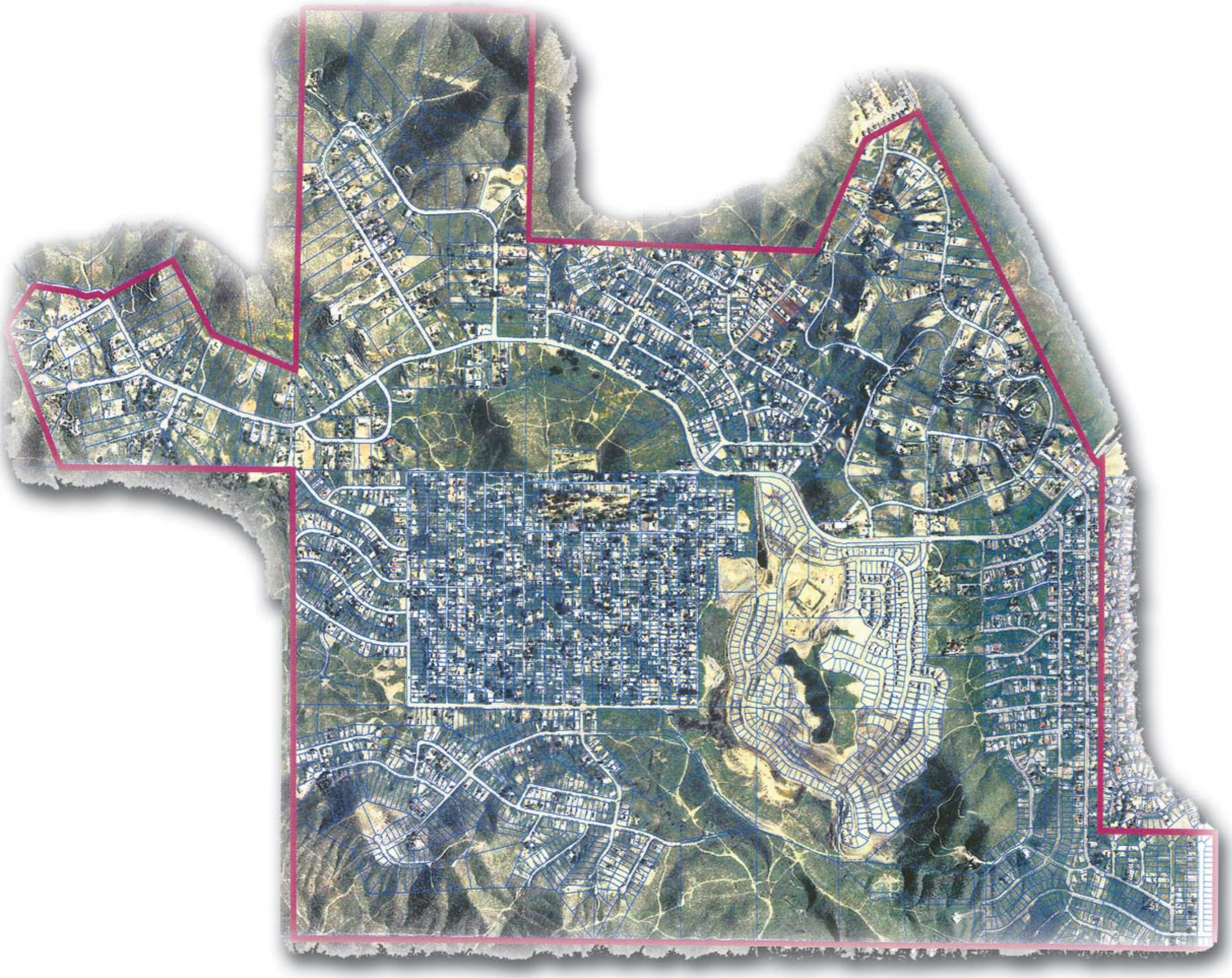
OCTOBER 2005

# QUAIL VALLEY SEWER IMPROVEMENTS ALTERNATIVES STUDY



Prepared For:

EASTERN MUNICIPAL WATER DISTRICT

SINCE 1950

Prepared By:

PBS&J

590127.01

# QUAIL VALLEY SEWER IMPROVEMENTS ALTERNATIVES STUDY

**August 2005**

**DRAFT**

**Prepared For:**

**EASTERN MUNICIPAL WATER DISTRICT**

**Prepared By:**

PBS&J

**10370 Hemet Street, Suite 200**
**Riverside, CA  92503**

PBS&J Project No.: 590127.01

______________________________

Gary H. Eikermann
Project Director

# Table of Contents

## Tables



## Figures



## Appendices



I:\admin\590127 Elsinore Valley MWD\Reports\SW Aug05 Quail Valley Draft Report 9-7- MARKUP.doc

# Chapter 1
# Introduction

## 1.1 GENERAL

The community of Quail Valley is located in southwestern Riverside County, immediately north of Canyon Lake (see Vicinity Map – **Figure 1**). It is accessible by both I-15 and I-215. The study area occupies an area of about 1.3 square miles. It is located at latitude 33° 42'24" N and longitude 117° 14' 46" W (Sections 19, 30 and 31 of T5SR3W and, sections 25 and 36 of T5SR4W of San Bernardino Meridian.. Current population is estimated at about 4,400 in approximately 1,400 homes. There is significant growth potential, since less than one-third of the 3981 residential parcels have existing homes. The community has a mixed ethnic population, with the median household income estimated to be below $38,000. Eastern Municipal Water District (EMWD) provides water for the community. All residences within the community are on individual septic systems.

Failing septic systems in Quail Valley have resulted in polluted water in the community and in downstream Canyon Lake, a potable water supply reservoir for Elsinore Valley Municipal Water District (EVMWD). A meeting was convened by the Santa Ana Regional Water Quality Control Board (SARWQCB) in February 2005, which involved entities including the County of Riverside, EMWD, EVMWD, and the cities of Lake Elsinore and Canyon Lake to discuss the problem and find possible solutions. EMWD has since initiated this *Quail Valley Sewer Improvements Alternatives Study* with the objective of finding a feasible solution to the problem of septic system pollution. The area encompassed by this study is shown in **Figure 2**.

Note that the subject study is limited to the identification and evaluation of the collection system within Quail Valley. Implementation of a Quail Valley sewer system would also entail the construction, expansion or shared use of several offsite facilities including lift stations, force mains, trunk sewers and the regional treatment plant. At the appropriate stage, the District will determine an appropriate connection fee and service charge to cover the construction modification, or use of these offsite facilities.

## 1.2 STUDY AREA CHARACTERISTICS

### 1.2.1 Topographic and Geologic Features

The topography of the study area is characterized by undulated land profile. The elevation range above Mean Sea Level (MSL) is 1,760 feet at the northeast corner to 1,420 feet at the southwest corner near Canyon Lake-an overall topographic relief of approximately 340 feet.

Quail Valley is located within the Peninsular Ranges Geomorphic Province of Southern California, more specifically in the southern portion of Perris Block. The Perris Block is an eroded mass of Cretaceous and older crystalline metamorphic rock. It has undergone extensive erosion and deposition in response to vertical movements of the Elsinore and San Jacinto Fault Zones. The bedrocks within the site are overlain by topsoil, alluvium, artificial fill and other deposits. The artificial fills are essentially associated with the development of streets and grading for residential developments. The topsoil is derived from weathering of bedrock and older alluvial deposits. In general, the slopes are stable.

