RECHARGE CHARACTERISTICS OF AN EFFLUENT DOMINATED STREAM NEAR TUCSON, ARIZONA

by
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As members of the Final Examination Committee, we certify that we have read the dissertation prepared by Laurel Jane Lacher entitled
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ABSTRACT

Almost 90% of the treated sewage effluent processed by the two treatment plants serving the greater Tucson area is available for passive recharge through the Santa Cruz River streambed north of Tucson. In the absence of any major disturbance of the effluent channel, the recharge capacity of the streambed materials decreases over time as microbial activity, and possibly suspended sediments settling out of solution, act to clog the surficial sediments under the effluent stream. Effluent stream transmission-loss measurements made over the period from November 1994 to August 1995 provided data used to determine the average vertical hydraulic conductivity of the low-flow channel in the study reach through simulations using the computer model known as KINEROS2. Saturated hydraulic conductivity (KSAT) served as the calibration parameter in the model. The appropriate KSAT value was chosen for each set of field data by matching the observed and simulated downstream hydrographs for the study reach. KSAT values were corrected for viscosity changes resulting from changing average daily surface water temperatures over the study period. Saturated hydraulic conductivity values for the effluent stream channel ranged from a maximum of 37 mm/hr in January, 1995, following several major winter storms, to a minimum of 11 mm/hr in August, 1995, after a nearly six-month interstorm period. The saturated hydraulic conductivity values decay exponentially with time after the last major winter storm. The mathematical model describing this decay may be used to estimate effluent recharge rates under similar future meteorological and climatological conditions.
CHAPTER I. INTRODUCTION

MOTIVATION FOR STUDY

Since Tucson's initial establishment as a permanent anglo settlement in the mid-1800's, water has been a pivotal facet of survival in this desert town. While Native American inhabitants of the area had long relied on the surface water of the Santa Cruz River for irrigation, groundwater use by Tucson settlers in the late 19th century, in conjunction with regional arroyo entrenchment during that time, quickly depleted the perennial surface water supply. Consequently, the greater Tucson area has grown into a metropolis of over 650,000 people that, until 1994 and the arrival of Colorado River water through the Central Arizona Project canal, has depended entirely on groundwater for survival.

In this environment of extreme heat (frequently exceeding 40°C) in the summer and very little precipitation (30.5 cm annual mean), conservation of water has evolved as a part of Tucson's culture. From the earliest days of centralized sewage treatment in the city (beginning in the 1940's), Tucsonan's have accepted and utilized treated effluent as an important water supply. In the mid-1960's, farmers bought treated effluent and used it to irrigate local farmlands. Over the last two decades, many parks and golf courses have converted to reclaimed-water irrigation, and interest in recharging effluent -- both passively and actively -- has inspired numerous scientific investigations on the topic. In spite of our
growing understanding of the mechanics of artificial recharge, however, its practical application remains fairly limited for a number of logistical and political reasons.

Since 1970, most of the treated effluent generated by the treatment plants for the greater Tucson area has been discharged directly to the ephemeral Santa Cruz River bed. Tucson's recent rapid population growth has led to a trend toward housing developments replacing farmlands near the treatment plants (and the river). The retirement of farmlands, and concerns over groundwater contamination, drastically reduced the market for treated effluent. This reduced market, combined with ever-increasing volumes of sewage being generated in Pima County, have led to a significant portion of the total treated effluent from the wastewater treatment plants being discharged directly to the Santa Cruz River. This vast supply of water (close to 190,000 cubic meters per day (CMD) (50 million gallons per day (MGD)) and increasing steadily) currently undergoes passive recharge or evaporation upon discharge into the normally dry riverbed.

In light of the tremendous value of water in the Tucson Basin, local city and county government agencies as well as private irrigation districts have expressed strong interest in quantifying the amount of recharge from the effluent stream (as well as from natural storm runoff) through the Santa Cruz River bed to the underlying unconfined aquifer. This study is primarily motivated by the issue of determining the quantity of effluent recharge in the Tucson basin.
OBJECTIVES

Quantifying streambed recharge involves understanding the various components of the local water budget as well as their relative magnitudes. This study seeks to improve the current level of knowledge about the central components of riverbed recharge in the study area. Specifically, this study aims to:

1) provide a general description of the time varying recharge process in the effluent stream of the Santa Cruz River and, specifically, to develop a mathematical model describing the magnitude and rate of change of the average vertical hydraulic conductivity of the effluent streambed during interstorm periods;

2) estimate the threshold flood event required to significantly improve infiltration in the low-flow (effluent) stream channel;

3) characterize the nature of the hydrologic connection between the river channel and the underlying aquifer;

4) evaluate the relative importance of effluent recharge in the study area with respect to total basin recharge;

5) contribute to the general bodies of literature on passive effluent recharge and on practical modeling applications of KINEROS2.

Ultimately, the results of this study should provide water resources managers with some guidance for estimating annual recharge of sewage effluent through the Santa Cruz River channel and, thus, aid in managing this important resource more effectively.
GENERAL APPROACH

The approach taken in this study for characterizing the recharge behavior of the middle Santa Cruz River relies on the following process: review of existing literature, collection of pertinent field data, aerial and land-based photographic analysis, consultation with other researchers as well as local residents familiar with the study area, and application of hydrologic theory and numerical models for estimating streambed hydraulic conductivity. The final objective of this research is to derive a mathematical model that describes the variation of streambed conductivity for the low-flow channel (during interstorm periods) with time and temperature. Subsequent chapters are devoted to detailed discussion of these individual activities.
CHAPTER II. BACKGROUND

HISTORY OF THE SANTA CRUZ RIVER NEAR TUCSON

The Santa Cruz River drains 22,225 square kilometers (km²) (8,581 square miles (mi²)) in southern Arizona and northern Sonora, Mexico at its confluence with the Gila River (Figure 1). The river originates just north of the U.S.-México border in the San Rafael Valley of southeastern Arizona. It takes a brief excursion southward into México before
heading west and then reversing course to flow north back into Arizona over a total distance of 360 kilometers. While portions of the river upstream of Tucson flow perennially or intermittently, the Santa Cruz near and downstream of Tucson is typical of large, ephemeral rivers in the desert southwest.

Like many other major valleys in the western United States, the Santa Cruz experienced a period of pronounced arroyo entrenchment in the late 19th century (Betancourt and Turner, 1988; Parker, 1993). Prior to that time, the Santa Cruz River upstream of Tucson was a "shallow, narrow channel in an active flood plain marked by gentle swales and ridges" (Parker, 1993). The arroyo cutting of the past century has produced a Santa Cruz River channel that is now up to 10 meters (m) (33 feet (ft)) below its historical flood plain and whose width exceeds 580 m (1900 ft) in places (Betancourt and Turner, 1988).

Prior to the mid-nineteenth century, Native Americans and a few Spanish missionaries comprised most of the human inhabitants of the Tucson basin area. Residents were drawn to the perennial sources of surface water south of present-day Tucson, and made use of gravity-feed ditches from cienega (marsh) areas to irrigate small crops. Early European settlers in the basin began developing local water supplies for purposes of irrigation and power generation (water wheels). They dammed the exits to cienegas along the river to convert the marsh areas to small lakes. Without occasional dredging, sediment began to fill these lakes, thereby steepening the gradient of the river channel. The lakes also
had the effect of allowing sediment to settle out of the surface water so that, when the dams broke, the clear, high velocity flows were highly erosive (Betancourt, 1988).

In 1887, Tucson resident Sam Hughes embarked on a scheme to tap the underground water supply and bring it to crops north of town. He dug a ditch, originally planned to be 6 m (20 ft) wide by about 26 km (16 mi) long, to intersect the water hidden beneath the dry surface of the Santa Cruz River channel. He never fully excavated the ditch, as he anticipated that natural flooding would finish his work for him. In late 1889, photographer Henry Buchman documented the initiation of erosion at the head of Hughes' ditch during a minor flood event. By the following summer (1890), extensive flooding along the Santa Cruz produced significant overbank flows covering areas up to 549 m (1800 ft) wide and 3.6 m (12 ft) deep adjacent to Tucson. On August 4, 1890, the flood waters began to widen Hughes' ditch, and residents reported watching in alarm as the headcut migrated upstream at a rate of 27 m (90 ft) per hour toward Congress Street (Betancourt and Turner, 1988).

Other headcuts in the previously unincised river channel were reported as early as 1871. As in the case of the small lakes, sediment eroding from these headcuts presumably filled in marsh areas downstream, steepening the river gradient. These areas of steeper gradient would be more susceptible to headcut migration (Parker, 1993). Consequently, the combination of headcut initiation, dams, and excavated ditches in the riverbed, precipitated a devastating and irreversible incision of the Santa Cruz channel. Not long after the dramatic
upstream migration of the headcut of the Sam Hughes ditch in 1890, residents began to recognize that their water supply was in peril:

*The shortage in the Tucson water supply is said to result from the deep wash in the Santa Cruz Valley which extends far above Silver Lake Hotel. The deep wash is drawing off the underflow which previously formed a vast underground reservoir and from which the city water wells drew their supplies.*

- Arizona Daily Star, June 24, 1905
  (cf. Betancourt and Turner, 1988)

The progressive deepening (degradation) of the channel initially sped the release of water downgradient from the Tucson area by conveying it along the surface instead of through the porous material of the aquifer bringing a rapid end to perennial surface water in the Tucson area. Native agriculturalists were forced to resort to their own water works: Papago Indians had an artificial channel constructed to connect the Santa Cruz channel with the entrenched Spring Branch. This channel's grade, about twice that of the natural channel, only compounded problems by allowing the Spring Branch headcut to migrate rapidly through the artificial channel and upstream on the Santa Cruz (Betancourt and Turner, 1988).

Parker (1993) notes that the Santa Cruz's long history of channel instability derives from the combined influences of channel morphology, "hydrologic and climatic factors such as magnitude, duration, intensity, and frequency of precipitation and floods," as well as topography, geology, hydraulics, and artificial controls. In studying channel change on the Santa Cruz, Parker (1993) observes that degradation dominates the vertical change, and that
meander migration, avulsion and meander cutoff, and channel widening contribute to lateral changes.

As a result of these dramatic geomorphologic changes on the Santa Cruz near the turn of the century, residents of Tucson began to react to the new and evolving status of the Santa Cruz. In place of the ill-defined arroyo with a broad, active flood plain and perennial surface flows in some areas, the Santa Cruz had become a deeply incised, strictly ephemeral channel. The prospect for urban development along the Santa Cruz in Tucson instigated efforts to control the unruly river prone to flash floods. As early as 1915, dynamite was used to redirect flood flows in the Santa Cruz in efforts to save the Congress Street bridge (Betancourt and Turner, 1988). In 1935, a federal work force straightened the channel between San Xavier Mission and Congress Street. As part of this effort, long pilot channels were constructed across six of the major bends. Revetments, constructed mostly of old automobile frames, diverted the flows out of the main channel. In 1950, the city of Tucson initiated a program of landfill disposal to replace the practice of incinerating garbage. As of the late 1980's, nearly a million tons of garbage, as well as overburden from highway construction in the 1960's, had been buried either in the channel or on the adjacent floodplain. Channel modification and stabilization were carried out in 1982 to increase the carrying capacity of the river at Congress Street to 1,274 cms (45,000 cubic feet per second (cfs)) (Betancourt and Turner, 1988).
Human alterations to the channel in and near Tucson have artificially narrowed the channel to roughly half of its natural width near Congress Street in downtown Tucson. Artificial channel narrowing, straightening, and bank protection have had several impacts on stream morphology. In general, removing meanders from a stream or river and narrowing its channel increases the longitudinal velocity of flood waters. Covering banks with concrete eliminates banks as a source of sediment, thereby leaving the channel bottom as the only remaining source. In spite of man's efforts, flood waters still adhere to the rules of nature. The sedimentary processes of aggradation and degradation express the conditions of the water at various spatial and temporal locations. Since the 1950's, the Santa Cruz channel has undergone severe (several meters) degradation in Tucson, while downstream reaches (Cortaro, Marana) have experienced a period of aggradation (Parker, 1993).

**HISTORY AND IMPACT OF EFFLUENT DISCHARGE TO THE SANTA CRUZ RIVER**

From the 1950's to the late 1960's, most of Tucson's treated sewage effluent (from the Roger Road Waste Water Treatment Plant) was used to irrigate farmland in the Cortaro area (Schmidt, 1972). Irrigation with effluent in the Cortaro area ceased in 1970, and in 1971, a second sewage treatment plant came into operation near Ina Road and the Santa Cruz River (Ina Road Wastewater Pollution Control Facility (WWPCF)). Without demand for the effluent from local farmers, most of the excess effluent was discharged into the Santa Cruz River channel. Tucson's effluent production during the 1980's and 90's has mirrored its soaring population growth, requiring both plants to operate at or near capacity for many
months of the year. Figure 2 shows historical rates for both effluent treatment and effluent discharge to the Santa Cruz River. As the figure shows, effluent discharges to the Santa Cruz in 1988 were nearly double the 1970 value, and comprised over 90% of the total effluent production of 74 million cubic meters per year (MCM/yr) (60,000 acre-feet per year) (Schmidt, et al, 1989; De Cook, 1970)).

The impact of perennial effluent flows downstream of Roger Road in Tucson on the Santa Cruz channel include increased channel roughness due to vegetation and increased incision in the low-flow channel. The perennial flow of effluent varies in longitudinal extent from roughly 6 kilometers (3.7 mi) in the storm season (winter or monsoonal) to over 40 km (24.8 mi) in the dry, interstorm periods (spring/early summer and/or fall). The early summer flows sustain vegetation growth along the low-flow channel that would otherwise not exist. While this vegetation is susceptible to elimination by very large flood flows, it does serve to slow flows of moderate size, and possibly enhances recharge rates in the process. Due to the relatively low sediment load of the effluent stream, the low-flow channels (especially within a few kilometers of the discharge outlets) tend to persist until disrupted by a large storm event. Ultimately, the low flows have become well entrenched along the banks in some areas partly because of the continued channelization efforts by sand and gravel pit operators at Ina Road. This entrenchment has produced significant erosion and undercutting of bank protected areas near Cortaro Road. In 1993, a large portion of the concrete protection on the west bank of the river collapsed. In order to repair the bank, workers dug
History of Effluent Production in Tucson, Arizona

Legend
- Total Treated Effluent
- Effluent Discharged to Santa Cruz River

Year
- 1940
- 1960
- 1980
- 2000

Effluent Volume (MCM)
- 80
- 70
- 60
- 50
- 40
- 30
- 20
- 10
- 0
a temporary trench to divert the low flows toward the center of the riverbed. Within a few months, winter floods had mobilized the channel bottom, and the low flow channel meandered back to its original position, immediately adjacent to the west bank.

**Climatological Impacts on the Santa Cruz**

The three types of flood events affecting the Tucson area are generated by three types of seasonal storms which occur in summer, winter, and early fall. The summer floods derive from monsoonal storms - violent but brief convective cells; the winter floods stem from slow-moving, frontal storm systems of low-intensity; and the fall floods, including the record-breaking flood of 1983, are generated by tropical storms or other unusual weather events. Betancourt and Turner (1988) report that for the 68-year period of record from 1915 - 1981, 72 percent of all annual flood peaks occurred during July and August, 19 percent during September and October, and 9 percent in the November-through-February period. The record of peak discharges on the Santa Cruz at Congress Street (near downtown Tucson) reflects a lack of uniformity in seasonal flood peaks, and a general increase in peak magnitudes since about 1960 (Hirschboeck, 1985). The seasonal variance in annual flood peaks complicates flood frequency analysis but may explain the general trend toward larger peak flows. Other factors, such as reduced channel storage as a result of progressive channelization and possibly greater runoff produced by urbanization, may also contribute to the trend.
Webb and Betancourt (1992) document three distinct periods of flooding history on the Santa Cruz this century. The first, prior to 1930, is characterized by generally variable flow conditions, with more than half of all floods resulting from fall or winter storms. The second period, from 1930 to 1959, exhibited less variability in flows, but with almost 90% of all floods generated by summer monsoons. The third period, 1960 to 1986, saw highly variable annual floods, with frontal systems or tropical cyclones generating almost half of all floods.

This apparent change in seasonality for flood-producing meteorological events carries with it the implication of a change in character of the floods themselves. Since winter storms are generally frontal in nature, they tend to last longer, are less intense, and cover a larger area than summer monsoon storms. These traits, combined with typically wetter antecedent moisture conditions than in summer, make winter floods less likely to be sediment laden and thus, more erosive than summer floods (Parker, 1993; Burkham, 1970a). The largest Tucson flood on record occurred in October of 1983. The U.S. Geological survey estimated the flood crest at approximately 1,492 cms (52,690 cfs), more than twice as large as the largest previous flood recorded in October of 1977. While the bank-protected reach of the river near downtown Tucson suffered relatively little damage, uncemented banks downstream experienced overbank flows up to 2 meters (6.6 ft) deep and some areas of the channel widened beyond the previous maximum widths recorded after the 1915 flood (Betancourt and Turner, 1988).
CHAPTER III. REVIEW OF RELEVANT BACKGROUND MATERIAL

SURFACE WATER

In desert environments, where virtually all groundwater recharge occurs either at mountain fronts or through alluvial riverbeds, research on natural recharge rates usually encompasses the study of transmission losses through streambeds. Interest in artificial recharge has produced another body of literature on ponded infiltration rates, where horizontal flow velocities and geomorphologic disruptions due to natural storm flows do not apply. Nevertheless, issues important to ponded infiltration projects, such as the development of a clogging layer on the sediment surface, and water quality transformations during recharge, have natural analogs in river recharge environments and will be considered in this review.

In arid region stream studies, "transmission losses," or abstractions from the flowing stream between two points of measurement, comprise the upper limit on streambed recharge. Unfortunately, transmission losses provide only an indirect measure of recharge because some water lost between upstream and downstream points may evaporate from the open channel or wetted soil or may be lost to plant transpiration. Even without evaporation and transpiration losses, the water that does infiltrate may not immediately penetrate vertically to the water table, but may instead be held in vadose zone storage before eventually draining.
through the soil profile (Wilson and DeCook, 1968). Regardless of these shortcomings, transmission losses provide the most direct measure of potential streambed recharge.

Research on recharge processes in arid environments focuses primarily on two topics: rate of infiltration into the soil zone, and water quality transformations during infiltration from the surface through the soil zone to the groundwater zone. Variations on these themes appear in studies on the propagation of flood waves down ephemeral alluvial channels (related directly to the rate of abstraction from the surface into the subsurface), and in studies related to tracing groundwater back to a surface water source via a chemical or isotopic signature (related to the chemical and isotopic transformations, or lack thereof, occurring during recharge). The surface water component of this study seeks to quantify the hydraulic conductivity of the streambed at specific points in time and to determine the magnitude and rate of change in this value in response to natural flood flows as well as during interstorm periods. While no water quality measurements were made in this study, the issue of water quality changes during streambed infiltration merits some discussion because it pertains directly to the mechanism for changing hydraulic conductivity of the streambed. Surficial clogging of the streambed limits infiltration during interstorm periods, and may play an important role in changing the quality of the water infiltrating from the surface to the groundwater zone.

While some authors distinguish between the vadose zone and the unsaturated zone by noting that the capillary fringe area is saturated but under negative pressure (therefore inappropriately relegated to part of the "unsaturated" zone), this paper will use the terms soil zone, vadose zone, and unsaturated zone interchangeably, as is customary in much of the literature.
**Surficial Clogging During Recharge**

A survey of the literature indicates that two general mechanisms apply to streambed clogging. The first is inorganic, and is generally described as a function of stream velocity, suspended solids concentration, suspended particle size distribution as well as of the hydraulic and geometric properties of the channel and bed material. The second clogging mechanism involves microbiological activity and/or chemical reactions.

**Velocity Effects**

Matlock (1965) conducted several tilting bed laboratory flume and field experiments with the objective of determining relationships between silt content of water, velocity of flow, and infiltration rate of sand bed. His flume experiments yielded the general conclusion that infiltration rate increases with velocity in the range of 0.6 - 1.5 meters per second (m/s) (2 to 5 feet per second (ft/s)), regardless of the suspended sediment concentration in the water. For velocities below 0.6 m/s (2 ft/s), he found that clear water infiltrated faster than at higher velocities (where bed movement occurred), but that infiltration rates decreased immediately in the presence of silty water. These findings generally agree with later work done by Marsh (1968), whose studies were based on field measurements, and Cunningham, *et al* (1987), who conducted laboratory flume experiments. Cunningham, *et al* (1987) found that infiltration ratios (final infiltration rate divided by initial rate) were highest at velocities below those which caused disturbance of the bed material.
Both Matlock (1965) and Cunningham, et al (1987) reported that high suspended sediment concentrations at low velocities may result in the deposition of a fine silt layer over the sand bed, eventually eliminating bed motion. Burkham (1970b) observed that the beds of ephemeral streams become coated with fine-grained material during periods of low flow, and surmised that this layer on the streambed surface probably limits infiltration velocity.

This fine layer of silt has an associated hysteresis effect in that a higher velocity flow than that which deposited it is required to subsequently remove it (Matlock, 1965 and Cox and Stephens, 1988). In related work, Burkham (1970b) noted that for a short duration flow event, initial infiltration velocities are low due to the layer of fines deposited by preceding flows. He further postulated that since infiltration velocity increases during the ascending limb of the flow event, a curve of infiltration velocity versus time should mimic the shape of the curve plotting stream depth (or stream velocity) versus time. He then used the assumption that infiltration velocity is proportional to stream depth to develop equations for relating infiltration rates to streamflow rates in perched streams (Burkham, 1970b).

Several authors (Matlock, 1965, Cunningham, et al, 1987, Bouwer, 1978, and others) note that at velocities above a certain threshold level (about 0.30 to 0.61 m/s (1 to 2 ft/s)), the mechanism of inorganic clogging changes as a result of turbulence in the system. At such velocities, clogging appears to derive from an "armoring" mechanism, whereby a thin suspended sediment layer is impacted into pore spaces (Cunningham, et al, 1987). Bouwer

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Infiltration velocity refers to the rate at which water infiltrates (vertically) into the underlying sediments.
and McDowell-Boyer, et al (1986) described sediment-clogging processes in terms of "straining" as fine particles filter through the soil until encountering a filter medium (such as a fine layer) or as larger sediment particles are mechanically intercepted by smaller pores. They also describe how a surface mat may develop on top of the sediment layer. This mat, in turn, strains out fines on top of the sediment layer which then contribute to the hydraulic resistance of the clogging layer. Bouwer (1989) further points out that higher hydraulic head above the clogging layer serves to compact the layer and reduce hydraulic conductivity.

**Suspended Solids Effects**

With flume experiments, Matlock (1965) and Cunningham, et al (1987) determined that the concentration of suspended solids in the surface water has little effect on infiltration rate as it relates to stream velocity. However, Matlock (1965) reported that, for constant velocity in an alluvial stream channel, infiltration rate is inversely proportional to suspended sediment concentration. Cunningham, et al (1987) observed that well-sorted, coarser suspended sediments produced less clogging at the same concentration and velocity than finer, better-graded suspended sediments. Rice (1974) observed significant soil clogging with secondary effluent when the suspended organic solids content exceeded 10 milligrams per liter (mg/l). In their column studies of infiltration of primary and secondary effluent, Quanrud, et al (1995) also attributed some soil clogging to suspended solids deposition.
Biological/Chemical Effects

Schuh and Shaver (1988) relate the clogging effects of microbiological activity in a recharge basin to: 1) biomass formation, 2) solid microbial byproducts, and 3) gases formed during respiration. Rice (1974) and Bassett (pers. comm. cf. Wilson and Gerba, 1993) also attributed soil clogging during recharge of effluent to entrapped soil gas produced by biological activity. Gases produced by chemical processes such as denitrification (\(N_2\), NO, and \(NO_2\)) may also contribute to soil clogging (Oberdorfer and Peterson, 1985). Several authors (Allison, 1947; Nevo and Mitchell, 1967; Wood and Bassett, 1975; Wilson and Gerba, 1993, and others) have observed the buildup of polyuronides and polysaccharides (components of bacterial cell capsules) in the soil matrix during recharge as a consequence of microbial activity. Wood and Bassett (1975) and Mitchell and Nevo (1964) conclude that the observed reduction in hydraulic conductivity associated with the buildup of polysaccharides in the soil is likely caused by the accumulation of dead cells and bacterial wastes in pore spaces, and not by the inorganic precipitation of ferrous sulfide.

