3.B.3 Intermediate Sites Summary

The water quality for intermediate sites is satisfactorily managed using overland flow in the safety and object free areas. These allow stormwater runoff from the vast majority of storms to infiltrate. Even those which do not fully infiltrate benefit from concentration changes caused by fractional infiltration. The simulation results validate the recommendations made in the *Florida Airports Stormwater Best Management Practices Manual* for runway and taxiway stormwater management for intermediate site conditions for Hydrologic Group B and C soils. This BMP recommendation may also apply to aprons depending strictly on use of overland flow as opposed to direct collection of runoff from paved areas.

3.C Wet Site Conditions

3.C.1 Pristine Site

3.C.1.a Hydrology

The wet site type D soil groups have a high runoff potential due to soil texture, groundwater elevation or a combination of these. Simulations indicate about 7.1 inches of runoff occurs from the pristine site for a continuous simulation with design storm superimposed. Table 3.C.1.a-1 provides a water balance summary.

Table 3.C.1.a – 1 Pristine Site Hydrology for Wet Site (Type D Soils) *

	SITE RAINFALL SETS		
	HISTORIC	HISTORIC W/DESIGN	DESIGN
PRECIPITATION (inches)	63.1	71.6	8.5
EVAPORATION (inches)	5.5	5.5	0.3
INFILTRATION (inches)	55.3	59.0	2.7
RUNOFF (inches)	2.3	7.1	5.5

* See Figures in Appendix E for additional water balance detail

3.A.1.b Runoff Water Quality

Runoff water quality from the pristine site is defined by the Event Mean Concentration data given in Table 2.B-1 for the pristine site. The information is also shown in Table 3.C.2.b-1 for comparison with post-development conditions.

3.C.2 Developed Site

3.C.2.a Hydrology

The developed wet site has more than twice the runoff of the pristine wet site, with nearly 25½ inches predicted by the continuous simulation with design storm superimposed. This is consistent with the rainfall distributions and the limited infiltration capacity for added runoff from pavement areas.

Table 3.C.2.a – 1 Developed Site Hydrology for Wet Site (Type D Soils) *

	SITE RAINFALL SETS		
	HISTORIC	HISTORIC W/DESIGN	DESIGN
PRECIPITATION (inches)	63.1	71.6	8.5
EVAPORATION (inches)	2.9	3.0	0.3
INFILTRATION (inches)	40.9	43.0	2.7
RUNOFF (inches)	19.3	25.6	5.5

* See Figures in Appendix E for additional water balance detail

3.C.2.b Runoff Water Quality

The increased runoff in the developed condition is of lower quality than the runoff in the pristine condition. Unlike the dry and intermediate cases, insufficient runoff infiltrates to create any diluting effect. The system is therefore additive for most event producing runoff. That is, pollutants from the pavement increase the EMC already present in runoff from the site.

Table 3.C.2.b-1 Comparison of Developed Site Runoff Water Quality for Wet Site (Type D Soils) with Pristine Site Quality

	EMC					
	TSS	TP	TN	Cu	Pb	Zn
	[mg/L]	[mg/L]	[mg/L]	[mg/L]	[mg/L]	[mg/L]
	Historical Rainfall					
Pristine Site	7.8	0.074	1.15	0.001	.005	0.006
Developed Airside	2.3	0.013	0.14	~0.000	0.001	0.002
	25-year, 24-hour Design Storm					
Pristine Site	7.8	0.074	1.15	0.001	.005	0.006
Developed Airside	20.3	0.142	1.67	0.018	0.010	0.045

3.C.3 Wet Site Summary

Wet sites represent a limiting condition where overland flow will not work for water quality management as a stand alone approach. The limiting condition is encountered when the water table is within 18 inches of the ground surface <u>and</u> hydraulic conductivity is 0.1 inches per hour or less. Some group D sites will not have both conditions, and some level of water quality management using overland flow will occur on those sites. The model results limit the recommendations made in the *Florida Airports Stormwater Best Management Practices Manual* to Group A, Group B and Group C hydrologic soil group conditions. D soils will require site specific testing before considering overland flow. A special case is where an existing D site will be filled with clean sands to elevate is above the water table. Such a condition will likely result in substantial, post development water quality improvement over existing conditions.

3.D Wet Pond Simulation Results

The wet pond simulation results are graphically presented in a series of figures following. The first series of figures are arranged by pond type and presented in order as:

- ☐ SWFWMD 14 day pond
- ☐ SJRWMMD 21 day pond
- ☐ FAA square pond
- ☐ FAA linear pond

Each of the first series of figures includes CFD outputs for two conditions for each pond. First, flows at Q_{p50} representing the average condition. Second, flows during a 25-year design rain event.

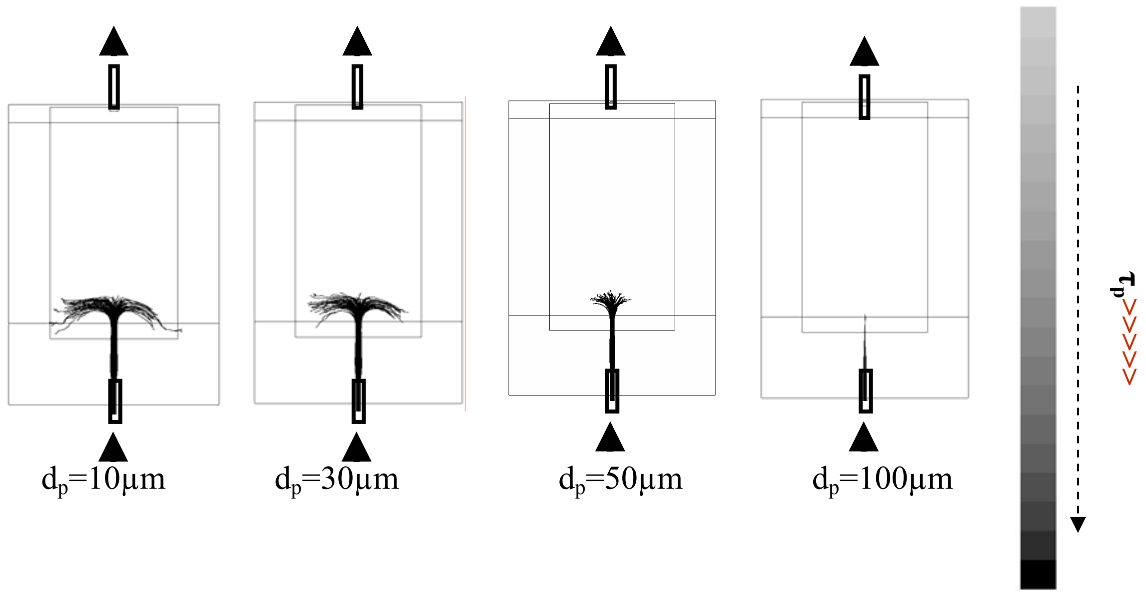

Figure 3.D-1a SWFWMD 14 day wet-pond particulate tracking for an influent flow rate of 3.8 cfs (Qp_{50} of median storm) and an influent concentration of 100 [mg/l]

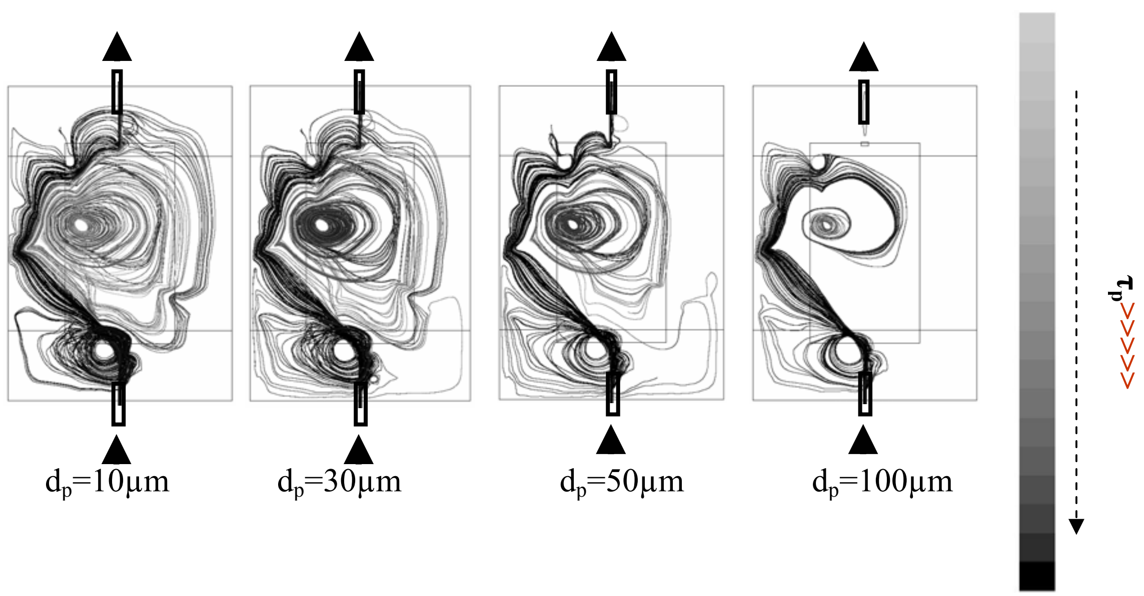

Figure 3.D-1b SWFWMD 14 day wet-pond particulate tracking for an influent flow rate of 60 cfs (25 year design storm peak flow) and an influent concentration of 100 [mg/l]

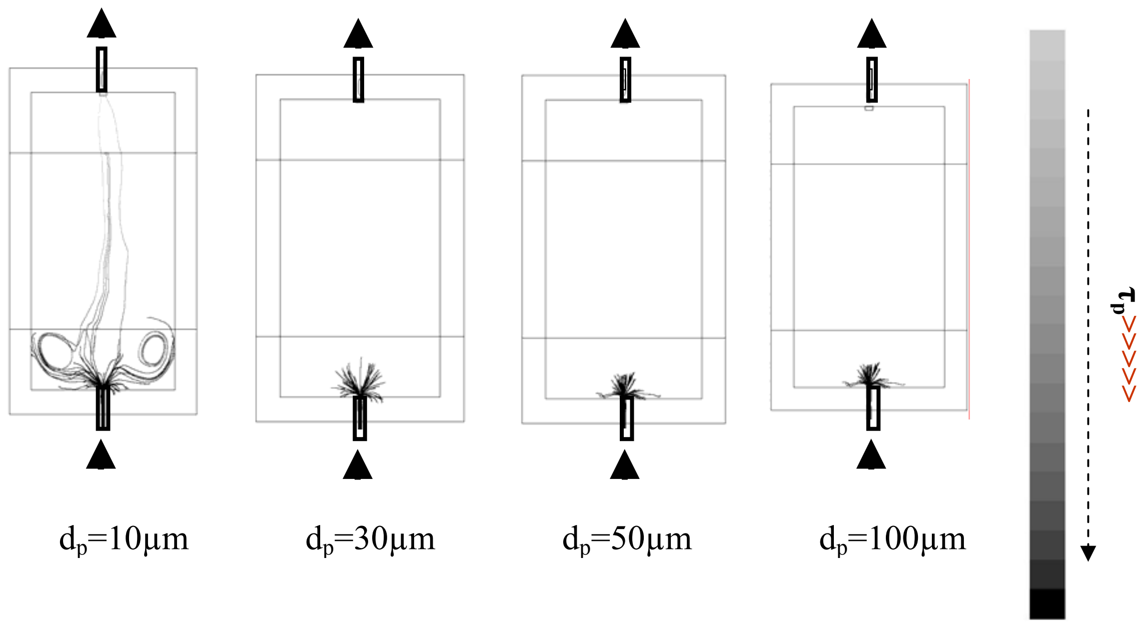

Figure 3D-2a SJRWMD 21 day wet-pond particulate tracking for an influent flow rate of 3.8 cfs (Qp_{50} of median storm) and an influent concentration of 100 [mg/l]

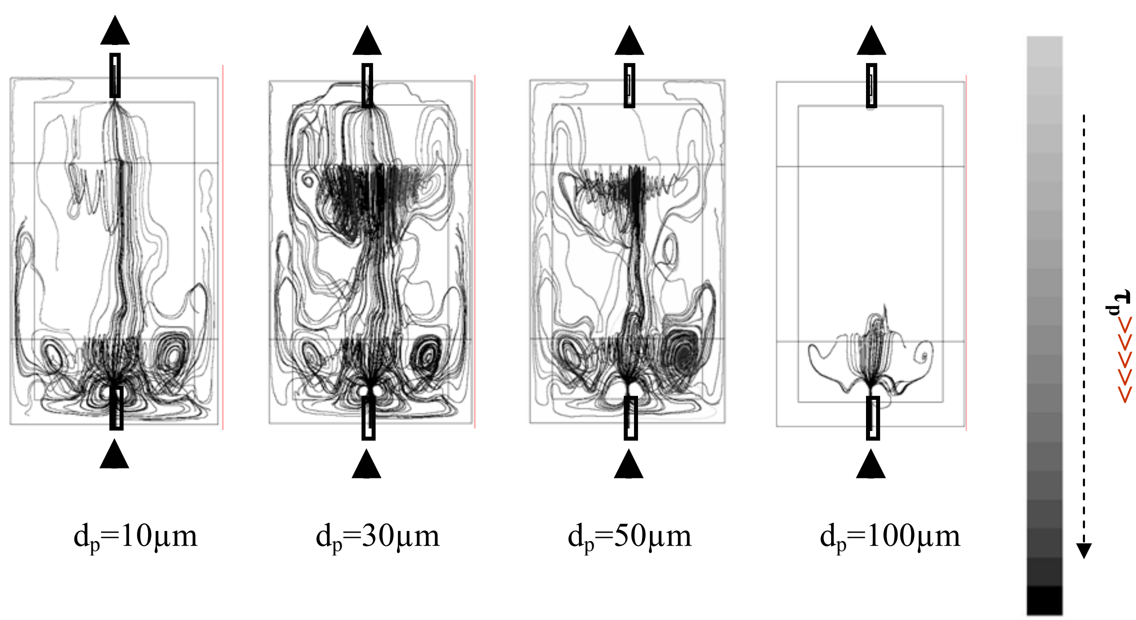

Figure 3D-2b SJRWMD 21 day wet-pond particulate tracking for an influent flow rate of 60 cfs (25 year design storm peak flow) and an influent concentration of 100 [mg/l]

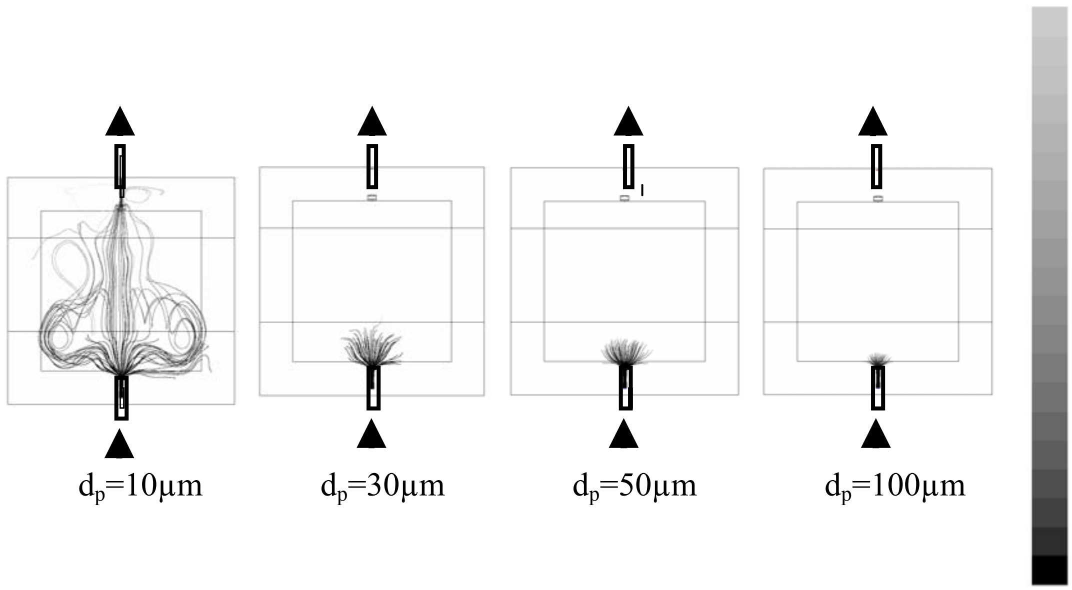

Figure 3D-2a: FAA 1:1 wet-pond particulate tracking for an influent flow rate of 3.8 cfs (Qp_{50} of median storm) and an influent concentration of 100 [mg/l]