The preliminary geotechnical feasibility investigation conducted as part of this study indicates that construction is feasible from a geotechnical engineering standpoint, but that shallow bedrock and groundwater are significant issues. Details are addressed in a subsequent section of this report.

### 1.2.2 Groundwater and Hydrology

The soil conditions and geological features of the project area indicate high ground water levels within the project area, at least seasonally and in some areas perennially. At some locations, groundwater may be encountered at depths as shallow as four feet below ground surface. The water table at times intersects the ground surface in some locations, resulting in ponded water. These conditions are verified by observation from borings and other excavations. The presence of high groundwater levels may be attributed to dense alluvial deposits and/or bedrock beneath the surface that does not allow the surface water to percolate easily. The depth to perched groundwater fluctuates depending on rainfall in the vicinity of the project area.

Surface runoff from the project area migrates generally in a southerly or southwesterly direction to Canyon Lake, located south of Quail Valley. Groundwater migration in the shallow soil profile is thought to be in the same general direction. The project area is located at the boundary of Flood Control District Zones 3 and 4 of the Riverside County Flood Control District. It is part of the San Jacinto Watershed. The San Jacinto River flows along the western side of the project area and drains into Canyon Lake.

### 1.2.3 Receiving Waters

Canyon Lake, the receiving water body for the surface and sub-surface flow from the study area, is located north of Railroad Canyon Road and west of Goetz Road. It falls under the jurisdiction of EVMWD. Specifically, the East Bay and Bass Cove portions of the Lake receive the majority of the surface flow from the project area. Any discharge into the Lake or any impact on the water quality of the Lake is governed by 401 permit (Corps of Engineers) and 404 permit (Regional Water Quality Control Board). Since the Lake is a potable water supply reservoir for EVMWD, the County Department of Environmental Health (DEH) and the State of California Department of Health Services (DHS) are also involved regulatory entities.

## 1.3 SCOPE OF STUDY

The scope of study includes four principal tasks, briefly described as follows:

**Task 1 Data Gathering:**
Assemble and review relevant data including aerial photographs, topographic features, geology, existing infrastructure, water use records, etc.

**Task 2 Geotechnical Investigation:**
Conduct a broad overview geotechnical investigation, including limited borings and seismic refraction surveys, to estimate subsurface conditions relevant to construction of a sewer system.

**Task 3 Alternatives Evaluation:**
Conduct a concept level investigation of alternatives to provide sewer service to the study area, and provide recommendations based on life-cycle cost of facilities, level of construction disruption, and operation/maintenance considerations  Alternatives to be considered include conventional gravity systems as well as pumped or vacuum systems and feasible combinations.

**Task 4 Report of Findings:**
Prepare a brief Project Report describing the investigation methodology, conditions, assumptions, alternatives considered, and recommendations.

# Chapter 2
# Existing Facilities and Conditions

## 2.1 EXISTING FACILITIES

As previously discussed, the homes in the area are served by septic systems and there are presently no facilities for collection and conveyance of wastewater from the project area to the EMWD regional treatment facility. The EMWD Perris Valley Regional Water Reclamation Facility (PVRWRF), east of the study area, is the logical candidate to treat wastewater conveyed through a future community sewer system. The current capacity of the PVRWRF is 11 MGD, to be expanded to 22 MGD by the year 2012. The District is planning to construct major trunk lines in the southerly portion of Goetz Road and along Newport Road. The design plans are on hold awaiting the results of this study. As presently envisioned, wastewater flows from the Quail Valley study area would flow through the trunk sewer in Goetz Road, along Old Newport Road to the Audie Murphy Ranch Regional Lift Station. Sewage would then be pumped to a manhole just west of Berea Road near its intersection with Newport Road. It would then flow by gravity to the Sun City lift station. The Sun City lift station has a current peak flow capacity of 8 MGD and is being expanded to 4 pumps for 10 MGD capacity. The Sun City lift station pumps the flow northward through a force main to the PVRWRF. In the near term, the planned community of Canyon Heights within the Quail Valley study area is to be served by a collection system flowing to an EMWD interim lift station and force main on Goetz Road.

## 2.2 EXISTING UTILITIES

There are a number of overhead and underground utilities within the study area that might interfere with the construction of new wastewater facilities. PBS&J used “Underground Service Alert Database for Southern California” and compiled a list of utilities agencies within the study area. These agencies were contacted for “no conflict” response. A listing of contacted utilities agencies and their responses can be seen in **Appendix C**. For existing water facilities within the study area, the GIS database of EMWD was used. The water facilities within the study area are generally aligned within the street rights-of-way. Further investigation and co-ordination with utility agencies will be required for specific location of facilities at the design stage. From preliminary research, following is a summary of utilities (existing and proposed) within the study area.

- Water – Existing Water Distribution Facilities by EMWD (see layout, **Appendix C**).
- Gas – Existing 8-inch gas line along Newport Road up to Goetz Road and a proposed 5,429 foot gas line along Goetz Road – Southern California Gas. Individual dwellings in majority of area are served by on-site propane tanks.
- Electrical – Overhead and underground electrical facilities, alignment to be determined by further investigation – Southern California Edison. Most services in older areas are by overhead lines.
- Cable – Underground cables, alignment to be determined by further investigation – Comcast Cable.
- Telephone Lines – Alignment to be determined by further investigation – Verizon.

## 2.3 SEWAGE AND GREY WATER COMPLAINTS

The Riverside County Department of Environmental Health (DEH) observed in its study (February 2005) that there are widespread instances and evidences of septic system failures in the Quail Valley area. The residents of the area have logged numerous complaints to the County about sewage flow or "grey water" flows on their property and in some cases into the streets. The problem areas and the complaints for the years 2003 and 2004 are shown in **Appendix A** of this report.

As previously mentioned, Canyon Lake has been seriously impacted by inflows that contain wastewater from the failed septic systems, a situation which was exacerbated by the abnormally wet winter of 2004-05. The City of Canyon Lake conducted independent tests on the storm water outfalls from the Lake and on incoming storm water from the Quail Valley area and found high levels of pathogens. The surfacing groundwater in the Quail Valley area also shows high pathogenic contamination.

## 2.4 EXISTING SEPTIC SYSTEMS

As previously discussed, existing homes in the study area are served exclusively by septic systems, since there is no community collection system. Disposal of septic effluent from individual unit septic tanks is by on-site leach fields (leach lines) or seepage pits. The study area soil conditions are generally not conducive to sustainable leaching of septic effluent, particularly in smaller parcels where there is inadequate leaching area. This is evidenced by the high rate of septic system failures and resulting pollution problems in the area. The larger lot subareas (lot sizes of one acre or greater) are less prone to septic system failures, since longer leach lines can be constructed within the parcel boundaries. From the County DEH complaints map, the higher density areas have apparently experienced the majority of reported problems with saturated soil profiles and surfacing of septic effluent. The primary problem is simply that there is inadequate land and leaching surface to accommodate the volume of effluent produced. The septic effluent thus mounds under the leach fields, and then surfaces or migrates along the bedrock interface and surfaces at some downstream point, resulting in aesthetic issues and health risks.

## 2.5 ENVIRONMENTAL AND HEALTH ISSUES

SARWQCB and County DEH officials are convinced that a significant contributor to contamination of the Lake is the failure of septic systems in the Quail Valley area and resulting inflow of surface and sub-surface contaminated water into the Lake, particularly during wet weather and prolonged storm events. This surfacing of septic effluent has created health issues for the residents of Quail Valley, the City of Canyon Lake, and the EVMWD. Canyon Lake was closed for swimming, wading and water-skiing due to high bacteria levels. This issue has drawn considerable media attention. The Lake has been listed as an impaired water body by the federal government, due to elevated levels of nitrates, phosphorous and pathogens.