The formation of an algal mat on the soil surface is the most conspicuous indication of biomass activity during recharge. Webb and Watson (1978) demonstrated a diurnal decrease in the hydraulic conductivity of a recharge basin due to oxygen production during algal photosynthesis in the surface mat. As noted earlier, an algal mat also has the capacity to act as a mechanical strain for fines settling out of suspension. The straining effect as well as the problem of entrapped gases have motivated some researchers to inhibit algal growth
in their attempts to enhance infiltration or isolate inorganic clogging effects in both column studies and in active recharge basins (Light, 1993; Okubo and Matsumoto, 1979). Furthermore, pH changes resulting from algal metabolism or sulfate-reducing bacteria (Wood and Bassett, 1975) may promote precipitation of carbonates and subsequent clogging (Bouwer, 1989).

Okubo and Matsumoto (1979) performed column experiments to test the importance of varying infiltration rates on biological clogging and water quality changes during artificial recharge of secondary effluent. Keeping everything else equal among three columns (filled with 0.25-0.42 millimeter (mm) sand), the authors applied different hydraulic gradients to the columns. The resulting infiltration rates progressed through four stages: 1) rapid decrease with reduction in dissolved oxygen (DO) in first 10 days due to aerobic microbial growth; 2) constant or slight increase from day 10 to day 20 resulting from DO-limited growth of aerobic microbes; 3) rapid decrease between days 20 and 30 coinciding with anaerobic degradation in the surface layer; 4) slow decrease after 30 days, with infiltration rate of less than 0.1 meter per day (m/d) (0.33 ft/d) in the slower columns. From their observations, Okubo and Matsumoto (1979) developed a biological clogging model based on the Hagen-Poiseuille's law for flow through tubes. This model incorporates assumptions of uniform narrowing in capillary tubes with development of biological clogging and of proportional decrease in porosity with cumulative discharge through the column.
Schumann and Galyean (1991) speculated that increased biological activity on the surface, "caused by nutrient-rich sewage effluent and increasing ambient air temperatures," was responsible for decreasing streambed infiltration capacity over time in the effluent stream of the Santa Cruz River near Tucson. They also acknowledged that infiltration capacity may be reduced by the deposition of suspended fine-grained organic or inert particles in the effluent on the surface of the streambed during periods of sustained effluent discharge.

**Schmutzdecke**

In their extensive review of recharge studies and recharge potential for the Southwest Alluvial Basins (SWAB/RASA) project, Wilson, et al (1980) note that:

"A black layer is frequently observed in shallow profiles underlying channels used for effluent disposal. This layer apparently consists of ferrous sulfide and other reduced compounds, as well as the filamentous byproducts of microbial metabolism."

This black, odoriferous layer is anaerobic (Sebenik, et al, 1972; Wilson, et al, 1975; Wood and Bassett, 1975; Okubo and Matsumoto, 1979) and has come to be known as a "schmutzdecke," which translates roughly from German to "dirty layer." Bouwer (Agric. Research Service, Phoenix, Arizona, 1996, pers. comm.) notes that the term schmutzdecke is well known in water treatment literature, and generally connotes a beneficial biofilm or biofilter. Considering this interpretation, schmutzdecke in open channels and spreading basins encompasses the entire suite of biological mat-building processes, and may be inseparable from the accompanying sedimentation and chemical precipitate effects that
contribute to the development of clogging layers.

The effects of this layer in alluvial stream systems and in artificial recharge basins remain somewhat uncertain, but researchers generally agree that it contributes to diminished infiltration rates, and that reducing bacteriological and chemical reactions take place in or near this zone. Laverty, et al (1961) found that such an anaerobic layer reduced flow by 88 percent in seven weeks. Herbert (1976) did a series of column experiments with secondary effluent to try to replicate the development of a \textit{schmutzdecke} in a controlled environment. Specifically, he sought to evaluate the effects of alluvial deposits on infiltration rates and on denitrification of effluent. In his first experiment, Herbert packed the columns with river sand and flooded them for 28 days. In his second experiment, the columns were packed with gravel and flooded for 64 days. In both cases, Herbert (1976) reported the development of a black layer in the columns shortly after introducing effluent to the columns. He also observed an inverse relationship between the buildup of the black layer and infiltration rates, but he attributed the reduced infiltration rates to clogging by suspended solids in the effluent, not to the black layer.

Wood and Bassett (1975) observed the development of a 10-20 centimeter (cm) (3.9 - 7.9 inch (in)) thick "black, gelatinous zone" two to five centimeters under the surface of the spreading grounds for an artificial recharge experiment in Lubbock, Texas. The recharge water was not secondary effluent, but rather sodium chloride, sodium sulfate water from a
reservoir in north Texas, with suspended solids concentrations rarely exceeding 10 mg/l. Infiltration rates for the experiment during the period April 12 - November 15, 1972 revealed the same three stages suggested by Allison (1947). In the early phase, the infiltration rate dropped, probably as a result of structural changes in the soil resulting from swelling and dispersion (Wood and Bassett, 1975). The maximum infiltration rate appeared in phase two (about 50 days after the minimum observed infiltration rate), as entrapped soil gases were dissolved, thereby increasing the effective pore size. The third phase (during the last 3 months) exhibited a rapid decrease in infiltration rate. Wood and Bassett (1975) use Allison's (1947) explanation of this decrease: a result of "the accumulation of bacterial cells and metabolic waste products, a slow disintegration of soil aggregates, and dispersion due to the attack of microorganisms on organic material that binds the soil into aggregates." Wood and Bassett (1975) also tied the sharp initial decline in infiltration rates to the growth of anaerobic bacteria, attributable to low dissolved oxygen content in the recharging water. Interstitial water extracted from the black reducing zone under the surface of the spreading basin revealed the presence of sulfate-reducing bacteria. No such bacteria were found in the light colored zone in the first 2-5 cm (0.8 - 2.0 in) below the soil surface.

In column experiments designed to simulate soil-aquifer treatment (SAT) of effluent during percolation through the unsaturated zone, Quanrud, et al (1995) applied effluent treated to varying degrees (primary, secondary, chlorinated secondary) to 1-meter (3.3-ft) bench scale columns over a two-year period. Their experiments focused mainly on water
quality improvements during SAT and are reviewed in the following section on water quality transformations during effluent recharge. During the course of their study, however, several observations were made regarding the development of *schmutzdecke* in the columns. The sediments in the columns consisted of either repacked or in-tact soil cores from recharge sites in southern Arizona. Effluent was brought to a temperature of 25°C before being introduced to the columns and maintained at 25-cm constant head (aerated with an aquarium pump) above the soil surfaces. The columns were operated in wet-dry cycles similar to those employed in artificial recharge basins (7 days wet followed by 7 days dry), and were exposed to 12 hours per day of artificial light to stimulate algal growth. *Schmutzdecke* formed in some of the Agua Fria River sand columns after about 4 to 6 wet-dry cycles, and in the Sweetwater sandy loam columns after one cycle, with researchers allowing 10 cycles to fully establish the layer in all columns. For the coarsest of the sands used (Agua Fria River sand), infiltration rate dropped from 3.0 meters per day (m/d) (9.8 ft/d) to 0.3 m/d (1.0 ft/d) over the course of one wetting cycle after the *schmutzdecke* was established. In the columns with finer-grained Sweetwater sandy loam (with saturated hydraulic conductivity roughly an order of magnitude lower than that of the Agua Fria River sand), initial infiltration rates of 0.6-1.2 m/d (2.0-3.9 ft/d) dropped to 0.1-0.3 m/d (0.3-1.0 ft/d) after 7 days of wetting. *Schmutzdecke* in the Agua Fria River sand was confined to the top two cm (0.8 in) of soil, whereas the Sweetwater sandy loam exhibited a zone of head loss over 12 cm (4.7 in). This finding suggests that the vertical extent of the clogging layer may be related to grain size in a manner somewhat analogous to the relationship between capillary length and grain size.
Esposito (1993) found elevated total iron and manganese in turbid water samples from the seepage zone underneath the effluent stream in the Santa Cruz River near Tucson. Since the concentrations of these metals were much lower in underlying groundwater, he suggests that the black staining in the shallow soil zone may result from precipitation of iron and manganese.

**Water Quality Transformations During Effluent Recharge**

Water quality improvements during effluent recharge have been widely documented in the literature. Because this study focuses on mechanical components of recharge and not on chemical aspects, the review of this topic will be fairly brief. For very thorough descriptions and literature reviews on the topic of soil-aquifer treatment in the context of effluent recharge, the reader should refer to Sebenik, et al (1972), Wilson, *et al* (1975), Sebenik (1975), Bouwer (1978), Schmidt (1980), Wilson, *et al* (1980), Esposito (1993), and Wilson, *et al* (1995).

Three types of studies describe most of the research done on water quality transformations during effluent recharge: 1) laboratory column studies, 2) spreading basin experiments, and 3) in situ stream-aquifer characterization studies. Okubo and Matsumoto (1979) and Quanrud, *et al* (1995) made detailed analyses of water quality parameters before and after recharge of effluent through soil columns. Herbert (1976) also performed column experiments to evaluate the effects of alluvial deposits on denitrification of effluent. Wilson,

**Nitrogen Transformations**

Review of the studies listed above reveals that the contaminants of primary interest (because of their toxicity to humans) during recharge of sewage effluent consist of nitrogen forms (mostly ammonia and nitrate), trace organics (especially organic halides), viruses, coliform, and other pathogens. Secondary effluent usually contains more than 20 mg/l total-N, with roughly 60% of that in the form of ammonium ion (NH₄⁺-N), and the remainder as
nitrate or organic nitrogen (Esposito, 1993; Schmidt, et al., 1989). Organic nitrogen in effluent undergoes ammonification by hydrolysis reaction to produce ammonium carbonate (Esposito, 1993). Because ammonium is toxic to aquatic life, conversion of ammonium to nitrate is desirable for effluent being discharged into a stream environment. Nitrification occurs in a two step process whereby ammonium is biologically oxidized to nitrite (NO$_2^-$) and then to nitrate (NO$_3^-$):

\[ 2\text{NH}_4^+ + 3\text{O}_2 = 2\text{NO}_2^- + 4\text{H}^+ + 2\text{H}_2\text{O} \]

\[ 2\text{NO}_2^- + \text{O}_2 = 2\text{NO}_3^- \]

Nitrification is relatively easy to accomplish in sewage treatment, and may range from minimal to virtually complete during treatment, but removal of total-N is very costly (Esposito, 1993). Ammonia volatilization, which produces ammonia (NH$_3$) gas, may also occur during transit within a treatment plant. This reaction, which is driven to the right by a pH increase but occurs minimally under conditions of neutral pH (typical of effluent), is expressed as:

\[ \text{NH}_4^+ + \text{OH}^- = \text{NH}_3 (\text{g}) + \text{H}_2\text{O} \]  

(Stratton, 1968).

Other proposed mechanisms for removal of NH$_4^+$ from secondary effluent in an open stream include uptake by plants and microbes, adsorption on the cation-exchange complex of clays and organic matter, and fixation within the crystal lattices of clay minerals and on organic matter (Wilson, et al., 1975). Under aerobic conditions, sorbed NH$_4^+$ may be oxidized to NH$_3^-$ which may then subsequently be leached from the exchange site and
undergo further reactions (Esposito, 1993).

While not as toxic to aquatic life as ammonium, nitrate (the product of nitrification of ammonium) presents a health hazard, in itself. The national safe drinking water standard for nitrate is 10 mg/l nitrate (NO$_3^-$) as nitrogen (N) (or 45 mg/l nitrate as nitrate). Schmidt, et al (1989) report nitrate concentrations of 14 mg/l and less than 1 mg/l for the two individual treatment plants that discharge treated effluent to the Santa Cruz River in Tucson. This problem has stimulated much interest in the potential effectiveness of soil-aquifer treatment (SAT) for removing nitrate during effluent recharge. Denitrification presents the most likely mechanism for nitrate removal in SAT. Denitrification occurs by the biological reduction of nitrate to nitrogen gas (N$_2$). In simplified forms, the denitrification reactions can be written as:

$$\text{NO}_3^- + \frac{1}{3} \text{CH}_3\text{OH} = \text{NO}_2^- + \frac{1}{3} \text{CO}_2 + \frac{2}{3} \text{H}_2\text{O}$$

$$\text{NO}_2^- + \frac{1}{2} \text{CH}_3\text{OH} = \frac{1}{2} \text{N}_2(\text{g}) + \frac{1}{2} \text{H}_2\text{O} + \text{OH}^- + \frac{1}{2} \text{CO}_2.$$

In the case of effluent, methanol provides a source of organic carbon for these reactions (Esposito, 1993). Denitrification also requires reducing conditions (absence of oxygen) and the presence of denitrifying, or reducing, bacteria.

Studies of groundwater quality in the vicinity of the Santa Cruz River northwest of Tucson indicate significant reductions (typically 60-80%) in nitrate (or total-N) and other nitrogen forms between the effluent stream and the receiving groundwater (Wilson, et al,
1975; Schmidt, 1988; Schmidt, et al, 1989; Esposito, 1993; Wilson, et al, 1995). Bouwer, et al (1980) reported consistent removal of 65% of total-N during artificial recharge at the Flushing Meadows Project in Phoenix as long as the hydraulic loading rate\(^3\) was limited to about 61 meters (200 ft) per year. The losses in ammonium and ammonia are generally attributed to sorption, ammonia volatilization, and nitrification. Esposito (1993) suggests that higher ammonium and organic nitrogen concentrations in shallow soil zone water under the effluent stream in the Santa Cruz River than in the underlying groundwater indicate that sorption is the mechanism for removal of these substances during recharge. Small quantities of both of these constituents in some groundwater samples from below the river suggest that the sorption capacity of the vadose zone may actually be exceeded in some areas (Esposito, 1993).

Several researchers have observed that nitrification rates in the effluent stream are directly related to physical and geometric properties of the channel, particularly as they relate to the variation in wetted area between high and low flows (Sebenik, et al, 1972; Sebenik, 1975; Wilson, et al, 1975).\(^4\) They propose that areas of the channel which undergo daily drying (during low diurnal streamflows) have a greater opportunity for oxygen transfer, and thus, for nitrification to occur.

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\(^3\) Hydraulic loading rate = recharge volume/(recharge area*total number of recharge days).

\(^4\) The fluctuating discharges from the sewage treatment plants produce strong diurnal fluctuations in effluent stream discharges.
Denitrification is the most prominent mechanism for observed nitrate removal between the effluent stream and underlying groundwater (Wilson, et al, 1975; Esposito, 1993; Wilson, et al, 1995). Higher nitrate levels in groundwater underlying areas that were irrigated with sewage effluent compared with nitrate concentrations in groundwater associated with effluent recharge from the Santa Cruz River offer some evidence that denitrification occurs during streambed recharge (Schmidt, 1972; Schmidt, 1973; Schultz, et al, 1976). Martin (1980) concluded from his examination of nitrogen isotopes that elevated nitrate and boron in the Cortaro area groundwater resulted from recharging effluent and not from artificial fertilizers.

Esposito (1993) states that the Santa Cruz River channel, in times when a perched water table exists above the black, anaerobic layer, constitutes an "effective denitrification reactor." By sampling both the effluent stream and water from the seepage zone above the subsurface clogging layer below the flowing stream, Esposito (1993) concluded that denitrification takes place in the first 0.3 to 0.6 meters (1 to 2 ft.) of percolation. The conditions required for this reaction -- an energy source (organic carbon), nitrate, and anaerobic conditions -- have all been documented in the seepage zone below the effluent stream. This result is corroborated by Wilson, et al (1995), who used $\text{NO}_3^-$ to $\text{Cl}^-$ ratios as an approximate measure of denitrification. They found that most nitrate loss occurred in the perched groundwater zone, in the upper 1.5 m (4.9 ft.) of their recharge basin floor.

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Schmidt (1973) also speculates that significant nitrification may occur during application of treated effluent for irrigation.
Esposito (1993) suggests that denitrification in the reducing zone below the effluent stream in the Santa Cruz River may be responsible for as much as 30% (or 5 mg/l) of all nitrogen losses between the surface water and the groundwater. Wilson, et al (1975) attributed a significant drop in the removal of total-N between the effluent stream and the groundwater to storm flows that scoured out the black, anaerobic clogging layer in the channel.

Several studies report a persistent "nitrate flush" phenomenon that occurs at the start of every wetting cycle in cases where artificial recharge is managed with wet/dry cycles (Bouwer, 1970; Wilson, et al, 1992; Wilson, et al, 1995; Quanrud, et al, 1995). Wilson, et al (1995) explain this flush as occurring in a three phase process: 1) NH$_4^+$-N is adsorbed to clays or organic matter during wetting; 2) sorbed NH$_4^+$-N is nitrified (converted to NO$_3^-$) during the drying cycle; and 3) NO$_3^-$ produced during nitrification is then leached at the beginning of the next wetting cycle. Bouwer (1975) suggests that no nitrogen would be removed if flooding periods were excessively long because the resulting absence of oxygen would prevent nitrification of ammonium, eventually causing the cation exchange capacity of the soil to be exceeded. Quanrud, et al (1995) report, however, that significant denitrification occurred in ponded effluent (primary effluent spiked with nitrate) prior to infiltration. Since nitrification occurs in the effluent stream of the Santa Cruz, this finding suggests that both denitrification and nitrification occur simultaneously. Even without denitrification in the surface water, continuous flooding of the stream channel promotes the anaerobic environment necessary for denitrification to occur in the shallow soil zone, so
nitrate production in the surface water indicates that both processes do occur simultaneously within the same stream-streambed profile (Sebenik, et al, 1972; Wilson, et al, 1975).

Other Water Quality Changes

Because chloride concentrations in effluent from the Tucson area sewage treatment plants average about 80 mg/l, high chloride combined with elevated nitrate levels, and in some cases, boron, in groundwater are associated with effluent recharge (Schmidt, 1972; Wilson, et al, 1975; Esposito, 1993).

Esposito (1993) reported a 50% reduction in detectable trace organics between the stream and the groundwater. He also reported a significant statistical difference between effluent and groundwater samples for all constituents analyzed except sulfate and nitrate. Conductance values for the stream effluent and the seepage zone water were essentially indistinguishable during sampling in May through August, 1992 (Esposito, 1993).

A significant reduction in dissolved oxygen (DO) content between the surface water and the seepage zone above the clogging layer in the Santa Cruz River confirms that anoxic conditions exist there (Esposito, 1993). In column studies, Okubo and Matsumoto (1979) observed an initial decrease in DO followed by an increase after about 20 days. From this result, they concluded that aerobic microbial growth occurred in the surface layer for the first 15 to 26 days, with facultative microorganisms becoming dominant thereafter. Chemical
oxygen demand (COD) provides a measure of readily oxidizable materials such as those produced during anaerobic degradation of microbial metabolite on the sand surface (NH$_4^+$, NH$_3$, etc.). Okubo and Matsumoto (1979) observed stable COD concentrations in the outflow from their columns over three months of infiltration, with increased COD removal in columns with lower infiltration rates. They also found that most COD removal occurred in the surface layer, but at lower infiltration rates, a higher percentage of COD removal occurred deeper in the column.

Esposito (1993) reported finding total and fecal coliform bacteria in nearly every sample of river effluent. None of the 98 groundwater samples taken from 12 wells adjacent to the Santa Cruz River contained detectable fecal coliform, and only three out of 98 samples (from three different wells) contained any total coliform. Although chlorinated effluent from the Tucson area sewage treatment plants reportedly contains enteric viruses, no virus was detected among the monitoring well samples except in one case where the well head was inundated after a large flood (Esposito, 1993). Wilson, et al, (1992) found that the most significant virus removal occurs near the soil surface. Esposito (1993) attributes removal of bacteria during recharge to sediment filtration. Removal of virus during recharge probably occurs by sorption and deactivation (Esposito, 1993; Quanrud, et al, 1995).

Quanrud, et al (1995) report that efficiency in removal of organics improved with time, and that it appeared to be independent of flow rate or mean hydraulic retention time.
They interpreted this result to mean that either organisms were very biodegradable or that sorption played a more important role than indicated in previous studies. They also found that organic removal efficiencies steadied out after about 6 wet/dry cycles (84 days), and surmised that these efficiencies were tied to the development of bacterial populations (Quanrud, et al., 1995). In a test to simulate rainwater moving through a recharge zone, Quanrud, et al. (1995) found that 8% of coliphage removed by the soil column was later desorbed by artificial rainwater.

Wilson, et al. (1995) found dissolved organic carbon (DOC) levels reduced by 80% after infiltration through 24.4 m (80 ft) of vadose zone, with most removal occurring in the upper 3.1 m (10 ft) (as a result of enhanced microbial activity). The same study revealed a major reduction in total organic halide (TOX) in the upper 5.2 m (17 ft) of vadose zone, with a total of 40% removal at 24.4 m (80.0 ft) below the soil surface. Groundwater samples from below the spreading basins (water table depth equal to 37 m (121 ft)) indicate a DOC removal of 92% and TOX removal of 85% (Wilson, et al., 1995). In a similar study for the Salt River Project, Wilson, et al. (1992) reported DOC removal of roughly 50% and TOX removal of about 40% for infiltration of secondary effluent through a 6 m (20 ft) profile, with the most significant changes occurring near the surface.
Transmission Losses in the Santa Cruz River

Natural Flows

Burkham (1970a) performed an extensive analysis of infiltration rates for main and tributary channels in the Tucson Basin over the period 1936-1963. For channel reaches with streamgaging stations, measured streamflows were used to: 1) determine relationships between inflow and infiltration rates, 2) produce flow-duration curves, 3) combine parts 1) and 2) to derive infiltration-duration curves, 4) use part 3) to calculate average annual infiltration volume, and 5) to verify these results with a water budget analysis for the individual channel reaches and the composite of all the reaches. For ungaged tributaries, synthetic flow-duration curves and estimated inflow-infiltration relations were derived from information from drainages with similar characteristics and from drainage area. Burkham (1970a) concluded that, on average, about 70% of the total annual inflow to the basin infiltrates and roughly 30% flows out of the basin downstream. He assumed evaporation losses from, and direct gains from precipitation on, the flowing surface waters to be negligible, and therefore attributed all streamflow depletions to infiltration. For the 19.7-km (12.2-mile (mi)) reach of the Santa Cruz River between Tucson (at Congress St.) and Cortaro Rd. (including a 6.9-km (4.3-mi) reach of the Rillito River tributary), Burkham (1970a) described the best-fit relationship between infiltration rate and inflow rate as:

\[ Q_f = 1.4 (Q_{inflow})^{0.8} \]  

(1)

where, \( Q_f \) = infiltration rate (cubic-feet per second (cfs))
\[ Q_{\text{inflow}} = \text{inflow rate (cfs)} \]

Burkham's (1970a) analysis also produced related equations including those for per mile infiltration rate (\( Q_f / \text{mile} = 0.11(Q_{\text{inflow}})^{0.8} \)) and infiltration as a percent of inflow rate (\( Q_f (%) = 140(Q_{\text{inflow}})^{0.2} \)). Burkham's formula yields a natural infiltration rate of about 367,910 m\(^3\)/km (480 ac-ft/mi) per year for a 19.7-km (12.25-mile) reach of the Santa Cruz River between Tucson and Cortaro Road (Burkham, 1970a). This result compares favorably with Osterkamp's (1973) estimated annual recharge rate of 383,240 m\(^3\)/km (500 ac-ft/mi) (for the Santa Cruz River downstream of the Rillito River confluence (upstream of Cortaro Road). Matlock (1965) estimated natural annual recharge through 185 km (115 miles) of stream channels in the Tucson area at 459,888 m\(^3\)/km (600 ac-ft/mi).

Katz (1987) analyzed steady-state infiltration rates in the Santa Cruz and Rillito Rivers in Tucson during February, 1985, after a prolonged wet season. She used video and still aerial photography to delineate the wetted areas of the rivers on February 11, and selected reaches where both upstream and downstream discharges were known for her analyses. After correcting for evaporation losses, she calculated a 672,011 m\(^3\)/d (544.8 ac-ft/day) streamflow loss (or 0.41 m/d (1.35 ft/d) infiltration rate) for a 48-km (30-mi) reach of the Santa Cruz between Continental and Tucson. Applying the February 11, 1985 flow conditions to Burkham's (1970a) formula yielded a volumetric infiltration rate of 352,781 m\(^3\)/d (286 ac-ft/d) and an infiltration velocity of 0.25 m/d (0.81 ft/d) (Katz, 1987). These
values correspond to \(8,431.3 \text{ m}^3/\text{km/d} \) (11 ac-ft/mi/d) or 4.95 million cubic meters (MCM)/km/yr (4,015 ac-ft/mi/year). Katz (1987) attributes the lower rates predicted by Burkham's (1970a) formula to the fact that his formula is based on curve-matching of historic flows (1936-63), and does not take into account any geomorphological channel changes since that time. For approximately the same reach, Matlock (1965) proposed infiltration velocities of 0.91 to 2.0 m/d (3 to 6.7 ft/day) based on estimated flow velocities of 0.61-0.91 and 1.5-2.4 meters per second (m/s) (2-3 and 5-8 ft/s), respectively.