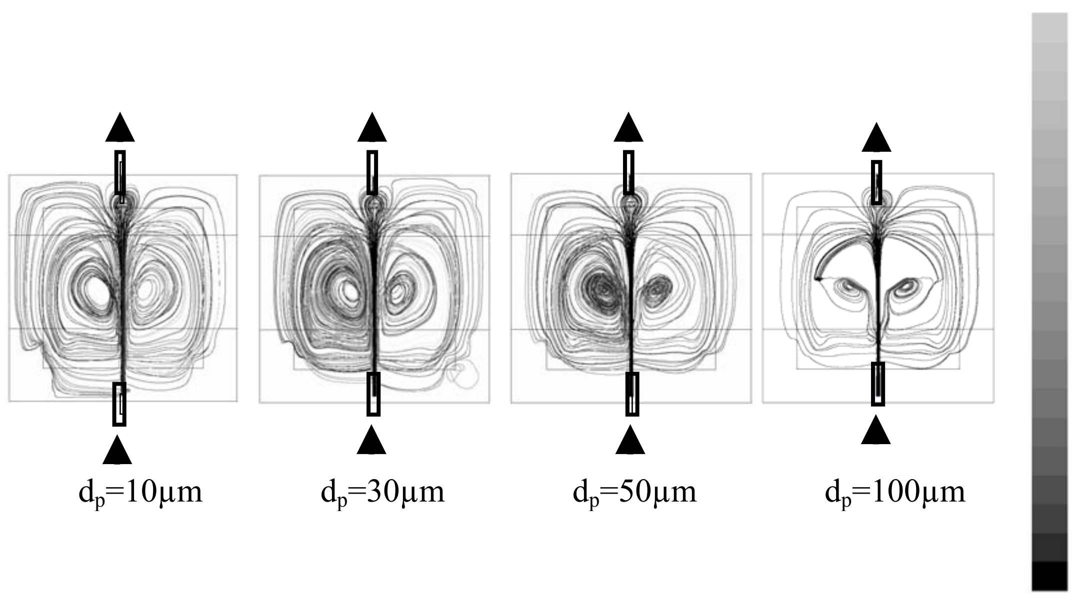

Figure 3D-2b: FAA 1:1 wet-pond particulate tracking for an influent flow rate of 60 cfs (25 year design storm peak flow) and an influent concentration of 100 [mg/l]

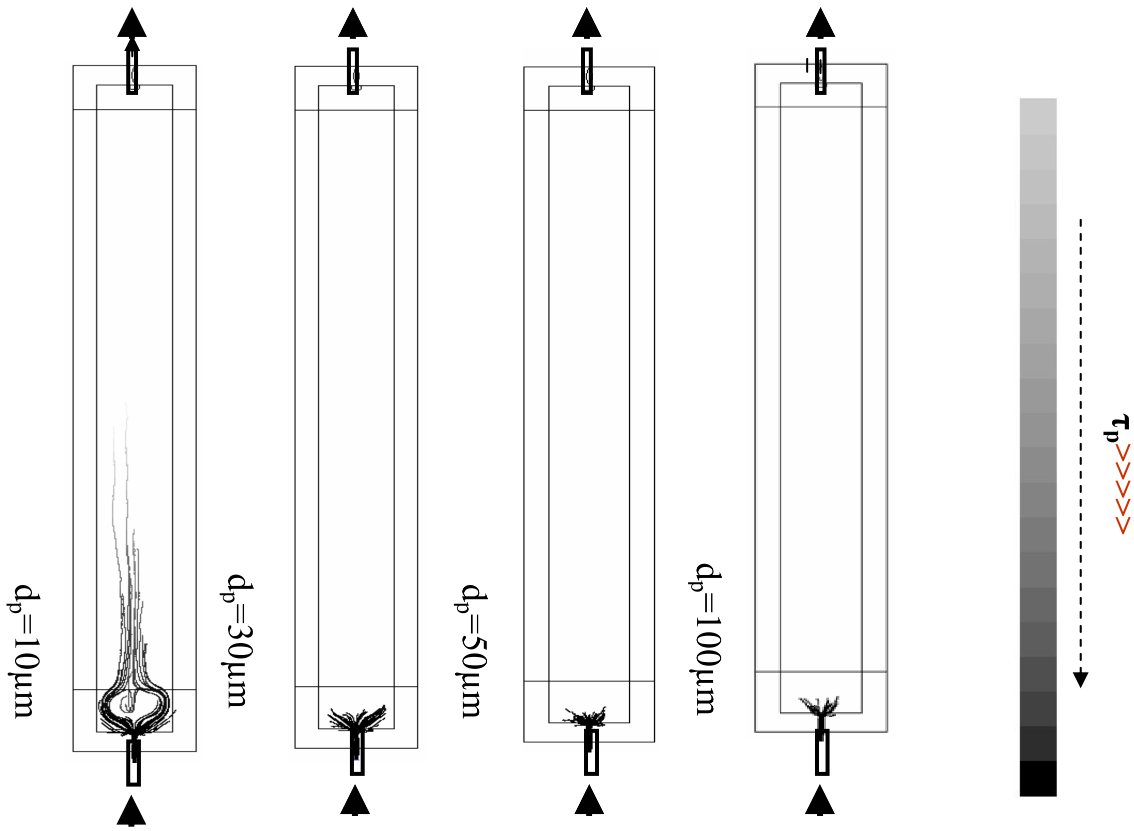

Figure 3D-4a FAA 10:1 wet-pond particulate tracking for an influent flow rate of 3.8 cfs (Qp_{50} of median storm) and an influent concentration of 100 [mg/l]

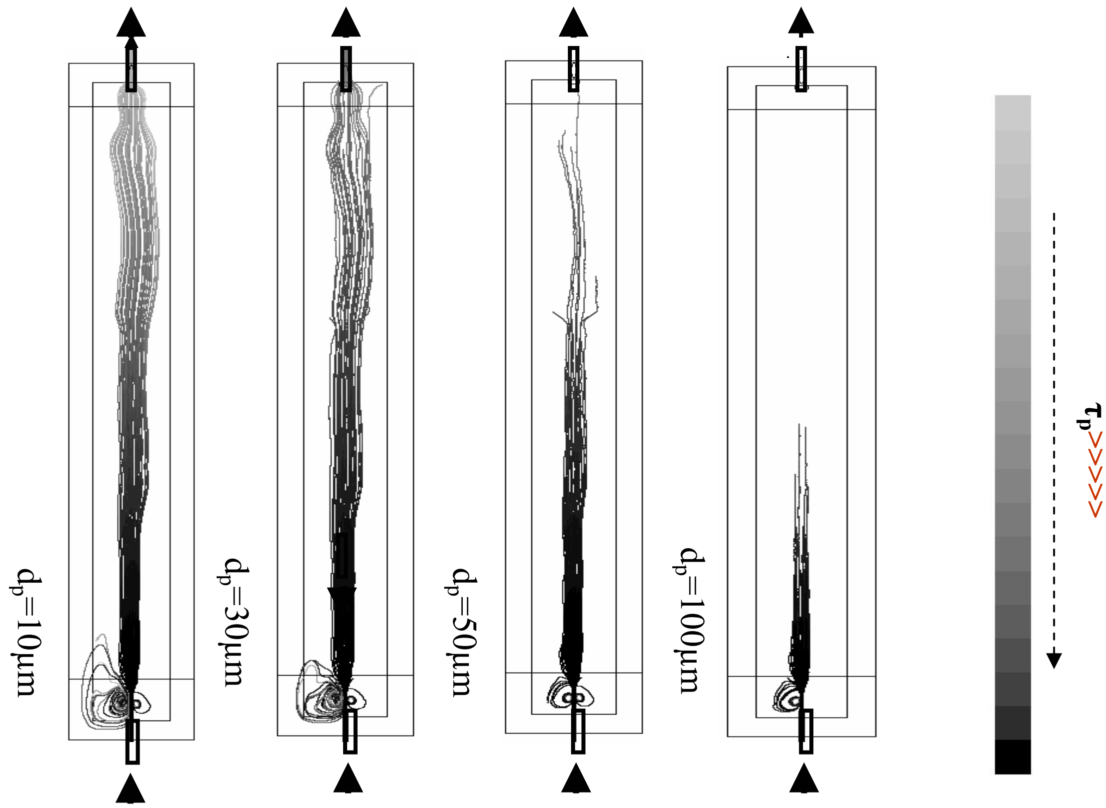

Figure 3D-4b FAA 10:1 wet-pond particulate tracking for an influent flow rate of 60 cfs (25 year design storm peak flow) and an influent concentration of 100 [mg/l]

The next series of figures illustrate the comparative removal efficiencies of the wet ponds modeled. All ponds have very good or excellent removal efficiencies for the median inflows expected each year. This is illustrated in Figure 3.D-5 following. However, efficiencies change substantially during the higher flow design events. This is illustrated in Figure 3.D-6 following.

Factors that influence the performance of wet pond particulate removal efficiency include the permanent pool volume, the permanent pool depth, the length to width aspect ratio, the effluent control structure, baffles, the longitudinal profile of the basin, the volumetric and particulate loading rate, and relative vegetation. Only the SWFWMD pond included the vegetated littoral shelf, which adds some removal capability, but is less effective than permanent pool, additional depth and increased linearity of the system. In order of predicted pollutant removal performance, from best to worst, the CFD simulations indicate:

1. FAA Linear Pond
2. SJRWMD 21 Day Pond
3. SWFWMD 14 Day Pond
4. FAA Square Pond

Particulate matter (PM) mass , in kilograms, removed by each of the wetponds necessitated the use of an influent volume equal to the total runoff treated. The treated volumes were set to be equal to the total permanent pool volume of each basin. The influent concentration for the PM loads was 100 [mg/l]. In so doing the influent mass loads for the FAA 1:1, FAA 10:1, SWFWMD, and SJRWMD were 781, 1130, 690 and 1450 kg respectively.

QP_{50} - QP_{50}:

For an influent flow rate of 3.8 cfs the SWFWMD, SJRWMD, and FAA 10:1 pond models demonstrated a very high treatment efficiency of all particles sizes down to 10 μm. The FAA 1:1 pond, however, generated effluent concentrations of 12.1 [mg/L] with particle sizes less than 31 μm. The results are illustrated in Figure 3D- 5 and Figure 3D- 8. The pond models resulted in load reductions of 681 kg (FAA 1:1), 1130 (FAA 10:1), 690 (SWFWMD) and 1450 kg (SJRWMD), as shown in Figure 3D- 8. While effluent concentrations are comparable since a concentration is a mass that is normalized per unit of aqueous volume, load reductions are a function of the specific basin design as well as the permanent pool volume.

Design Storm- Design Storm

For an influent flow rate of 60 cfs from the apron to the each wet pond, the FAA 10:1 wet pond model illustrated the greatest reduction in particulate concentration from 100 to 7.31 [mg/L] as summarized in Figure 3D- 7. The SJRWMD, SWFWMD, and FAA 1:1 wet pond models demonstrated effluent concentrations of 14.9, 38.7 and 55.1 [mg/L], respectively. The particulate matter removal characteristics corresponded to the concentration trends in that no particle sizes greater than 15 μm are eluted from the FAA 10:1 effluent, while the FAA 1:1 eluted particle sizes as large as 225 μm as shown in Figure 3D- 6. Figure 3D-7 illustrates load efficiency.

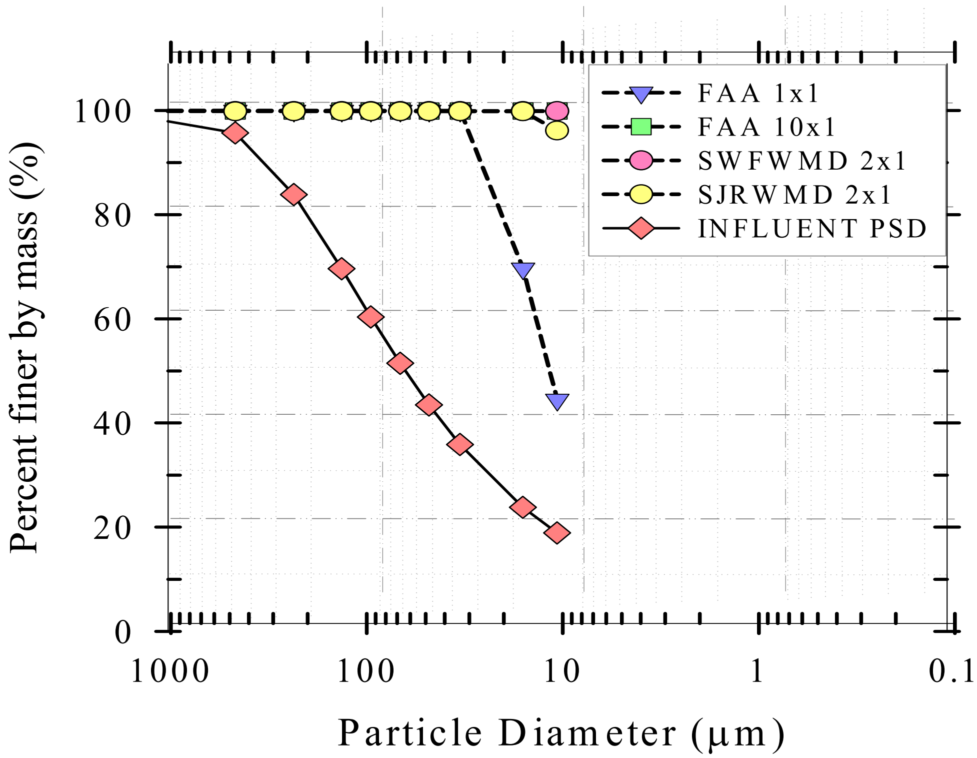

Figure 3D-5 Influent and effluent particulate loading for all of the wet-ponds in the comparative study flow rate 3.8 cfs (influent conc. = 100 [mg/l])

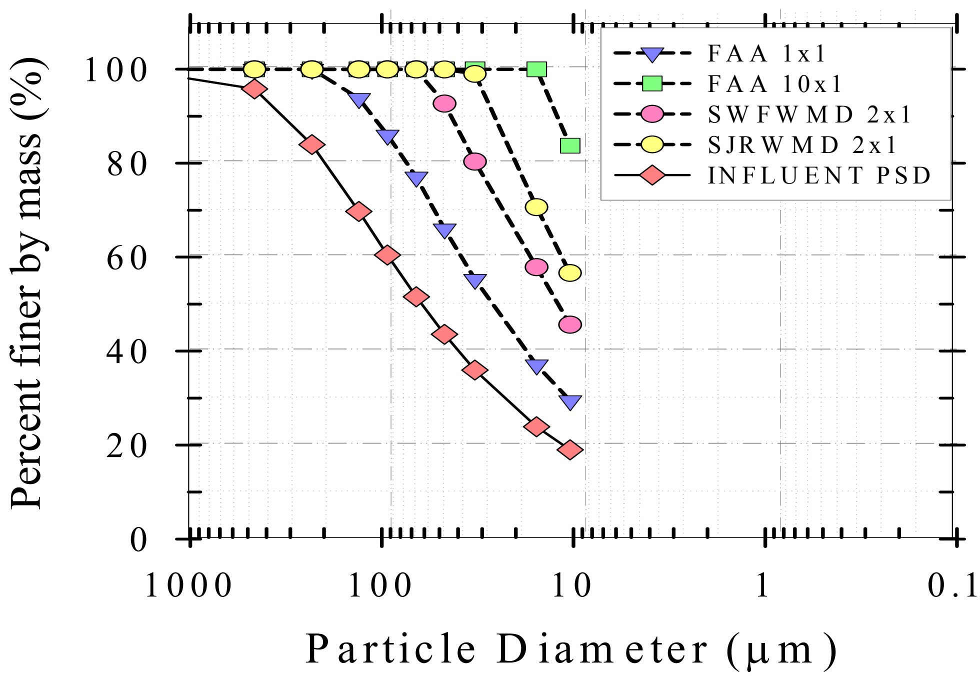

Figure 3D-6 Influent and effluent particulate loading for all of the wet-ponds in the comparative study flow rate 60 cfs (influent concentrations = 100 [mg/l])