As previously mentioned, the SARWQCB recently convened a meeting of various agencies that have a stake in this issue. These agencies include the County of Riverside, EMWD, EVMWD, the City of Lake Elsinore and City of Canyon Lake. One of the possible solutions discussed in the meeting was to investigate the feasibility of installing a publicly-owned and maintained sewer system in the area. Since the project area falls under it's jurisdiction, EMWD has assumed the primary investigational responsibility.

Since the Quail Valley area is only partially developed (about one-third of the zoned residential parcels are currently occupied), the prospect of continued development in the absence of a community sewer system is particularly troubling to many residents, downstream parties, and regulatory entities.

# Chapter 3
# Design Considerations

## 3.0 GENERAL

Identification of a feasible concept for a community sewer system to serve the study area poses a significant challenge due to several circumstances, some of which were previously mentioned, as follows:

- Retrofit construction in a partially developed area with existing homes, streets and utilities.
- High density, crowded conditions in portions of the community, with very small lots and narrow streets.
- The need to consider potential flows from ultimate development in the study area, which could be more than triple the existing dwelling units and wastewater generation rates.
- Lack of a major trunk system or treatment facility in the immediate vicinity.
- Difficult terrain, with highly undulating relief; including valleys and ridges generally sloping away from the nearest planned trunk system collection point.
- Challenging soil conditions in much of the area, due to shallow bedrock and/or boulders, as well as high groundwater.

The challenge is to define an affordable system which would not cause undue disruption during construction or result in an untenable operation/maintenance burden.

## 3.1 WASTEWATER GENERATION

The starting point for an analysis of sewer system concepts is a reasonably reliable estimate of wastewater discharges from development in the study area. Although the impetus for this sewer feasibility study is pollution caused by failing septic systems in existing development, any community sewer system concept must take into account probable future development.

### 3.1.1 Existing and Ultimate Development

For convenience of analysis, the study area is divided into nine subareas, each having reasonably common development characteristics (see Figure 4). From the parcel layer obtained from the County, a total of 3,585 parcels are counted for the entire study area, making the Quail Valley area potentially a very populous residential community. From a count of existing dwelling units (DU) made using the year 2005 aerial, a total of 1,390 dwelling units is estimated. In viewing the parcel map, it is apparent that the great majority of the study area has been subdivided to the extent feasible, given the constraints of terrain and zoning. Development densities and lot sizes vary greatly from subarea to subarea. Subareas 1, 6, and 8 feature lot sizes of one acre or greater with a few exceptions. Subareas 2, 3, 5, and 9 feature lot sizes corresponding generally to medium-density residential, ranging from about 4 to 5 dwelling units per acre (with a few exceptionally large parcels). Subarea 4 is unique in that it is subdivided into exceptionally small lots, averaging about 5000 square feet. The streets are also unusually narrow, making this subarea very crowded and subject to unusual construction impacts. Subarea 7 is the planned community of Canyon Heights, which is currently under construction (Tract Maps 30330 and 30330-1). A total of 334 homes are planned, and it is assumed that no additional development will take place in this subarea.

There are a few remaining large parcels in certain subareas that may potentially be divided into smaller parcels in the future. In order to arrive at realistic estimates of ultimate development, some of these large parcels are assumed to eventually be divided into several smaller ones. The following table shows current and projected ultimate dwelling units in the various subareas.

**Table 3-1**
**Existing and Ultimate Development**

| Subarea | Estimated Existing DU | Ultimate DU's | | |
|---|---|---|---|---|
| | | Parcels | Additional Lots Assumed | Total |
| 1 | 59 | 95 | 0 | 95 |
| 2 | 201 | 422 | 40 | 462 |
| 3 | 73 | 210 | 22 | 232 |
| 4 | 382 | 1400 | -280* | 1120 |
| 5 | 215 | 445 | 100 | 545 |
| 6 | 45 | 87 | 0 | 87 |
| 7 | 0 | 394 | 0 | 394 |
| 8 | 162 | 163 | 0 | 163 |
| 9 | 253 | 765 | 55 | 820 |
| Total | 1390 | 3,981 | -63 | 3,918 |

*Area 4 buildout assumes that some small lots will be combined and occupied by a single DU

### 3.1.2 Flow Generation Factors

Using the dwelling unit information developed in Table 3-1, existing and ultimate wastewater flows can be estimated by applying unit flow coefficients expressed as average daily flow (ADF) per capita or per dwelling unit, and using standard peaking factors to compute peak flow rates. Although the number of existing and ultimate dwelling units can be reasonably ascertained, abnormal occupancy patterns could render the use of standard per dwelling unit flow factors to be inappropriate. For instance, high vacancy factors or situations where several families occupy a single dwelling would result in unusually low or high wastewater flows, respectively. Because the Quail Valley study area exclusively utilizes onsite septic systems for wastewater disposal, it is not possible to physically measure the wastewater flows being generated. For this reason, an indirect approach using wet weather water meter data in the study area is employed to estimate the existing wastewater flows. The process began with a spatial selection of the meters within the study area utilizing EMWD's GIS database. Then usage data for selected meters was extracted from EMWD's billing database. Demands during the 2002/3 and 2003/4 winter months (December, January, and February) were used because irrigation use during these months is typically at a minimum. Meters are read throughout the month and reflect usage for the period preceding the read date. For ease of statistical analysis, all meters read on any date in a given month are assumed to reflect the preceding month (i.e., a meter read on January 10 is assumed to reflect usage for the Month of December when it actually reflects usage from the previous read date through January 10). This assumption is required to produce discreet monthly averages and is not expected to have a large effect on the data. Using the above-described data, a statistical summary table is developed, as follows.

**Table 3-2**
**Historical Winter (Minimum) Water Usage**
**and**
**Estimated Wastewater Generation Rates**

| Month* | Total No. Meters | Total Water Use** | Avg. Water Use | Max single meter Water Use | Min single meter Water Use | Est. Wastewater generation (90% of Wtr Use) |
|---|---|---|---|---|---|---|
| | | gpd | gpd/mtr | gpd/mtr | gpd/mtr | gpd/du |
| **Dec-02** | 1,079 | 310,733 | 287.98 | 6,805 | 24 | 259.18 |
| **Jan-03** | 1,092 | 303,566 | 277.99 | 8,518 | 24 | 250.19 |
| **Feb-03** | 700 | 157,333 | 224.76 | 8,349 | 24 | 202.29 |
| **Dec-03** | 1,159 | 346,205 | 298.71 | 4,488 | 24 | 268.84 |
| **Jan-04** | 1,176 | 301,877 | 256.70 | 3,499 | 24 | 231.03 |
| **Feb-04** | 1,175 | 293,551 | 249.83 | 2,220 | 24 | 224.85 |
| **Avg.** | | | **266.00** | | | **239.40** |

* Based on meter reads from the month following, i.e. Dec data based on Jan meter reads

** Monthly usage divided by number of days per month

Table 3-2 shows that the number of meters read for each usage period is relatively consistent, with the exception of the February 2003 usage period. The table also shows that no more than 2% of the meters had usage greater than 1,000 gallons per day (gpd) and no more than 6% had usage less than 25 gpd. This demonstrates that less than 8% of the meters fall outside of expected range for typical domestic use. The average water usage for the winter months examined is 266 gpd per meter. Making the assumption that each meter serves a single dwelling unit, the average use is 266 gpd/DU. Because the usage data examined is only for the winter months and due to the limited amount of irrigated landscaping in the study area, an indoor water usage factor of 266 gpd/DU appears reasonable. To estimate the wastewater generation rate, it is assumed that about 90% of the water used during the wet periods is returned as sewage. This results in an average sewage generation rate of about 240 gpd/DU.