**Effluent Flows**

Table 1 summarizes comparable published transmission loss values for the effluent stream of the Santa Cruz River between the Roger Road Waste Water Treatment Plant in Tucson and the Rillito Narrows (at Avra Valley Road) roughly 19.3 km (12 mi) downstream (north). Matlock (1966) measured effluent flows during a 20-day period in the fall of 1964 when all excess effluent from the Roger Road facility was temporarily diverted to the Santa Cruz River. Aside from residual soil moisture from a large natural flow event about six weeks earlier, the riverbed was dry at the start of the study. The effluent channel near Cortaro Road (about 10.5 km (6.5 mi)) north of the treatment plant) was modified to concentrate flow into an area suitable for developing a stream flow rating curve. Discharges from the treatment plant were measured by Parshall flume and hourly stages taken from a continuous recorder graph. Matlock (1966) determined the wetted area of the channel by conducting periodic traverses across and along the study reach. He based infiltration rates
Table 1.

Published Transmission Loss Rates for Santa Cruz River Effluent Stream North of Tucson, Arizona.†

<table>
<thead>
<tr>
<th>Source</th>
<th>Roger Rd. to Cortaro Rd. (approx. 10.5 km (6.5 mi.))</th>
<th>Cortaro Rd. to Rillito Narrows (approx. 8.8 km (5.5 mi.))</th>
<th>Roger Rd. to Rillito Narrows (approx. 19km (12 mi.))</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>cfs/mi</td>
<td>ac-ft/day/mi</td>
<td>cfs/mi</td>
</tr>
<tr>
<td>Matlock, 1966†</td>
<td>3.3</td>
<td>6.5</td>
<td>1.0</td>
</tr>
<tr>
<td>Sebenik, Cluff and DeCook, 1972b</td>
<td>2.2-3.1</td>
<td>4.4-6.1</td>
<td>3.4-3.4</td>
</tr>
<tr>
<td>Sebenik, Cluff, and DeCook, 1972c</td>
<td>N/A†</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Sebenik, 1975d</td>
<td>4.4</td>
<td>8.7</td>
<td>2.6</td>
</tr>
<tr>
<td>Schumann and Gaylean, 1991e</td>
<td>N/A</td>
<td>N/A</td>
<td>1.4, 1.8, 3.4</td>
</tr>
<tr>
<td>Gaylean, 1996f</td>
<td>3.4</td>
<td>6.7</td>
<td>2.9</td>
</tr>
</tbody>
</table>

†English units are reported in the literature. Conversions can be made by: cfs/mi = 0.0176 cms/km; ac-ft/day/mi = 766.5 m³/day/km.

*Measurements taken over 20-day period in October-November, 1964, roughly 6 weeks after natural flow event.

* Ranges represent measurements taken before and after a summer 85-cms (3000-cfs) peak flow event.

* Measured over the period November 18 - December 3, 1971, following a 14-cms (500-cfs) peak flow event.


* Measurements made on March 23, April 26, and October 24, 1990 during daily minimum flows.

* Values are averages of monthly values from March 1991 to December 1992.

* Values derived from weighted sum of the two segment reaches.

† N/A: not applicable; no data.
on an effective wetted area of about 12% less than the total wetted area because he assumed that essentially no infiltration occurred in areas of silt deposits and algae growth. He corrected for evaporation losses by applying pan evaporation rates with a coefficient of 0.8. Average daily discharges ranged from a minimum of 0.42 cubic meters per second (cms) (15 cubic feet per second (cfs)) to a maximum of 1.1 cms (40 cfs). The effluent stream extended for roughly 22.5 to 25.7 km (14 - 16 mi) downstream of the Roger Road plant (3.2 - 6.4 km (2-4 mi.) past Avra Valley Road) with approximately 64% of upstream discharge infiltrating in the reach above Cortaro Road (Matlock, 1966).

Sebenik, et al (1972) measured effluent flows at the Rillito Narrows (roughly 19.3 km (12 miles)) downstream of the Roger Road treatment plant) by routing the flow through a 1.2-m (4-foot) H-L flume. Upstream flows were recorded with a flume at the treatment plant, and intermediate discharges were obtained from the U.S. Geological Survey's gaging station at Cortaro Road (about 10.5 km (6.5 mi) from the Roger Road facility). Sebenik, et al (1972) observed that the effluent stream length varied from about 17.7 km (11 mi) long during the trough of the diurnal discharge fluctuation to over 40.2 km (25 mi) long during high flow periods. As indicated in Table 1, the authors report streamflow losses for two different periods. For the period November 18 to December 3, 1971, following a natural flow event with a peak discharge of about 14.2 cms (500 cfs) at Cortaro Road, approximately 85% of the effluent discharged from the plant infiltrated between the plant and the Rillito Narrows (at Avra Valley Road).
Flows measured by Sebenik, *et al* (1972) before and after a storm flow of 84.9 cms (3000-cfs) peak flow (at Cortaro Rd.) in the summer of 1972 demonstrate the impact of that natural flow on effluent infiltration rates. They used an additional H-L flume installed at Cortaro Road to measure transmission losses between the Roger Road plant and Cortaro Road, and between Cortaro Road and the Rillito Narrows. Although they observed a 40% increase in infiltration rates in the reach above Cortaro Road after the flood, Sebenik, *et al* (1972) reported no infiltration rate change in the lower reach (Cortaro Rd. to Avra Valley Rd.).

During the following summer (June and July, 1973), Sebenik conducted another series of streamflow loss measurements using the same flumes employed by Sebenik, *et al* (1972) (Sebenik, 1975). In contrast to the values reported by Sebenik, *et al* (1972) (but in keeping with the findings of Matlock (1966)), Sebenik (1975) reported significantly higher infiltration rates in the reach between Roger Road and Cortaro Road than between Cortaro Road and the Rillito Narrows. He also observed that the infiltration rate at high flows (about 2.54 cm/hr (1 in/hr)) was about twice as high as the average infiltration rate (1.2 cm/hr (0.5 in/hr)).

Wilson and Small (1973) measured infiltration losses in a short segment (0.43 km (0.27 mi)) of the effluent stream near the Ina Road Wastewater Pollution Control Facility (WWPCF) outfall to the Santa Cruz River (about 2.0 km (1.25 mi) upstream of Cortaro Rd).
They report an average infiltration velocity of 0.063 cubic meters per second per kilometer (cms/km) (3.6 cfs/mi) which corresponds to a volumetric infiltration rate of 5,442 m³/day/km (7.1 ac-ft/day/mi). This value falls within the range of the values listed in Table 1 for the reach between Roger Road Wastewater Treatment Plant (WWTP) and Cortaro Road.

Schumann and Galyean (1991) conducted a preliminary transmission loss study from March to October of 1990 in the reach between Cortaro Road and the Trico Road (26.2 km (16.3 mi)). This study consisted of manual streamgaging at seven locations in the study reach on three different dates. The large diurnal fluctuations in effluent discharges from the treatment plants complicate the process of measuring streamflow losses. In an effort to measure the same packet of water as it progressed downstream, Schumann and Galyean (1991) gaged the effluent stream during the easily recognizable daily flow minima. Consequently, the transmission losses they recorded represent lower limits for those days because of smaller vertical hydraulic gradients, lower velocities, and smaller wetted perimeters in the stream sections. The authors considered evaporation losses during the roughly 12-hour measurement periods (required for the trough to progress to farthest downstream measurement point) to be negligible compared to the volume of water lost to infiltration during that period. As shown in Table 1 infiltration losses increased slightly between the March and April measurements, even though upstream discharges diminished from 0.81 cms (28.5 cfs) on March 23 to 0.55 cms (19.5 cfs) on April 26. The October values marked a dramatic (~86%) increase over the April values, with the upstream discharge
on October 24 reported as 0.87 cms (30.9 cfs). Galyean (1996) reports that the October measurements were made following a period of high flows, and attribute the high streamflow losses to scouring by flood flows in early October. In addition to specific discharge measurements, Schumann and Galyean (1991) observed a general increase in channel infiltration capacity following high flows in early March. During the summer months, they also observed a general increase followed by a progressive reduction in the infiltration capacity of the streambed indicated by the manner in which the limit of the effluent stream moved progressively downstream.

Galyean (1996) reports the results of a continuation of the streamflow loss study conducted in 1990. Using low-flow streamgaging stations installed near Ina Road, Avra Valley Road, and at Sanders Road (near Marana, Arizona), he monitored streamflow losses on a continuous basis from October 1991 through September 1993. The final product of the study consisted of a 5-part water budget including: 1) volume of effluent discharge, 2) discharge through and past the reach, 3) evapotranspiration losses, 4) open-channel evaporation losses, and 5) infiltration through the streambed.

Galyean (1996) reports that 88.4 - 90.2\% of the effluent discharged to the channel during the study period infiltrated in the 37-km (23-mi) reach between the Roger Road treatment facility and Trico Road. During the period of study, discharges from the sewage treatment plants to the Santa Cruz River increased 8.7\% and total infiltration increased 8.9\%
(Galyean, 1996). Again, Galyean (1996) reported a marked decrease in downstream flows following storm flows. Following these periods, the downstream flows gradually increased with a corresponding decrease in infiltration losses. Galyean (1996) considered open-channel evaporation and plant transpiration losses in the study reach. By using the Blaney-Morin equation to estimate evapotranspiration, he included information on phreatophyte area, open-channel area, proportion and density of vegetation types, water consumption coefficients of phreatophytes and riparian species, annual daytime hours, temperature, and humidity. He applied monthly class A pan evaporation values to open channel area by multiplying by a 0.69 correction factor. Combined calculated evaporation and evapotranspiration losses totaled 3.2-3.9% of the effluent discharged to the stream, with potential open-channel evaporation generally exceeding potential phreatophyte transpiration. These losses were negligible compared to the infiltration losses (72-99%) in the study reach, especially after storm flows. Galyean (1996) reports a 2-8% measurement error associated with standard streamgaging techniques, and additional measurement errors associated with vertical fluctuations in channel geometry and gage-pool control, mainly due to storm flows and human activities in the channel. He proposes, however, that all of these combined errors are much smaller than the reduction in stream discharge downstream (Galyean, 1996).

Transmission Losses - General

The topic of streamflow abstractions (transmission losses) in arid environments and how to estimate recharge from those abstractions has spawned a broad body of literature.
For more a more general survey of these topics, the reader should refer to: Lane, et al (1971), Besbes, et al (1978), Lane, et al (1980), Lane (1972, 1985, 1990), Walters (1990), and Sharma and Murthy (1994a, 1994b).

**SURFACE WATER/GROUNDWATER INTERACTIONS**

In the winter of 1965-66, Wilson and DeCook (1968) conducted field studies to characterize changes in subsurface moisture during influent seepage in the Santa Cruz River, upstream from the effluent discharge point near Roger Road. The experiments entailed observing soil moisture changes via neutron probe access tubes extending from the surface to the water table (average depth to water was 21.3 m (70 ft)) and observing changes in groundwater elevations in observation wells and piezometers. Five natural runoff events occurred in the Santa Cruz River immediately adjacent to the field study site during the period of study (December to February). The total discharge volumes of the individual flow events ranged from 0.090 to 4.54 MCM (73 to 3680 ac-ft), and the longest sustained flow occurred from February 7 to 15, 1966. The unexpected finding that one observation well farther from the river responded more dramatically to river flows than those close to the river demonstrated the influence of geology and permeability on subsurface flow paths. Neutron logs and piezometers indicated the presence of saturated zones in the intermediate vadose zone well above the local water table. Observation well water level changes accounted for only 33% of transmission losses from the river, so roughly 67% of infiltrated water was held in the vadose zone. Moisture logs allowed the authors to monitor the buildup and the
dissipation of the near-saturated "mounds." They observed a recharge wave celerity of roughly 30.5-45.7 m/d (100-150 ft/d) from the peak discharge event. For weeks after the cessation of river flows, the mounds gradually moved vertically down through the soil profile while maintaining high moisture contents. Wilson and DeCook (1968) postulated that slow drainage from the lower mound (after the upper mound quickly drained into the lower mound) sustained groundwater levels in the immediate vicinity of the mound, and cautioned that such "in-transit storage" should be accounted for in water balance studies.

Cox and Stephens (1988) conducted two sets of stream/aquifer recharge experiments in ephemeral streams in New Mexico. While the depth to water in both cases was much shallower than in the case of the Santa Cruz near Tucson, the authors observed similar vadose zone responses to surface water flow events as those described by Wilson and DeCook (1968) for the Santa Cruz. By installing neutron probe access tubes at an angle underneath the streambed surface, Cox and Stephens (1988) were able to track the downward progression of water in the vadose zone during and after a flow event. They observed mounding at the water table in response to unsaturated flow underneath the flowing stream, and the maintenance of unsaturated conditions at a small (2 m (6.6 ft)) lateral distance from the flowing stream. One of the streams showed evidence of a clogging layer (no increase in soil moisture under the stream during a large flow event) while the other did not. Cox and Stephens (1988) attribute this distinction to variation in bed load grain size (fewer fines prevented a clogging layer from developing).
Marie (1985) describes results of two experiments conducted in conjunction with a 23-day aquifer test in south-central Arizona. The experiments were intended to determine 1) water loss from a natural channel, and 2) flow through a 100-m (330-ft) thick unsaturated zone overlying an unconfined aquifer. Water produced by the well (0.155 cms (5.46 cfs)) was piped into the ephemeral stream channel and streamflow losses were recorded. Transmission losses increased steadily until stabilizing at a maximum value for each stream reach. Neutron logs in wells adjacent to the stream revealed a progressive increase in soil moisture with depth. After about one week, soil in the 15 m (50 ft) above the water table became nearly saturated, and after two weeks, the near-saturated area extended up to about 30.5 m (100 ft) above the original water table. The piezometric surface raised just over 0.3 m (1 ft) after 10 days and nearly 0.6 m (2 ft) by day 16. The first five days after streamflow ceased saw a rapid decrease in soil moisture above the water table, and within 20 days, the entire soil profile had returned to its original soil moisture conditions (Marie, 1985).

Depths to groundwater in the vicinity of the Santa Cruz River northwest of Tucson averaged several meters below the surface in the early 1900's (Smith, 1910) and about 6.7 to 15.2 m (22 to 50 ft) by 1920 (Schmidt, 1973; Schwalen and Shaw, 1957). Water levels proceeded to drop sharply until 1965 (averaging 24.4-36.6 km (80-120 ft)), then stabilized and began rising (on average 0.9 m (3 ft) per year) by 1972 (Schmidt, 1973). Matlock, et al (1972) noted that the stabilization of groundwater levels in the presence of large agricultural pumpages in the Cortaro area provides testament to the "excellent rechargeability of the
aquifer through continued effluent flows."

The Cortaro Marana Irrigation District (CMID) and the Cortaro Water Users Association (CWUA) operate 50 active wells along the Santa Cruz River Corridor (74 km (46 mi)) from Ina Road to the Pima/Pinal County Line) that supply 80% of all the pumped water within their combined service areas northwest of Tucson. The CMID holds appropriative rights to 36.0 MCM/yr (29,210 ac-ft/yr) of surface water on the Santa Cruz River, and to 2.22 MCM/yr (1800 ac-ft/yr) of effluent (Environmental Resource Consultants, 1995).

Schultz, et al (1976) reported that an order of magnitude difference in hydraulic conductivity between the channel alluvium and the underlying Fort Lowell formation causes recharging water from the Santa Cruz River to mound at the interface of the two formations and move laterally faster than downward. They also observed the presence of chlorofluorocarbon in groundwater near the Santa Cruz River, indicating that recharge had taken place since the 1930’s.

Bostick (1978) used stable isotopes to discern that groundwater in wells near the effluent-dominated portion of the Santa Cruz River reflects a mixture of river recharge and deeper groundwater from other recharge sources, with about 70% coming from effluent

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6"Surface water rights," in this case, include infiltrated river water known as "subflow" in Arizona water law. For a detailed discussion on subflow, refer to Glennon and Maddock, 1995.
recharge and 30% coming from deeper groundwater. He estimated that effluent recharge is limited to about 3.2 lateral kilometers (2 mi) from the river.
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CHAPTER IV. FIELD STUDY

TUCSON BASIN GEOGRAPHY AND GEOLOGY

Tucson falls within the Basin and Range Province of the western United States, and the Tucson basin exhibits characteristic Basin and Range structural features resulting from a block faulting disturbance which began locally 12 million years ago (Anderson, 1987). Elevations within the basin vary from about 914 m (3,000 ft) on the southern end to 610 m (2,000 ft) in the northwest corner. The Santa Catalina Mountains, the Rincon Mountains, and the Santa Rita Mountains (with peak elevations ranging from 2,590-2,865 m (8,500-9,400 ft) above mean sea level) form the northern, eastern, and southern boundaries of the Tucson basin, respectively. The Santa Catalinas and the Rincon Mountains consist primarily of metamorphic and igneous rocks, while the Santa Ritas contain significant igneous and sedimentary components. The comparatively small and arid Tucson Mountains (volcanic and sedimentary) bound the basin to the west.

Davidson (1973) and Anderson (1987) provide extensive reviews of the basin fill material in the Tucson basin. Davidson (1973) defines four geologic units that overly the crystalline basement. The Pantano Formation of Tertiary age unconformably overlies the basement rock. Its mudstone, sandstone, and conglomerate components are saturated and is reported to have a maximum thickness of about up to 3,200 m (10,500 ft) south of Tucson, but extend to only about 305 m (1000 ft) thick in the central area of the Tucson basin (Davidson, 1973). Davidson (1973) dates most of the unit at 26-38 million years old.
The Tinaja Formation (Tertiary) unconformably overlies the Pantano Formation in most of the Tucson basin. It is generally divided into three units called the Lower, Middle, and Upper Tinaja Beds, but the Middle beds thin and, in some areas, disappear near the basin margins. The Tinaja grades from moderately cemented gray to grayish brown sandy gravel into reddish brown gypsiferous clayey silt and mudstone near the basin center. The entire formation dates from 2 to 29 million years old, with the upper beds ranging from 2 to 6 million years old. The Tinaja was deposited in a closed basin environment and exhibits evaporitic facies in the Middle and Lower beds. The Lower and Middle beds represent a combined saturated thickness of nearly 1,067 m (3,500 ft). The Upper Tinaja Beds represent the lower portion of the most productive water-bearing strata in the basin. These beds are largely unconsolidated and may be up to 335 m (1,100 ft) thick in the center of the basin (CH2M Hill, et al, 1987).

The Quaternary Fort Lowell Formation (1.3 to 2.5 million years old) unconformably overlies the Tinaja Beds and comprises the main aquifer in the Tucson basin (Davidson, 1973). With a maximum thickness of about 131 m (430 ft) (thinning toward the basin margins), the weakly consolidated to unconsolidated clayey silts, sandy silts, and sands and gravels that make up the Fort Lowell may be saturated up to about 61 m (200 ft) above the base of the formation (CH2M Hill, et al, 1987). Formed in a closed basin environment, the Fort Lowell is the most productive aquifer unit in the Tucson basin because it is generally coarser than the underlying units (Davidson, 1973; Anderson, 1987).
Above the Fort Lowell lies the Recent Alluvium. This formation, dated at less than 11,000 years old, contains coarse-grained, highly permeable deposits from stream channels, flood plains, and alluvial fans (Davidson, 1973). Most of this alluvium consists of unconsolidated coarse sand and gravel with a small percentage of silt and clay. Estimates of the thickness of this alluvium fall below 30.5 m (100 ft) throughout the basin (CH2M Hill, et al, 1987).

HYDROLOGY OF THE TUCSON BASIN

The Santa Cruz River drains the Tucson basin to the north and bisects the basin into east and west portions. The east portion provides most of the tributary flow to the Santa Cruz River through the Rillito River and Cañada del Oro Wash. Between downtown Tucson and just downstream of the confluence with the Rillito River, at the town of Cortaro (about 13 river miles), the Santa Cruz River drains approximately 3,416 m² (1319 square miles (sqmi)) (Burkham, 1970a). While depths to water are still relatively shallow under the Rillito River on the far east side of the basin, groundwater levels in the remainder of the basin are typically more than 30.5 m (100 ft), and frequently more than 91 m (300 ft), below the surface with the exception of the area immediately adjacent to the Santa Cruz River downstream of the sewage effluent outfalls in northwest Tucson (City of Tucson, 1990). Recharge to the aquifer occurs primarily through the main river channels (Santa Cruz, Rillito, Pantano, Cañada del Oro) and at the mountain fronts (Burkham, 1970a).
Published hydraulic conductivities in the deepest basin fill unit, the Pantano Formation, range from 0.2 to 4.1 m/d (5-100 gpd/ft² or 0.7-13 ft/d). Davidson (1973) reports porosities between 20% and 27% and estimates a long term storage coefficient equal to about 10% of the total saturate volume. Although this unit is not presently considered an important aquifer in the Tucson basin, it can produce appreciable well yields (0.25-0.50 m²/min (20-40 gpm/ft)) if a significant saturated thickness is penetrated. The Tinaja Formation, while not as productive as the Fort Lowell, is still an important aquifer in the Tucson basin. Transmissivity estimates range from 124 to 1860 m²/d (10,000 to 150,000 gpd/ft) with corresponding hydraulic conductivities of 0.41 to 16 m/d (10-400 gpd/ft² or 1-53 ft/d). Porosities vary from 24 to 35% and specific capacities in wells penetrating the Tinaja range from 0.01 to 0.50 m²/d (1-40 gpm/ft). The Fort Lowell Formation, sometimes considered to include the Recent Alluvium, is the most productive aquifer in the Tucson basin. Hydraulic conductivities of the Fort Lowell Formation range from 6.1 to 28.5 m/d (150-700 gpd/ft² or 20-93 ft/d), and porosities from 26 to 34%. Wells penetrating significant saturated thickness in this formation may yield 2,725 to 8,175 m³/d (500-1500 gpm) with specific capacities of roughly 0.1 to 1.2 m³/min (10-100 gpm/ft) (Davidson, 1973; Anderson, 1987).

SELECTION OF A FIELD SITE

This study concerns changes in the shallow soil regime of the effluent channel in the Santa Cruz River northwest of Tucson. For this reason, the study area lies downstream of both sewage treatment plants serving the greater Tucson area and discharging excess effluent
to the river (Figure 3). In order to simplify transmission loss measurements, the upstream end of the study reach was positioned near Ina Road, downstream of the two major tributaries feeding the Santa Cruz River in this area: the Rillito River and Cañado del Oro Wash. The Rillito Narrows (near Avra Valley Road) has served as a traditional downstream study boundary for previous researchers, largely because it separates the Upper Santa Cruz sub-basin from the Avra Valley sub-basin and marks a dramatic increase in depth (an added 61 m (200 ft)) to water north of the Narrows. Accessibility and consistency with previous researchers motivated the selection of Avra Valley Road as the downstream boundary of the area in this study.

FIELD MEASUREMENTS

Surface Water

Although several researchers have demonstrated that infiltration rates for sewage effluent in the Santa Cruz River vary over time and generally increase in response to natural storm flows, none has quantified the rate of change in streambed hydraulic conductivities. This issue is important to the estimation of recharge from the effluent stream. If the meteorological circumstances that lead to various streambed conditions are understood, then one can predict the infiltration capacity of the streambed under various potential conditions. The first step in quantifying the hydraulic conductivity of the streambed entails measuring streamflow abstractions and distributing those between losses to the subsurface (infiltration)
adapted from: Esposito, 1993.
and losses to the atmosphere (evaporation). An important distinction separates infiltration from recharge. Infiltration includes all water passing from the surface water system down through the sediments that comprise the streambed surface. The depth of these surficial streambed materials is somewhat arguable but, for the purpose of this study, it probably extends down a maximum of about 0.5 meter (1.6 ft). This definition suggests, then, that water taken up by plant roots penetrating several feet down into the soil zone adjacent to the stream is counted in infiltration. Recharge, however, implies that water makes the complete progression from the surface of the streambed to the groundwater table. In the case of a stream separated from the saturated zone by several tens or hundreds of feet, the process of recharging the aquifer by unsaturated flow can take weeks or months (Wilson and DeCook, 1968). In estimating recharge, plant transpiration from the root zone and water lost to vadose zone storage must be subtracted from the infiltration volume.