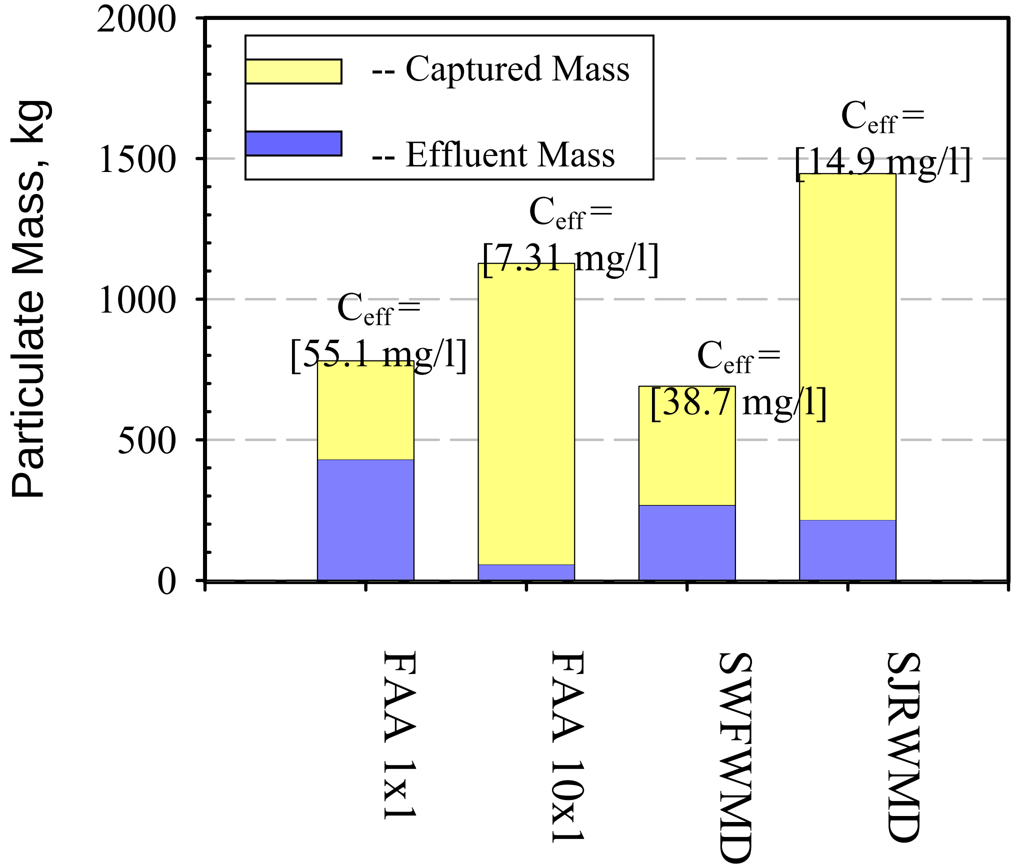

Figure 3D-7 25 year design storm effluent concentrations and loads with an influent load rate of 60.1 cfs and a particulate loading concentration of [100 mg/L]

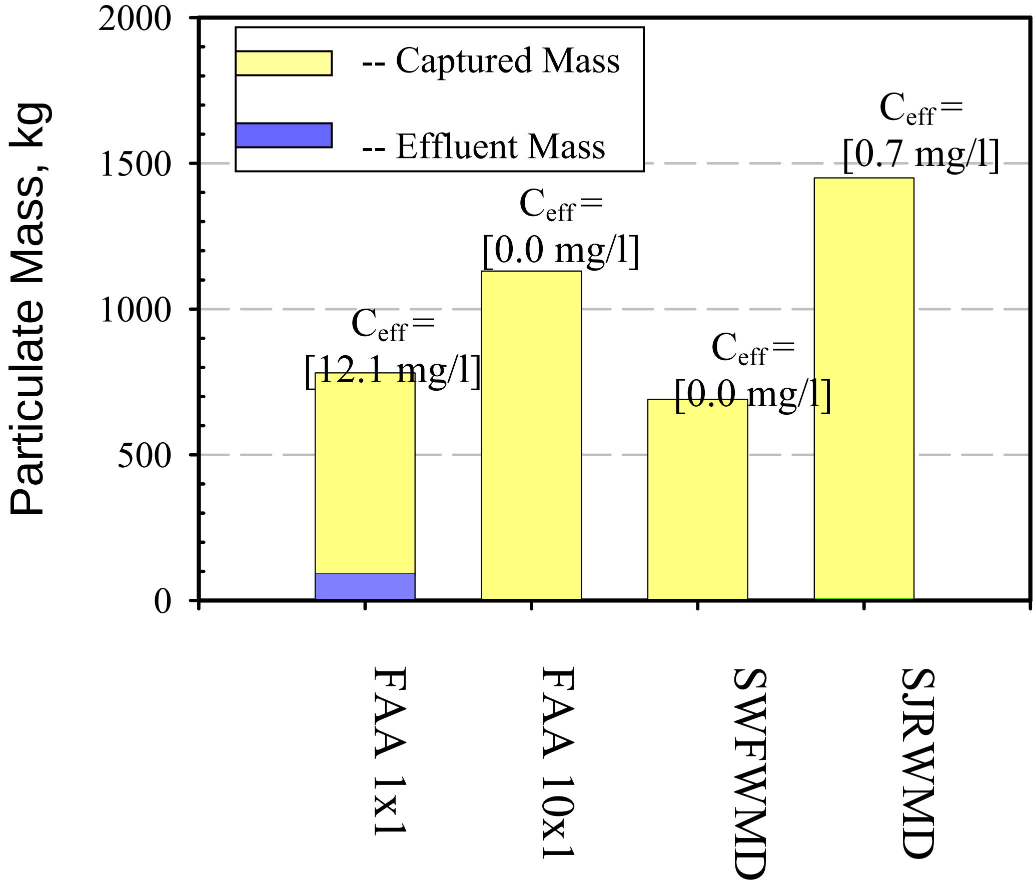

Figure 3D-8 Qp50 effluent load with an influent load rate of 3.8 cfs and a particulate loading concentration of [100 mg/l]

4. CONCLUSIONS

1. SWMM simulations indicate the overland flow Best Management Practice will result in water quality equal to or better than pristine site runoff quality for Hydrologic Group A, B and C conditions on the airport airside. The Best Management Practices recommendations for these soils provide reasonable assurance of exceeding current state water quality standards.

2. SWMM simulations indicate soils with groundwater tables within 18 inches of the ground surface and hydraulic conductivity rates of 0.1 inches per hour or less in the vertical direction will not provide adequate water quality management through infiltrating overland flow. These are generally soils in a Hydrologic Group D condition as described by NRCS. Site modification, for example soil

replacement or engineered modification to provide infiltration; or another form of water quality management will be required.

3. Design based on physical, measured parameters using Green-Ampt infiltration equations and continuous simulation hyetographs provides a physically-based, more accurate and precise evaluation of the incremental and cumulative airside infiltration abstraction (and therefore stormwater runoff) as compared to more empirical and rudimentary methods. Rainfall data in 5 minute increments is required for best estimates.

4. If there are airside conditions where infiltrating overland flow BMPs cannot successfully resolve runoff quality and quantity, a linear FAA pond with design features as described in this report has a high probability of exceeding the performance of current presumptive design wet ponds. These features should be the basis for initial FAA ponds if a surface water impoundment such as a wet ponds must be built on an airport. However, infiltrating overland flow should be the preferred BMP for both hydrologic restoration and reduced risk of bird-strikes as potentially generated by surface water impoundments.

5. Computational Fluid Dynamics (CFD) is likely the most powerful design and analysis tool available for wet pond design in order to examine the performance of wet ponds or other BMP units. CFD is also a tool that can be used to optimize performance features of a BMP unit such as a wet pond for specific project needs. CFD can be considered as a means of evaluating relative performance of various options; but can be calibrated and validated to provide absolute values of performance. CFD should not be misused or misinterpreted as providing direct answers when used to examine relative differences between design scenarios. Calibration and validation is essential when examining actual performance verification for any BMP unit.

Appendix A
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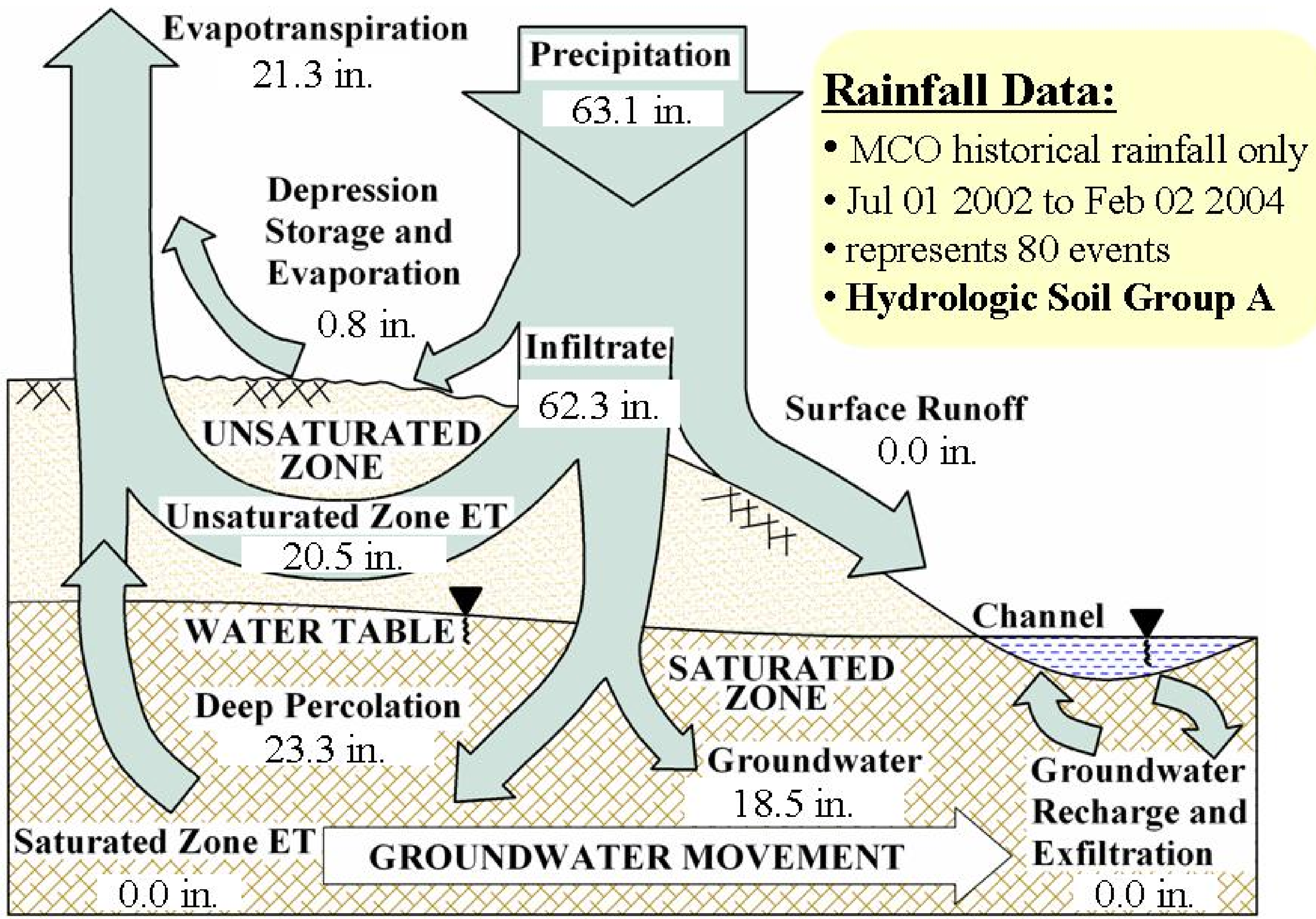
Appendix B
Dry Case Water Balance



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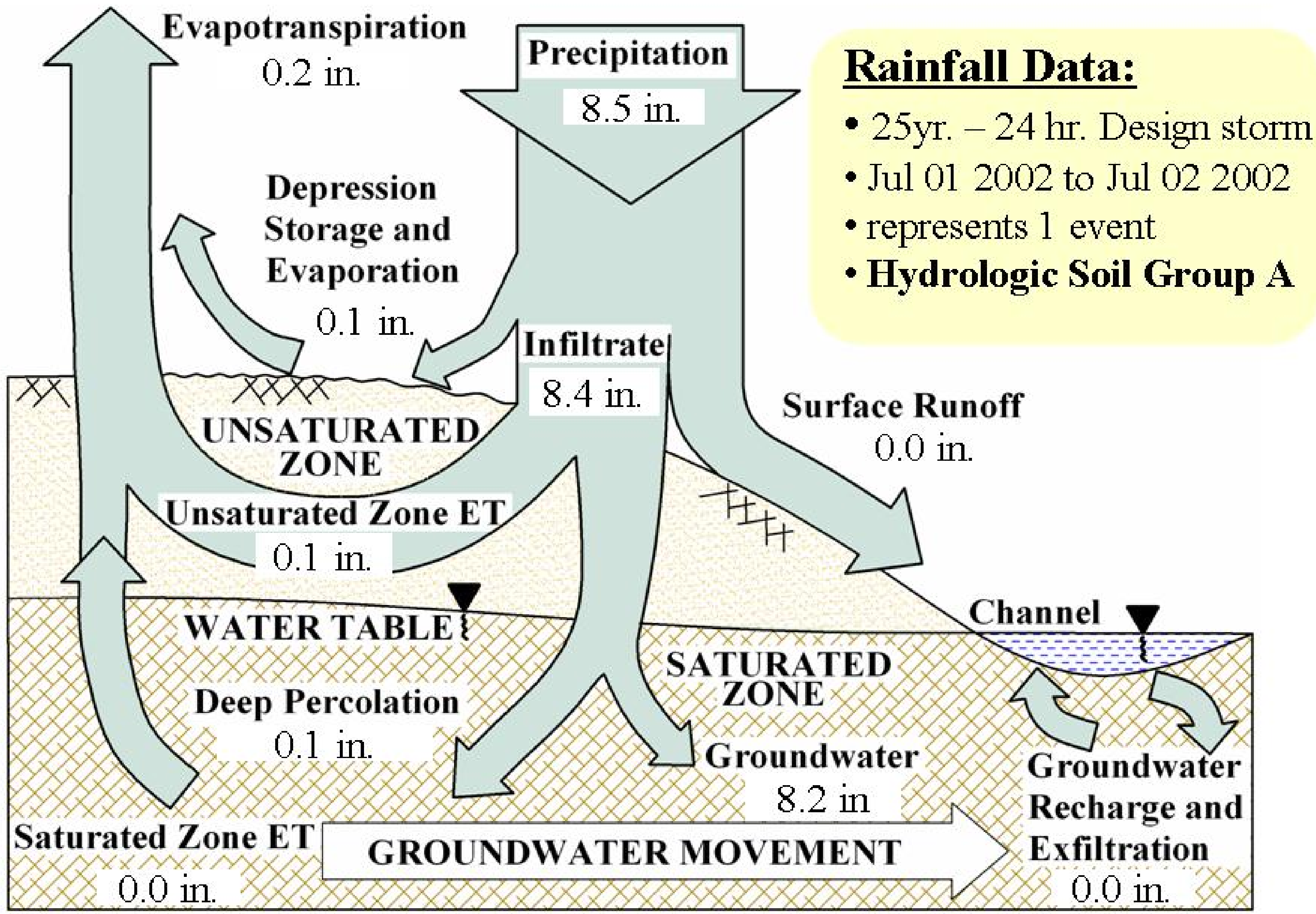