It is noted, however, that both of the December readings generate higher indoor use numbers (288 gpd/DU in 2002 and 299 gpd/DU in 2004). This could represent an influx of visitors during the holidays surrounding Christmas. Since the purpose of this evaluation is to estimate maximum wastewater flows to be accommodated in a community sewer system, the December 2002 and 2003 numbers are used to approximate maximum day wastewater flow coefficients (259 gpd/DU and 269 gpd/DU, respectively).

From the above, it is concluded that household maximum day wastewater generation rates in the study area are not abnormal, and in fact are very close to the District's design standard of 300 gpd/DU ADF. For this reason, it is decided to use the District's standard to estimate design wastewater flows from the study area. Note that although this factor is appropriate for sizing of the collection system, it would result in overestimation of average inflows to the District's treatment facility.

Also, for this preliminary evaluation, a peaking factor of 2.5 times ADF is used to estimate peak hour flows in the smaller or less populated subareas. This peaking factor is appropriate for small tributary areas, but becomes quite conservative as flows move downstream and are attenuated by staggered inflows and storage in the pipelines. For the Goetz Road Trunk and the downstream sewer lines serving the larger subareas, EMWD standard peaking factors based on tributary population are used to size the various reaches.

### 3.1.3 Estimated Sewer System Flows from Subareas

The following tables show current and future computed flows for each subarea based on existing and ultimate dwelling units.

**Table 3-3**
**Estimated Existing Wastewater Flow**

| | | | | | **Peak Flow** | |
|---|---|---|---|---|---|---|
| Subarea | DU | Per DU Flow (gpd/DU) | Average Flow (gpd) | Peak Factor | (cfs) | (gpm) |
| 1 | 61 | 300 | 18,300 | 2.5 | 0.071 | 32 |
| 2 | 213 | 300 | 63,900 | 2.5 | 0.247 | 111 |
| 3 | 79 | 300 | 23,700 | 2.5 | 0.092 | 41 |
| 4 | 407 | 300 | 122,100 | 2.4 | 0.453 | 204 |
| 5 | 220 | 300 | 66,000 | 2.5 | 0.255 | 115 |
| 6 | 45 | 300 | 13,500 | 2.5 | 0.052 | 23 |
| 7 (Canyon Heights) | 0 | 300 | 0 | 2.5 | 0 | 0 |
| 8 | 168 | 300 | 50,400 | 2.5 | 0.195 | 88 |
| 9 | 270 | 300 | 81,000 | 2.5 | 0.313 | 141 |
| **Subtotal for Quail Valley*** | **1,463** | | **438,900** | **2.0** | **1.358** | **610** |

The sum of the subarea peak flow exceeds the peak flows to be experienced in the Goetz Trunk, since the composite flow peak factor is less than the individual subarea peak factors.

Abbreviations:

DU- Dwelling Units
gpd - gallons per day
cfs - cubic feet per second
gpm - gallons per minute

**Table 3-4**
**Estimated Ultimate Wastewater Flow**

| Subarea | DU | Per DU Flow (gpd/DU) | Average Flow (gpd) | Peak Factor | Peak Flow (cfs) | Peak Flow (gpm) |
|---|---|---|---|---|---|---|
| | | | | | | |
| 1 | 95 | 300 | 28,500 | 2.5 | 0.110 | 49 |
| 2 | 462 | 300 | 138,600 | 2.4 | 0.515 | 231 |
| 3 | 232 | 300 | 69,600 | 2.5 | 0.269 | 121 |
| 4 | 1120 | 300 | 336,000 | 2.1 | 1.092 | 490 |
| 5 | 445 | 300 | 133,500 | 2.4 | 0.496 | 223 |
| 6 | 87 | 300 | 26,100 | 2.5 | 0.101 | 45 |
| 7 (Canyon Heights) | 394 | 300 | 118,200 | 2.4 | 0.439 | 197 |
| 8 | 163 | 300 | 48,900 | 2.5 | 0.189 | 85 |
| 9 | 820 | 300 | 246,000 | 2.2 | 0.837 | 376 |
| | | | | | | |
| **Subtotal for Quail Valley*** | **3,818** | | **1,145,400** | **1.8** | **3.189** | **1432** |

The sum of the subarea peak flow exceeds the peak flows to be experienced in the Goetz Trunk, since the composite flow peak factor is less than the individual subarea peak factors.

Abbreviations:

DU- Dwelling Units
gpd - gallons per day
cfs - cubic feet per second
gpm - gallons per minute

### 3.1.4 Deletion of Low Density Areas

The study objective is to explore the feasibility of a community sewer system which could serve the entire Quail Valley study area. However, it is prudent to look into the appropriateness of sewering the lower density subareas. It is obvious from viewing the parcel map that the subareas differ greatly in character of development. Three of the subareas—1, 6, and 8, feature large lots, respectively having 85%, 74%, and 84% of the parcels exceeding one acre in size. The remaining subareas are much higher density; having large lots in ratios ranging from only 2% to 6% of their totals.

It is assumed, and reinforced by the "complaints" map furnished by the County, that the great majority of the septic problems being experienced derive from the subareas featuring a predominance of small parcels. It is thus recommended that the majority of the low-density subareas 1, 6, and 8 be excluded from consideration in the layout of the conceptual Quail Valley sewer system being developed in this study. It would be very costly to sewer the entirety of these three subareas to serve the few undersized lots, with nebulous pollution control benefits. It is proposed, however, that the downstream trunk system be oversized to be able to accommodate flows from ultimate development in these three subareas. The relatively modest incremental cost may be justified by the contingency flexibility to sewer in the future if needed. Also, provision is made in the proposed collection system to accommodate a small number of units in the south part of subarea 1 and the extreme westerly part of subarea 8 that may be contributing to septic effluent complaints along Goetz Road and Conejo Drive.

### 3.1.5 Possible Service of Subarea 9 by EVMWD System

EVMWD has studied the possibility of serving a portion of Subarea 9, immediately north of Canyon Lake, by flow into its Canyon Lake collection system. A recent report indicates that the Canyon Lake system could potentially accept flow from 143 DU in the south and east portion of Subarea 9 to a "…specified point of connection." However, the report indicates that there would be impacts to the EVMWD system. Evaluations herein indicate that there is a low-lying area in the west portion of Subarea 9 which could flow by gravity to EVMWD's system. Alternatively, it would have to be pumped to be served by EMWD. This area includes about 370 parcels.

Meetings have been held regarding possible service, and EVMWD has initiated an evaluation of the impacts on their system and the cost of service. However, since no decision or agreement to serve some or all of these units is likely in the time frame of this study, it is assumed for purposes of this study that the entirety of Subarea 9 would be served by EMWD.

## 3.2 GEOTECHNICAL FEASIBILITY

In recognition of the challenging subsurface conditions in the study area, a limited geotechnical investigation was conducted to estimate the feasibility and degree of difficulty of construction of the proposed sewer system.

A report by Inland Foundation Engineering, Inc. was prepared and included as Appendix B. Following are the conclusions and recommendations quoted from the subject report.