Surface water discharges can be measured either directly or indirectly. The direct method, usually referred to as manual streamgaging, involves the manual placement of a current meter (mounted on a rod or cable) at specific depth and width intervals across the stream cross-section. The current meter provides a velocity measurement, and the stream velocities are averaged over the distance between approximately twenty measurement points dividing the cross-section. Vertical variations in velocity are averaged by measuring velocities at depths of $0.6 \times \text{depth of water}$ below the surface. Ideally, the twenty measurement points divide the stream discharge into equal increments across the section.
In most natural channels, achieving this goal requires that the points be concentrated (more closely spaced) in high flow velocity regions and more widely spaced in the lower velocity areas (typically near the stream banks) (World Meteorological Org., 1980; Herschy, 1985).

Indirect discharge measurements involve the development of a "rating curve" for the specific stream section being measured. This rating curve defines the unique relationship between stream stage and stream discharge at that site. For natural streams (versus idealized weirs), rating curves must be determined through manual stage and discharge measurements. Once the rating curve is established, and provided that the geometry of the stream section is invariant with time, the stage-discharge relationship can be used to estimate discharge from stage measurements at a single location. Obviously, this method lends itself to automation. By placing a single stage measuring device (such as a nitrogen bubbling tube, pressure transducer, or float recorder) in a pool of calm water (by use of a stilling well) in the stream along the designated cross-section, continuously-measured stream stages may be extrapolated to equally continuous discharge values.

Cross-sections in the portion of the Santa Cruz River measured in this study vary from nearly ideal to nearly impossible for streamgaging. In some upper parts of the reach (particularly upstream of Cortaro Road), the channel has been repeatedly dredged and continuous diversions by the sand and gravel pit operators near the upstream end of the reach maintain a consistent, nearly ideal cross-section for stream gaging. This reach of the
effluent-dominated stream is well entrenched in a straight channel with a rocky bottom, so that the channel geometry remains fairly constant over time. Downstream of Cortaro Road, however, the effluent stream wanders in and out of well-defined channels, often spreading into very shallow, broad flows which vary in width by more than 100% between high and low diurnal flow cycles. These areas defy even manual streamgaging efforts because of continuously shifting streambottom sediments, quickly varying stage and stream width, and flow depths shallower than the 0.1 m (0.3 ft) tolerance of a pygmy meter. Wherever possible, stream sections with the fewest of these complications were chosen for measurement, but in most cases, some degree of expertise on the part of the measurer was essential for obtaining reasonably reliable discharge measurements. Special techniques required for handling difficult sections included: 1) adjusting the elevation of the pygmy meter on the rod as the sediments shifted underneath the rod's foot during a measurement (in order to maintain the 0.6 x depth elevation); 2) correcting the pygmy meter velocities for measurements made in water shallower than 0.1 m (0.3 ft) (see Pierce, 1941); 3) estimating velocity in areas of flow too shallow to measure (by use of a visual tracer on the surface); and 4) measuring velocities for 20 to 30 seconds instead of the full 40 seconds (standard) in order to complete the entire cross-section of measurements before the stage changed too dramatically.

With the goal of obtaining long-term, continuous discharge and stage measurements, an experimental automated streamgaging system was designed and during the first year of
In an effort to cover the entire width of the Santa Cruz River at each end of the study reach, two separate arrays of pressure transducers (Motorola MPX-2200AS, 0-30 pounds per square inch (psi) (0-2,068 millibars), temperature-compensated) equipped with thermocouples and encased in water-tight housings were anchored to 3 m x 1.3 cm (10-ft x 1/2-in) rebar rods hammered 1.8-2.4 m (6 to 8 ft) into the streambed. The sensors were then connected to a data logger (Campbell Scientific, Inc. CR-10) buried in an insulated cannister on the bank of the river. Voltage/temperature paired readings were taken and recorded every 30 minutes. The voltages from the individual transducers (10 installed across the river width) were later converted to equivalent pressures by use of a multiple regression equation (Carpenter, 1994) that describes voltage/temperature pairs corresponding to specific pressures for each transducer (see Appendix A for details on pressure transducer calibration). Pressure can then be directly translated into water depth (millibar (mb) = 0.98 cm), which can be interpreted as stream stage as long as the pressure transducer's port rests on the surface of the streambed. Because effluent flow only covered a small percentage of the riverbed's width (approximately 10-20%) in the array transects, the transducers which fell outside of the wetted area were buried approximately 10 to 30 cm (3.9-11.8 in) below the soil surface to insulate them from extreme daily temperature swings.

The Motorola MPX-2200AS sensors (Figure 4) read absolute pressure, so barometric pressure variations must be subtracted from the water sensor readings in order to measure submersion depth. A similar transducer with a pressure range of 0-1,034 mb (0-15 psi)
4-conductor cable, Belden 8723 (4 millimeters)

4-millimeter hole

Potting relief hole (3 millimeters)

PVC coupling

Potting (TAPI-1)

Thermocouple

Sensor leads soldered to 4-conductor cable

Pressure sensor (Motorola MPX2200AS)

PVC pipe plug drilled 6.248 millimeters for press fit ("D" bit)

Sensor port

Figure 4.
Submersible Pressure/Temperature Sensor.

coupled with a temperature sensor and buried with the data logger (with an "breathing tube" connecting to the open air) served as a barometer at each of the three gaging stations. For each station, this sensor's voltage was converted to pressure with the regression equation procedure described in Appendix A before its pressure value was subtracted from that read by the water sensors.

The success of the array system of transducers was severely limited by the dynamic nature of the alluvial river system. Heavy vegetation in the Avra Valley Road transect (downstream end) encouraged in-filling of the low-flow channels and prevented the development of new stage-discharge rating curves after several weeks. Although the upstream array (near Ina Road) proved successful for a short time, it sustained significant damage during the first winter storm of the season (November 11, 1994) and clearly could not hold up to further storm flows. While storm flows scoured the upstream portion of the study reach, they exhibited depositional behavior in the downstream end of the reach. Consequently, transducers in the downstream reach became significantly buried (beyond 1 meter (3.3 ft), in some cases) after the winter storm season, and were rendered useless.

In order to get a more accurate measure of the transmission losses occurring in the study reach, four manual synoptic streamgaging runs were conducted over the course of the following year. Each synoptic run took place over 24 hours, with the upstream measurements beginning two hours prior to those downstream. The streamgaging "stations"
were carefully chosen for their channel characteristics and for accessibility. All of the synoptic runs were conducted at the same upstream and downstream sites, near the beginning and end of the study reach, approximately 8.9 km (5.5 mi) apart. Four people conducted the discharge measurements (two upstream and two downstream), with each person measuring for 12 hours. The variety of flow conditions at the upstream end required the use of both a standard Price current meter as well as a pygmy meter. During some of the synoptic runs, Marsh-McBirney electronic current meters were used in place of the mechanical (Price and pygmy) meters. The electronic meters were checked against the mechanical meters for accuracy. In all but the first synoptic run, top-setting rods facilitated the discharge measurements.

The four synoptic runs were conducted on November 19, 1994, January 25, 1995, March 25, 1995 and May 1, 1995. As shown in Figure 5, the first synoptic run immediately followed the first winter storm of the season (11/11/94). While comparatively small, that storm marked the first significant (natural) runoff in the study reach for almost 20 months (since January of 1993). The second synoptic run (1/25/95) followed two of the largest storms of the season (12/5/94 and 1/5/95). The third synoptic run (3/25/96) followed the last and largest runoff event of the winter season (2/15/95). No natural runoff occurred after the February storm until August 12, 1995.

At the end of the winter storm season, new surface water pressure/temperature
Figure 5
Total Discharge in Santa Cruz River at Cortaro Road

Mean Daily Discharge (cms)

Day of Year

Oct. 1, 1993

Sept. 30, 1995

0.0

100.0

20.0

40.0

60.0

80.0

100.0

Nov. 19, 1994
Jan. 25, 1995
Mar. 25, 1995
May 1, 1995
Jul. 12, 1995
Aug. 3, 1995

synoptic run dates
sensors were installed to supplement the synoptic run data. Three new sensor sites were selected for their security, channel geometry, and accessibility. Figure 3 locates the three sites, referred to as INAB (near Ina Road), CORT1 (roughly 2.25 km (1.4 mi) downstream of Cortaro Road), and AVRAB (near Avra Valley Road). Each site housed a redundant, double sensor (two transducers, one thermocouple) near the edge of the flowing stream and anchored by a steel rod. The sensor was carefully positioned so that the port remained stationary at the average streambed surface. A very fine mesh screen over the sensor ports prevented sediment from clogging the ports. In all three sites, the sensors were positioned in a deep part of the channel cross-section to keep the sensors submerged and to provide some degree of insulation from the ambient air temperature.

The upstream station (INAB) was located within the dredged, well-confined effluent channel between Ina and Cortaro Roads, approximately 0.6 km (.4 mi) downstream of Ina Road. While this channel cross-section had very good channel geometry and stability, several factors precluded the collection of reliable stream depth data with the automated sensors without frequent site visits for manual validation of the measurements. For all three sites, the accuracy of the water depth values measured with the sensors remained questionable due to debris build-up on the sensors (mostly plant material which then caught sediment) and wide daily temperature fluctuations (typically 23°C (40°F)). During site visits, manual stage and discharge measurements were taken over a period of several hours, usually coinciding with a period of rapidly changing stream stage, to provide "truth" points
for sensor calibration and to define the current stage-discharge relationship. These truth values were subsequently added to the regression values for each sensor (see Appendix A).

In order to achieve accurate discharge measurements at both upstream and downstream sensor sites (i.e., at INAB and at either CORT1 or AVRAB) for the purpose of estimating transmission losses, manual streamgaging was conducted during periodic field visits. During field visits, measurements of stream stage and discharge were made over the course of several hours (usually during a period of rapid stage change) coincidentally with the automated sensor readings. Ideally, these sensor calibrations would have remained fixed, or would have experienced only a "zero drift" which could be corrected with a linear offset of the calibration curves. Unfortunately, the field conditions mentioned above produced inconsistent results in the pressure transducers. Ultimately, the stage values calculated from the sensors (based on the original calibration equations) were used to develop stage-discharge rating curves for the three stations, even though the calculated stages were not accurate in some cases. By using the calculated stages, however, the sensor data could be used to determine reliable discharge values for short periods of time (up to approximately 24 hours) surrounding manual discharge measurements. In addition, shifting channel bottoms at the three stations resulted in stage-discharge relationships which changed too rapidly to provide accurate discharge estimates for more than about one day.
Atmosphere

Table 2 lists values provided by researchers at the Constructed Ecosystems Research Facility near Roger Road and the Santa Cruz River roughly 8.8 km (5.5 miles upstream from Ina Road) for average daily Class A pan evaporation rates, total monthly evaporation, and total monthly rain (Office of Arid Land Studies, 1995). Pan evaporation rates,

Table 2
Summary of Pan Evaporation, Open Channel Evaporation, and Total Monthly Rainfall for the Study Area in 1995

<table>
<thead>
<tr>
<th>1995 month</th>
<th>Average Daily Pan Evaporation</th>
<th>Total Monthly Pan Evaporation</th>
<th>Total Monthly Open Channel Evaporation</th>
<th>Total Monthly Rain</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>mm in</td>
<td>mm in</td>
<td>mm in</td>
<td>mm in</td>
</tr>
<tr>
<td>January</td>
<td>2.7 0.11</td>
<td>83.8 3.30</td>
<td>58.7 2.31</td>
<td>61 2.4</td>
</tr>
<tr>
<td>February</td>
<td>3.8 0.15</td>
<td>107 4.21</td>
<td>75 2.95</td>
<td>32 1.26</td>
</tr>
<tr>
<td>March</td>
<td>6.8 0.27</td>
<td>211 8.31</td>
<td>148 5.82</td>
<td>10 0.39</td>
</tr>
<tr>
<td>April</td>
<td>9.7 0.38</td>
<td>291 11.5</td>
<td>203 8.05</td>
<td>11.2 0.44</td>
</tr>
<tr>
<td>May</td>
<td>11.9 0.47</td>
<td>368 14.5</td>
<td>258 10.2</td>
<td>2.5 0.1</td>
</tr>
<tr>
<td>June</td>
<td>14.3 0.56</td>
<td>430 16.9</td>
<td>301 11.8</td>
<td>0 0</td>
</tr>
<tr>
<td>July</td>
<td>14.3 0.56</td>
<td>445 17.5</td>
<td>311 12.3</td>
<td>10.5 0.41</td>
</tr>
<tr>
<td>August</td>
<td>12.8 0.50</td>
<td>396 15.6</td>
<td>277 10.9</td>
<td>114 4.49</td>
</tr>
<tr>
<td>September</td>
<td>11.4 0.45</td>
<td>341 13.4</td>
<td>239 9.38</td>
<td>6 0.24</td>
</tr>
<tr>
<td>October</td>
<td>8.9 0.35</td>
<td>276 10.9</td>
<td>193 7.63</td>
<td>0 0</td>
</tr>
<tr>
<td>November</td>
<td>6.2 0.24</td>
<td>187 7.36</td>
<td>131 5.15</td>
<td>32.5 1.28</td>
</tr>
<tr>
<td>December</td>
<td>5.7 0.22</td>
<td>177 6.97</td>
<td>124 4.88</td>
<td>4 0.16</td>
</tr>
</tbody>
</table>

* Values from Constructed Ecosystems Research Facility Near Roger Road and the Santa Cruz River, approximately 8.8 km (5.5 mi) upstream from Ina Road.

* computed as (0.7 * total monthly pan evaporation)
when corrected by the appropriate "pan coefficient" (typically 0.7-0.8), provide good approximations for open-channel evaporation (Nokes, 1995). Galyean (1996) provides monthly evapotranspiration estimates for the study reach (Ina Road to Avra Valley Road) for water years 1991-1993 (Table 3). Galyean used aerial photographs and field studies to determine the areal coverage and plant type in the study area. He then applied the Blaney-Morin Equation (Blaney and Morin, 1942) for estimating evapotranspiration based on number of daylight hours, relative humidity, average temperature, and consumptive water-use coefficients of native vegetation derived from the Blaney-Criddle (Blaney and Criddle, 1950) formula. Galyean's (1996) values provide reasonable evapotranspiration estimates for this study.

Table 3
Average Monthly Evapotranspiration (mm) for Water Years 1991-1993

<table>
<thead>
<tr>
<th>Month</th>
<th>Ina Rd. to Cortaro Rd.</th>
<th>Cortaro Rd. to Avra Valley Rd.</th>
<th>Ina Rd. to Avra Valley Rd.</th>
</tr>
</thead>
<tbody>
<tr>
<td>October</td>
<td>96.5</td>
<td>89.9</td>
<td>93.2</td>
</tr>
<tr>
<td>November</td>
<td>63.4</td>
<td>59.3</td>
<td>61.3</td>
</tr>
<tr>
<td>December</td>
<td>30.1</td>
<td>27.3</td>
<td>28.7</td>
</tr>
<tr>
<td>January</td>
<td>31.6</td>
<td>28.8</td>
<td>30.2</td>
</tr>
<tr>
<td>February</td>
<td>44.9</td>
<td>42.0</td>
<td>43.4</td>
</tr>
<tr>
<td>March</td>
<td>53.6</td>
<td>49.2</td>
<td>51.4</td>
</tr>
<tr>
<td>April</td>
<td>104.1</td>
<td>92.1</td>
<td>98.1</td>
</tr>
<tr>
<td>May</td>
<td>136.2</td>
<td>120.8</td>
<td>128.5</td>
</tr>
<tr>
<td>June</td>
<td>166.4</td>
<td>152.1</td>
<td>159.2</td>
</tr>
<tr>
<td>July</td>
<td>143.8</td>
<td>127.5</td>
<td>135.6</td>
</tr>
</tbody>
</table>
Precipitation records are available from the Pima County Department of Transportation and Flood Control District raingage network. However, river discharge measurements provide direct information on transmission losses and therefore obviate the need for precipitation data in this study.

Land Surface

Streambed topography, particularly the angle of the side slopes of the channel, strongly influences infiltration and open-channel evaporation rates. Low angle side slopes permit water to spread laterally at high stage, wetting a relatively large area with very shallow water. Increased wetted area promotes more evaporation and lowers the average hydraulic head above the streambed surface, decreasing the vertical hydraulic gradient across the width of the channel.

Low side slopes may also allow water to flow out of the normal low-flow channel during high stage. Such an occurrence complicates transmission loss analysis in that added surface area outside the normal flow channel must be considered when evaluating infiltration rates. Water moving out of the normal low-flow channel may also encounter areas of higher vertical hydraulic conductivity where no streambed clogging layer exists. Finally, any water
that infiltrates into comparatively dry alluvium (outside of the main channel) is not likely to infiltrate to any significant depth. Rather, sorptive forces in the shallow soil on the perimeter of the wetted area would initially hold the water in the shallow soil zone. Over time, evaporative forces may overcome the sorptive forces to remove the soil water from the shallow soil zone.

Channel side slope angles were manually measured at various locations along the study reach, and in other areas, slopes were estimated by the variation in width of the flowing stream between high and low stage by means of aerial photographs (Cooper Aerial, 1994) and by the presence or absence of concrete bank protection. Riverbed topography and width were measured by occasional transit and stadia transect surveys near the upper and lower ends of the study reach. Low flow channel widths were measured during manual streamgaging runs and estimated from aerial photos. The overall slope of the river within the study reach was determined in segments from ortho-corrected aerial photo maps with topographic contours overlain (Pima County Transportation Department and Flood Control District, 1984, 1990).

Near Surface

*Hydraulic Conductivity by Tension Infiltrometer*

In most environments, direct, in situ measurements of saturated hydraulic conductivity can be obtained by a variety of methods (double or single ring infiltrometer, air-
entry permeameter, bore-hole permeameter, and others). In the case of a flowing stream, however, scour due to the stream's lateral velocity precludes the use of most direct conductivity measurement techniques. While Matlock (1965) reports the use of a double ring infiltrometer in the flowing stream of the Rillito River in Tucson, attempts to use the same device in the effluent stream of the Santa Cruz River during this study were entirely unsuccessful due to scour under the upstream edge of the cylinder.

During the winter of 1994-95, heavy rains and resultant large natural discharges in the Santa Cruz River increased infiltration rates so dramatically that the low-flow stream ceased flowing near Avra Valley Road for several hours each day during a few weeks in January, 1995. During these no-flow periods, the perched water table in the streambed fell to approximately 0.5 to 1 meter (1.6-3.3 ft) below the surface. With only a few exceptions, most days saw the abrupt return of effluent flows near Avra Valley Road after the onset of the peak discharge (from the treatment plants) period in late afternoon. The 2.5 to 4-hour period of no flow permitted the in situ measurement of hydraulic conductivity in the low-flow stream channel. In situ measurements of unsaturated hydraulic conductivities were achieved by the use of a 20-cm (7.9-in) diameter tension infiltrometer.

Figure 6 illustrates the major components of a disc tension infiltrometer. This device provides a rapid, in situ, non-invasive method for measuring hydraulic conductivities at

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1 Tension infiltrometer provided on loan from Bruce Bailey, Dept. of Hydrology and Water Resources, Univ. of Arizona.
Figure 6.

Tension Infiltrometer
various soil tensions, which is analogous to measuring conductivities at different soil moisture values. The tension infiltrometer can also provide other hydraulic properties of the wet region including: saturated hydraulic conductivity ($K_s$), sorptivity ($S$), and the macropore capillary length ($\lambda_c$) (White and Sully, 1987; Ankeny, et al, 1988; Smettem and Clothier, 1989; Husjen, 1991; White, et al, 1992; Husjen and Warrick, 1993a,b). While slight variations in design exist, all tension infiltrometers share the three major parts described below.

The "Bubble Tower" consists of a marriotte tube device with one or more small air entry tubes, each with a pinch clamp on top. The tubes extend through a rubber stopper into the water at different depths, each corresponding to the desired tension to be imposed at the disc membrane-soil interface ($P_s$) (Hussen and Warrick, 1995). This pressure is described by:

$$P_s = P_a - \rho g (h_1 - h_2)$$

where,

$P_a = $ atmospheric pressure,

$h_1 = $ length of inlet tube below the water,

$h_2 = $ height of connection tube from soil surface,

$\rho = $ density of water,

---

2 Bailey (Dept. of Hydrology and Water Resources, Univ. of Arizona, 1994, pers. comm.) devised an analogous system for manipulating supply pressure through the use of a large-diameter syringe graduated in 0.5-cm increments.
and \( g \) = acceleration due to gravity.

The second term on the right side of Equation (1) is equivalent to \( h_{we} \) and represents the pressure head. When \( h_1 > h_2 \), \( h_{we} \) is negative and a suction is applied at the membrane-soil interface.

The "Water Reservoir Tube" supplies water to the disc. The rate of infiltration may be monitored by manually observing the water level change with time in the graduated cylinder or by the use of pressure transducers.

A porous baseplate disc at the bottom of the infiltrometer is covered on one side with a fine-mesh (usually nylon) membrane. The membrane establishes hydraulic conductivity with the soil through a thin sand cap (between the disc and the soil) which is leveled to promote good contact with the membrane. The membrane's bubbling pressure (air-entry pressure) depends on its mesh size and must exceed the highest tension to be applied to the soil being tested. Typical air entry pressures for these membranes range from 25 to 100 cm (9.8 - 39 in) (Hussen and Warrick, 1995; White, et al, 1992).

**Theory of Disc Tension Infiltrometers**

The excellent overview of tension infiltrometer applications and implementation provided by Hussen and Warrick (1995) serves as a basis for most of the theoretical
discussion presented here. Water flows out of a tension infiltrometer in response to gravity (downward) and capillary forces (three dimensional), and the geometry of the infiltrometer. Philip (1969) described the flow at early times (nearly one-dimensional) by:

\[ I = St^{0.5} \]  

(2)

where \( I = \) cumulative intake per unit area (L) and \( t \) is time. The sorptivity (\( S \)) has units of length/(time)^{0.5} and equals the slope of \( I \) vs. \( t^{0.5} \). At large times, Wooding’s (1968) equation represents a solution for steady-state flow per unit area, \( Q \) (L/T):

\[ Q = K_{\text{wet}} \left[ 1 + \frac{4\lambda_c}{\pi r_o} \right] \]  

(3)

where, \( r_o \) = radius of the disc infiltrometer (L), 
\( K_{\text{wet}} \) = hydraulic conductivity (L/T) corresponding to the water supply potential, \( h_{\text{wet}} \), and \( \lambda_c \) = macropore capillary length (L).

The first term on the right side of Wooding’s Equation (3) represents gravitational flow and the second term represents the contribution to flow by capillarity and source geometry. This equation assumes uniform, homogenous, non-swelling soil with an exponential hydraulic conductivity function described by Gardner (1958) as:
where $K$ is the unsaturated hydraulic conductivity at matric potential $h$ (L) and $K_s$ is the saturated hydraulic conductivity.

White and Sully (1987) showed that the macropore capillary length ($\lambda_c$) is a function of sorptivity and hydraulic conductivity described by:

$$\lambda_c = \frac{b S^2}{(\theta_{wet} - \theta_{dry})(K_{wet} - K_{dry})}$$

where, $\theta_{wet} = \text{volumetric water content} [L^3/L^3]$ corresponding to supply potential $h_{wet}$,

$\theta_{dry} = \text{initial volumetric water content}$,

$K_{wet} = \text{hydraulic conductivity} [L/T]$ at $\theta_{wet}$,

$K_{dry} = \text{hydraulic conductivity} [L/T]$ at $\theta_{dry}$,

$b = \text{constant} [\text{]}$, usually taken to be 0.55 (White and Sully, 1987; Warrick and Broadbridge, 1992).

Substituting $b = 0.55$ and for $K_{wet} >> K_{dry}$, equations (3) and (5) yields
\[ Q = K_{\text{wet}} + \frac{2.2S^2}{\pi r_o (\theta_{\text{wet}} - \theta_{\text{dry}})} \]  

(6)

If \( S \) is found from early time measurements (Equation 2) and \( Q \) from large time measurements, \( K_{\text{wet}} \) can be determined from Equation (6). Wooding's Equation (3) can be solved for two unknowns (\( K_{\text{wet}} \) and \( \lambda_c \)) if steady-state \( Q \) values are known for two different disc radii \( (r_o) \) (Scotter, et al, 1982; Yitayew and Watson, 1986; Smettem and Clothier, 1989; Hussen, 1991). Alternatively, the same principle may be applied for multiple tensions and a single disc (Lien, 1989; Ankeny, et al, 1991; Hussen, 1991; Reynolds and Elrick, 1991). Substituting the Gardner Equation (4) into the Wooding Equation (3) gives:

\[ Q = K_s \exp \left( \frac{h}{\lambda_c} \right) \left[ 1 + \frac{4\lambda_c}{\pi r_o} \right] \]  

(7)

Measuring steady-state flows (\( Q \)) for \( n \) tensions with a single disc at one site yields \( n \) equations, each with two unknowns (\( K_s \) and \( \lambda_c \)). For two tensions (\( n = 2 \)), \( \lambda_c \) is determined by dividing two equations from Equation (7) for different supply potentials \( (h) \), giving:

\[ \lambda_c = \frac{|h_2 - h_1|}{|\ln(Q_2/Q_1)|} \]  

(8)

where \( Q_1 \) and \( Q_2 \) are steady-state flow rates at supply tensions \( h_1 \) and \( h_2 \). \( K_s \) follows from substituting \( \lambda_c \) back into Equation (7). Using three or more supply tensions allows \( K_s \) and
λ to be evaluated using a best-fit procedure (Hussen, 1991; Hussen and Warrick, 1993b).