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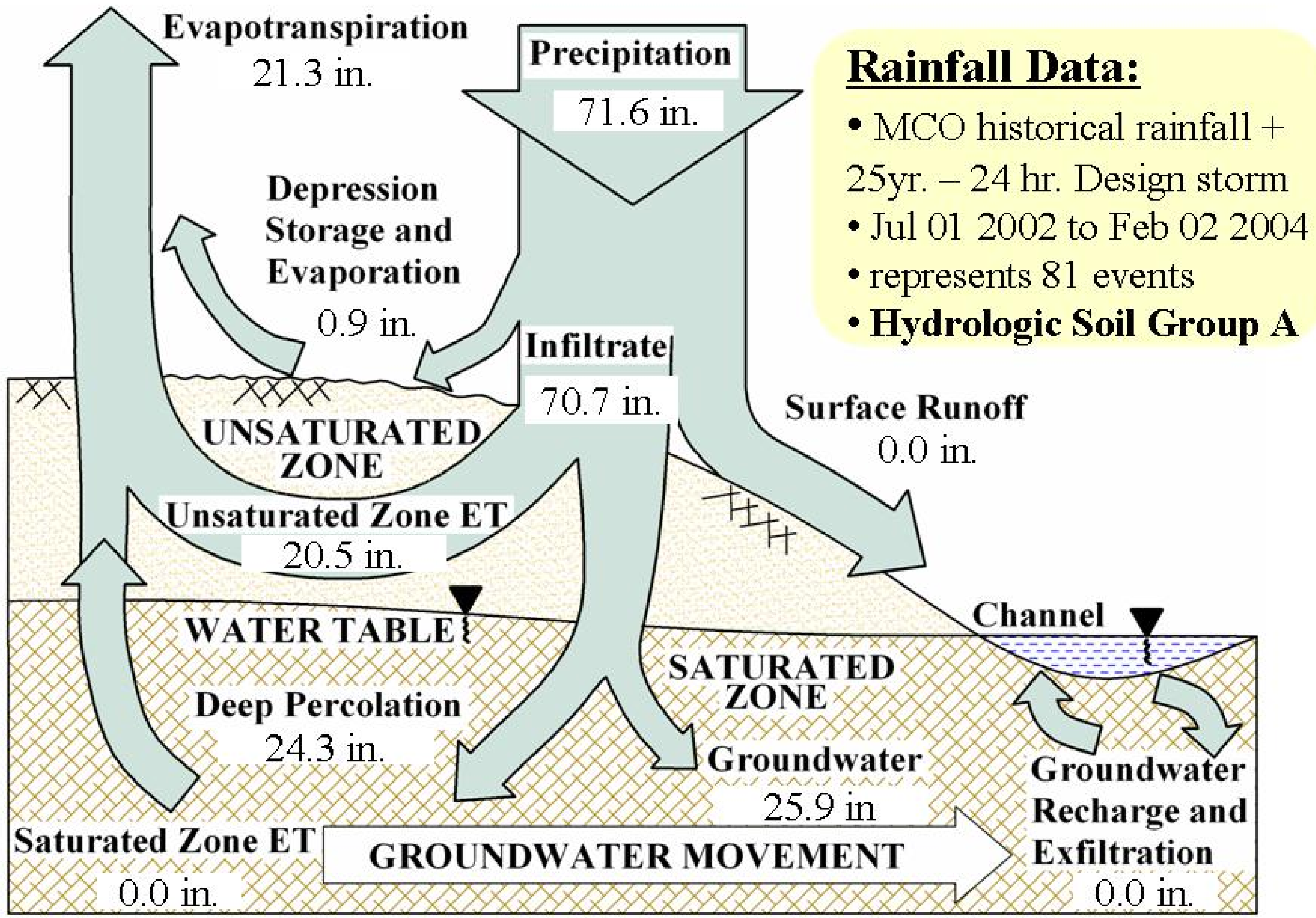

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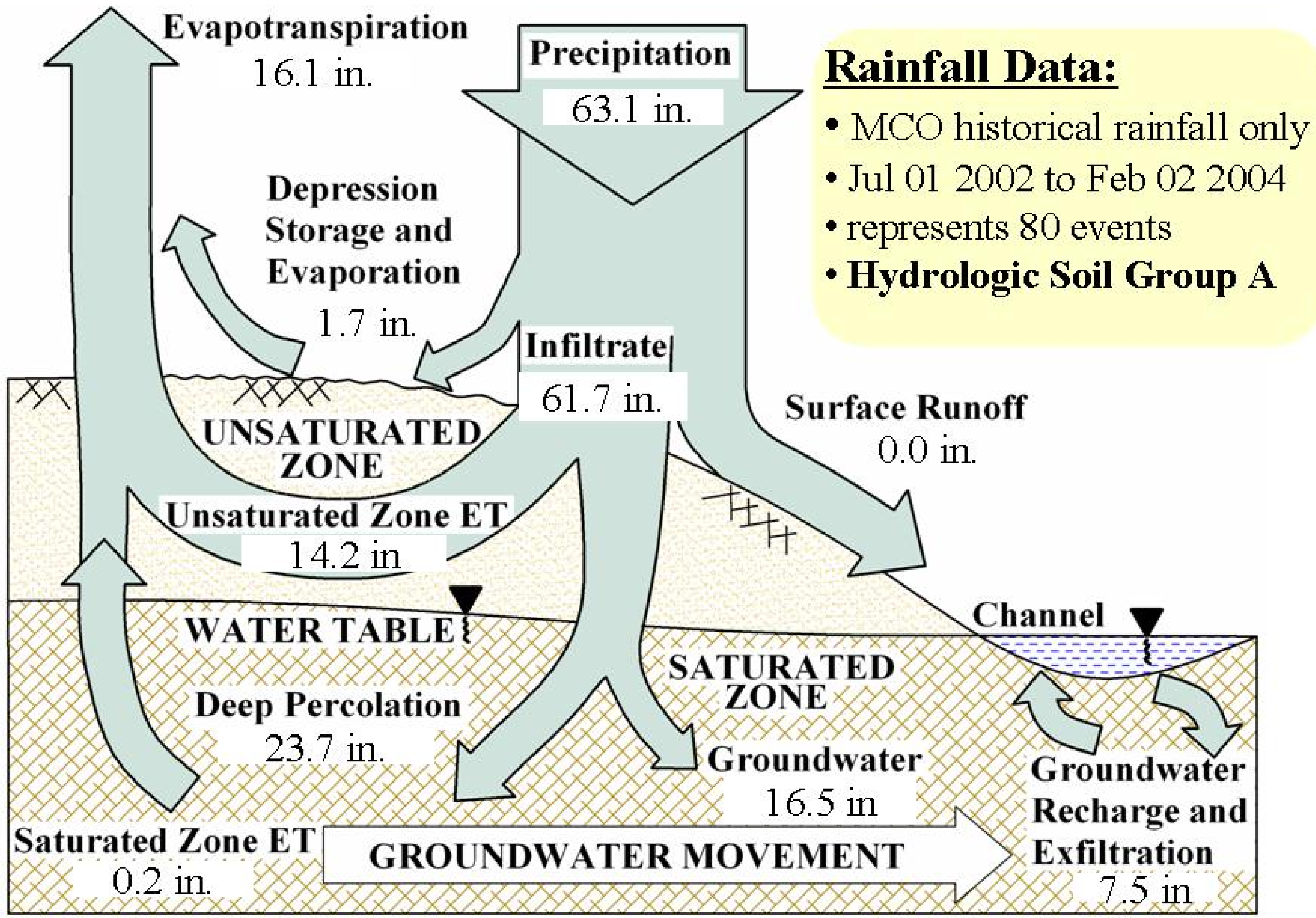

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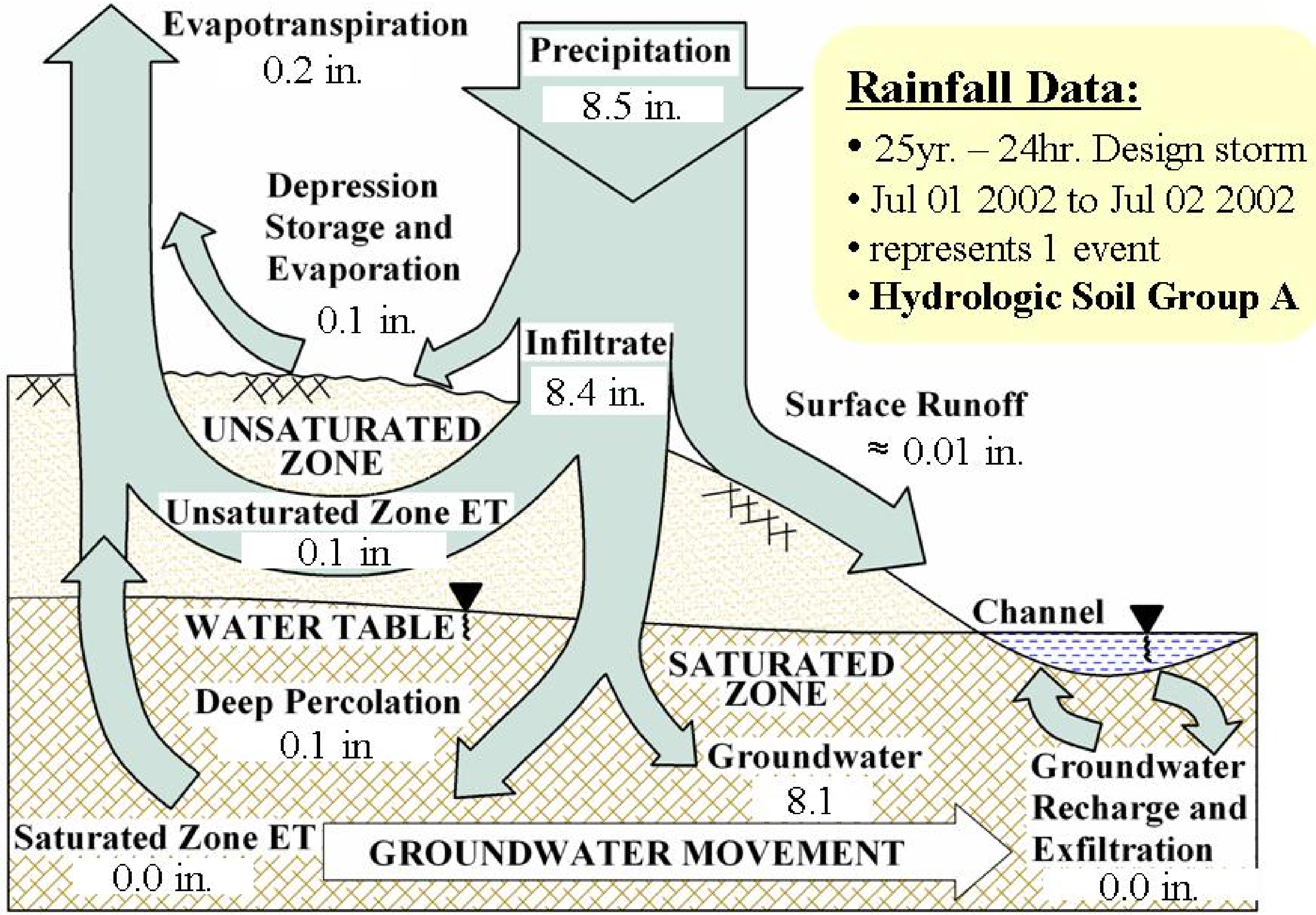

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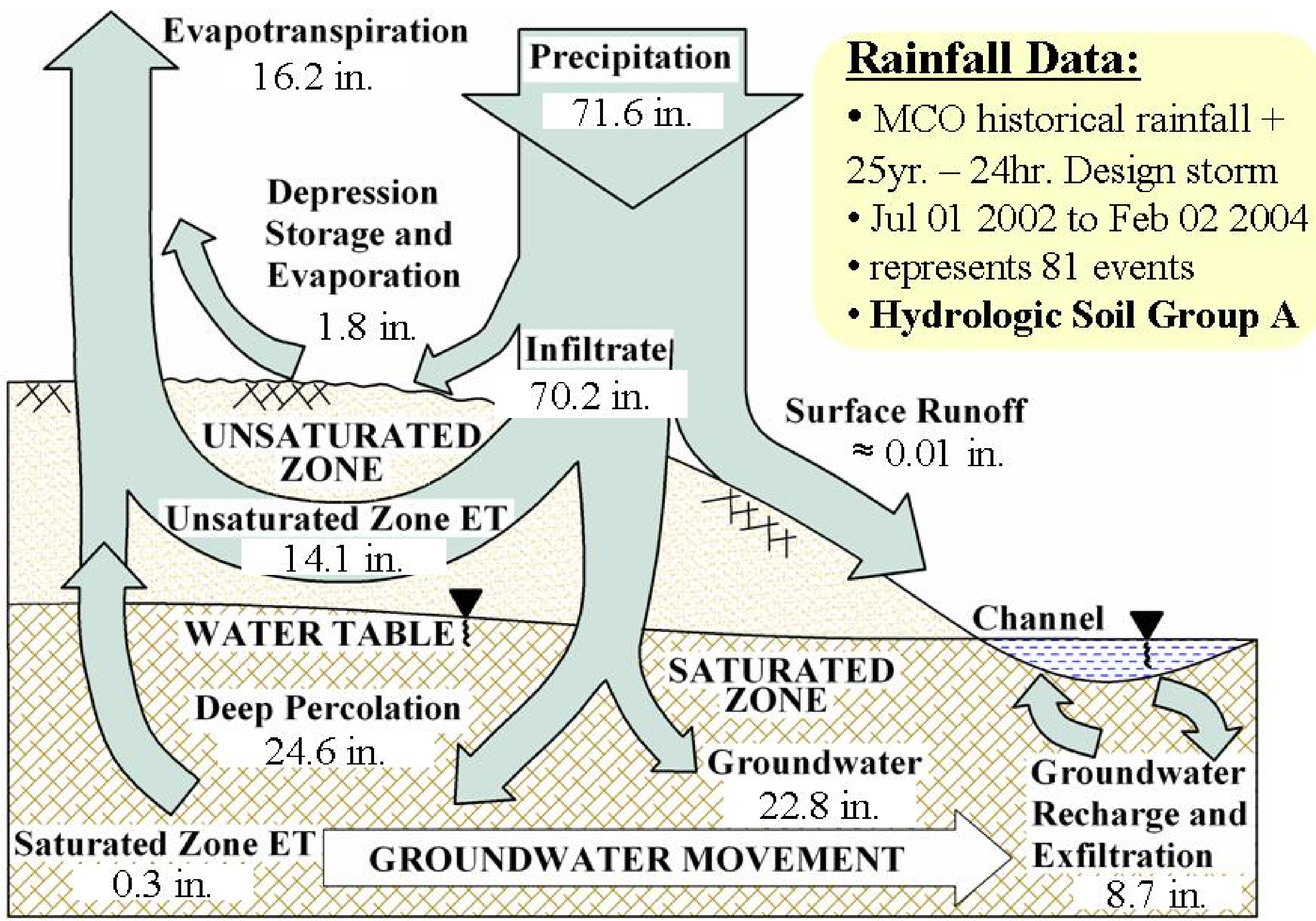
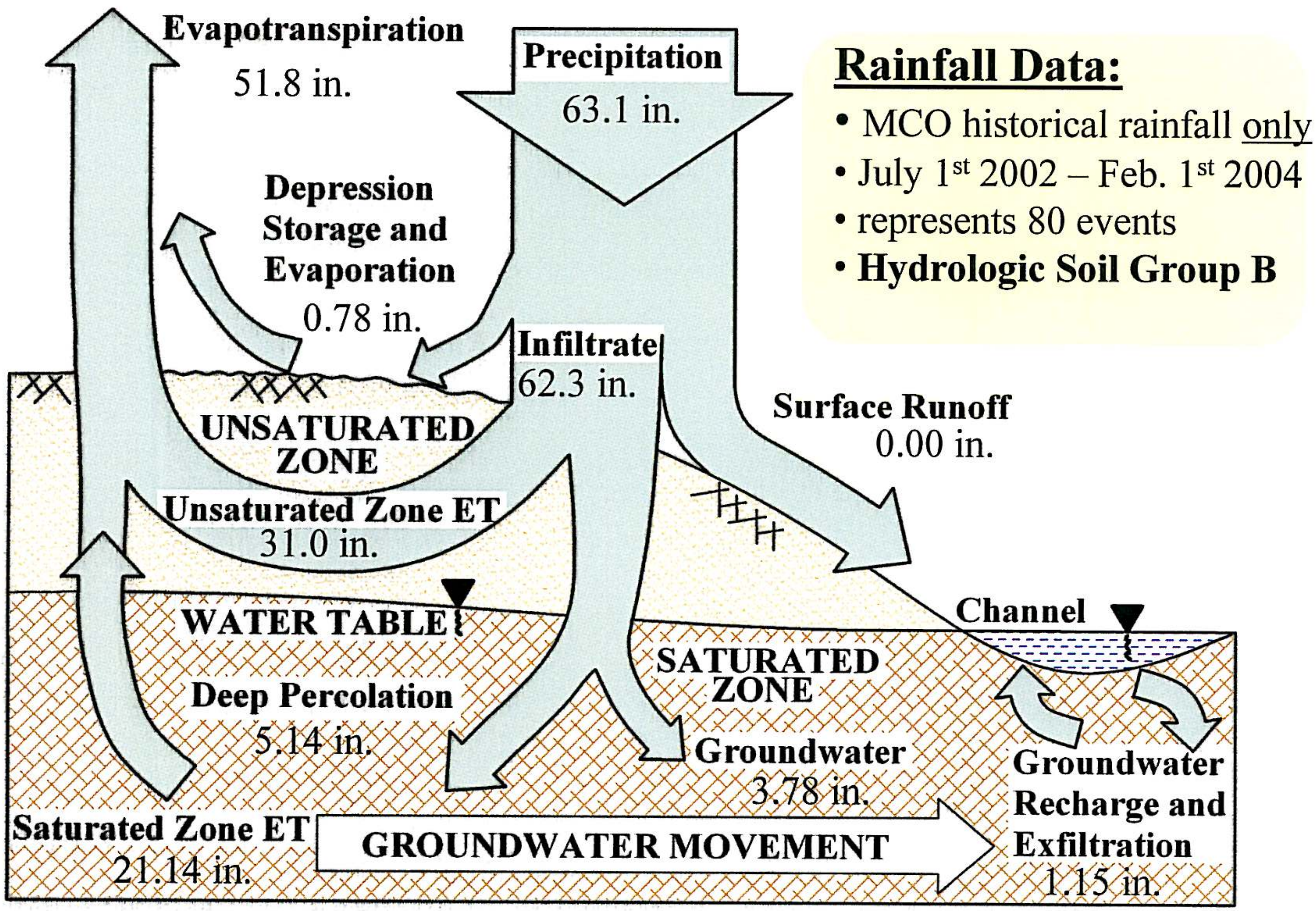