*On the basis of our limited study, it appears that the construction will be feasible from a Geotechnical Engineering standpoint. The primary issues affecting the proposed sewer project are related to shallow bedrock and groundwater. Bedrock may be difficult to excavate using large conventional excavators below a depth of five to ten feet. Blasting should be anticipated along portions of the alignment, particularly within deeper zones of the pipeline.*

*Where groundwater is encountered in the alluvium, it will destabilize excavation sidewalls and should be removed from outside the trench. Groundwater will also be encountered within deeper excavations made in the bedrock. This should not present stability problems in the bedrock and can probably be removed from within the excavations. Due to the likelihood of groundwater within the pipe zone, earthen trench dams are recommended. In addition, piping will occur where the pipe zone lies within the alluvial soils.*

*The following recommendations have been developed for use in the design of pipelines on this project.*

***Bedding:*** *We recommend a minimum bedding thickness of 6 inches be placed to provide uniform and adequate longitudinal support under the pipe.*

*The bedding material should not be compacted. Blocking should not be used to bring the pipe to grade. Bell holes at each joint should be provided to permit the joint to be assembled properly while maintaining uniform pipe support.*

***Embedment:*** *Processed native materials should provide suitable support for the pipe where cover thicknesses are greater than three feet. However, screening of oversized particles may be required for use in the embedment zone. A Lateral Modulus of Subgrade Reaction (E') of 3000 pounds per square inch may be assumed where the pipe zone is below the bedrock surface. This is expected to be the condition throughout the project area. If designs are based upon the use of imported granular embedment materials, we recommend that a granular free-draining soil be used. The actual thickness should be determined by the pipeline engineer or manufacturer. We recommend that if imported granular embedment material is used, it have a minimum Sand Equivalent (SE) of 30 and be free of particles greater than two inches diameter.*

***Lateral Design:*** *On the basis of laboratory testing, we propose a lateral bearing capacity of 2000 pounds per square foot below a depth of four feet. This should be modified on the basis of the anticipated trench type. This modification may be based upon silty sand (SM), assuming native backfill.*

***Trench Wall Stability:*** *Significant caving did not occur within our exploratory borings. However, all excavations should be configured in accordance with the requirements of CalOSHA. We would classify the residual soils and bedrock as Type A. Within the alluvial areas, special consideration may be necessary depending upon seasonal groundwater conditions. The classification of the soil and the shoring and/or slope configuration should be the responsibility of the contractor on the basis of the trench depth and the soil encountered. The contractor should have a "competent person" on-site for the purpose of assuring safety within and about all construction excavations.*

***Excavatability:*** *Soils along the alignment generally consist of shallow alluvial and residual soils underlain by moderately weathered bedrock. Within the bedrock areas, excavations below a depth of five feet may be very difficult. Blasting may be required for deeper excavations made using conventional excavating equipment such as large track-mounted backhoes.*

***Compaction Characteristics:*** *In general, we anticipate that the soils that are excavated and replaced as controlled compacted backfill will respond to mechanical compaction. Depending upon the seasonal groundwater levels, some of the material may be in a very moist condition and require air-drying and manipulation prior to use as backfill. Laboratory testing suggests that jetting and compacting may not be a feasible alternative for achieving compaction. The soils should be brought to near optimum moisture content as they are excavated and placed in the stockpile. As the soils are placed, they should be compacted in shallow lifts. We estimate that the shrinkage of the native soils used in the backfill will be less than five percent. We expect that most of the bedrock will break up sufficiently for use as backfill. The use of excavated native soils may not be appropriate for use in the embedment zone due to oversized particles. Some losses will occur due to oversized particles. Blasting of unweathered bedrock may also result in the generation of oversized particles that will not be suitable for backfill.*

***Dewatering:*** *Depending upon seasonal groundwater conditions, dewatering could present unique challenges. This is due to the likelihood that the groundwater will contain partially treated sewage. Handling and discharging water that is produced during dewatering processes should be carefully planned with respect to this characteristic. Health and safety concerns should be paramount in the planning and permitting processes associated with dewatering.*

Based on the Geotechnical Report conclusions, estimated construction costs and construction impacts are adjusted to account for rock excavation, particularly in the deeper reaches. Scheduling of construction should be in the dry season to minimize groundwater impacts. These issues are addressed in the subsequent "Constructability" discussion (Section 4.5.1).

# Chapter 4
# Evaluation of Alternatives

## 4.1 General Description of Alternative System Concepts

The evaluation of sewer system alternatives includes consideration of several sewer system concepts that will be analyzed based on many factors including but not limited to topography, soil conditions, right of ways, overhead and underground utilities, constructability, cost and operational burden. The study area is characterized by highly undulated terrain with an overall topographic relief of about 340 feet. Most of the existing paved streets do not have sidewalks and have minimum right of way. Thus, the alignment of a sewer line within the right of way or adjacent to a street will have to be considered very carefully with respect to disruption of access and property impacts.

The focus of this alternatives evaluation is the identification of the most feasible and cost effective system. Considering the various constraints, a preliminary screening identified the following four concepts for further evaluation:

- Conventional Gravity/Force Main System
- Vacuum Sewer System
- Low Pressure System
- Combination Sewer System

These concepts are generally described in the following sub sections.

### 4.1.1 Conventional Gravity/Force Main Sewer System

A typical gravity and force main system generally consists of laterals from residences, tying into a local collection system or gravity sewers flowing down gradient to larger trunk lines or lift stations. Manholes are placed at regular intervals for cleaning and maintenance. Where wastewater cannot flow by gravity, lift stations are provided in the system along with force mains for pressure flow. This system is generally the first choice of sewering agencies, and in typical circumstances is the lowest cost and easiest to maintain. The construction cost of a conventional gravity/force main system is significantly influenced by the depth of sewer lines and soil conditions in the area, as well as the number and size of the lift stations required.

The main components of a gravity and force main system are further described as follows:

**Laterals:** Laterals from dwelling units are typically 4 inches in diameter. The lateral begins at the building sanitary plumbing discharge point and is installed at a downward slope to the sewer main. The District typically is responsible for the portion that is not on the private property. Laterals over 200 feet in length will have cleanouts installed. The lateral connects with a wye to the top of main sewer line.

**Mainline Piping:** The mainline piping is laid on a downward slope to allow the wastewater to flow by gravity. The downward slope of the pipe should be such that there is no sediment accumulation in the pipeline and the flow velocity is sufficient for self cleansing (more than 2 feet per second). Usually a minimum slope of 0.4% is maintained for an 8-inch pipe to obtain a self cleansing velocity. The larger diameter pipes might have lesser slopes if the allowable minimum velocity can be attained for a flow condition. The design depth of flow in the pipeline is dependent on pipe size and is governed by agency guidelines. Manholes are installed at regular intervals, at changes in grade, changes in alignment, and in long sections to allow cleaning and maintenance operations.

**Lift Stations:** When topography does not permit a sewer alignment to drain downstream by gravity without excessive cost, a pump or lift station is provided. A lift station has a wet well to collect wastewater and pumps to convey the wastewater through a force main to a point where it can join the gravity system or to the treatment facilities. Lift stations have emergency power and storage features, telemetry, noise, and odor control devices.