As with any measurement technique, hydraulic conductivity values measured with the tension infiltrometer must be corrected for temperature. For this study, a spring thermometer inserted at an angle into the soil directly under the disc plate of the tension infiltrometer measured the temperature of water infiltrating from the infiltrometer into the soil. The conductivity values derived by the multiple tension method described above were then corrected for the effects of temperature on the viscosity of the water.

While the disc tension infiltrometer presents several advantages (in situ, portable, repeatable, rapid) over other hydraulic conductivity measurement techniques, it still suffers some important limitations. The principal limitation to the application of tension infiltrometer measurements of hydraulic conductivity involves the simplifying assumptions behind the equations used in the analysis of the raw data. The Wooding Equation (3) assumes uniform, homogeneous, and nonswelling soil. In practice, non-uniform soil water contents, bulk densities, layering, and texture in near surface soils can yield negative calculated hydraulic conductivity values. Other restrictions pertinent to the Santa Cruz River application include limits on the hydraulic conductivity of the sand cap used to establish contact between the disc membrane and the soil, and the proximity of shallow perched water under the streambed surface. The hydraulic conductivity of the sand cap must exceed that of the underlying soil in order to prevent its impeding flow out of the tensiometer. The
conductivities of some of the well-sorted alluvium were certainly similar to those of the sand cap (well-sorted clean sand taken from the riverbed), but any restriction on flow imposed by the sand cap should have been negligible. The proximity of shallow perched water could have introduced error into hydraulic conductivity measurements taken (by any type of infiltrometer) in the low-flow streambed during the several hours each day when the stream ceased to flow. Excavation into the alluvium revealed that water often pooled at less than one meter below the surface, presumably due to a restricting layer in the subsurface. Once encountered by water moving out of the tensiometer, this shallow water table would impede further downward progress of the infiltrating water and force water to flow laterally through the higher conductivity region above the perched water zone. This lateral diversion would produce artificially high infiltration rates measured at the surface (Bouwer, 1986).

Soil Core Samples

Soil samples collected by means of a JMC Tube Sampler\(^3\) provided soil texture and composition information. The transparent plastic sampling tubes measure 2.1 cm (0.83 in) in diameter by 30 cm (11.8 in) in length and are pressed (with open ends) into the soil by the tube sampler. After removing the filled sampling tube from the tube sampler, each tube is labeled and sealed with caps on both ends for easy transport and storage. While most of the surficial sediments of the riverbed are roughly of similar composition throughout the study reach, soil samples could only be obtained with the tube sampler across the entire width of

\(^3\)Provided on loan from Cindy Sailor, Soils/Plant Analysis Laboratory, University of Arizona.
the riverbed near the downstream end of the study reach because of the higher percentage of cobbles upstream. Soil samples were collected at twelve locations in a transect across the entire riverbed roughly 100 meters (328 ft) just upstream (south) of Avra Valley Road. The location of the low-flow stream(s) governed the areal intensity of sampling (more samples per lateral distance than in the dry sandbar areas). Where the soil type and depth to water permitted, two or more 30-cm (11.8 in) samples were collected in a vertical sequence.

The stratigraphy of each sample was described by visual analysis and each integrated (30-cm) sample was analyzed for wet and dry bulk density, volumetric water content, porosity and grain-size distribution (see discussion under “Soil Parameters” in Chapter V and Appendix B). Figure 7 illustrates the locations of the soil samples across the riverbed and indicates the depth of sample collection at each location.

The soil core samples were collected for a qualitative comparison with the results of the tension infiltrometer survey. Comparing the stratigraphy and soil type coinciding with each tension infiltrometer measurement allowed for some judgment of the validity of the conductivity values derived from the infiltrometer tests. In addition to contributing to the general body of data on alluvial streambeds in the southwest, the soil parameters derived from these samples provided porosity and starting hydraulic conductivity values for the modeling work (see Chapter VI).
Figure 7: Schematic Map of Soil and Tension Infiltration Tracer Surveys Near Avra Valley Road

- Effluent Flow
- Large Island
- Soil Sample Sites: 4, 5, 6, 7, 8, 9, 10, 11
- Concrete Bank Protection
- Not drawn to scale

Avra Valley Road

Dirt Bank

250 meters
Determining the nature of the hydrologic connection between the surface water (effluent) stream and the underlying water table requires some information on the degree of saturation and the hydraulic gradient beneath the flowing stream. Although tensiometers typically can provide these values, installing tensiometers in a flowing stream is problematic for several reasons. First, in order to measure the characteristics of the soil below the perched surface water, the tensiometer cup must be emplaced at the appropriate depth but without leakage from the overlying stream. Second, the ceramic porous cup at the tip of the tensiometer is delicate and can easily fracture if pushed forcefully into rocky soil. This constraint severely limits the application of tensiometers in alluvial sediments because of the inherent rockiness of those soils. Finally, the narrow-diameter (1.9 cm (0.75 in)) PVC tubing construction of the tensiometers offers very little shear strength against the horizontal force of streamflow, and leaves the tensiometers vulnerable to breakage or prying out of the soil.

In an effort to address the aforementioned problems, a new drive-point access tube technique was developed for installing tensiometers in a flowing stream (Figure 8). A prescribed length (cut to fit each tensiometer) of 2.54-cm (1-in) diameter EMT steel conduit was fitted with a hardened steel sacrificial drive point at one end and a hammering cap on the other. After driving the conduit to the desired depth into the streambed with a sledgehammer, a smaller steel tube was inserted down through the conduit and used to tap the drive point tip out of the end of the conduit, leaving a gap between the steel point and the
Figure 8
Schematic Diagram of Tensiometer Access Tube System

- Rubber Stopper
- Glass Window
- Plummer's putty
- Tensiometer body
- Porous Ceramic Cup
- Small diameter steel push rod
- Drive point pushed out
- Machined steel hammer cap
- 1' (2.54 cm) EMT steel conduit
- Sacrificial machined steel drive point
end of the conduit approximately equal in length to that of the porous cup on a tensiometer (6.3 cm (2.5 in)). A slurry of native sediment and water poured down through the conduit served to "seat" the tensiometer cup in the gap below the bottom of the conduit and the top of the drive point. Although the slurry helped establish a good contact between the porous cup and the surrounding soil, adding water to the system (where it was unsaturated) necessarily altered the ambient moisture content at that point. Consequently, several days were allowed for soil moisture equilibration to occur. After the tensiometers were emplaced down through the conduit tubes, plumber's putty was used to seal the gap between the PVC tensiometer and the larger diameter steel conduit to prevent any overflow from the stream or any rain water from leaking down to the level of the porous cup. This method provided added rigidity to the tensiometer tubes (by supporting them with 2.54-cm (1-in) steel conduit), prevented downward leakage from the flowing stream, and easily penetrated through rocky sediments to depths of more than one meter below the streambed surface. The most significant limitation to this method is the propensity for debris buildup behind the nested tubes (usually in groups of three).

Figure 9 illustrates the basic components of a tensiometer. The type of tensiometer used in this study consists of a sealed plastic (PVC) tube with a porous ceramic cup at the tip and a rubber stopper above a short (5-cm (2-in)) plexiglass window at the other end. The tube is filled with deaerated water and installed in close contact with the soil. The pressure in the tube's water (or in the air space above the water) can be measured directly with a
Figure 9
Tensiometer Construction

- Rubber Stopper
- Glass Window
- Water Surface
- PVC Tubing
- Water inside tensiometer
- Ceramic Porous Cup
- Pa
- Ps
- L
portable pressure transducer instrument known as a 'tensimeter.'

Tensiometers are used to measure the soil water matric potential. As the water content of the soil surrounding the water-filled porous cup decreases, the energy level of the soil water decreases relative to that of the water in the tensiometer cup, and water moves out of the tensiometer (through the porous cup) and into the soil. This evacuation of tensiometer water lowers the pressure on the water still inside the tensiometer. If the surrounding soil gets wetter, on the other hand, the soil water pressure increases and forces water back through the porous cup walls into the tensiometer. Once the water in the soil and in the tensiometer cup have equilibrated, the pressure inside the tensiometer reflects the pressure in the soil water adjacent to the tensiometer cup. For a detailed discussion of the theory of tensiometry, the reader should refer to Yeh and Guzman-Guzman (1995) and Cassel and Klute (1986).

**Subsurface**

Figure 3 shows the locations of the two wells whose hydrographs were measured for the duration of the field study. The wells were constructed by Kenneth D. Schmidt and Associates (1988) as a part of the study conducted by Esposito (1993). These two wells, designated SC-3 and SC-7, comprised part of a suite of twelve observation wells installed along the banks of the Santa Cruz River in 1988. The monitoring wells were drilled by the reverse rotary method and ranged in depth from 43 m (140 ft) in the Cortaro area to a
maximum of nearly 122 m (400 ft) in the Marana area. Esposito (1993) used data from these wells in his analysis of recharging effluent water quality. Pima County Wastewater Management Division also monitored these wells for chemical constituents and took water levels on a quarterly basis for several years (approximately 1988-1993), but regular monitoring of the wells was discontinued in 1994 (C. Tinney, Pima County Wastewater Management, 1994, pers. comm.).

Appendix C provides construction details and driller's logs for the two wells used in this study (SC-3 and SC-7). The well near the upstream end of the study area (SC-3) lies approximately 30 meters (100 ft) from the west bank of the Santa Cruz River bed, just north of Ina Road. During the period of water level measurements (summer 1994-fall 1995), the low-flow channel migrated from adjacent to the west bank to near the center of the riverbed. The well was completed to a depth of 43 m (140 ft) below ground surface (roughly 5 meters (16 ft) above the adjacent riverbed surface), and screened over the 12-m (40-ft) interval of 27 to 40 m (90 to 130 ft). The well near the downstream end of the study area (SC-7) lies approximately 20 meters (66 ft) from the east bank of the Santa Cruz, just south of Avra Valley Road. This well penetrates to a depth of 56.4 m (185 ft), and is screened over the 18.3-m (60-ft) interval 36.6 to 54.9 m (120 to 180 ft). During the course of this study, the low-flow channel became more focused (instead of bifurcating into west and east channels) near the east bank of the river. Kenneth D. Schmidt and Associates (1988) report that the wells tapped coarse-grained, stream-channel deposits, and that the deposits generally became
finer grained with increased layering toward the north (downstream). Two pump tests conducted by Kenneth D. Schmidt and Associates (1988) yielded transmissivity estimates of 1,612 to 1,984 m²/d (130,000-160,000 gpd/ft) and hydraulic conductivity estimates of 126 to 151 m/d (3,100-3,700 gpd/ft²).

The groundwater hydrographs measured in this study were obtained by installing pressure transducer/temperature sensor probes⁴ (similar to those described in the Surface Water section above) into the 2.5-cm (1-in) sounding tubes in each well. The wells were selected because of their proximity to the ends of the study reach and because of accessibility.⁵ The pressure transducer/temperature sensor probes were calibrated in situ by taking several voltage readings (via a Campbell Scientific, Inc. CR-10 data logger connected to a portable computer) at different submersion depths and recording the associated temperature at each depth. These readings were then regressed to develop a mathematical equation relating transducer voltage and temperature to submersion depth, as described in Appendix A. Each well was equipped with one water-level probe, one barometer probe (0-1,034 mb (0-15 psi) transducer buried in shallow soil adjacent to the well), and a data logger and storage module. Periodic (at one to three month intervals) manual water level measurements in the wells provided truth points for verification and adjustment of the sensor calibrations.

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⁴ The probes were provided on loan from Mike Carpenter of the U.S. Geological Survey-Water Resources Division, Tucson, Arizona.

⁵ Curved sounding tubes in other wells prevented probe installation.
CHAPTER V. RESULTS OF FIELD STUDY

SOIL PARAMETERS

Tension infiltrometer measurements taken in the Santa Cruz River bed near Avra Valley Road (see Figure 3) yielded values for in situ saturated and unsaturated hydraulic conductivities for surficial streambed deposits. Figure 7 illustrates the spatial distribution of the disc tension infiltrometer measurements and corresponding soil core samples. In some sampling locations, two or more flat-bottomed pits were excavated at approximately 30-cm (11.8 in) intervals below the surface for the purpose of obtaining infiltrometer measurements at those depths.

The raw data obtained from the tension infiltrometer survey consisted of infiltration rates measured over time (usually 10 minutes to 1 hour, depending on the soil characteristics) corresponding to several different applied tensions. The applied tensions ranged from 0 to 20 cm (0-7.9 in), and three or more tensions were applied at each sampling point. Although the early-time infiltration rates generally decayed rapidly to approach steady-state infiltration rates, determining when steady-state infiltration had been achieved involved a highly subjective evaluation of the real-time data. In practice, when the time elapsed between several successive observation points (graduations on the storage reservoir of the infiltrometer) remained steady relative to early- and intermediate-time values, or the successive averages for several groups of observations remained steady, steady-state infiltration was assumed to have occurred and the test was terminated for that tension.
Because the tests had no direct bearing on the ultimate objective of determining the time-varying average saturated hydraulic conductivity of the effluent channel, no tests were conducted to evaluate the repeatability of these measurements.

A computer program called "NFIT," distributed by Island Products in Galveston, Texas (Univ. of Texas, 1990), provides an automated calculation of hydraulic conductivity from raw tension infiltrometer data based on the technique described for multiple tensions in Chapter IV. The program allows the user to choose which mathematical model to apply; in this case, the Wooding Equation (1968) (Equation 3, Chapter IV) with the Gardner formula (1958) (Equation 4, Chapter IV) relating saturated and unsaturated hydraulic conductivities. NFIT uses a non-linear best fitting technique to derive an exponential function that describes the hydraulic conductivity-matric potential ($K-\psi$) relationship for the soil. Extrapolating this function back to $\psi = 0$ (which corresponds to saturated conditions), provides an estimate for the value of saturated hydraulic conductivity ($K_{sat}$). In some cases (if the conductivity is low enough), the tension infiltrometer can accommodate a zero-tension measurement, eliminating the need for estimating $K_{sat}$.

Table B1 in Appendix B describes the stratigraphy, soil parameters and corresponding $K_{sat}$ values for samples collected in the streambed transect survey near Avra Valley Road in February 1995 (see Figure 7). Table B1 also provides Gardner Equation (1968) $\lambda_c$’s for each tension infiltrometer sample. Figure 10 illustrates the combined results
of soil sieve analysis data for all of the soil samples collected in the transect survey. Although the samples were sieved in groups of five, the classic gravelly sand (de Marsily, 1986) soil texture curve shown in Figure 10 was very representative of the smaller sample groups.

Figure 10. Lumped Soil Sieve Analyses.

Soil Texture: All Samples
\[ d_{10} = 134.4 \text{ micrometers} \]

SURFACE WATER/GROUNDWATER INTERACTION

Shallow Subsurface

Tensiometer nests installed at two locations in the flowing effluent stream ('Upstream T-Site' and 'Downstream T-Site' in Figure 3) and at one location in a sandbar adjacent to the low-flow stream near Avra Valley Road ('Avra T-Site' in Figure 3) were used to collect
information on vertical hydraulic gradients below the streambed. Three nests of three (0.91-, 1.2-, and 1.5-m-long) tensiometers were installed at each instream site in an effort to get an average characterization across the flowing stream at each location. The tensiometer nest in the sandbar near Avra Valley Road contained four tensiometers penetrating to depths of 1.45, 1.16, 0.88, and 0.70 meters (4.75, 3.81, 2.89, and 2.30 ft) below the surface. These tensiometers were equipped with pressure transducers installed in the side walls of the tensiometer tubes and wired to a data logger buried on the west bank of the river. The transducers were housed in serviceable PVC casings to prevent water from contacting the transducers or shorting the wire connections. In order to mitigate the detrimental impact of severe temperature swings on the transducer measurements, the transducers were emplaced roughly 0.3 m (1 ft) down from the top of each tensiometer. The tensiometers were installed so that all of the transducer housings rested at the same elevation (Figure 11). The soil was excavated to about 0.3 m (1 ft) to allow the transducer housings to rest on the excavated platform. The area above the transducer housings was then back-filled with native sediment and tamped somewhat in an effort to recreate a representative surface.

Figure 12 shows the installation configurations for two of the tensiometer nests at the upstream site. Nest #2 (near the center of the low-flow stream) suffered damage soon after installation and was not used. Nests #1 and #3 at the upstream site were near opposite edges of the low-flow stream and were quickly overgrown by vegetation on the banks of the
Figure 11
Cross-sectional and Plan Views of Avra Valley Road Tensiometer Nest

CROSS-SECTION

PLANVIEW
Figure 12
Cross-Sectional and Plan Views of Upstream Tensiometer Nests

CROSS-SECTION

Plan View

Dry Riverbed

~ 865 m

Effluent Stream

Cortaro Road

Not drawn to scale

Legend
- casing depth
- depth of porous cup

Not drawn to scale

tensiometer nests
stream during the summer of 1995.

Figure 13 illustrates the three tensiometer nest configurations at the downstream site. Unlike the upstream installation, these nests were not installed in a linear transect across the stream channel. The individual nests locations were chosen for protection (from vandals and from high stream flows) and to characterize the entire width of the stream channel.

Figures 14 and 15 illustrate the vertical hydraulic gradients measured in the upstream and downstream tensiometer sites, where the land surface served as the datum for gradient calculations. For the upstream nest (Figure 14), both nests exhibited virtually constant unit gradients for the duration of the 50-day study period (6/2/95-7/21/95). The streambed at that site is very rocky with little or no loose sediment on the surface. Algal growth and coincidental silt entrapment were readily visible at the onset of very warm summer temperatures and persisted throughout the remainder of the study period.

Figure 15 shows the gradients for the three downstream nests. Nests #1 and #3 exhibit a wide diversion in gradients between the upper two tensiometers and the lower two, with the average gradient in both nests lying near -2.0 (downward). The gradients between the upper two and the lower two tensiometers in Nest #2 in the downstream site were similar, with an average gradient of about -0.5. The variation in gradients among the nests and between individual tensiometer pairs in Nests #1 and #3 reflects the pattern of mud lenses
Figure 13
Cross-Sectional and Plan Views of Downstream Tensionmeter Nests

CROSS-SECTION

PLANVIEW

Not drawn to scale
Figure 14.
Vertical Gradients in Upstream Tensiometer Nests

Hydraulic Gradients in Upstream Tensiometer Nest 1

Hydraulic Gradients in Upstream Tensiometer Nest 3

Legend
a 1 to 1.3 m
b 1.3 to 1.6 m
c 1 to 1.6 m
Figure 15.
Vertical Gradients in Downstream Tensiometer Nests

Hydraulic Gradients in Downstream Tensiometer Nest 1

Legend

Day of Year (1=1999)

Gradient

0.00
-0.00
-2.00
-3.00
500.00
550.00
600.00
650.00
700.00

Hydraulic Gradients in Downstream Tensiometer Nest 2

Legend

Day of Year (1=1999)

Gradient

0.00
-0.00
-2.00
-3.00
500.00
550.00
600.00
650.00
700.00

Hydraulic Gradients in Downstream Tensiometer Nest 3

Legend

Day of Year (1=1999)

Gradient

0.00
-0.00
-2.00
-3.00
500.00
550.00
600.00
650.00
700.00

Legend

Day of Year (1=1999)

Gradient

0.00
-0.00
-2.00
-3.00
500.00
550.00
600.00
650.00
700.00
interspersed within the shallow unconsolidated sand channel sediments. During installation, several of the access tubes filled with water when the drive points were pushed out, indicating the presence of shallow (pressurized) confining units under the streambed. The very small gradients (approaching -3.0) in Nests #1 and #3 indicate low matric potentials (high soil suction values) just a few feet under the flowing stream and the absence of free gravity flow (i.e., the presence of impeding layers with strong capillary forces).

Measurements were conducted at the tensiometer nest installed near Avra Valley Road (see Figure 3) from June 1 to mid November, 1994. Although these tensiometers were automated (equipped with pressure transducers), high surface temperatures and occasional flooding over the sandbar which held the tensiometers rendered some of the transducer readings unreliable after a few weeks. As a precaution, manual pressure head readings were taken with a tensimeter throughout the study period. Manually measured values for total head for each of the four tensiometers are shown in Figure 16. Figure 17 illustrates the incremental and total gradients between the tensiometers in this nest. In both figures, the only significant change over time occurs in the shallowest two tensiometers. Figure 16 shows a strong increase in total hydraulic head (reflecting a reduction in suction potential) from summer to late fall in the shallowest tensiometer. This trend most likely represents a response to lower evaporative demands in the upper half meter of soil as air temperatures decreased from summer to fall. The decreasing gradients shown in Figure 17 between the shallowest two tensiometers and between the shallowest and deepest tensiometers also reflect
Figure 16.
Total Head in Tensiometer Nest Near Avra Valley Rd.

Legend:
- d 5' tensiometer
- c 4' tensiometer
- b 3' tensiometer
- a 2' tensiometer
the changing (increasing) pressure head in the shallowest tensiometer. Local variations in response to flood events are most apparent in the upper two tensiometers, indicating relatively stable soil moisture conditions below about 1.3 m (4.3 ft).

As shown in Figure 11, the tensiometers in the nest near Avra Valley Road were installed roughly 1.5 meters (5 ft) from the flowing effluent stream. Extremely rocky soil precluded the augering of tensiometer holes. Instead, a large area was excavated to a depth of about 0.75 m (2.5 ft) and augers were used to complete the deepest tensiometer installations below that level. The shallower tensiometers were emplaced as soil was backfilled. In spite of the proximity of the flowing effluent stream, no water seeped into the excavated hole during the tensiometer installation. This fact suggests that very little lateral flow occurs below the low-flow streambed.

**Groundwater Storm Response**

Figures 18 and 19 show the entire length of record for hydrographs measured in wells SC-3 (near Ina Road) and SC-7 (near Avra Valley Road), respectively. Depths to water in well SC-3 varied from 23.7 to 14.3 m (78 to 47 ft.) below the ground surface, while those in the downstream well (SC-7) ranged from 40.7 to 35.9 m (134 to 118 ft.) below the ground surface between January 1994 and June 1995. Figures 18 and 19 also show surface water discharges for the Santa Cruz River at Cortaro Road. At first observation, the plots reveal that the upstream well (SC-3) exhibits a much more dynamic response to storm events than
Figure 19.
SC-7 Well (Near Avra Valley Road) Hydrograph

Santa Cruz River Discharge (cms)

Day of Year (1=1/1/94)

Depth to Water (neg. meters)
does the well near Avra Valley Road. Since the SC-3 well is shallower and has a shallower average depth to groundwater, this difference is not surprising. The timing of the groundwater response in SC-3 to storm flows is also slightly more immediate than that for SC-7. Although the well hydrograph for SC-3 shows a distinct rebound around day 600, the Santa Cruz River hydrograph shows only a minimal perturbation. However, the first and most significant monsoon rainfall of the season occurred on August 11-12, 1995 (over 2.5" (6.3 cm) in most areas and significantly more near the Santa Cruz River north of Tucson) (National Weather Service, 1995, pers. comm.). In light of the observed (by the author) storm flow in the Santa Cruz at Cortaro Road on August 12, 1995 in response to this precipitation event, the stream hydrograph shown in Figures 18 and 19 is considered to be inaccurate for days 588 and 589.

TRANSMISSION LOSSES

As described in Chapter IV, four manual synoptic runs were conducted during the period November 19, 1994 to May 1, 1995 and automated streamgaging measurements were used to evaluate transmission losses between May and August 1995. Figure 20 illustrates the results of the four manual synoptic runs. In each case the two hydrographs were aligned with a three-hour offset (i.e., the 12:00 measurement at the upstream station corresponds to the 3:00 reading at the downstream station) to attempt to match the peaks and troughs of the diurnal effluent discharge cycle. When the hydrographs are overlain in such a manner, integrating over the area between the two curves gives the volume of water lost to infiltration
Figure 20.
Summary of Manual Synoptic Streamgaging Data;
Area Between Upstream and Downstream Hydrographs Indicates Volume of Transmission Loss Between Stations INAB and AVRAB
and evaporation between the upstream and downstream stations. Some interesting seasonal features of the effluent stream system can be detected from these plots, as well. When interpreting the seasonal behavior of the system, one must consider the pattern of storm flows in the Santa Cruz as illustrated in Figure 5 (Chapter II). This figure illustrates the temporal relationship between the first four (manual) synoptic runs and the storm events on the Santa Cruz River during the winter of 1994-95. Only one relatively minor storm occurred (after more than a year with no significant storm events) before the first synoptic run in November of 1994. Two major storm events (12/5/94 and 1/25/95) occurred between the first and second synoptic runs, and the largest runoff event of the year (2/14/95) separated the second and third synoptic runs. From February to mid August, no runoff-producing rain fell in the Tucson Basin, and conditions in the effluent stream reflected those of a long-term dry period.