Developed Condition

APPENDIX C
INTERMEDIATE CASE (B) WATER BALANCE



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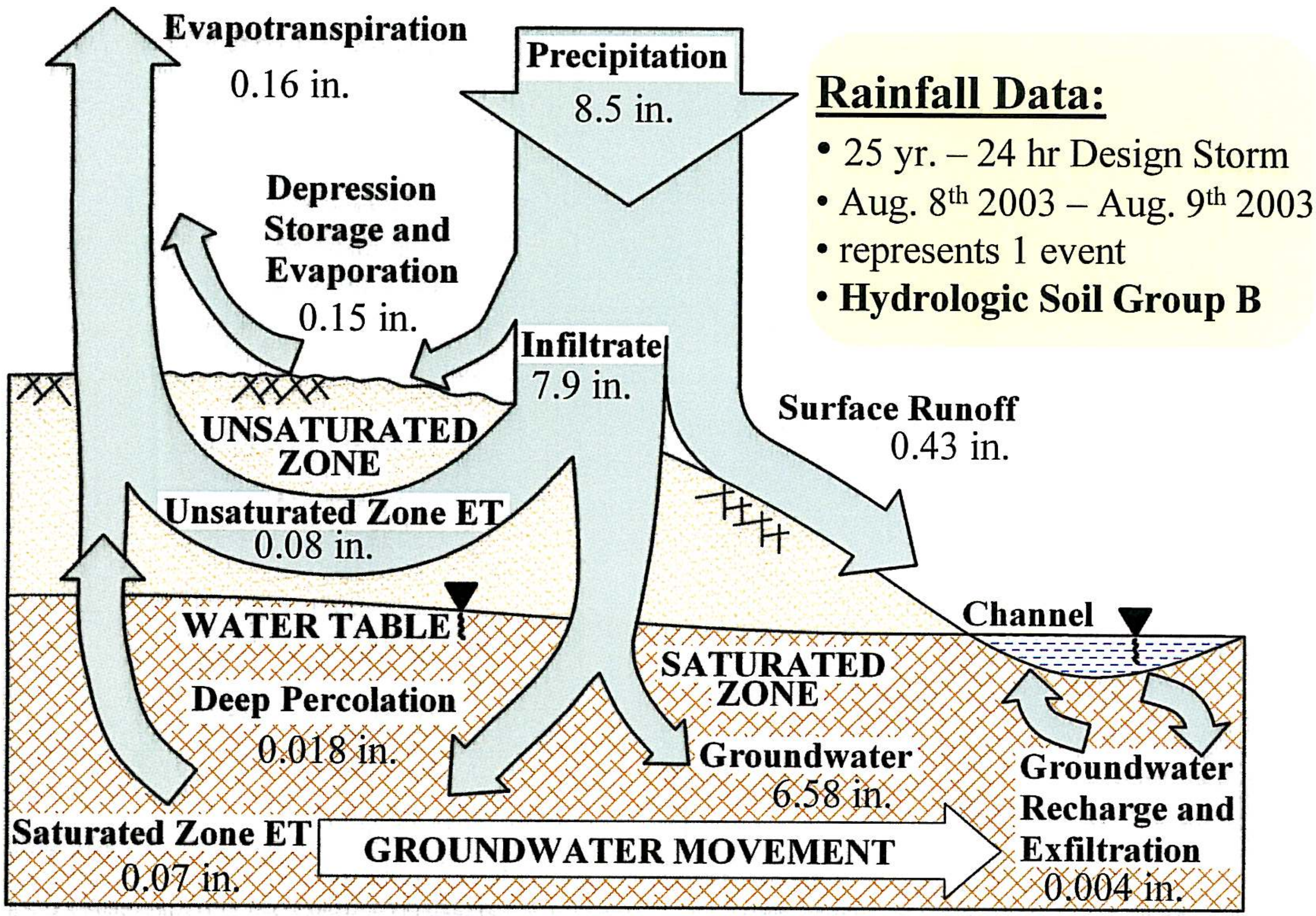

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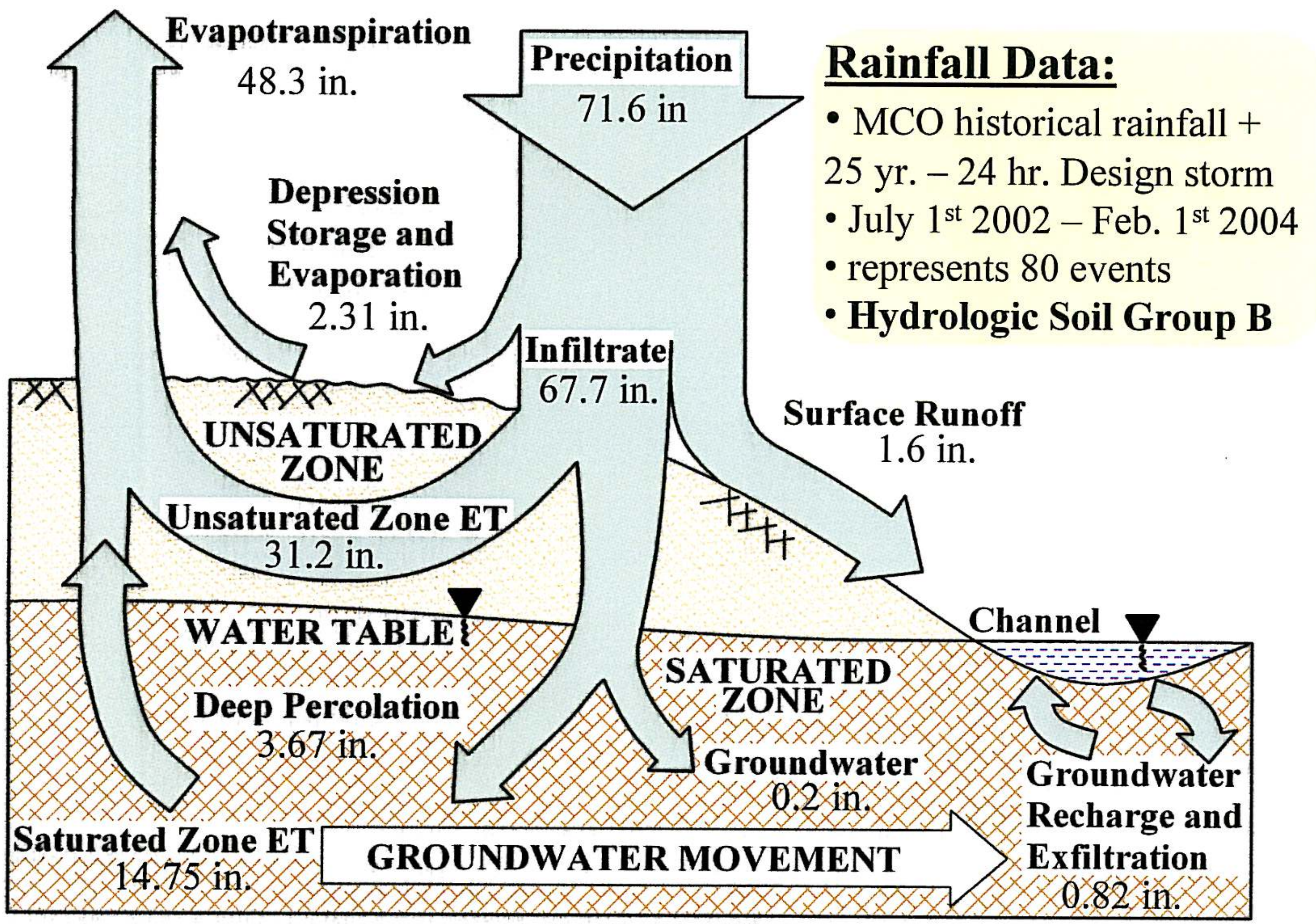

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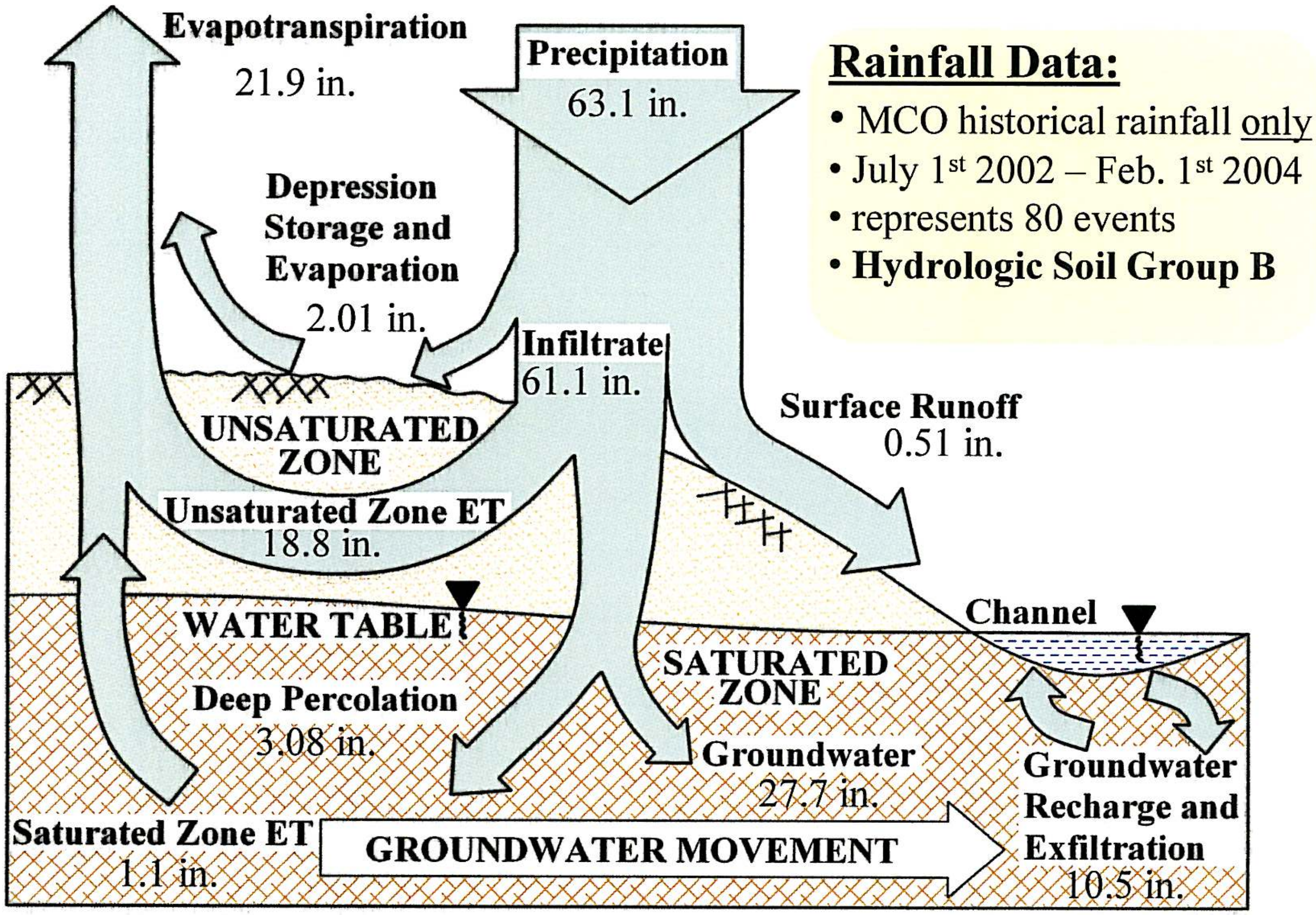

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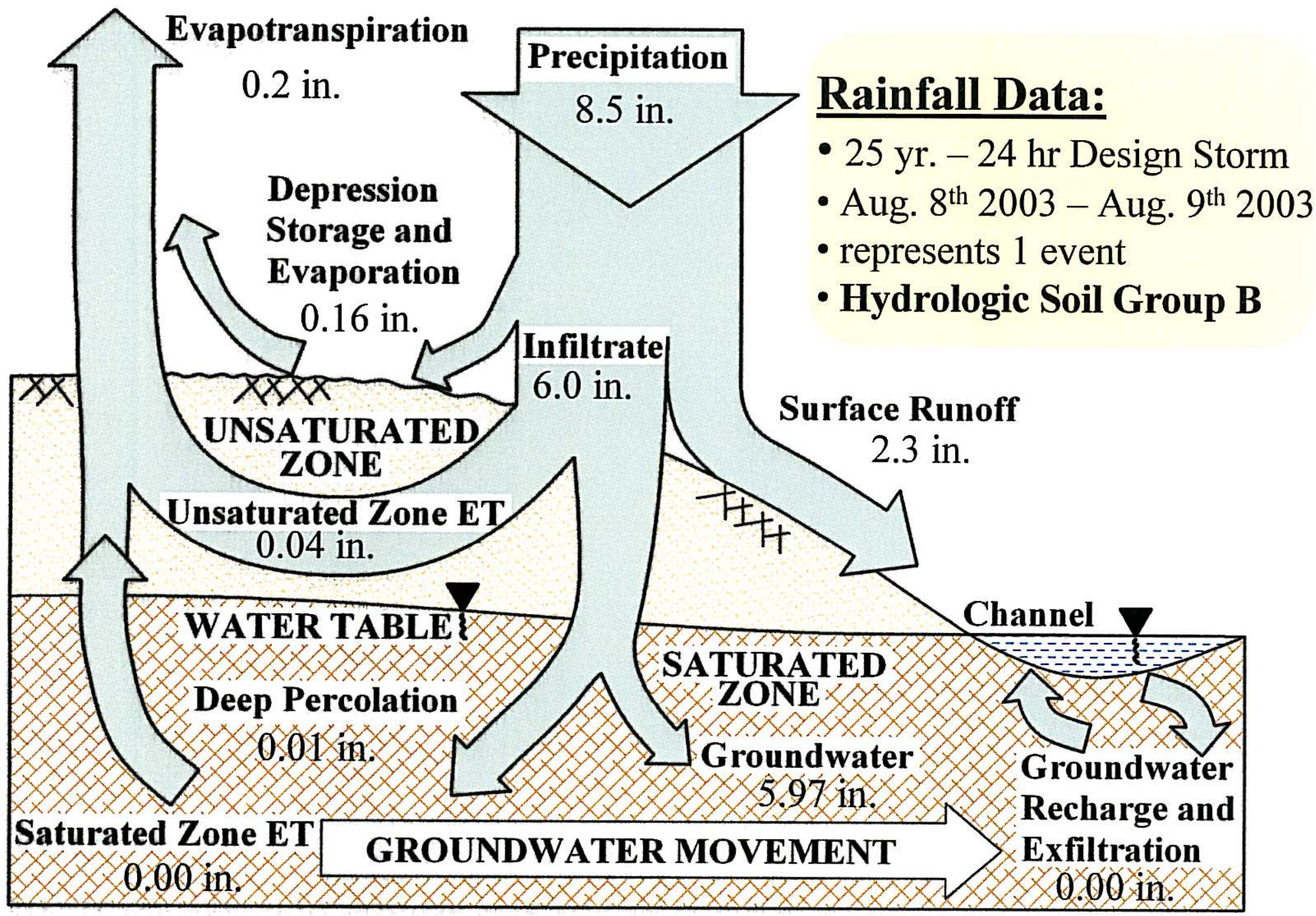