### 4.1.2 Vacuum Sewer System

A vacuum sewer system operates by the differential pressure between the open atmosphere and the lower of the contained environment of collection system. System components include collection piping, valve pits, division valves, and vacuum stations. From the vacuum station the wastewater is pumped to the treatment facility or the gravity system through a force main. After entering the system, the wastewater is only exposed to the open air at the discharge point of the force main. The components of a typical vacuum system are further described as follows:

**Valve Pit:** The valve pit interfaces with the residences being served by the vacuum system. The wastewater flow from residences traverses to the valve pit by gravity through conventional sewer pipe, where it accumulates to approximately 10 gallons. As the level of wastewater rises in the pit, it pneumatically activates the valve to the vacuum system. The valve remains open to the system for about 10 seconds. The fluid is forced into the collection system along with atmospheric air from pipe vents until the valve closes. The wastewater mixture normally travels at an approximate velocity of 18 feet per second. The air is driven so fast through the pit that a second air vent is required on the house side pipe to prevent the house p-traps on the fixture drains from emptying.

**Collection Piping:** The collection piping is sized to convey flow from the valve pit to the vacuum station. The size of the line increases as the number of pits contributing flow increases. Typical pipe sizes are 4-inch at the beginning of the system to a maximum 10-inch at the vacuum station. A vacuum system uses the high velocity in the system to create fast moving foam that keeps the pipe clean. It relies on gravity to advance wastewater flow towards the vacuum station when the pit valves are closed. The vacuum lines are installed at a shallow depth of 3 to 5 feet. The pipe is generally laid at a 0.2 percent slope, with a lift installed at approximately 500 feet. The lift is a grade break that uses a small section of pipe and bends to raise the grade break up and, then starts a long slope down at 0.2 percent to the next lift. This process is repeated until the pipe enters the vacuum station. The wastewater collects at the base of the lift until a burst of expanding air from open-pit valves lifts the wastewater up and over the lift. Once over the lift, gravity will assist the flow downwards until the next lift is encountered. The division valves are placed at branch lines and intermediate lengths to allow isolation of line segments for maintenance, location of line breaks and pit valve malfunctions.

**Vacuum Station:** The vacuum station is similar to a conventional wastewater lift station with the addition of vacuum pumps. These stations are typically at grade and use a large tank to receive wastewater from the collection system. The wastewater enters the tank near the top. Vacuum pump suctions are mounted on the top of tank and conventional wastewater pump suctions are drawn from the bottom of the tank. The vacuum pumps provide the operating energy for the system. When the vacuum pumps are off, the receiving tank normally operates at 20 inches of mercury. As the valve pits force air into the system and the pressure rises to 16 inches of mercury, the pumps start. This energy operates the interface valves in the pits and provides all the energy required at the residences. The reliability is ensured by an onsite emergency generator at the vacuum station.

### 4.1.3 Low Pressure Sewer System

Low pressure systems operate by pumping residential sewage using a small pump station at each residence or clusters of residences, through small diameter pipes into a force main conveyance system. The primary reason for considering a low pressure sewer system for the Quail Valley application is the advantage of being able to install a shallow, terrain-following system of small diameter pressure pipes rather than larger, deeper gravity sewer lines. There are two general types of low pressure systems: effluent and grinder pump. An effluent system is commonly referred to as a septic tank effluent pump (STEP) pressure sewer system. It consists of a small pump in a pump vault that receives septic tank effluent, an access riser and lid, and an alarm/control panel.

The grinder pump low-pressure systems pump raw sewage, rather than septic tank effluent, through a small grinder pump at each residence. Following is a brief description of typical components of STEP and grinder pump low pressure sewer systems.

**STEP System**

**Riser and Lids:** Risers are required for access into internal vaults and access into the septic tanks for septage pumping. Lids are provided with each access riser.

**Screened Pump Vault:** A pump vault is installed in the septic or "interceptor" tanks to include a 1/8-inch mesh screen filter. The effluent enters the vault through holes that are spaced around the perimeter between the sludge and scum layers.

**Discharge Hose and Valve Assembly:** This assembly consists of a ball valve, check valve, flex hose and PVC pipe.

**Float Switch Assembly:** This assembly includes three switch floats mounted on a PVC stem that is attached to a filter cartridge. The floats are adjustable and must be removable without removing the pump vault. Each float lead is secured with a nylon strain relief bushing at the splice box.

**High-Head Effluent Pump:** Stainless steel turbine effluent pump system, 1/2 to 3/4 hp with a 4 to 6 gpm preferred pumping rate.

**Electrical Splice Box:** Splice box approved for wet locations, equipped with four electrical cord grips and an outlet fitting.

**Alarm/Control Panel:** A control panel equipped with a motor start contactor, toggle switch, controls circuit breaker, pump circuit breaker, audio alarm, visual alarm, panel enclosure, S1RO panel ratings, S2RO panel ratings, event counter and an elapsed time meter. A recent innovation available at additional cost is a remote telemetry panel coupled with a web-based monitoring system.

**Grinder Pump System**

**Pump Vault**: Cylindrical HDPE buried tank with low profile cover

**Grinder Pump:** A semi-positive displacement progressing cavity pump with fairly constant flow under varying TDH conditions. Pump includes hardened steel grinder wheel and shredder ring.

**Pressure Switch Level Control:** Self-cleaning level sensor.

## 4.2 QUAIL VALLEY SEWER SYSTEM OPTIONS

### 4.2.1 Methodology

The approach utilized in defining and evaluating the various sewer system options is as follows:

- Determine optimum configuration of a conventional gravity/force main system or systems, define facilities and estimate capital costs for each subarea and entire study area.
- Capital cost estimates are based on quantity takeoffs and unit costs developed by an in-depth assessment of terrain, geotechnical, and development conditions as they affect labor, equipment, production rates, and system components and materials.
- Assess the applicability of alternative systems (i.e., vacuum or low-pressure STEP or grinder pump systems) in their potential to significantly reduce capital costs for all or selected portions of the study area.
- Define facilities and estimate capital costs for identified technically feasible alternative systems, to include combinations with conventional gravity facilities.

- For the competing sewer system alternatives for which capital costs have been developed, conduct a further comparative assessment of life-cycle costs (i.e., long term O&M. rehabilitation and replacement) constructability and general operational burden.

### 4.2.2 Capital Cost Methodology

Capital cost is the primary factor in determination of the fiscal viability of constructing a community sewer system in Quail Valley. As such, and given the unusually challenging site conditions, it would be inappropriate and perhaps misleading to estimate costs based on normal planning-level costing procedures. Thus, a more refined cost opinion for pipeline construction has been prepared based on research of recent construction history in comparable conditions. Lift station costs are based on recent bids received by the District. Costs for low pressure system components are based on vendor-provided cost estimates factored upward to account for idiosyncrasies of the study area. A value estimate report (October 3, 2005) was prepared for the District by the Schooler Company to assist in determining the appropriate costs for easements and acquisitions of fee-simple property for pipelines and lift stations. The results indicate a range for vacant property of $8 to $13 per square foot for pipeline easements, and $84,000 to $130,000 for lift station sites. This information corroborates the appropriateness of the $12 per square foot value used in the capital cost estimates in this report.

Following is a brief explanation of the methodology and assumptions employed in the "Preliminary Opinion of Probable Cost" for the collection system (see Appendix E). To prepare the Cost Opinion, the geotechnical feasibility study (Study) prepared by Inland Foundation Engineering, Inc., dated August 23, 2005 was reviewed. The Study provides limited data in the form of six (6) borings and four (4) seismic soundings performed in various locations within the project area.

The compiled data and the report text do not provide definitive conclusions as to the overall rippability of the bedrock. All of the borings met with refusal at various depths between five and twenty feet in depth. Maximum Seismic velocities were recorded at approximately 5000 - 5600 ft/sec. The seismic soundings suggest that the bedrock will be rippable with a large excavator. However, experience has demonstrated that seismic data is not always reliable. While the borings met refusal at various depths, this does not necessarily mean that the bedrock is unrippable.