Figure 20a shows the measured hydrographs for the synoptic run conducted on November 19, 1994. Equipment failure precluded the completion of an entire 24-hour data collection cycle, but 14 hours of data at the upstream station (on the hour from 00:00 to 13:00)\(^1\) and 21 hours at the downstream station (on the hour from 00:00 to 20:00) were collected. These data, in conjunction with data from subsequent streamgaging sessions and discharge data from the two sewage treatment plants were used to estimate the missing values for the upstream station for the period 14:00 to 23:00. Discharges at the upstream station (INAB) were calculated as a percentage of combined discharges from the sewage

\(^1\)A military time convention expresses a 24-hour based time keeping system. For example, 00:00 corresponds to 12:00 a.m. (midnight), 08:00 is 8:00 a.m., 12:00 is 12:00 p.m., and 18:00 is 6:00 p.m.
treatment plant while considering an appropriate temporal offset for discharges from Roger Road and Ina Road (approximately 8.8 km (5.5 miles) apart). The hydrograph shown for the upstream station in Figure 20a contains those estimated values.

Comparison of the four pairs of synoptic run hydrographs in Figure 20 reveals a trend of increased infiltration during the storm season and gradually decreased infiltration upon the cessation of storm flows. The most dramatic evidence of this trend appears in the contrast between the hydrographs for synoptic runs 1 and 2. In the first synoptic run, the stream flowed continuously at both stations, and the small area between the upstream and downstream curves reveals the comparatively small transmission losses occurring between the two stations. Conversely, the downstream hydrograph for synoptic run 2 (Figure 20b) shows that zero flow occurred at the downstream station for at least six hours (08:00 - 13:00), followed by an abrupt onset of flow by 14:00. The upstream hydrograph in the same figure exhibits a tremendous range of discharges (0.28 to 2.5 cms (10 to 90 cfs)), with the peak discharge exceeding all others for all four synoptic runs shown. This high peak flow at the upstream station reflects the large demands on the sewage treatment plants associated with Tucson's large winter tourist (and temporary resident) population as well as low evaporation losses. The low minimum upstream flow and the wide gap between the upstream and downstream discharges during this synoptic run demonstrate that infiltration losses were quite high.
Figure 20c shows that by late March, the downstream station still experienced several hours (four) of no flow, but gap between the upstream and downstream hydrographs narrowed from that in synoptic run 2. Figure 20d illustrates a marked change in infiltration conditions between late March and early May. By May 1st, the downstream station experienced continuous flow throughout the day and the distance between the upstream and downstream hydrograph curves continued to decrease.

As stated in Chapter IV, automated streamgaging stations were established at three locations in the study reach (INAB, CORT1, and AVRAB on Figure 3). Several logistical problems precluded the use of these continuous data, however, leaving only a selected few points in time amenable to transmission loss calculations. Figure 21 shows the two 24-hour hydrographs used for estimating transmission losses in the study reach for July 12-13, 1995. The upstream hydrograph in this figure derives from automated streamgaging measurements, verified by several manual measurements, made at the CORT1 station roughly 4 km (2.5 mi) downstream from the regular upstream station (INAB). This sensor's data proved to be more reliable and verifiable than the sensors at the INAB station at that time. Superimposing this upstream hydrograph on the downstream hydrograph (measured at the AVRAB station) with a 2-hour offset (rather than the 3 hours used for INAB and AVRAB to account for the shorter flow distance between CORT1 and AVRAB) permits the same transmission loss calculation for the reach downstream of the CORT1 station.
Figure 21.

Upstream (CORT1) and Downstream (AVRAB) 24-hour Hydrographs for July 12, 1995

(2-hour offset applied)

Legend
- CORT1 Hydrograph (1000-0900)
- AVRAB Hydrograph (1200-1100)
Figure 22
Upstream (INAB) and Downstream (AVRAB) 24-Hour Hydrographs for August 2-3, 1995

Discharge (cms)

Time of Day (hour; 0000=midnight)

Legend
- Reconstructed INAB Curve
- AVRAB Hydrograph

(3-hour offset applied)
Figure 22 shows the 24-hour hydrographs used for evaluating transmission losses on August 2-3, 1995. In this case, neither the INAB nor the CORT1 sensors provided reliable data for the upstream hydrograph. Several manual measurements were made on August 2 (Figure 23), however. These values, along with other manual streamgaging values and discharge data from the sewage treatment plants, were used to reconstruct an upstream hydrograph for this date. Figure 24 shows the hydrographs recorded at the INAB and AVRAB stations on August 3, 1995. The fact that the two curves intersect indicates that the sensor-derived upstream hydrograph is inaccurate. The process of reconstructing a curve to replace the recorded INAB hydrograph started with the development of a hydrograph for the combined discharges of the Roger Road and Ina Road sewage treatment plants with the appropriate temporal offset (to account for the 8 km flow distance between the two plants) incorporated into the sum of the two individual discharge curves. This combined treatment plant curve is shown as the top curve in Figure 24, and is labeled "580 recon" in the legend. The curve marked "67.7% of 580 recon" shows values equal to 67% of the combined sewage treatment plant discharges. This percentage (67.7%) was determined by evaluating the relationship between discharges at INAB and the combined sewage treatment plants (with the appropriate offset) for other days when many manual streamgaging values were available (i.e., for the synoptic run on May 1 and for the manual streamgaging done on August 2, 1995).\(^2\) This August 3, 1995 reconstructed curve, and the one reconstructed for August 2, (Figure 23) replaced the automated measurements of discharge at INAB for the purpose of

\(^2\)This method does not directly account for transmission losses between the two plants, but uses the combined plant discharges as a basis for estimating discharges at INAB.
Figure 23

Reconstructed Discharges for August 2, 1995

Discharge (cms)

manual point
manual point
manual point
manual point

Treatment Plants – INAB Station

Time of Day (hour; 0000 = midnight)
evaluating transmission losses in the study reach.

Table 4 summarizes the transmission loss rates determined from the four manual and two automated synoptic runs described above. Table 4 also lists transmission loss rates published by other authors for effluent flows in the same reach of the Santa Cruz River.

Certain constraints govern the interpretation of the results listed in Table 4, however. For the results of this study, all but one of the manual synoptic run transmission loss rates were calculated from 22-hour volumetric losses normalized to 24-hour loss rates (i.e., (22-hour loss rate + 22 hours) / 24 hours). The first synoptic run (11/19/94) only had 18 hours of data on which to base a transmission loss calculation, however, so the normalization to a 24-hour volumetric loss introduces more error due to the cyclic nature of the effluent flows. The daily transmission loss rates calculated from the automated synoptic runs (7/12/95 and 8/3/95) were based on 24 hours of data so no normalization was required. Variations in the length of the study reach also influence the measured transmission losses, even though the results are displayed in "per mile" units. Schumann and Galyean (1991) observed the highest infiltration rates in the entire reach from Roger Road to Avra Valley Road within the short segment between Ina Road and Cortaro Road. For the synoptic run on July 12, 1995, which measured transmission losses downstream from the CORT1 station (lower 3.6 miles of the study reach), the value expressed in Table 4 may be lower than the transmission loss rate on the same date for the full reach length (INAB to AVRAB - 5.5 mi).
Table 4

Transmission Losses from Synoptic Runs (INAB to AVRAB - 8.8 km (5.5 mi))

<table>
<thead>
<tr>
<th>Date</th>
<th>Volume</th>
<th>Loss Rate</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ac-ft</td>
<td>m³</td>
<td>ft/day</td>
<td>m/day</td>
<td>cfs/mi</td>
</tr>
<tr>
<td>11/19/94</td>
<td>27.5</td>
<td>33,974</td>
<td>1.2</td>
<td>0.36</td>
<td>2.5</td>
</tr>
<tr>
<td>1/25/95</td>
<td>52.9</td>
<td>65,298</td>
<td>2.3</td>
<td>0.67</td>
<td>4.8</td>
</tr>
<tr>
<td>3/25/95</td>
<td>38.4</td>
<td>47,402</td>
<td>1.7</td>
<td>0.52</td>
<td>3.5</td>
</tr>
<tr>
<td>5/1/95</td>
<td>37.1</td>
<td>45,731</td>
<td>1.6</td>
<td>0.49</td>
<td>3.4</td>
</tr>
<tr>
<td>7/12/95*</td>
<td>18.7</td>
<td>23,077</td>
<td>1.1</td>
<td>0.33</td>
<td>2.6</td>
</tr>
<tr>
<td>8/2/95</td>
<td>31.0</td>
<td>38,273</td>
<td>1.3</td>
<td>0.39</td>
<td>2.8</td>
</tr>
</tbody>
</table>

Other Published Transmission Loss Rates

<table>
<thead>
<tr>
<th>Author</th>
<th>Reach Length (mi)</th>
<th>Loss Rate (cfs/mi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Matlock, 1966</td>
<td>6.1a</td>
<td>3</td>
</tr>
<tr>
<td>Sebenik, et al, 1972</td>
<td>12.0b</td>
<td>2.2 - 3.5</td>
</tr>
<tr>
<td>Sebenik, 1975</td>
<td>6.5c</td>
<td>4.2</td>
</tr>
<tr>
<td>Sebenik, 1975</td>
<td>5.5d</td>
<td>2.6</td>
</tr>
<tr>
<td>Schumann and Galyean, 1991</td>
<td>5.5d</td>
<td>1.3</td>
</tr>
<tr>
<td>Galyean, 1996</td>
<td>6.5e</td>
<td>3.4</td>
</tr>
<tr>
<td>Galyean, 1996</td>
<td>5.5d</td>
<td>2.9</td>
</tr>
</tbody>
</table>

† Channel area = 93,336 m² (1,005,000 ft²).
* Measured in shorter reach from CORT1 to AVRAB station (5.8 km (3.6 mi)).
† See Table 1 for details on measurements by other authors.

a Ina Rd. to Avra Valley Rd.
b Roger Rd. to Avra Valley Rd.
c Roger Rd. to Cortaro Rd.
d Cortaro Rd. to Avra Valley Rd.
Table 4 also lists the 24-hour-equivalent transmission loss volumes for each synoptic run. The significantly smaller value for the 7/12/95 synoptic run reflects the abbreviated river reach length for that measurement. Comparable volumes were not available from the work by other authors shown in Table 4.
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CHAPTER VI. MODELING

MOTIVATION AND OBJECTIVE

Transmission loss data provide the most direct evidence of time-varying infiltration rates, but, as the results in Table 4 in Chapter V demonstrate, several complexities plague the interpretation of these data for the effluent stream of the Santa Cruz River. Clearly, the time period of measurement, as well as the location and reach length in which the measurement is obtained, strongly influence the value of transmission loss volume and rate. Experience from this study shows that eliminating nonuniformity in all field measurements is, in practice, an unrealistic goal and should not be requisite for interpreting the hydrologic characteristics of the system.

Additional complications in interpreting transmission loss rates derive from the head and area dependence of infiltration. The Green-Ampt (1911) theory describes infiltration of water into soil from surface inundations. This theory assumes that infiltrated water advances downward as piston flow (implying uniform hydraulic conductivity in the wetted zone and constant pressure head at the wetting front). Applying Darcy's Law to this flow system gives:

$$v_i = \frac{K_w (H_w + L_f - h_f)}{L_f}$$

(9)

where, \( v_i \) = infiltration rate [L/T],
\( K_w \) = hydraulic conductivity of wetted zone \([L/T]\),
\( H_w \) = water depth above soil \([L]\),
\( L_f \) = depth of wetting front \([L]\),
and \( h_f \) = pressure head of water at wetting front \([L]\).

Bouwer (1966 and 1986) notes that the soil in the wetted zone is rarely, if ever, fully saturated and that \( K_w \) can be taken as roughly \( 0.5K_{sat} \). Equation (9) shows that the depth of water above the soil surface affects infiltration rate most in early time infiltration, when \( L_f \) is still small. As \( L_f \) increases, its influence eventually outweighs that of \( H_w \), making the depth of water's influence on infiltration negligible (Bouwer, 1986). The Green-Ampt (1911) Equation (9) also shows that the initial infiltration rate (when \( L_f \) is small) greatly exceeds \( K_w \), but that infiltration eventually becomes constant as it approaches, and then equals, \( K_w \). The importance of these early time infiltration effects is uncertain for the effluent stream in the Santa Cruz. While certain areas of the stream "dry out" (experience no surface flow) during the daily trough of the diurnal discharge cycle and then rewet as the peak flow period approaches, these areas never desaturate to a great extent. Some capillary suction effects may influence infiltration in these transition zones at the onset of high flows, but most likely, the residual degree of wetting is high enough to preclude significant suction. Thus, infiltration during the rewetting phase of flow probably approaches \( K_w \) very quickly, and \( L_f \) (the depth of the wetting front) is essentially large from the onset of rewetting because the

\[1\) due to capillary effects (sorptivity); see Philip Equation (2) in Chapter IV.
underlying soil suffered little drainage during the no flow period. In this case, the depth of water above the soil surface ($H_u$) should have little impact on infiltration rate shortly after the immediate onset of flooding and the refilling of soil pores.

Variable discharges create other problematic influences on infiltration. Stream velocity affects the stream's capacity to retain sediment and mobilize sediments on the streambed surface (Matlock, 1965; see discussion in Chapter III). Strong diurnal variations in the effluent stream's velocities may lead to unusual patterns of scour and deposition of fines on the streambed surface, possibly altering the conductivity of the streambed in reaches from hour to hour.

Variable discharges also produce variable wetted surface areas in the stream. Since evaporation from an open body of water is directly related to the surface area of the water body, these fluctuating wetted areas strongly impact the volume of water lost to evaporation during each hour of the day. Perhaps most significantly, wide fluctuations in wetted stream surface area produce correspondingly large changes in the channel surface area for infiltrating water. Darcy's Law describes the movement of water from a higher energy state to a lower energy state through a saturated soil:

$$Q = -KA \frac{dh}{dz}$$ (10)
where, \( Q \) = total discharge through the flow volume \([L^3/T]\),

\( K \) = saturated hydraulic conductivity \([L/T]\),

\( A \) = cross-sectional flow area \([L^2]\),

\( \frac{dh}{dz} \) = hydraulic gradient (change in head per change in flow distance).[ ]

Equation (10) shows that the surface area through which water flows has a large bearing on the total discharge through the flow volume \((Adz)\). As the flow width of the effluent stream fluctuates, the value for \( A \) in that system varies, necessarily impacting the total flow rate through the soil.

In light of the aforementioned complications, interpreting transmission loss data for the purpose of describing time varying streambed infiltration rates presents some difficulty. Figure 25 shows the pattern of discharges for the combined sewage treatment plants, the upstream synoptic streamgaging station (INAB), and the downstream synoptic streamgaging station (AVRAB). The trend in the difference between the combined discharges from the sewage treatment plants and the discharges at the upstream synoptic station (INAB) qualitatively demonstrates the phenomenon of decreasing infiltration through the streambed in warm months. The combined discharges from the treatment plants gradually decrease from mid-winter (January 1995) through late summer (August 1995). By contrast, the measured discharges at INAB follow a more steady trend. The gap between the treatment plants' discharges and the INAB discharge is considerably smaller in July/August than in
January, indicating that less water is lost to infiltration between the two stations during the summer than during the winter. This trend persists in spite of increased evaporation and evapotranspiration demands in the summer months (which would enhance transmission losses upstream of INAB). The gap between discharges at INAB and AVRAB also decreases from January to July/August, indicating the same trend toward lower transmission losses in the summer than in the winter.

Schumann and Galyean (1991) and Galyean (1996) describe transmission losses as a percent of upstream discharge in an effort to normalize infiltration rates for different discharges. Figure 26 illustrates the results of this type of analysis for the synoptic runs in this study. The surface water hydrograph for total flow in the Santa Cruz (at Cortaro Road) is superimposed on the transmission loss plot to provide insight into the significance of storm events in the behavior of streambed infiltration. Figure 27 shows the 24-hour volumetric transmission losses in cubic feet. For both graphs, the points represent measurements of transmission losses between the INAB and AVRAB stations except for the July 12, 1995 point, which reflects losses measured between the CORT1 and AVRAB stations. Both graphs reflect the tremendous increase in streambed infiltration capacity between November 1994 and January 1995 following two major storm runoff events. The plots differ in their respective representations of the interstorm period after the last major winter storm event (February 14-15, 1995). The March 25, 1995 point plots considerably closer to the January 25 point in terms of transmission loss volume (Figure 27) than in terms of percent of
Figure 26
Transmission Losses in Santa Cruz River Effluent Stream as a Percentage of Upstream Flow Volume

Total Discharge (cms)
Figure 27
Volumetric Transmission Losses in Santa Cruz River Effluent Stream

Transmission Loss Volume (cubic meters)

Total Discharge (cms)

Nov. 19, 1994
Jan. 25, 1995
Mar. 25, 1995
May 1, 1995
Aug. 3, 1995
July 12, 1995
Santa Cruz Total Discharge
upstream discharge (Figure 27). Both plots show little distinction in streambed infiltration behavior for the May and August points, and show the July 12, 1995 point as falling well below all other transmission loss values. The trend of decreasing transmission losses, as either a percent of upstream discharge or as an absolute volume, for the interstorm period (February through early August) makes intuitive sense. Based on the literature cited in Chapter III describing the development of *schmutzdecke* on the floor of artificial recharge basins, in laboratory columns, and in the Santa Cruz River bed, the observed trend in streambed clogging during interstorm periods might have been anticipated.

The July 12, 1995 transmission loss data remain fairly incomparable to the other points, however, due to the different upstream station (CORT1) location and correspondingly different upstream to downstream discharge ratios and absolute volumes. Even though infiltration rates were similar for July and August (see Table 4 in Chapter V), the graphical analyses provided in Figures 26 and 27 falsely suggest a significantly different stream recharge behavior for that date.

All of the complications associated with variable discharge rates, inconsistent study reach lengths, and nonuniform measurement times on the interpretation of transmission losses in the effluent stream motivate the search for an alternative form of analysis for the hydraulics of the low-flow stream channel. Determination of the hydraulic conductivity of the streambed through numerical simulation provides a means for normalizing the irregular
transmission loss data to reflect the hydraulics of the streambed at different points in time. The objective in modeling the effluent stream behavior is to derive an average value for saturated hydraulic conductivity of the streambed for the entire reach at a given point in time by matching observed downstream hydrographs with observed upstream hydrographs as the input to the model.

**OVERVIEW of KINEROS AND KINEMATIC WAVE THEORY**

Developed by the United States Department of Agriculture Agricultural Research Service for modeling small agricultural and urban watersheds, KINEROS is a distributed, "event-oriented, physically based model describing the processes of interception, infiltration, surface runoff, and erosion" (Woolhiser, et al, 1990). The acronym "KINEROS" stands for 'Kinematic Runoff and Erosion Model.' The model represents watersheds by a cascade of planes and channels (Figure 28). KINEROS uses finite-difference techniques to solve the differential equations describing channel flow and erosion, overland flow, and sediment transport. The program is also designed to handle spatial variability of rainfall and infiltration, runoff, and erosion parameters. The authors of the model suggest that it may be useful in determining the effects of "various artificial features such as urban developments, small detention reservoirs, or lined channels on flood hydrographs and sediment yield" (Woolhiser, et al, 1990). Because KINEROS' capabilities far exceed the needs for this study, only those portions of the program pertinent to the modeling work done for this study will be described here. For more details on kinematic wave theory and on specifics regarding the
use of KINEROS, the reader should refer to Miller, 1984 and Woolhiser, et al, 1990.

Figure 28
Example of KINEROS Plane and Channel Conceptualization.


The complete de Saint Venant equations for unsteady, nonuniform flow contain expressions for continuity and momentum as shown below.
Continuity:

\[ \frac{\partial (A)}{\partial (t)} + \frac{\partial (uA)}{\partial (x)} = q_c(x,t) \]  \hspace{1cm} (11)

Momentum:\n
\[ \frac{\partial (u)}{\partial (t)} + u \frac{\partial (u)}{\partial (x)} + g \frac{\partial (y)}{\partial (x)} = g(S_o - S_f) + \frac{q_c}{A}(v - u) \]  \hspace{1cm} (12)

where,
- \( A \) = cross-sectional area [L^2]
- \( u \) = average velocity [L/t]
- \( S_o \) = bed slope [L/L]
- \( S_f \) = friction slope [L/L]
- \( q_c \) = lateral inflow rate [L^2/t]
- \( v \) = x-ward component of velocity of lateral inflow [L/t]
- \( g \) = gravitational acceleration [L/t^2]
- \( t \) = time
- \( x \) = space coordinate
- \( y \) = water surface elevation [L]

\(^\dagger\) This formula assumes the momentum coefficient (\( \beta \)) equals 1.

 Instead of solving the entire de Saint Venant equations to model flow through a channel, KINEROS uses the kinematic wave approximation to simulate overland flow and
flow through small channels. In the kinematic wave approximation to channel flow, all of the dynamic (left-side) terms in the Momentum Equation (12) are neglected. In fact, Miller (1984) refers to *kinematics* as "the description of motion without considering the forces giving rise to motion." Waves whose motion is adequately described by these simplifications are called *kinematic* waves as opposed to *dynamic* waves. The assumption that the dynamic terms of Equation 12 are negligible arises from assuming that the shallow-water wave is long and flat so that the friction slope \( S_f \) is nearly equal to the bed slope \( S_o \). Thus, for no lateral inflow, Equation 12 reduces to:

\[
0 = g(S_o - S_f)
\]  

(13)

or

\[
S_o = S_f
\]  

(14)

(Miller, 1984)

With this simplification, the kinematic wave model is based primarily on the Continuity Equation 11. Discharge becomes strictly a function of water height (stage) or area, and can be evaluated using a uniform-flow formula such as the Chezy or Manning formulas. Expressing \( Q \) as a unique function of \( A \) (by replacing \( \partial Q/\partial x \) with \( dQ/dA \cdot \partial A/\partial x \)), noting that \( Q = uA \), and using the definition of kinematic-wave celerity, \( c = dQ/dA \), in the Continuity Equation 11 yields the kinematic wave equation:
\[ \frac{\partial A}{\partial t} + \frac{1}{c} \frac{\partial A}{\partial x} = q_c(x,t) \]  \hspace{1cm} (15)

The kinematic assumption is embodied in the relationship between channel discharge and cross-sectional area:

\[ Q = \alpha R^{m-1} A \]  \hspace{1cm} (16)

where \( R \) is hydraulic radius (Woolhiser, et al, 1990).

KINEROS provides several options for the determination of \( \alpha \) and \( m \) which describe channel roughness or resistance to flow. Manning's Law of hydraulic resistance is given by:

\[ \alpha = 1.49 \frac{S^{1/2}}{n} \]  \hspace{1cm} (17)

and \( m = 5/3 \), where \( S \) is the slope, \( n \) is Manning's roughness coefficient, and English units are used. The Chezy Law gives:

\[ \alpha = CS^{1/2} \]  \hspace{1cm} (18)

where \( C \) is the Chezy friction coefficient and \( m = 3/2 \).

The infiltration expression developed by Smith and Parlange (1978) provides the
basis for infiltration calculations in KINEROS:

\[ f_c = K_s \frac{e^{FB}}{e^{FB} - 1} \]

where, \( K_s \) = saturated hydraulic conductivity [L/T],
\( f_c \) = infiltration capacity [L/T],
\( B \) = \( G (\theta_s - \theta_i) = G \phi (S_{max} - S_i) \) [L]

where \( G \) = net capillary drive,
\( \theta_s \) and \( \theta_i \) = saturated and initial soil moisture contents,
\( \phi \) = soil porosity,
\( S_{max} \) and \( S_i \) = maximum and initial values of relative saturation,
defined as \( \theta/\phi \),

and \( F \) = cumulative infiltration depth [L].