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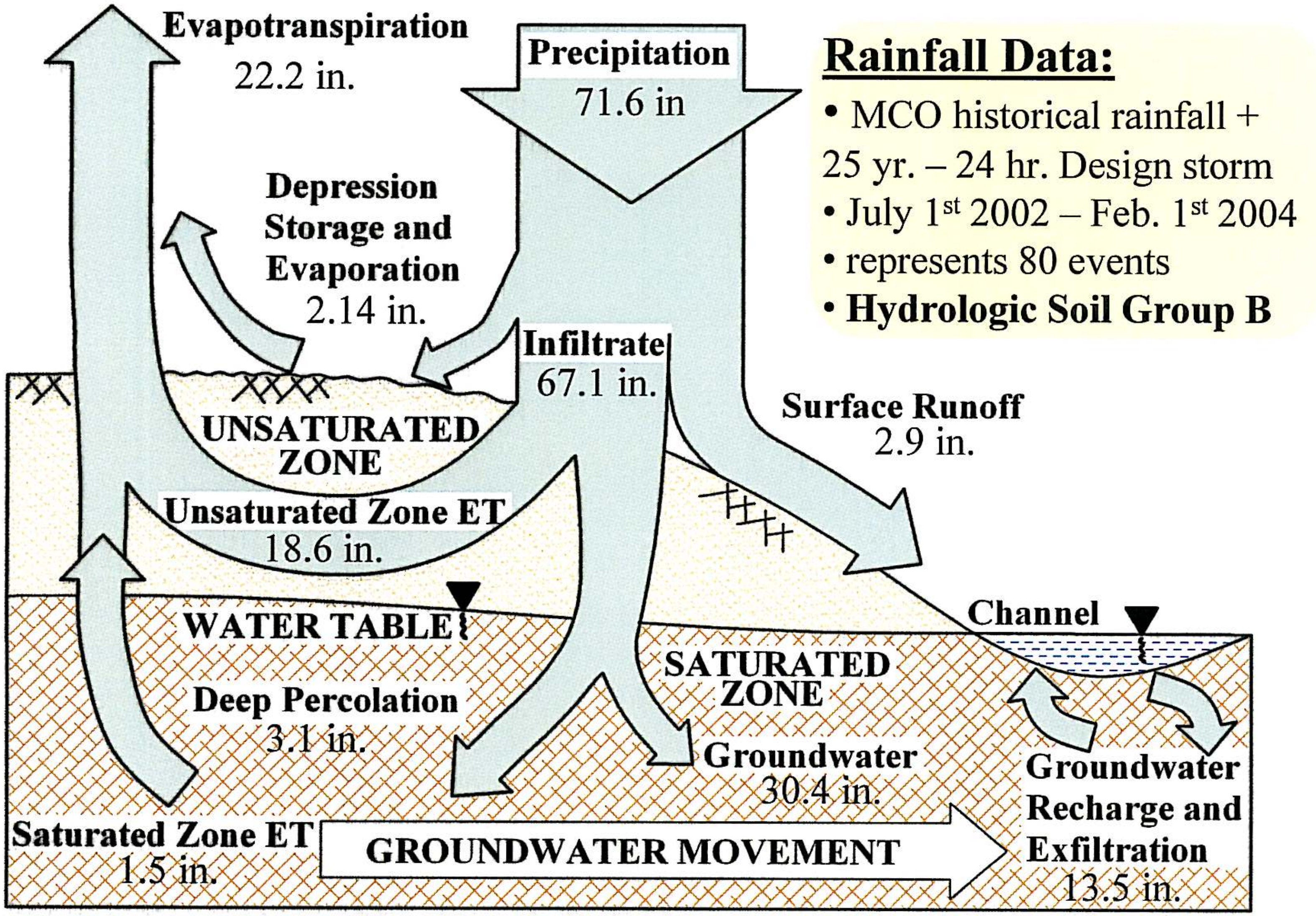
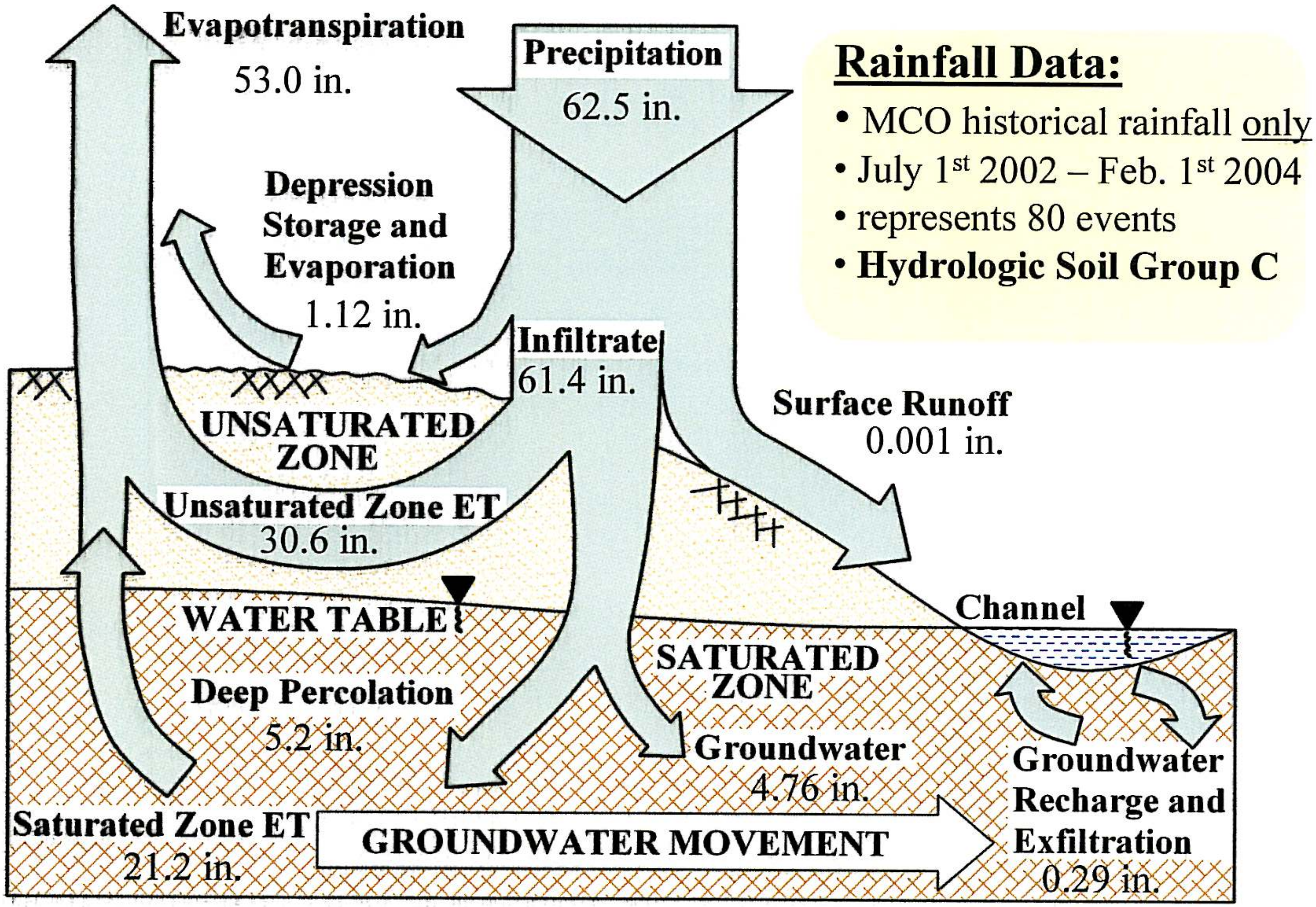


Developed Airside

Appendix D
Intermediate Case (C) Water Balance



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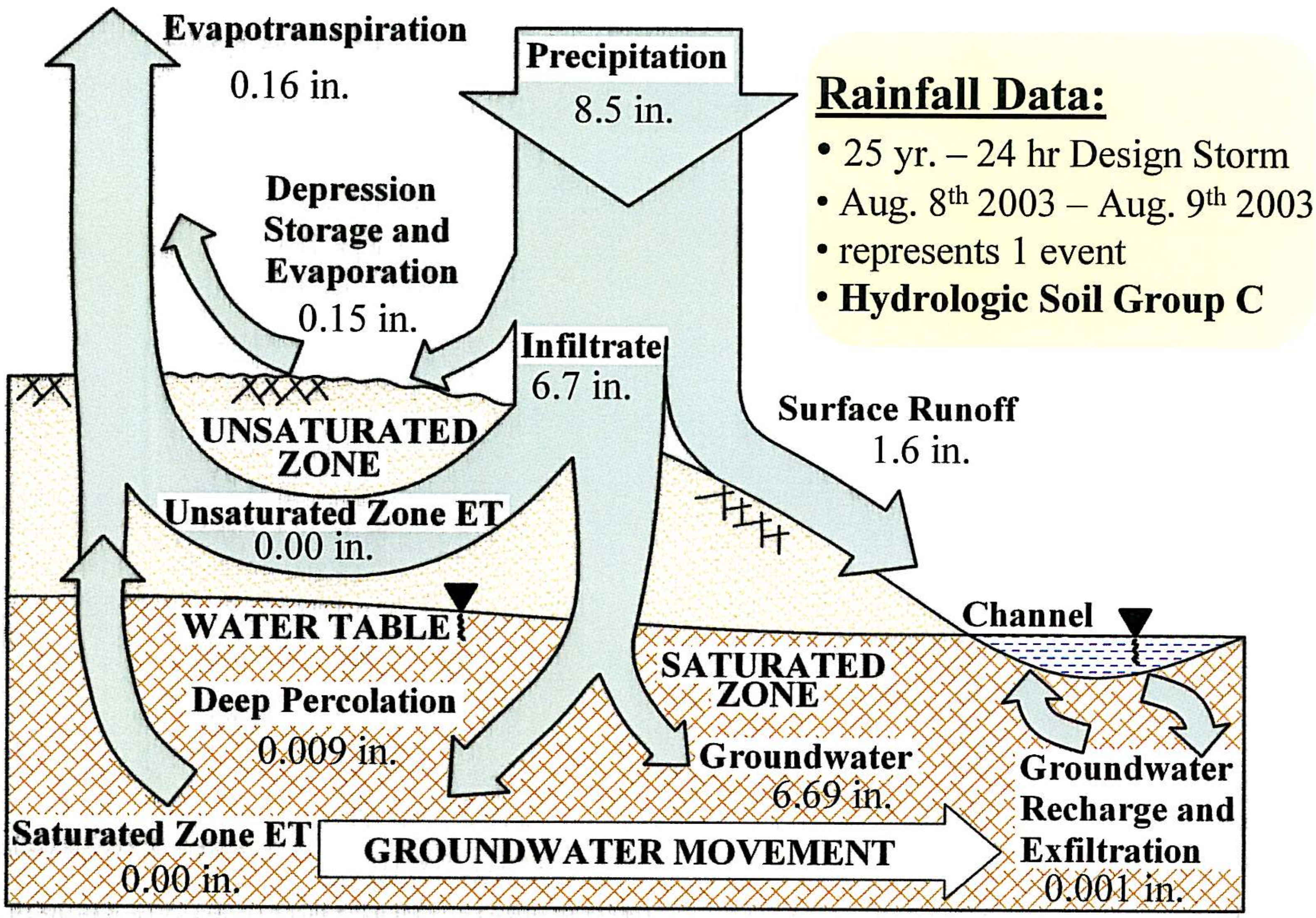

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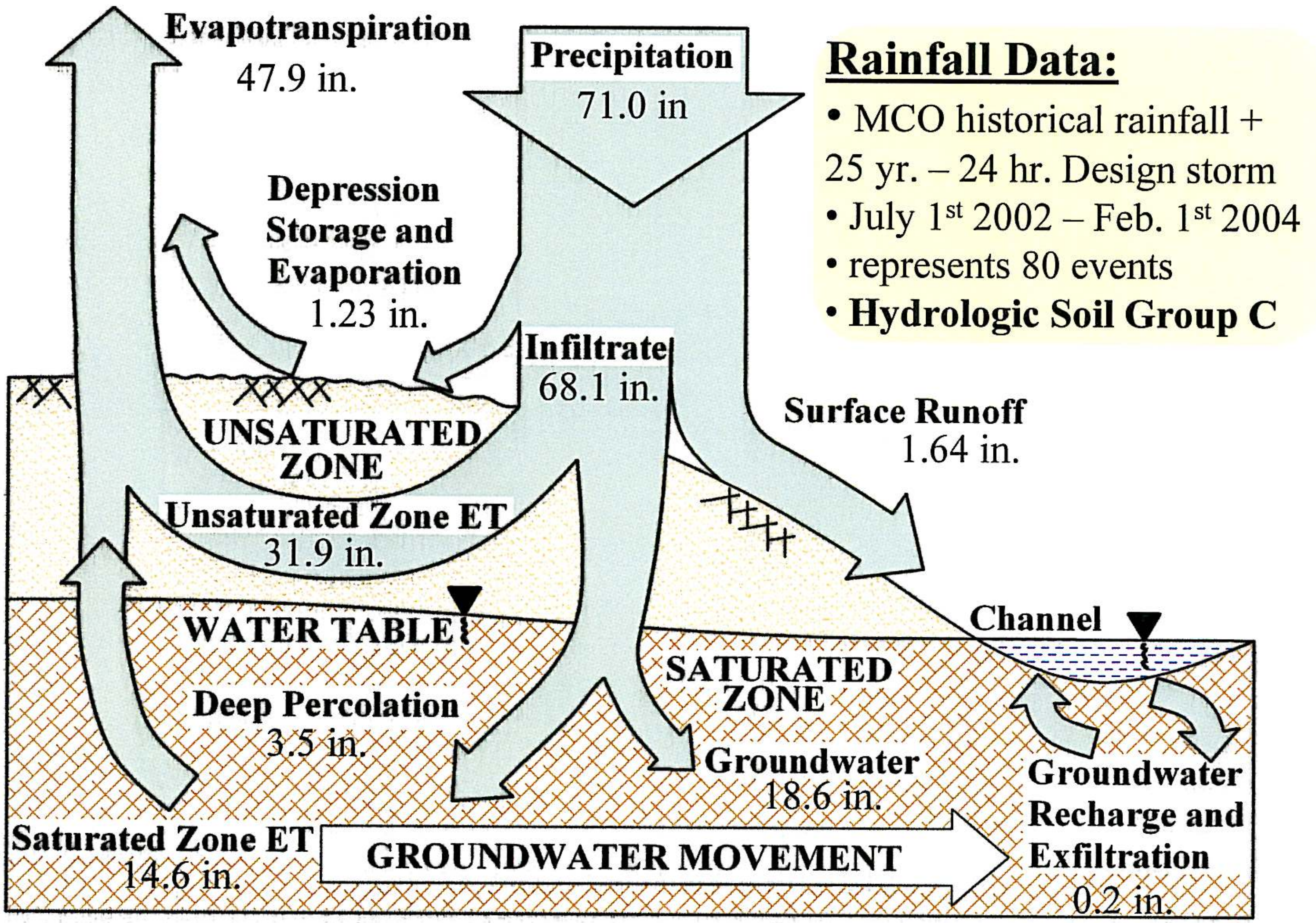