For the purposes of this Cost Opinion, it is assumed that blasting is not a feasible alternative. Blasting, if required, would disturb adjacent utilities and improvements along an approximate 1:1 angle of repose from the depth of the charge up to the surface.

Since the geotechnical data available to date does not indicate that bedrock is definitively unrippable, the Cost Opinion assumes a rippable condition and provides a range of costs associated with various levels of productivity that can be reasonably anticipated. The Cost Opinion also assumes that there will be localized areas where rock breakers will be required to remove floaters or pockets of hard rock. This level of analysis furnishes the District with a preliminary level of data necessary to evaluate deep sewer construction alternatives against the lift station and shallow force main construction alternatives.

Additional geotechnical investigation will be required to ascertain and map the rippability of the bedrock. There are a number of alternatives available to the District that would furnish additional data and reliability, which include additional soundings, borings and test pit excavations. The utilization of an Air-track Drill Rig is recommended to perform additional site investigation, particularly where deep sewer reaches are recommended. These drill rigs are capable of furnishing a consistent and constant drill pressure and can evaluate the bedrock hardness by tracking the observed drilling rates. This type of evaluation can provide a significant amount of reliable data for a minimal investment.

Note that two assumed production rates are computed (see Appendix E). The more optimistic production rates are used for the estimates in this report, but a construction contingency factor is included.

Groundwater considerations are also evaluated as they relate to cost. Based upon available information, it appears that groundwater encountered will be "perched" in nature over nearly all of the project area. This condition is fairly easily addressed in construction. It is also anticipated that short portions of the alignment will cross under natural drainages and that these areas will encounter a more significant dewatering approach. These areas, however, are a small percentage of the entire project area and therefore their cumulative impact on costs will also be small. For the purposes of this cost opinion, a nominal allocation has been incorporated.

No cost has been incorporated for the potential treatment or removal of contaminated groundwater. This issue has not been defined at a level that would allow a reasonable attempt at defining the cost. However, a significant construction contingency factor has been included which should be adequate to account for this and other possible undefined or unforeseen costs.

### 4.2.3 Concept 1 – Conventional Gravity Lift Station System

For the conventional gravity sewer option, the feasibility of a system consisting primarily of gravity mains is considered in terms of cost and constructability. For purposes of this analysis, it is assumed that the District will allow sewer lines at depths ranging to as shallow a 5 feet, to avoid hard-rock excavation. In general, the flow is directed from each subarea under consideration to a major trunk line in Goetz Road serving the overall study area. Because of terrain, it is not possible to serve the entire area by gravity collection; thus, a limited number of lift stations/force mains are required. With County DEH concurrence, the large-lot subareas (1, 6, and 8) are considered to remain on septic systems, with the exception of a few lots along Goetz Road in south subarea 1, south subarea 6, and along Goetz Road and Conejo Drive in subarea 8. A gravity sewer system for subareas 2, 3, 4, 5 and 9, as well as along a stretch of road in subarea 8, is shown in Figures 5, 6 and 7. Costs associated with each subarea are shown in Tables 4-2 through 4-10. Subarea 7 will be served by a developer-constructed system also flowing to the Goetz Road Trunk.

Two gravity sewer system alternatives are considered for subarea 4, as shown in Figures 6 and 7. The difference between the alternatives for subarea 4 is that Alternative 4A features extra-depth sewers in order to limit the number of lift stations. A brief description of the Goetz trunk line and each subarea is given below, along with quantities and costs of each item.

### Goetz Road Trunk

The alignment for the trunk is proposed to follow Goetz Road from north to south, beginning at its intersection with South Canyon Road and ending with a connection to the offsite interim pump station located off of Goetz Road just south of the overall study area, as seen in Figure 5. The trunk for the overall study area was evaluated along several reaches of pipe, ranging progressively in size from 8-inch to 18-inch in diameter. To avoid the construction of a lift station for directing flow along Goetz Road, an extra-depth sewer (ranging to about 26 feet) was required for a reach beginning at Avenida Robles and running south for approximately 1540 feet.

**Table 4-1**
**Goetz Road Trunk Sewer**
**Quantities and Costs**

| Item | Quantity | Unit | Unit Price | Amount |
|---|---|---|---|---|
| 8-inch Main | | | | |
| Depth < 10 | 3500 | LF | $137 | $479,500 |
| 15-inch Main | | | | |
| Depth 10-15 | 190 | LF | $192 | $36,480 |
| 18-inch Main | | | | |
| Depth < 10 | 2972 | LF | $139 | $413,108 |
| Depth 10-15 | 78 | LF | $192 | $14,976 |
| Depth 15-20 | 465 | LF | $400 | $186,000 |
| Depth 20-30 | 995 | LF | $578 | $575,110 |
| 48" Diameter MH | | | | |
| Depth < 10 | 22 | EA | $11,700 | $257,400 |
| Depth 10-20 | 5 | EA | $18,400 | $92,000 |
| Depth 20+ | 5 | EA | $25,400 | $127,000 |
| Sub-total = Construction + Contingency (20%): | | | | $2,617,889 |
| Engineering, Administration, Legal & CM (25%): | | | | $654,472 |
| Total: | | | | $3,272,361 |

### Subarea 2

A conventional gravity layout for subarea 2 is feasible. A conventional gravity system with normal depth sewers is practical throughout the subarea, because the majority of the flow collects at a low point on La Bertha Lane.

However, due to the rolling topography of subarea 2, two lift stations are required. One lift station is required to pump flow from 29 lots from a low point at Idaho Place and Hampshire Drive to connect with the gravity system at La Bertha Lane and Hampshire Drive. A second lift station is required to pump flow from the entire subarea system in order to utilize the gravity system of subarea 4 to direct the flow to the Goetz Road Trunk.

The flow from the entire subarea 2 is directed through the gravity system of subarea 4 to the lift station at Mountain View Place and Newport Drive. From there it is pumped to a collection manhole located at the intersection of Goetz Road at Avenida Robles.

**Table 4-2**
**Subarea 2 System**

| Item | Quantity | Unit | Unit Price | Amount |
|---|---|---|---|---|
| 8-inch Main | | | | |
| Depth < 10 | 13600 | LF | $102 | $1,387,200 |
| 48" Diameter MH | 55 | EA | $8,550 | $470,250 |
| Sewer Lateral | | | | |
| SB-177 (Minimum Depth) | 422 | EA | $4,850 | $2,046,700 |
| Lift Station | | | | |
| La Bertha Lane<br>Units Served: 462 UNITS | 1 | EA | $600,000 | $600,000 |
| Idaho Place<br>Units Served: 69 UNITS | 1 | EA | $320,000 | $320,000 |
| Lift Station Eminent Domain | | | | |
| Lift Station Site | 2 | EA | $70,150 | $140,300 |
| Force Main | | | | |
| 6" - La Bertha Lane to Newsport Drive | 1500 | LF | $103 | $154,500 |
| 6" - Idaho Place to Hampshire Drive | 500 | LF | $103 | $51,500 |
| Decommission Existing Septic Tank | 213 | EA | $500 | $106,500 |
| Sub-total = Construction + Contingency (20%): | | | | $6,332,340 |
| Engineering, Administration, Legal & CM (25%): | | | | $1,583,085 |
| Total: | | | | $7,915,425 |

### Subarea 3

A conventional gravity sewer system for subarea 3 is feasible and will connect with the sewer system of subarea 4 at several intersections on San Jacinto Road. The flow from the entire View Place and Newport Drive. From there it is pumped to a collection manhole located at the intersection with Goetz Road at Avenida Robles. There it is connected with the Goetz Road Trunk.