Equation 19 describes infiltration capacity \( f_c \) as a function of initial water content \( \theta_i \) and the amount of water already absorbed into the soil \( F \). Smith and Parlange (1978) derived Equation 19 from simplifying assumptions allowing an analytic solution of the underlying Richards flow equation and continuity of water across the surface. As in many infiltration models, effective saturated hydraulic conductivity \( K_s \) and effective net capillary drive \( G \) are key parameters in the Smith-Parlange infiltration model. The net capillary drive is defined as:
\[ G = \frac{1}{K_s} \int_{-\infty}^{0} K(\psi) \, d\psi \]  

(20)

where, \( \psi \) = soil matric potential [L],

and \( K(\psi) \) = hydraulic conductivity function [L/T].

The net capillary drive is similar to the sorptivity term described by White and Sully (1987) in that it is conceptually a soil characteristic and does not incorporate the effect of initial water content. Equation 20 permits the determination of \( G \) from the \( K(\psi) \) curve for the soil, which is available from disc tension infiltrometer data.

For early times after the onset of flooding or rainfall (generally when \( F/B < 0.1 \)), when sorption dominates the system, Equation 19 approaches the gravity-free infiltration relation:

\[ f_c = \frac{BK_s}{F} \]  

(21)

Figure 29 illustrates the three-phase theoretical infiltration process for uniform soils. The first phase is supply limited by incoming water (rainfall, overland or channel flow), and \( F \) accumulates at the rainfall (or input) rate. When \( f_c \) equals the becomes limiting (input rate \( \geq f_c \)), ponding occurs and \( F = F_p \) (ponding depth). After that time, infiltration rate, \( f \), is
Figure 29.
Graphical Representation of KINEROS Infiltration Expression.

\[ \log f = \frac{B K_s}{F} \]


described by Equation 19 and is soils limited (limited by \( f_c \)) (Woolhiser, et al, 1990).

KINEROS accommodates varying width stream segments as indicated in Figure 30. Figure 31 illustrates the conceptual channel cross-section used in KINEROS. \( BW \) represents the channel bottom width, \( ZL \) and \( ZR \) are normalized left and right channel slope ratios and \( h \) is stage.
Conceptualized River System

upstream width of reach R1

length of reach R1

downstream width of reach R1

Downstream Width R(n) = Upstream Width R(n+1)

Natural River System

Sandbar

Figure 30
KINEROS2 Channel Conceptualization

GENERALIZED CROSS-SECTION

SS1  h  BW  SS2
Woolhiser, et al. (1990) incorporated an empirical "effective wetted perimeter" ($p_e$) function to compensate for error in the calculated wetted perimeter of the channel caused by the trapezoidal channel simplification:

$$p_e = \min\left(\frac{h}{0.15\sqrt{BW}}, 1.0\right)p \quad (22)$$

Equation 22 was generally found to improve simulations of transmission losses for ephemeral streams in the Walnut Gulch Experimental Watershed. Figure 32 illustrates the significance of the wetted perimeter correction (Equation 22) in very rough channels.
KINEROS2 Program Modifications

Smith, et al (1993) modified the infiltration expression in KINEROS to accommodate a multilayered soil system and to provide a physically-based approximation for the redistribution of soil water, including recovery of infiltration capacity during a hiatus, and a method that more accurately determines infiltration rates following a hiatus. Because the middle Santa Cruz channel system was modeled as a simple, single layer with no rainfall (and therefore, no hiatus for soil moisture redistribution), these options were not utilized for this study. For details on the multilayered infiltration expression, the reader is referred to Smith (1990), Smith, et al (1993), and Corradini, et al (1994).

Other modifications incorporated into KINEROS2 add flexibility to the channel representation in the model. El-Shinnawy (1994) a dual hydraulic conductivity representation developed a method for incorporating overbank flow in the form of a compound channel into KINEROS by means of. Unkrich (U.S. Dept. of Agriculture - Agricultural Research Station, 1996, pers. comm.) modified KINEROS to allow for an upstream “injection” of flow into the channel, rather than requiring channel flow to derive from runoff.

The major assumptions and limitations of KINEROS, as they affect application to the effluent stream in the middle Santa Cruz River, are summarized below.
The kinematic wave approximation to the de Saint Venant equations neglects the dynamic terms of the Momentum Equation (12), so that the solution for discharge depends primarily on the Continuity Equation (11). Discharge is a function of stage (or area) alone (Equation 116), and the friction slope is equal to the bed slope. Physically, use of the kinematic wave approximation means that neither backwater nor pressurization (as in a circular conduit) may exist and waves propagate only downstream and without attenuating (although they do distort). The 0.3% average channel slope of the middle Santa Cruz River falls within the practical limits for application of the kinematic approximation to stream systems (D. Goodrich, U.S. Dept. of Agriculture - Agricultural Research Station, 1995, pers. comm.)

The same assumptions incorporated in the de Saint Venant equations (hydrostatic pressure distribution, one-dimensional flow, approximately straight channel, small channel slope and fixed channel bed, resistance coefficients for steady uniform turbulent flow apply, incompressible fluid) (Chow, et al, 1988) also apply to the kinematic wave approximation.

The event-based nature of KINEROS is reflected in its inability to describe evapotranspiration and soil water movement between storms (other than during rainfall recession). Given initial soil moisture conditions, it calculates surface runoff (and erosion) for a single event, but it cannot maintain a hydrologic water balance
between storms.

- KINEROS uses the Smith-Parlange (1978) infiltration equation to calculate cumulative infiltration through a channel bed. While this solution may be appropriate for the progression of a wetting front down through a previously dry (ephemeral) streambed, it has no capacity to reflect feedback from the groundwater system. If the soil column under the stream becomes fully saturated and hydraulic connection is achieved between the stream and the aquifer, this infiltration model would be inappropriate. The new two-layer scheme in KINEROS2 addresses this issue.

KINEROS APPLICATION

The channel “injection” option in KINEROS2 allows simulation of channel flood routing without simulating the overland flow processes that typically initiate streamflow or stream flooding in a natural watershed. For the Santa Cruz effluent stream application, water was “injected” into the channel at the INAB station and routed through the channel with abstractions calculated by the KINEROS infiltration expression. The upstream hydrograph input file consisted of the measured hydrograph with an artificial “buildup” from zero flow to the first measured discharge value to provide a smoother input function for the simulation. Several tests were conducted to evaluate the need for an added cycle of discharge through the river system to establish a wetted condition prior to the formal simulation. The tests
revealed that priming the system in such a way had no impact on the simulated downstream hydrograph since the channel was specified as being saturated (parameter SATURATION=1.0) from the start of the simulation. Thus, with no backwater effects and no sorption effects, the flood wave propagates down the “empty” channel with the same velocity and magnitude as it would in a water-filled channel.

A KINEROS “parameter” file incorporates the effluent channel geometry determined from aerial photos (see Chapter IV). This parameter file also includes estimated values for saturated hydraulic conductivity (KSAT), effective net capillary drive (G), pore size distribution index (DIST), porosity (POR), and volumetric rock fraction (ROCK), initial degree of saturation (SAT), and a flag (WOOL) for implementing an “effective wetted perimeter” expression for each channel segment. The following excerpt from a KINEROS2 parameter file illustrates the format and values for the parameter file.

Sample Parameter File for KINEROS2:

```
BEGIN GLOBAL,,,,
UNITS = METRIC, CLEN = 1068 ! m,,,,
END GLOBAL,,,,
!----------------------,,,,BEGIN INJECT,,,,
ID=1,,,,! block #1,,,,
FILE = 3231B.dat,,,,
END INJECT,,,,
!----------------------,,,,
BEGIN CHANNEL,,,,
ID=2,,,,! block #2,,,,
UPSTREAM = 1,,,,! channel reach 1,,,,
LENGTH = 298.5,,,,! m,,,,
WOOL = YES,,,,! effective wetted perimeter function,,,,
WIDTH,SLOPE,MANNING,DEPTH,SS1,SS2
```
KSAT was held constant for the entire study reach for simplicity. In reality, the development of *schmutzdecke* in the streambed may vary in some systematic fashion with distance from the treatment plant outfalls, but this trend was not clearly apparent from field observations. While unit vertical gradients were clearly present under rocky upstream channel deposits, downstream vertical gradients reflected the presence of shallow subsurface impeding layers that appeared to be mud lenses instead of surficial schmutzdecke deposits. The fining of streambed sediments with distance downstream likely affects the streambed's transmissive capacity as least as much as any systematic variation in the development of schmutzdecke as a result of changing water chemistry (i.e., ammonia volatilization, sorption, etc.).
Initial Simulations

Table 5 shows the model parameter values used in the initial synoptic run simulations. The initial KSAT value used for the KINEROS2 simulations came from the average KSAT value obtained from tension infiltrometer tests on the surface and shallow subsurface near Avra Valley Road (see Chapter IV and Appendix B). After discarding one extreme outlier (907 mm/hr) from the tension infiltrometer data set, the resultant average saturated hydraulic conductivity value was 52 mm/hr (refer to Table B2 in Appendix B). These values compared relatively well (larger by about a factor of two) with average infiltration rates computed from the cumulative 24-hour synoptic run data (see Table 4 in Chapter V) and provided satisfactory simulation results after some experimentation.

<table>
<thead>
<tr>
<th>WOOL</th>
<th>MAN</th>
<th>KSAT</th>
<th>G</th>
<th>DIST</th>
<th>POR</th>
<th>ROCK</th>
<th>SAT</th>
</tr>
</thead>
<tbody>
<tr>
<td>min.</td>
<td>YES</td>
<td>0.03</td>
<td>50 mm/hr</td>
<td>18</td>
<td>0.4</td>
<td>0.40</td>
<td>0.7</td>
</tr>
<tr>
<td>max.</td>
<td>0.03</td>
<td>50 mm/hr</td>
<td>21</td>
<td>0.7</td>
<td>0.45</td>
<td>0.2</td>
<td>1.0</td>
</tr>
</tbody>
</table>

The initial Manning’s $n$ value used in the KINEROS2 simulations (0.030) was obtained from tables of typical roughness coefficients for sand and gravel in natural stream channels (Shen and Julien, 1993; Chow, 1959). By comparison, Sebenik (1975) chose a Chezy roughness coefficient of 40. The KINEROS User’s Manual reports that a Manning’s $n$ of 0.012 approximately equals a Chezy C of 38 (Woolhiser, et al, 1990).
Analysis of the soil core samples collected from the riverbed transect survey near Avra Valley Road yielded an average porosity value of about 0.40 (see Chapter IV and Table B1 in Appendix B). The porosity value was increased from 0.40 to 0.42 and then 0.45 in the last two-thirds of the model reach to reflect increasing clay content in the streambed. The values for ROCK (0.2 - 0.7) and DIST (0.4 - 0.7) were estimated strictly from a visual analysis of the stream bottom at various locations throughout the study reach with additional insight provided by Smith (1995). The value for ROCK was decreased from 0.7 to 0.4 below Cortaro Road to reflect a significant change in channel character from a very rocky-bottomed reach to a more anastomosing, sandier reach. The DISTRIBUTION parameter was increased with distance downstream to reflect the wider grain size distribution in the downstream half of the study reach. Estimates for effective net capillary drive length (18 \leq G \leq 21) were based on measured soil properties tabulated in Smith (1995). SATURATION was set to 1.0 (100\%) in every stream segment to mimic continuous flow conditions. The effective wetted perimeter correction flag (WOOL) was turn on (set to “YES”) on the recommendation of experienced KINEROS users (D. Goodrich, C. Unkrich, USDA-ARS, 1996, pers. comm.) (see discussion of Equation 22 earlier in this chapter).

Results of Initial Simulations

Figure 33 shows the resultant downstream hydrographs from the initial simulations described above for November 19, 1994. The three separate graphs show the response of KINEROS2 to changes in KSAT. The simulated downstream hydrograph in Simulation 1
Figure 33
November 19, 1994 KINEROS2 Simulations

Legend

- input (upstream) hydrograph
- measured downstream hydrograph
- simulated downstream hydrograph

a) KSAT = 20 mm/hr

b) KSAT = 30 mm/hr

c) KSAT = 40 mm/hr
(KSAT=20 mm/hr) lies above the measured hydrograph, indicating that the infiltration rate (and corresponding KSAT value) is too low. The third simulation (KSAT = 40 mm/hr) produces a simulated hydrograph that falls below the measured downstream discharge curve, indicating that infiltration is too large (KSAT is too high). The simulation with KSAT = 30 mm/hr (Simulation 2) comes closest to matching the observed downstream hydrograph, and is clearly bracketed by the KSAT values in the Simulations 1 and 3.

Figures 34 through 38 show the same process for bracketing KSAT in the initial KINEROS2 simulations for the remaining synoptic runs. In some cases (see Figure 34, Simulations b and c), determining the best fit between simulations with different KSAT values is highly subjective. In general, the simulated hydrographs resulting from these initial simulations shared the following characteristics: 1) most of the curves preceded the actual hydrographs by several minutes (up to about an hour); 2) the simulated curves generally underpredicted the peak flow and overpredicted the minimum flow of the real downstream hydrographs.

Sensitivity Analysis and Detailed Calibration

In an effort to improve the match between simulated and measured downstream hydrographs, the various parameters and channel geometry values included in the KINEROS2 parameter files were examined for their respective impacts on the simulations. Because stream width and side slope dimensions were estimated from 1:12,000 aerial photos
Figure 34
January 25, 1995 KINEROS2 Simulations
Figure 37
July 12, 1995 KINEROS2 Simulations
Figure 38
August 3, 1995 KINEROS2 Simulations

Legend
- input (upstream) hydrograph
- measured downstream hydrograph
- simulated downstream hydrograph
and a few field measurements and then generalized for upstream and downstream ends of each stream segment, the channel geometry described by the KINEROS parameter file may introduce some error. Additional error may be introduced in estimating a bottom width for a natural stream with extremely low-angle side slopes. The conceptual representation of this natural configuration and of small sandbars within very wide, shallow stream segments is highly subjective. Two simulations were conducted to evaluate the potential impact of channel geometry representation errors on the simulation of downstream flows; one with channel widths increased by 1 meter in every stream segment and one with channel widths reduced by 1 meter in every segment. Figure 39 shows the results of the channel width sensitivity analysis for the July 12, 1995 synoptic run. This figure illustrates that increasing or reducing the surface area of the stream by roughly 10% has a negligible effect on the simulations.

Figure 40 shows simulated curves using values of $G$ equal to 10, 21, and 30. All three simulated downstream hydrographs are virtually indistinguishable, indicating the model's lack of sensitivity to this parameter. Reference to Equation 19 reveals that, for initially saturated conditions, where $\theta_i = \theta_s$, $G$ has no impact on the value of infiltration capacity ($f_e$). In other words, the sorptive forces represented by the parameter $G$ are negligible under saturated conditions.

The model was also insensitive to changes in the value of the volumetric rock
Figure 39
July 12, 1995 KINEROS2 Simulation - Bottom Width Sensitivity

KSAT = 30 mm/hr

Legend
- Input (upstream) hydrograph
- Measured downstream hydrograph
- Width increased by 1 m
- Width decreased by 1 m
- Original width

Discharge (cms)

Time (min)
Figure 40
July 12, 1995 KINEROS2 Simulation - G Sensitivity
KSAT = 30 mm/hr
fraction (ROCK) parameter. Varying ROCK from 0.6 to 0.1 had no noticeable impact on the simulated downstream hydrograph (Figure 41). Since ROCK only modifies the value of net capillary drive (G), its impact should be negligible under saturated conditions.

Figure 42 illustrates the effect of varying Manning’s n from 0.02 to 0.035. These values represent typical limits on published values for sand-and-gravel-bottom stream channels, either natural or dredged. The results shown in Figure 42 demonstrate that a lower roughness coefficient raises the peak discharge and accelerates the flood wave’s arrival downstream.

Determining that the simulations were essentially insensitive to all of the parameters and geometric descriptors described above prompted a closer examination of the effective wetted perimeter correction factor (WOOL flag). Woolhiser, et al (1990) derived this empirical relationship from extensive experience in simulating ephemeral stream flows in the Walnut Gulch Experimental Watershed in southeastern Arizona. Equation 2 can be interpreted as follows: the wetted perimeter, p, for a 100-m wide channel is reduced (multiplied) by a factor equal to the stage height (h) over 1.5 until the stream stage reaches a depth of 1.5 m. For \( h \geq 1.5 \) m, the wetted perimeter is calculated by adding the bottom width to the submerged lengths on the side slopes of the channel. This formula (Equation 22) implies that the specified bottom width \( (BW) \) of the channel segment is not entirely submerged until \( h/(0.15*BW^{0.5}) = 1.0 \). Thus, for a typical segment of the effluent stream in
Figure 41
July 12, 1995 KINEROS2 Simulation - ROCK Sensitivity
KSAT = 30 mm/hr

Legend
- Input (upstream) hydrograph
- Measured downstream hydrograph
- ROCK = 0.6
- ROCK = 0.1

Discharge (cms)

Time (min)
the Santa Cruz (12 m wide), setting WOOL = YES means that the effective wetted perimeter correction applies until \( h \geq 52 \text{ cm} \). Field experience suggests that this conceptualization does not accurately represent conditions in the low-flow channel. The effluent channel between Ina Road and Avra Valley Road is generally flat-bottomed, typically requiring only about 10 cm of water to submerge the entire bottom width. Some exceptions do occur, but even in the most uneven transects, 25 cm of water would be more than sufficient to submerge the channel bottom.

Figures 43 through 48 show the results of repeating the initial simulations but without the effective wetted perimeter correction, \( p_e \) (i.e., WOOL = NO). In general, eliminating \( p_e \) produced closer matches between the simulated and observed downstream hydrographs. In most cases, simulating without the effective wetted perimeter correction improved the model’s ability to match both the peak and the minimum discharges and improved the timing of (delayed) the simulated wave’s arrival at the downstream station (see Figures 35 and 45, for example).

Removing the effective wetted perimeter correction had a tremendous impact on the calibrated value of KSAT required to match the simulated and observed downstream hydrographs. Figure 49 illustrates the “best fit” KSAT values for the two groups of simulations (with and without \( p_e \)). While the trend in the two groups of KSAT values is similar, the KSAT magnitudes vary greatly between the two simulation sets. The effective
Figure 4.4
January 25, 1995 KINEROS2 Simulations
No Wetted Perimeter Correction

b) KsAT = 40 mm/hr

c) KsAT = 50 mm/hr

Legend
- input upstream hydrograph
- measured downstream hydrograph
- simulated downstream hydrograph
Figure 4.5
March 25, 1995 KINEROS2 Simulations
No Wetted Perimeter Correction

Legend
- input (upstream) hydrograph
- measured downstream hydrograph
- simulated downstream hydrograph

(a) $K_{SAT} = 15$ mm/hr
(b) $K_{SAT} = 25$ mm/hr
(c) $K_{SAT} = 35$ mm/hr

Discharge (cms) vs. Time (min)
May 1, 1995 KINEROS2 Simulations
No Wetted Perimeter Correction

Figure 46
Figure 47
July 12, 1995 KINEROS2 Simulations
No Wetted Perimeter Correction

a) KSAT = 10 mm/hr

b) KSAT = 13 mm/hr

c) KSAT = 16 mm/hr

Legend
- input (upstream) hydrograph
- measured downstream hydrograph
- simulated downstream hydrograph
Figure 48: August 3, 1995 KINEROS2 Simulations
No Wetted Perimeter Correction

Legend:
- input (upstream) hydrograph
- measured downstream hydrograph
- simulated downstream hydrograph

b) $KSAT = 14 \text{ mm/hr}$

c) $KSAT = 20 \text{ mm/hr}$

Discharge (cm)

Time (min)

Legend:
- input (upstream) hydrograph
- measured downstream hydrograph
- simulated downstream hydrograph
wetted perimeter correction reduces the available recharge area in the streambed, thereby forcing KSAT values to be higher in order to allow the same volume of water to infiltrate.

Since KINEROS has no provision for modeling evaporation or evapotranspiration losses, the potential impacts of these losses on the overall transmission loss during each synoptic run were evaluated by imposing the entire estimated volume of daily evaporation on the upstream hydrograph. Average daily Class A pan evaporation rates (see Table 2 in Chapter IV) were multiplied by a factor of 0.70 (Nokes, 1995) to approximate open channel evaporation losses. The appropriate evaporative volumetric losses were then distributed over the course of 24 hours for each synoptic run date in proportion to the air temperature increase from daily minimum. Figure 50 illustrates the effect of adding 10.0 mm/day (the highest average daily rate for all of the synoptic run dates) evaporation losses (over the entire surface area of the channel) to the upstream hydrograph for August 3, 1995. Even this diminution in upstream flow had only a negligible impact on simulated downstream flows, and was therefore considered insignificant to the determination of streambed conductivity. Galyean (1996) evaluated evapotranspiration potential for the effluent stream in the Santa Cruz and concluded that total evapotranspiration and evaporation losses averaged about 3% of the total effluent discharge.
Figure 50
Impact of Open-Channel Evaporation on Effluent Discharges

Legend
- Upstream Discharge
- Evaporation
- Net Discharge

Evaporation = 10.01 mm/day
Average wetted area = 93,336 sq. meters

Discharge (cms)

Time (hr)
CHAPTER VII. DISCUSSION

SURFACE WATER

Figure 49 in Chapter VI illustrates the discrepancy in saturated hydraulic conductivity (KSAT) values determined by two different wetted channel computation options in KINEROS2. Comparison of the KSAT values determined by model calibration with observed infiltration rates provides a means for choosing the appropriate group of KSAT values. Table 5 in Chapter V shows that infiltration rates calculated from the raw synoptic run data range from 0.33 m/d (13.75 mm/hr (0.54 in/hr)) to 0.67 m/d (27.9 mm/hr (1.1 in/hr))

\[ 0.33 \text{ m/d (13.75 mm/hr (0.54 in/hr)) to 0.67 m/d (27.9 mm/hr (1.1 in/hr))} \]

These limits correspond closely to the range of KSAT values (12 to 31 mm/hr (0.47-1.2 in/hr)) determined from KINEROS2 without the effective wetted perimeter correction, suggesting that the \( p_e \) correction (Equation 22 in Chapter VI) is inappropriate for this stream system.

The effect of temperature on hydraulic conductivity in the streambed must also be considered in an evaluation of the magnitude and rate of change of KSAT. Hydraulic conductivity is a function of the intrinsic permeability \( k \) of the porous medium and the kinematic viscosity \( \nu \) of the fluid flowing through the medium and can be expressed as:

\[
K = \frac{k \rho g}{\mu} = \frac{k g}{\nu}
\]  

\[ (23) \]

---

\[ ^{1}\text{Sebenik (1975) reported a maximum hourly infiltration rate of 1 in/hr and an average rate of about 0.5 in/hr (see Chapter III).} \]
where, $K$ = hydraulic conductivity [L/T],

$k$ = intrinsic permeability [L²/T],

$\rho$ = fluid density [M/L³],

$g$ = acceleration due to gravity [L/T²],

$\mu$ = dynamic viscosity [M/(L·T)],

and $v$ = kinematic viscosity [L²/T].

Since the kinematic (fluid) viscosity ($v$) is inversely related to temperature, hydraulic conductivity increases with increasing temperature. Figure 51 shows KSAT values derived from KINEROS2 simulations before and after correction for temperature-related viscosity changes in the stream water. In order to compare the KSAT values measured at different temperatures, the original KSAT values were all normalized to an equivalent KSAT at 20°C (68°F). Thus, the calibrated KSAT values (derived from KINEROS2) for the cold months (November and January) were suppressed and those for the warm months (May through August) were increased because of variations in the stream water temperature. The normalized (temperature-corrected KSAT values) represent more accurately the transmissive capacity of the streambed at that point in time. Figure 52 plots the normalized KSAT values with average daily stream water temperatures for the same dates. This graph illustrates the strong correspondence of streambed clogging (decreasing KSAT) and warmer temperatures. This relationship likely derives from increased microbial activity in the streambed during the warmer months (see discussion in Chapter III).
Figure 51
KSAT Calculated In KINEROS2 Simulations Before and After Temperature/Viscosity Correction

KSAT (mm/hr)

Day of Year (1 = 1/1/94)

Legend
- Uncorrected
- Temperature Corrected
Figure 52
KSAT and Water Temperature

(no wetted perimeter correction; temperature corrected KSAT)
Figure 53 illustrates the relationship between the temperature-corrected KSAT values derived from KINEROS2 simulations and storm/interstorm periods. The four winter storms of 1994-95 served to "reset" the hydraulic conductivity of the effluent streambed system, increasing KSAT nearly three-fold (13 to 37 mm/hr, 0.51-1.4 in/hr) from November 19, 1994 to January 25, 1995. An estimated value of KSAT (equal to the January 25, 1995 value but corrected for the temperature on February 15, 1995) is plotted on February 15, 1995 to demonstrate the probable maximum KSAT after the last major storm of the season (Feb. 14-15, 1995). This point marks the last streambed disturbance by storm flow until mid August 1995. The KSAT values for February 15 through August 3, 1995 exhibit an exponential decay in response to increasing average daily temperatures and lack of streambed scour. Figures 54 (linear scale) and 55 (semi-log scale) show the best-fit curve through the KSAT vs. time data derived from the KINEROS simulations. As shown in the figures, the decay of saturated hydraulic conductivity with time since the last major storm event (Δt) for this study period is described by the equation:

\[ KSAT(t) = -5.11835 \cdot \ln(\Delta t) + 37.3032 \]  

(24)

with a coefficient of determination (R²) of 0.995 and a residual mean square (σ²) of 0.767.