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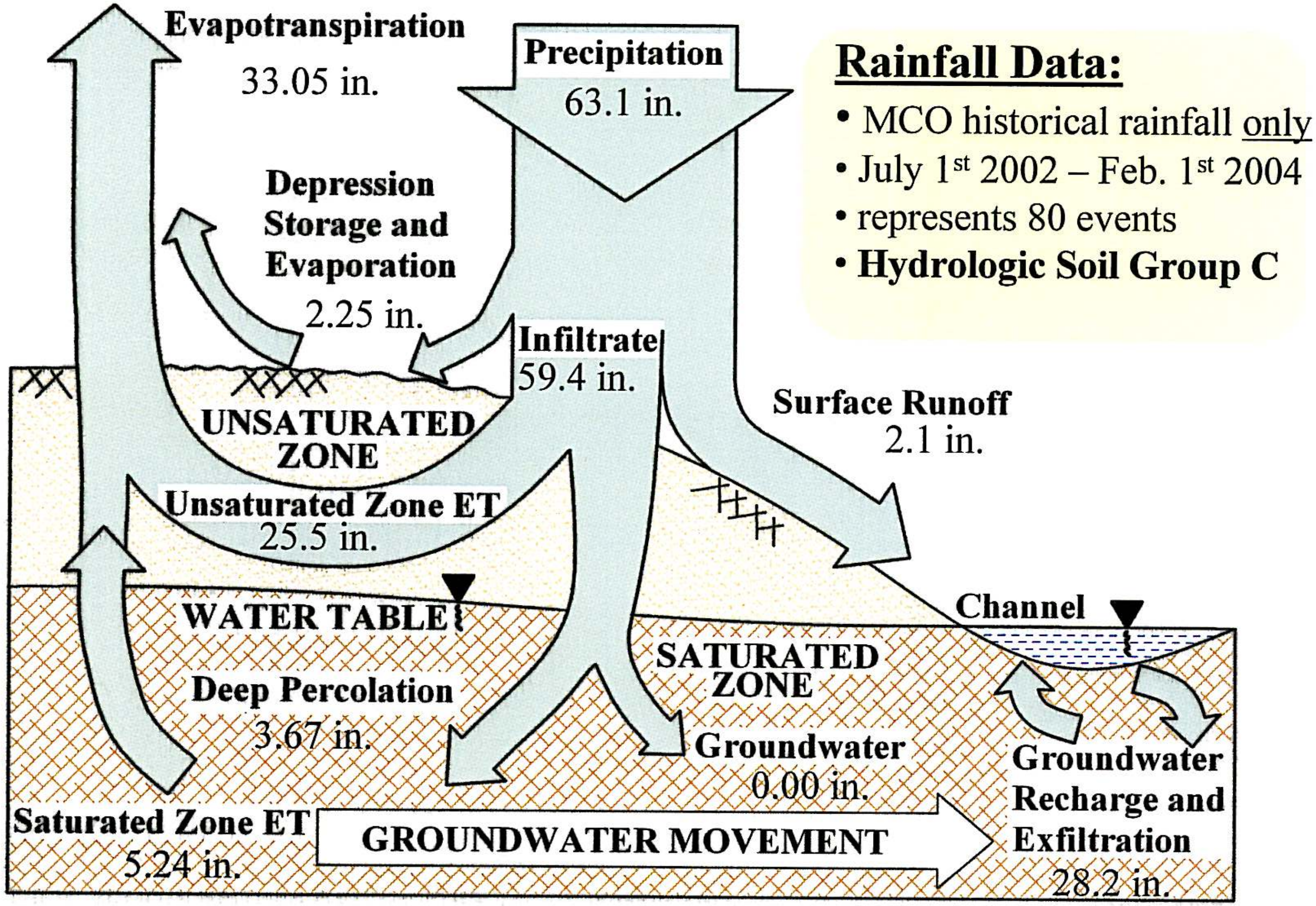

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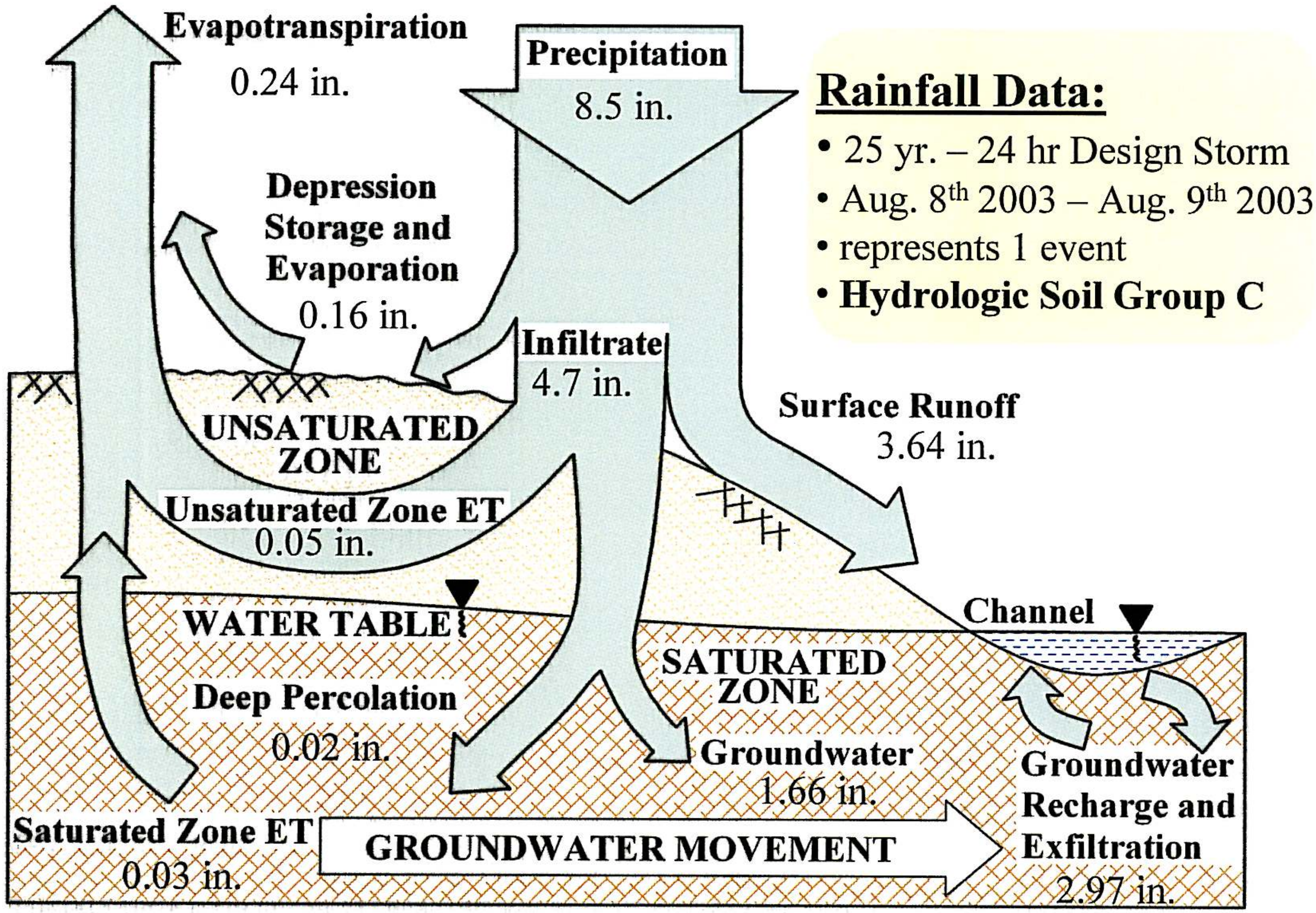

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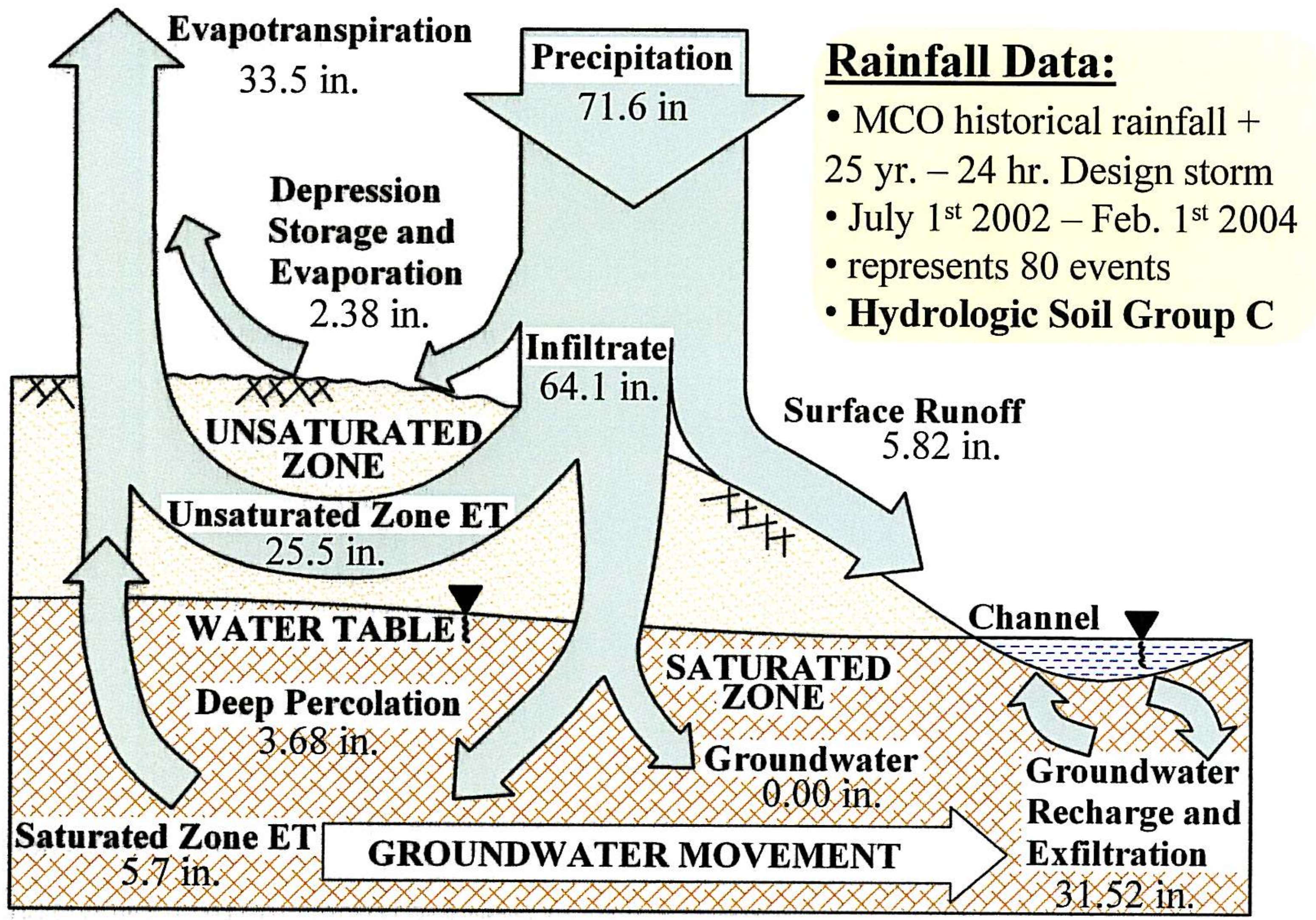
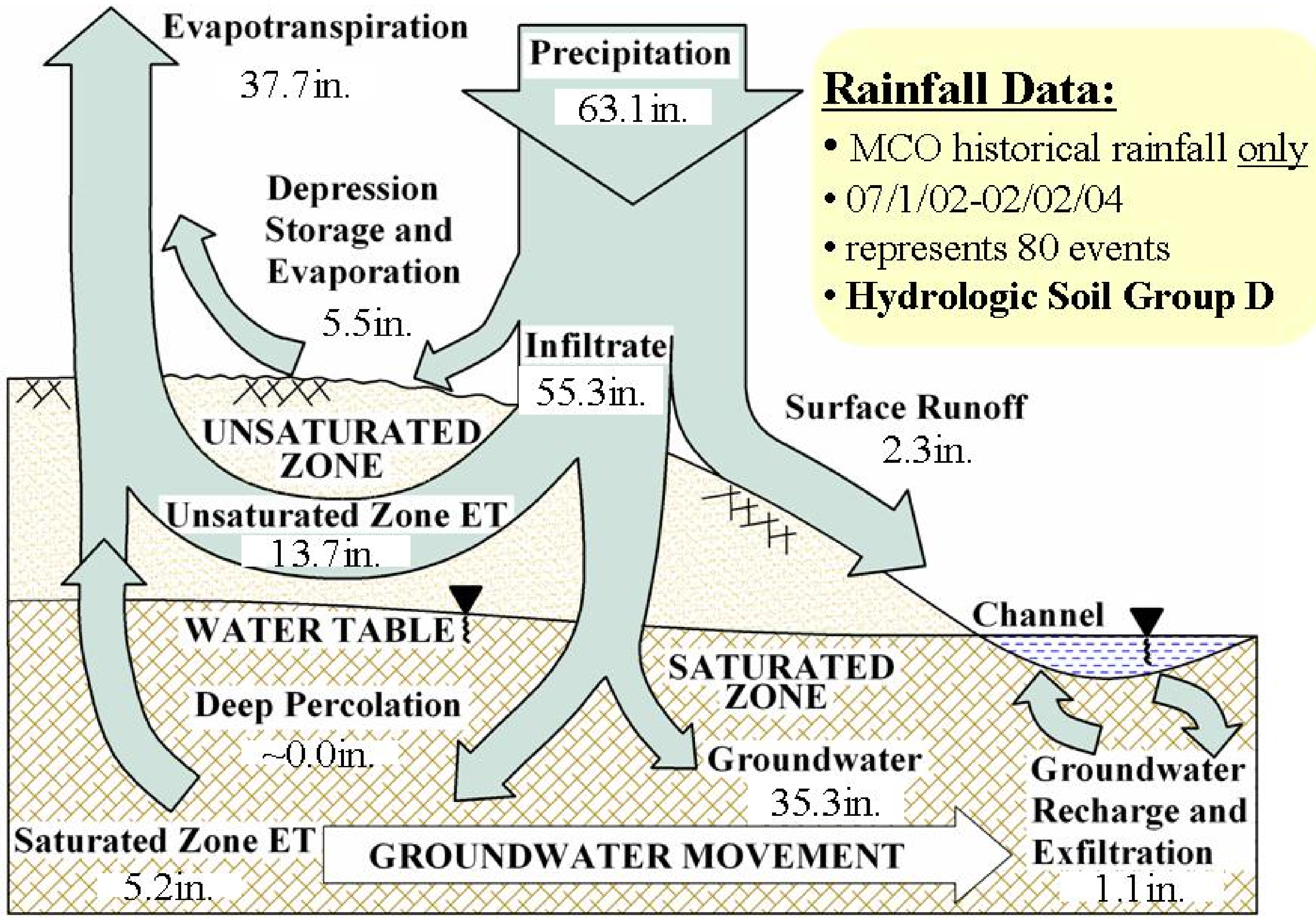


Developed Airside

APPENDIX E
WET CASE WATER BALANCE



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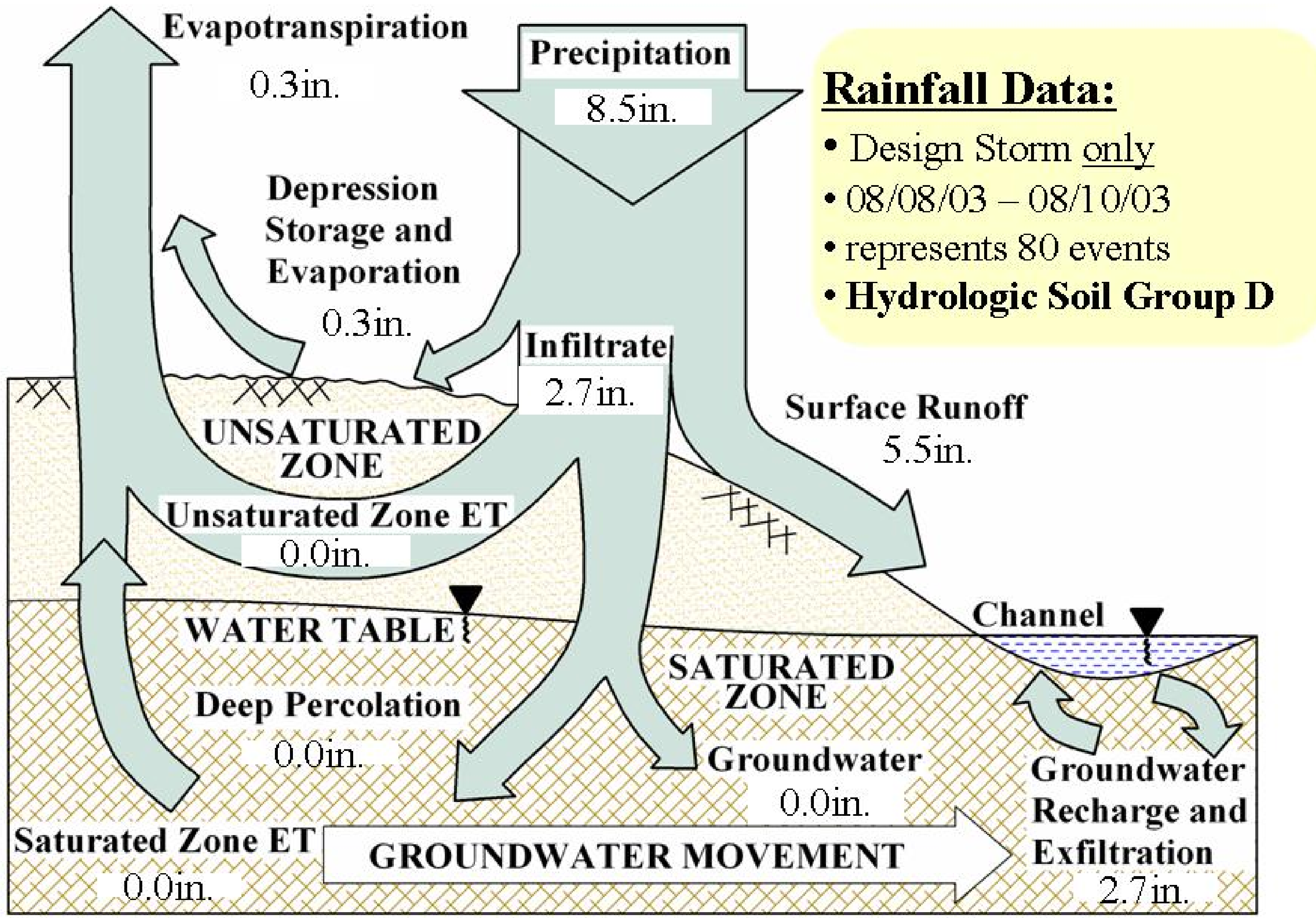

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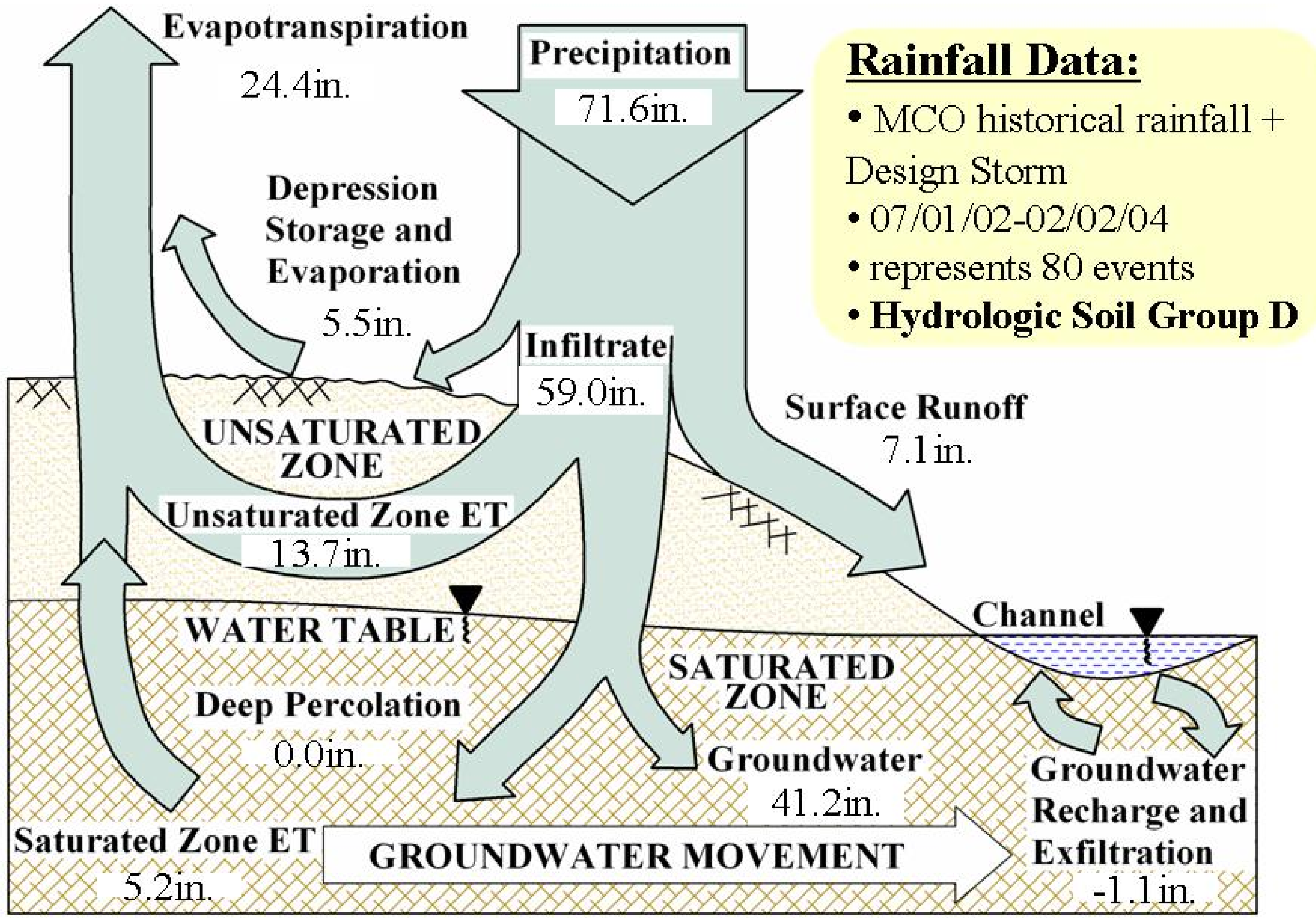

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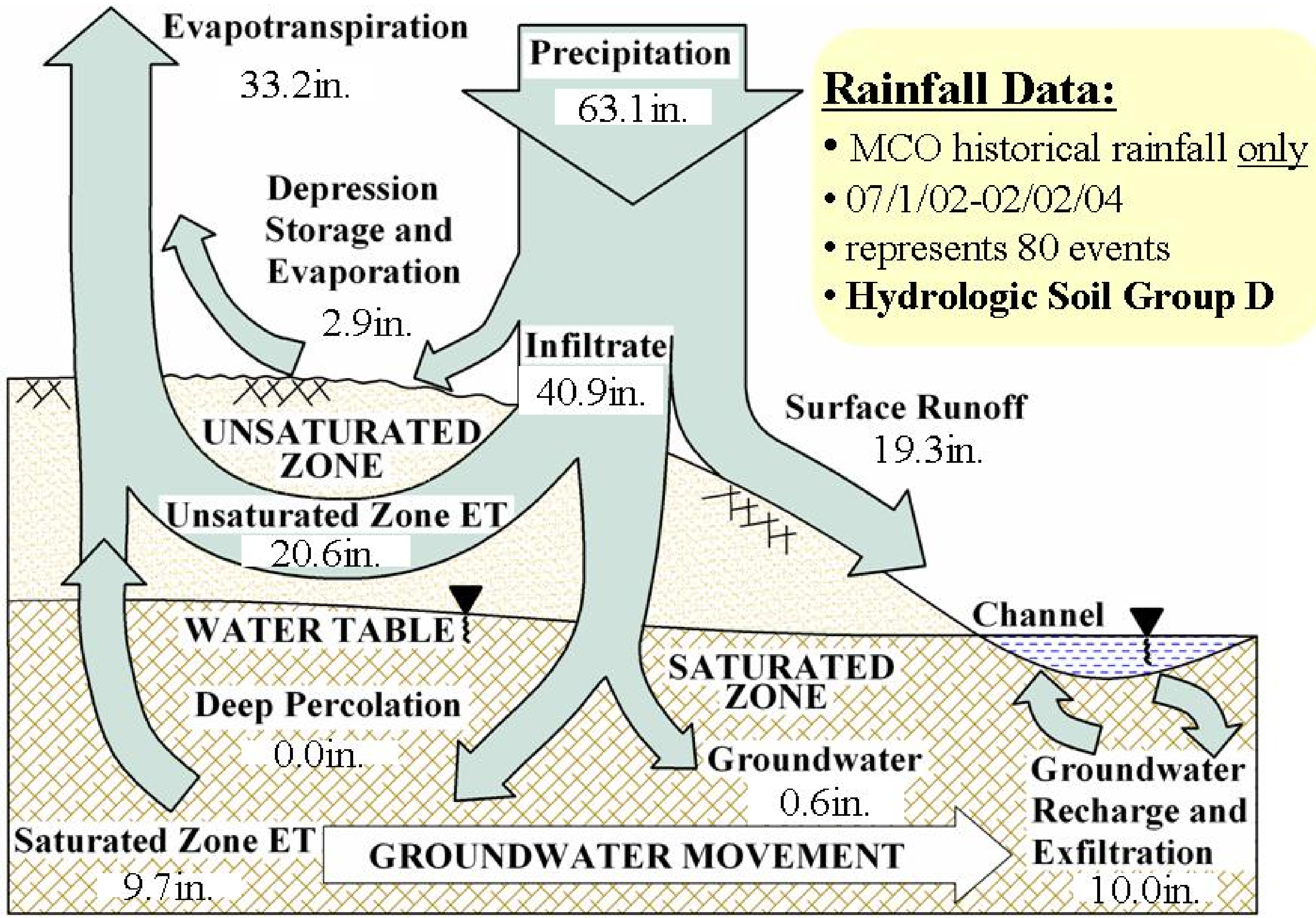

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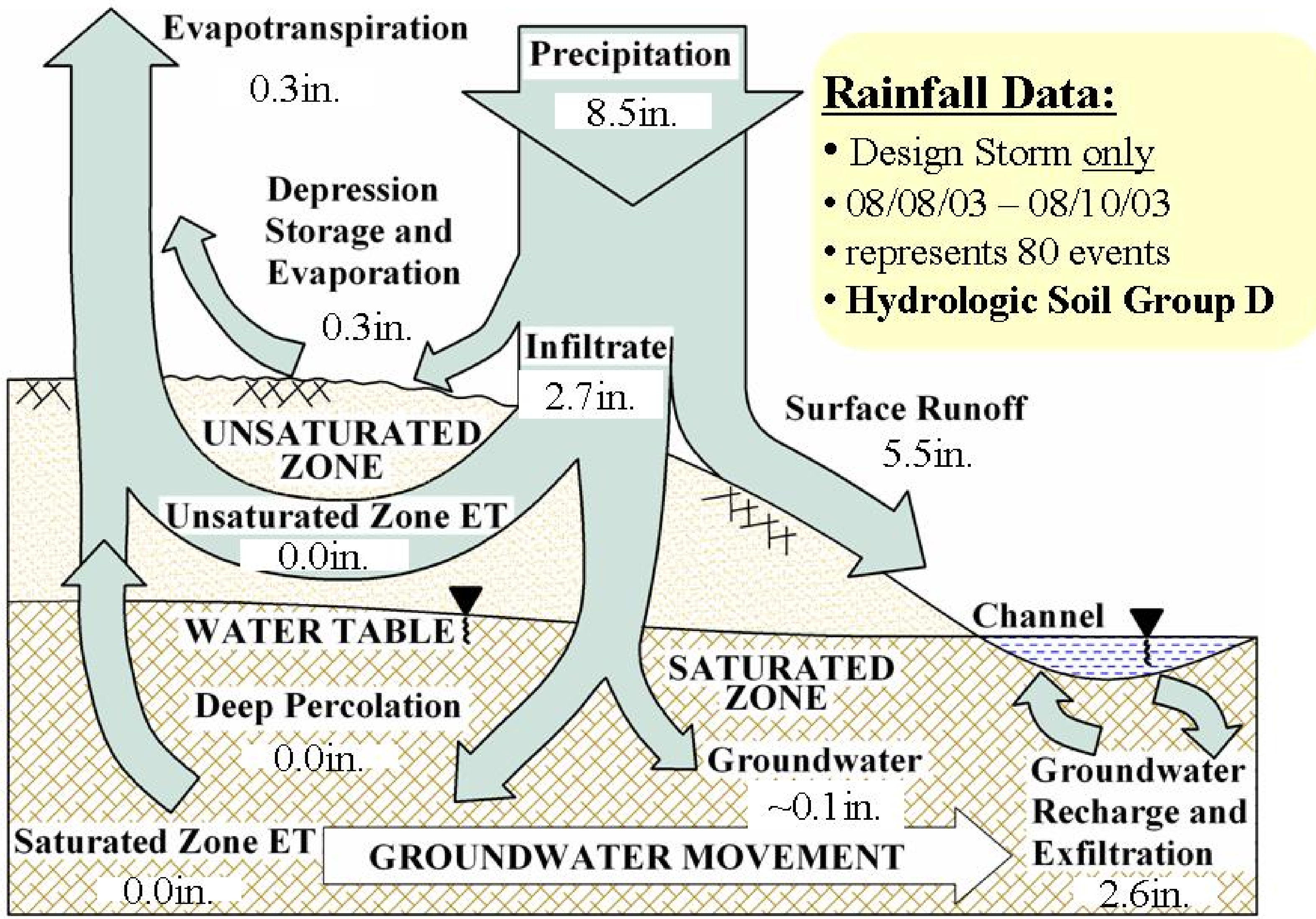

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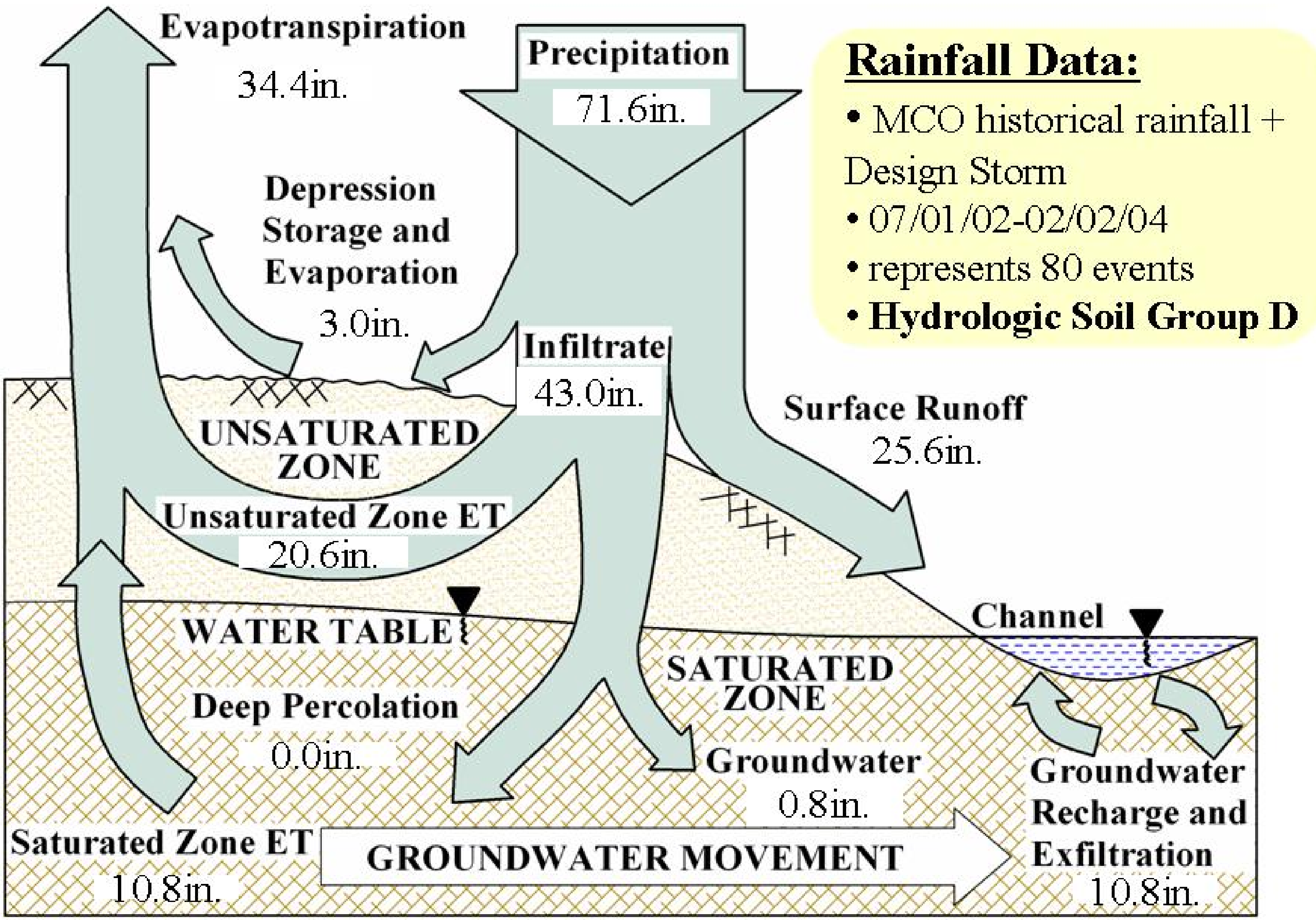

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