Understanding the driving mechanisms controlling the rate and magnitude of change in saturated hydraulic conductivities in the effluent stream provides important insight to the recharge potential of the effluent stream system. The approximate combined discharge
Figure 53
KSAT and Santa Cruz River Discharge at Cortaro Road
(no wetted perimeter correction, temperature corrected KSAT)
Figure 54
KSAT vs. Time Since Last Major Storm Event (Feb. 15, 1995 to Aug. 3, 1995)

Fit Results

Equation:
\[ Y = -5.11835 \times \ln(X) + 37.3032 \]
Number of data points used = 5
Average \( \ln(X) \) = 3.61508
Average \( Y \) = 18.8
Regression sum of squares = 464.978
Residual sum of squares = 2.30187
Coef of determination, R-squared = 0.995074
Residual mean square, sigma-hat-sq'd = 0.767292
Figure 55
Semi-log Scale Plot of KSAT vs. Time

Fit Results
Equation:
Y = -5.11835 * ln(X) + 37.3032
Number of data points used = 5
Average ln(X) = 3.61508
Average Y = 18.8
Regression sum of squares = 464.978
Residual sum of squares = 2.30187
Coef of determination, R-squared = 0.995074
Residual mean square, sigma-hat-sq'd = 0.767292
capacity of the two sewage treatment plants upstream of the study area was 176,300 m$^3$/d or 64 MCM/yr (46 MGD) in Water Year 1995 (Pima County Wastewater Management data for 1994, 1995). Figure 56 illustrates the potential effluent infiltration volumes for the 8.8-km (5.5-mi) study reach associated with different values of streambed hydraulic conductivity. The minimum, maximum, and weighted mean KSAT values (normalized to 20°C (68°F)) determined from the KINEROS2 simulations in this study are shown as vertical bars. The height of each KSAT bar indicates the corresponding volume of secondary effluent that would infiltrate into the streambed (with no evaporation/transpiration losses) within the study reach (area = 93,336 m$^2$ (10$^6$ ft$^2$)) if that conductivity rate applied for the entire period of study (257 days). The horizontal line marks the approximate volume of water that would infiltrate in the study reach (under the meteorologic conditions of this study period) according to the more realistic conductivity distribution shown in Figure 57. Figure 56 demonstrates that even using the weighted mean conductivity value$^2$ yields a significantly different total recharge value (18.2 MCM (14,700 ac-ft)) than the infiltration volume predicted from the more realistic (step-function) KSAT distribution (12.1 MCM (9,800 ac-ft)). Curiously, this latter infiltration volume corresponds to an equivalent KSAT value of 14.8 mm/hr (0.58 in/hr), which is exactly the KSAT value derived from KINEROS2 simulations and synoptic run data for May 1, 1995.$^3$

$^2$The weighted mean KSAT value is calculated from the step-function shown in Figure 57 by the following formula: $\sum_n (\Delta t)_n/(257 \text{ days}) \cdot \text{KSAT}(n)$, where $n$ is the number of different KSAT values in the step-function distribution and (\Delta t)$_n$ is the duration of the single value, KSAT$_n$.

$^3$The 12.1 MCM recharge estimate for Water Year 1995 (Oct.-Sept.) is likely somewhat conservative, since the period of study stops short of two major storm events that occurred in August and September of 1995.
Figure 56: Potential Infiltration Volumes for Different KSAT Values

- Ksat (max) = 37.3 mm/hr
- Ksat (weighted mean) = 23.4 mm/hr
- Ksat (min) = 11.0 mm/hr

Infiltration for Realistic KSAT Distribution (over 257 days)

Million Cubic Meters

KSAT (mm/hr)

0 5 10 15 20 25 30 35

30.00 20.00 10.00 0.00
In terms of basin-scale water management, determining the actual hydraulic conductivity distribution for the effluent channel in the Santa Cruz River holds enormous importance. Burkham (1970a) and Schwalen and Shaw (1961) estimate total natural recharge to the Tucson basin at 58-61 MCM/yr (47,000 - 49,000 ac-ft/yr). Thus, the estimated effluent recharge volume of 12.1 MCM/yr (9,800 ac-ft/yr) (based on only 257 days of observation) in the 8.8-km (5.5-mi) reach in this study amounts to roughly 19% of the basin's natural recharge. Using the maximum KSAT value (37.3 mm/hr (1.47 in/hr)) determined in this study yields a recharge estimate of 30.6 MCM/yr (24,800 ac-ft/yr), while using the minimum KSAT value (11.0 mm/hr (0.43 in/hr) results in an estimate of only 9.0 MCM/yr (7,300 ac-ft/yr) effluent recharge in the study reach. These recharge estimates vary from 14 to 48% of natural basin recharge. Clearly, significant error could be introduced into an estimate of effluent recharge if only one or two infiltration rates (or KSAT values) were measured at random in a given year.

Equation 24 provides water resource managers a tool for estimating the hydraulic conductivity of the effluent streambed in this study reach given the storm and temperature history of the area. While the relative contributions of time since the last major storm event and ambient air temperature on the development of streambed clogging cannot be separated without more data, the graphs in Figure 52 suggest that time is more important than increasing temperature for this particular data set. Caution must be exercised in extrapolating these results to different climatological conditions, however. The response of
hydraulic conductivities to time after the last storm event of the winter season may be significantly different than it would be to time after the last summer storm of the season. In the first case (as in this study period), daily temperatures exceeded 17°C (63°F) after the last storm in February and continued to rise for several months. After a major summer storm season (ending perhaps as late as mid-September), daily temperatures would initially be high but would rapidly decrease over the next several months. Since warm temperatures promote microbial activity, this change in temperature regime after a major storm event might be accompanied by less dramatic clogging effects than those observed in this study.

The general impact of climate change on the recharge capacity of the effluent stream cannot be ignored. As discussed in Chapter II, the Tucson basin and the Santa Cruz River, in particular, have exhibited at least three distinct flooding periods over the past century. A cyclic shift from wetter fall/winters and drier summers to drier fall/winters and wetter summers, and back, was clearly documented with a periodicity of about 30 years since the turn of the century (Betancourt and Webb, 1992). This type of climatological change could significantly modify the recharge characteristics of the effluent stream in the Santa Cruz by changing the frequency, magnitude and distribution of storm events that disturb or eliminate the schmutzdecke in the streambed.

Figure 53 provides a framework for estimating the threshold storm event required to significantly improve streambed conductivity in the low-flow channel of the study reach.
The figure shows the first runoff-producing storm in over one year occurring on day 316 (Nov. 12, 1994). The U.S. Geological Survey (USGS, 1996) estimates this storm's peak average daily discharge at 21.7 cms (766 cfs). The first synoptic run of the study was conducted just one week later (Nov. 19, 1994). Casual observation of the stream channel pre- and post-storm revealed only minimal disturbance to the streambed sediments as a result of this storm. The hydraulic conductivity value derived from the synoptic run data and KINEROS2 modeling was 13.7 mm/hr (0.54 in/hr), only 2.7 mm/hr (0.10 in/hr) more than the lowest KSAT value for the entire study period. This fact substantiates the notion that this small storm runoff did not significantly improve streambed infiltration in the low-flow channel. The next two storm events, on Dec. 5-6, 1994 62 cms (2200 cfs) and Jan. 5, 1995 87 cms (3070 cfs) markedly improved the recharge capacity of the low-flow streambed. The Jan. 25, 1995 KSAT value is the highest of the study period (although no synoptic run data were available immediately following the larger storm in February, 1995). Without more detailed synoptic run data, one can only guess the relative importance of the December and January storms in terms of improving streambed conductivity in the effluent channel. While a 62-cms (2200-cfs) average daily discharge storm event might have been sufficient to maximize KSAT in this reach, other scenarios are probably more realistic. The combined impact of two consecutive major storms is likely to be larger than the individual impact of either storm. For the purposes of water resources management, a threshold average daily discharge of 80-85 cms (2800-3000 cfs) is probably a reasonable estimate for determining the onset of significantly higher (near maximum) infiltration rates after a long interstorm
The rapid response of the groundwater system adjacent to the river to storm flows in the Santa Cruz indicates the tremendous recharge capacity of that system. Streambed conductivities in the effluent stream decreased in the dry, warm months (in the absence of storm runoff) diminishing the recharge capacity of the stream during peak groundwater pumping months. By contrast, the storms in the winter of 1994-95 greatly enhanced the conductivity of the effluent stream, permitting maximum recharge during the season of minimal groundwater pumping and low evapotranspiration demands.

The presence of unit gradient conditions under the flowing effluent stream in summer months indicates that the stream water is perched at the surface due to a clogging layer. Field observations and KSAT-temperature relationships indicate that this clogging layer develops rapidly with the onset of warmer temperatures. The lowest KSAT value of the entire study period (11 mm/hr (0.43 in/hr)) was observed on August 3, 1995 during the period of warmest average daily surface water temperatures and after 5.5 months with no storm flows. Separating the relative importance of increasing temperature and time between storm events cannot be readily achieved without more data. While laboratory experiments to evaluate the temperature dependence of *schmutzdecke* development in soil columns supplied with effluent are currently underway in the Environmental Engineering Department at the University of
Arizona (G. Wilson and M. Conklin, Dept. of Hydrology and Water Resources, Univ. of Arizona, 1996, pers. comm.), the results of that study are not yet available.
CHAPTER VIII. SUMMARY AND CONCLUSIONS

A significant portion (currently approaching 90%) of the treated sewage effluent processed by the two treatment plants serving the greater Tucson area is available for passive recharge through the Santa Cruz River streambed north of Tucson. In the absence of any major disturbance of the streambed, the recharge capacity of the streambed materials decreases over time as microbial activity, and possibly suspended sediments settling out of solution, act to clog the surficial sediments under the effluent stream. Effluent stream transmission-loss measurements made over the period from November 1994 to August 1995 provided data used to determine the average vertical hydraulic conductivity of the low-flow channel in the study reach with the computer model known as KINEROS2. Saturated hydraulic conductivity (KSAT) served as the calibration parameter in the model. The appropriate KSAT value was chosen for each set of field data by matching the observed and simulated downstream hydrographs for the study reach. KSAT values were corrected for viscosity changes resulting from changing average daily surface water temperatures over the study period.

The following conclusions can be drawn from this study:

1) Streambed hydraulic conductivities for the effluent channel ranged from 37.3 mm/hr (1.5 in/hr) in late January, 1995 after three winter storm events (two exceeding 62 cms (2200 cfs) average daily discharge) in 54 days to 11.0 mm/hr (0.43 in/hr) in early
August, 1995 (after 169 days of no storm runoff). KSAT values decreased sharply after the last winter storm of the season (February 15, 1995) and the exponential decay of KSAT with time is well described by the equation:

\[
KSAT(t) = -5.11835 \ln(\Delta t) + 37.3032
\]  

where \(\Delta t\) is time since the last major storm event. This relationship is not entirely general and should not be extrapolated to time periods where meteorological/climatic conditions differ significantly from those observed in this study. The exponential decrease in KSAT values described by Equation (24) incorporates the effects of both time and temperature on the development of a clogging layer in the effluent streambed. While many column studies have documented the development of a clogging layer in effluent-flooded sand columns under constant temperature conditions, data indicating the relative importance of temperature on the formation of the clogging layer are not yet available.

2) A flow of approximately 85 cms (3000 cfs) average daily discharge measured at Cortaro Road bridge is a conservative estimate for the threshold magnitude of storm flow required to achieve significant improvement in the hydraulic conductivity of the effluent stream channel.

3) Estimates of recharge based on the maximum and minimum KSAT values derived in this study yield estimates of effluent recharge in the study reach from over 48% to only 14% of total natural recharge to the Tucson basin, respectively. A more realistic (step-
function) distribution of KSAT over the study period provides a recharge estimate of 12.1 MCM (9,800 ac-ft), or about 19% of total annual natural recharge to the Tucson basin.

4) Groundwater levels in wells adjacent to the Santa Cruz River at the upstream and downstream ends of the study reach reflected very abrupt and rapid responses to storm flows during the winter storm period. The groundwater response to daily effluent flows was masked by heavy groundwater pumping in the area and the buffering effects of a 20-40 meter (65-130 ft) thick vadose zone underlying the effluent stream.

5) KINEROS2 provides a useful tool for interpreting transmission loss data in the effluent stream by accounting for the effects of channel geometry and varying upstream discharges. The data derived from both the field work and modeling efforts in this study should contribute significantly to the bodies of literature on passive recharge of treated effluent and on implementing a computer model to evaluate the recharge characteristics of a stream with time varying infiltration rates.

In light of the rapid population growth Tucson is expected to experience in the next several decades, gaining a better understanding of the recharge potential for treated effluent should remain a high priority. As the population increases, the supply of secondary effluent available for passive recharge in the Santa Cruz River channel will increase. This study provides the first quantitative delineation of the rate and magnitude of change in hydraulic conductivity of the effluent channel sediments. Continued, systematic efforts to obtain more
data like those described in this study will ultimately provide a good understanding of the rate limiting processes for effluent recharge and should greatly improve prospects for managing this important resource.
APPENDIX A

Calibration of Submersible Pressure Transducer/Thermocouple Sensors
CALIBRATION OF SUBMERSIBLE PRESSURE TRANSDUCER/Thermocouple Sensors

Although the Motorola MPX2200AS pressure transducers used for water stage measurements in this study are rated as "temperature compensated" by the manufacturer, the residual error in the regression used to determine stage from sensor output (voltage) can be significantly improved by accounting for temperature effects in the manual calibration procedure (Carpenter, 1994). The typical calibration procedure involves lowering the sensor into a standpipe and recording the output voltages corresponding to different depths. The straight-line fit (linear regression) for the plot of submersion depth versus sensor output voltage is described by Equation 1:

\[ V = a + bP \]  \hspace{1cm} (A1)

where, \( V \) = sensor output voltage (dependent variable),
\( P \) = pressure corresponding to submersion depth (independent variable),
and \( a \) and \( b \) are regression coefficients. Solving Equation A1 for \( P \) gives:

\[ P = \frac{V - a}{b} \]  \hspace{1cm} (A2)

Temperature effects were incorporated into the calibration procedure by measuring
the sensors' output voltages at different submersion depths and different temperatures at each submersion depth. For this study, the sensors were submerged to at least four different depths in a constant-temperature warm water bath by bailing the water bath to sequentially lower water levels. After completing one set of constant temperature, variable depth measurements, the bath was refilled and the temperature allowed to reequilibrate at a new (usually higher) value before repeating the submersion process.

The multiple regression equation used to relate sensor output voltage to the corresponding water depth and temperature is written as:

$$V = a + bP + cP^2 + dT + eT^2 + fTP$$  \hspace{1cm} \text{(A3)}$$

where, \(V\) = sensor output voltage, 
\(P\) = pressure, 
\(T\) = water temperature, 
and \(a, b, c, d, e,\) and \(f\) are regression coefficients. Carpenter (1994) reports that using this multiple regression calibration procedure reduces the residual error by an order of magnitude (from \(\pm 30\) mm to \(\pm 3\) mm) over the procedure described by Equations A1 and A2 for a calibration submersion depth range of 10 meters of water. Solving Equation A3 for pressure gives:

$$P = \frac{- (fT + b)}{2c} \pm \frac{\sqrt{(fT + b)^2 - 4c(a + dT + eT^2 - V)}}{2c}$$  \hspace{1cm} \text{(A4)}$$
Using Equation 4 requires that water temperature be measured simultaneously with pressure. Figure 4 in Chapter IV illustrates the thermocouple placement within the submersible sensor housing near the pressure transducer. Both the thermocouple and pressure transducer outputs were recorded with a Campbell Scientific, Inc. CR-10 data logger. The CR-10 has an internal program to convert thermocouple output to temperature (in °C), but the pressure transducer outputs were recorded as raw voltages and later regressed to determine submersion depth.
APPENDIX B

Soil Sample Analyses
<table>
<thead>
<tr>
<th>Core Sample #</th>
<th>Depth below surface (cm)</th>
<th>Description</th>
<th>Dry Bulk Density (g/cm³)</th>
<th>Porosity</th>
<th>Tension Infiltrometer Sample #</th>
<th>Ksat (cm/hr)</th>
<th>ρc (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1a</td>
<td>0-19</td>
<td>tan fine, well sorted sand</td>
<td>1.38</td>
<td>0.48</td>
<td>1a</td>
<td>0.24</td>
<td>0.39</td>
</tr>
<tr>
<td></td>
<td>19-21</td>
<td>tan fine to coarse sand, poorly sorted sand</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td></td>
<td>21-25</td>
<td>tan v fine to med., well sorted sand</td>
<td></td>
<td></td>
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<tr>
<td></td>
<td>25-44</td>
<td>tan/grey fine to med., well sorted sand</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>44-49.5</td>
<td>tan med., well sorted sand</td>
<td></td>
<td></td>
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<td>Porosity</td>
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Table B2
Saturated Hydraulic Conductivity Values from Tension Infiltrometer

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Ksat Values from Tension Infiltrometer

All Samples

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<td>Sample Variance</td>
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<td>2800.702</td>
</tr>
<tr>
<td>Sum</td>
<td>1690.52</td>
<td>783.6426</td>
</tr>
<tr>
<td>Count</td>
<td>16</td>
<td>15</td>
</tr>
<tr>
<td>Confidence Level(95.0%)</td>
<td>117.065</td>
<td>29.30706</td>
</tr>
</tbody>
</table>

Without Sample #16

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
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</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td></td>
<td>52.24284</td>
</tr>
<tr>
<td>Standard Error</td>
<td></td>
<td>13.66431</td>
</tr>
<tr>
<td>Median</td>
<td></td>
<td>32.436</td>
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<tr>
<td>Standard Deviation</td>
<td></td>
<td>52.92166</td>
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<tr>
<td>Sample Variance</td>
<td></td>
<td>2800.702</td>
</tr>
<tr>
<td>Sum</td>
<td></td>
<td>783.6426</td>
</tr>
<tr>
<td>Count</td>
<td></td>
<td>15</td>
</tr>
<tr>
<td>Confidence Level(95.0%)</td>
<td></td>
<td>29.30706</td>
</tr>
</tbody>
</table>
APPENDIX C

Well Logs and Construction Details for SC-3 and SC-7
WELL LOGS AND CONSTRUCTION DETAILS FOR SC-3 AND SC-7

The following diagrams of well construction and driller's logs for Pima County Wastewater Management wells SC-3 and SC-7 were obtained from Kenneth D. Schmidt and Associates, 1988.
SCHEMATIC DIAGRAM OF SHALLOW MONITOR WELL SC-3
SCHEMATIC DIAGRAM OF SHALLOW MONITOR WELL SC-7

- Land Surface
- 18-inch Diameter Hole
- 14-inch Diameter Steel Conductor Casing
- 12 1/4-inch Diameter Hole
- Blank 6-inch Diameter Steel Casing
- Cement Grout
- Centralizers
- Tacna No. 4 Gravel Pack
- Perforated 6-inch Diameter Schedule 80 PVC Casing (8 rows of 0.05-inch slot at 72 slots per foot) with flush joint threaded ends
- Bottom Cap

DEPTH (FEET)

0
42
96
120
180
185

Schematic Diagram of Shallow Monitor Well SC-7
GEOLOGIC LOG FOR WELL SC-3 (Near Ina Rd.)

Elevation: 2166 feet above mean sea level

State Well Location: (D-12-12) 35 ddd

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-5</td>
<td>Fill. Pale yellow-brown fine sand and silt.</td>
</tr>
<tr>
<td>5-10</td>
<td>Sand with some pebbles. Light brown medium to coarse sand uncemented with subrounded pebbles.</td>
</tr>
<tr>
<td>10-15</td>
<td>Sand. Light brown uncemented fine to medium sand.</td>
</tr>
<tr>
<td>15-20</td>
<td>Sand and gravel with some pebbles. Pale brown uncemented medium to coarse sand and gravel. Pebbles subrounded to rounded and primarily volcanic rocks.</td>
</tr>
<tr>
<td>20-40</td>
<td>Sand and pebbly gravel. Pale brown medium to coarse sand and gravel with subrounded to rounded pebbles.</td>
</tr>
<tr>
<td>40-50</td>
<td>Sand and pebbly gravel. Yellow brown medium to coarse uncemented sand and gravel with pebbles. Gravel and pebbles are primarily volcanic rocks, but also metamorphic and sedimentary rocks are present. Pebbles are subangular to well rounded.</td>
</tr>
<tr>
<td>50-60</td>
<td>Clayey, silty sand with pebbly, cobbly gravel. Brown fine to medium uncemented sand and gravel with silt, clay, and cobbles. Gravel, pebbles, and cobbles are volcanic, metamorphic, and sedimentary rocks, fairly evenly distributed.</td>
</tr>
<tr>
<td>60-85</td>
<td>Sand and pebbly gravel. Brown medium to coarse uncemented sand and gravel with pebbles. Gravel and pebbles are primarily volcanic rocks.</td>
</tr>
</tbody>
</table>
| 85-90      | Silty sand and pebbly, cobbly gravel. Brown fine to medium uncemented sand and gravel with silt, pebbles, and cobbles. Gravel, pebbles and cobbles,
are primarily volcanic rocks.

90-110

**Sand with some pebbles and cobbles.** Brown medium to coarse uncemented well sorted sand with some pebbles and cobbles. Pebbles and cobbles are primarily volcanic rocks.

110-125

**Clayey, silty sand.** Brown fine to medium sand uncemented with silt and clay and some pebbles and cobbles.

125-140

**Sand and pebbly, cobbly gravel.** Brown medium to coarse uncemented sand and gravel, with pebbles and cobbles. Gravel, pebbles, and cobbles are both volcanic and sedimentary rocks.
GEOLOGIC LOG FOR SC-7 (Near Avra Valley Road)

Elevation: 2068 feet above mean sea level

State Well Location: (D-12-12) 8 cba

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-10</td>
<td><strong>Fill.</strong> Silty sand and generally subrounded to well rounded pebbles and small cobbles.</td>
</tr>
<tr>
<td>10-25</td>
<td><strong>Silty sand and pebbly gravel.</strong> Brown medium to coarse uncemented sand and gravel with silt and pebbles. Gravel and pebbles generally subrounded to well rounded and of many rock types.</td>
</tr>
<tr>
<td>25-35</td>
<td><strong>Clayey, silty sand and pebbly, cobbly gravel.</strong> Brown fine to medium uncemented sand and gravel with clay, silt, pebbles and cobbles. Gravel, pebbles, and cobbles are of diverse rock types.</td>
</tr>
<tr>
<td>35-45</td>
<td><strong>Silty clay with sand and pebbly gravel.</strong> Brown clay and silt with uncemented sand, gravel, and pebbles.</td>
</tr>
<tr>
<td>45-60</td>
<td><strong>Sand and pebbly gravel.</strong> Yellow brown clean fine to coarse sand and gravel with subangular to rounded pebbles. Gravel and pebbles are of diverse rock types.</td>
</tr>
<tr>
<td>60-92</td>
<td><strong>Sand and pebbly gravel with silty clay.</strong> Brown medium to coarse uncemented sand and gravel with pebbles, silt, and some clay.</td>
</tr>
<tr>
<td>92-119</td>
<td><strong>Sand and pebbly, cobbly gravel.</strong> Brown medium to coarse sand and gravel with pebbles and cobbles. Pebbles range from subangular to well rounded. Gravel, pebbles, and cobbles are primarily volcanic and metamorphic rocks.</td>
</tr>
<tr>
<td>119-125</td>
<td><strong>Silty clay.</strong> Brown soft clay and silt, with some sand, gravel, and pebbles.</td>
</tr>
<tr>
<td>125-140</td>
<td><strong>Sand and pebbly gravel.</strong> Brown clean medium to coarse uncemented sand and gravel with pebbles. Pebbles are angular to well rounded and if diverse</td>
</tr>
</tbody>
</table>
rock types.

140-173  **Sand and pebbly gravel with some silty clay.** Brown medium to coarse sand and gravel with pebbles and some silty clay.

173-184  **Sandy clay.** Yellow brown sticky sandy clay with some pebbles and gravel.

184-185  **Sand and pebbly gravel.** Brown medium to coarse sand and gravel with pebbles and some silt and clay.
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