Although the decision has been made to exclude subarea 1 because of the predominance of large lots, provision has been made to serve a few parcels in the southwestern part of the subarea by a gravity connection to the subarea 3 system at the extreme northeast corner of subarea 3. This may be necessary to remediate the County-reported complaints along a short reach of Goetz Road in south subarea 1.

**Table 4-3**
**Subarea 3 System**

| Item | Quantity | Unit | Unit Price | Amount |
|---|---|---|---|---|
| 8-inch Main | | | | |
| Depth < 10 | 8500 | LF | $107 | $909,500 |
| 48" Diameter MH | 30 | EA | $9,000 | $270,000 |
| Sewer Lateral | | | | |
| SB-177 (Minimum Depth) | 232 | EA | $4,050 | $939,600 |
| Decommission Existing Septic Tank | 79 | EA | $500 | $39,500 |
| Sub-total = Construction + Contingency (20%): | | | | $2,590,320 |
| Engineering, Administration, Legal & CM (25%): | | | | $647,580 |
| Total: | | | | $3,237,900 |

## Subarea 4 - Alternatives A and B

This subarea presents the greatest challenges because of the terrain and crowded conditions. Two alternatives, (A and B) for the gravity sewer system were laid out for subarea 4. Due to the undulating topography, extra depth gravity sewers versus additional lift stations are the two options considered for the conventional gravity sewer system.

It is also noted that because of the small parcels, many existing residences occupy more than one lot. Thus, as discussed in Chapter 3, it is assumed that only 80% of the total parcels will ultimately be occupied by a dwelling unit. Flows from these lots, in addition to all the lots in subareas 2 and 3, were used to determine the sizes of the sewer lines and the capacities of the lift stations.

Alternative A ("Extra Depth") features deep sewers as opposed to additional lift stations (see in Figure 6). Extra depth sewers are proposed for several reaches in order to limit the number of required lift stations to three.

Due to the rocky terrain and crowded conditions in subarea 4, the cost and disruption associated with trenching and possible tunneling or blasting for extra depth sewers might render this option unfeasible. However, if disruption impacts are not extreme, the higher capital costs for extra depth sewers might be more than offset by higher long-term operation and maintenance costs associated with additional lift stations, which would be required for normal depth sewers.

Alternative B ("Normal Depth") is shown in Figure 7. Normal depth sewers are proposed, and flows from subareas 2, 3 and 4 are directed to five lift stations. Three lift stations are required to direct the flow to the lowest point of subarea 4 at the southwest corner of the subarea. The greater number of lift stations would require additional costs in terms of long term operation and maintenance, while limiting the requirement for extra depth sewers.

For both alternatives, (except for the flows from 20 lots in the southeast corner of subarea 4, which are collected and pumped at a lift station at Quail Place and Welles Place to a collection manhole at Welles Place and Goetz Drive), all of subareas 2, 3 and 4 are pumped from the lift station located at Mountain View Place and Newport Drive to a collection manhole at Avenida Robles and Goetz Road. From there the combined flow is pumped to a collection manhole at Avenida Robles and Goetz Road. There the combined flow of the three subareas is joined with the Goetz Road Trunk.

**Table 4-4**
**Subarea 4 - Alternative A (Extra Depth) System**

| Item | Quantity | Unit | Unit Price | Amount |
|---|---|---|---|---|
| 8-inch Main | | | | |
| Depth < 10 | 44580 | LF | $110 | $4,903,800 |
| Depth 10-15 | 790 | LF | $152 | $120,080 |
| Depth 15-20 | 790 | LF | $318 | $251,220 |
| Depth 20+ | 210 | LF | $456 | $95,760 |
| 10-inch Main | | | | |
| Depth < 10 | 3140 | LF | $110 | $345,400 |
| Depth 10-15 | 140 | LF | $152 | $21,280 |
| Depth 15-20 | 140 | LF | $318 | $44,520 |
| 12-inch Main | | | | |
| Depth < 10 | 1360 | LF | $110 | $149,600 |
| Depth 10-15 | 125 | LF | $153 | $19,125 |
| Depth 15-20 | 125 | LF | $317 | $39,625 |
| 15-inch Main | | | | |
| Depth < 10 | 430 | LF | $110 | $47,300 |
| 48" Diameter MH | | | | |
| Depth < 10 | 183 | EA | $9,200 | $1,683,600 |
| Depth 10-20 | 10 | EA | $14,650 | $146,500 |
| Depth 20+ | 2 | EA | $20,050 | $40,100 |
| Sewer Lateral | | | | |
| SB-177 (Minimum Depth) | 1043 | EA | $4,150 | $4,328,450 |
| SA-87 (Chimney) | 77 | EA | $6,200 | $477,400 |
| Pipeline Easements | | | | |
| Vacant | 34 | EA | $24,000 | $816,000 |
| Lift Station | | | | |
| 4A<br>Units Served: 292 UNITS (SA4) & 232 UNITS (SA3) | 1 | EA | $600,000 | $600,000 |
| 4B<br>Units Served: 1332 UNITS (SA4) & 462 UNITS (SA2) | 1 | EA | $1,100,000 | $1,100,000 |
| 4C<br>Units Served: 20 UNITS (SA4) | 1 | EA | $320,000 | $320,000 |
| Lift Station Eminent Domain | | | | |
| Lift Station Site | 3 | EA | $57,500 | $172,500 |
| Force Main | | | | |
| 6" - Goetz Dirve to Clara Place | 1100 | LF | $110 | $121,000 |
| 8" - Newport Drive to Goetz Road | 3000 | LF | $147 | $441,000 |
| Decommission Existing Septic Tank | 407 | EA | $500 | $203,500 |
| Sub-total = Construction + Contingency (20%): | | | | $19,785,312 |
| Engineering, Administration, Legal & CM (25%): | | | | $4,946,328 |
| Total: | | | | $24,731,640 |

**Table 4-5**
**Subarea 4 - Alternative B (Normal Depth) System**

| Item | Quantity | Unit | Unit Price | Amount |
|---|---|---|---|---|
| 8-inch Main | | | | |
| Depth < 10 | 50750 | LF | $110 | $5,582,500 |
| 48" Diameter MH | 195 | EA | $9,200 | $1,794,000 |
| Sewer Lateral | | | | |
| SB-177 (Minimum Depth) | 1120 | EA | $4,150 | $4,648,000 |
| Pipeline Easements | | | | |
| Vacant | 44 | EA | $24,000 | $1,056,000 |
| Lift Station | | | | |
| 4A<br>Units Served: 76 UNITS (SA4 - 4A) & 216 UNITS (SA3) | 1 | EA | $500,000 | $500,000 |
| 4B<br>Units Served: 198 UNITS (SA4 - 4B) | 1 | EA | $500,000 | $500,000 |
| 4C<br>Units Served: 270 UNITS (SA4 - 4C) & 16 UNITS (SA3) | 1 | EA | $500,000 | $500,000 |
| 4D<br>Units Served:1332 UNITS (SA4) & 462 UNITS (SA2) | 1 | EA | $1,100,000 | $1,100,000 |
| 4E<br>Units Served: 20 UNITS (SA4) | 1 | EA | $320,000 | $320,000 |
| Lift Station Eminent Domain | | | | |
| | 5 | EA | $57,500 | $287,500 |
| Force Main | | | | |
| 6" - 4A | 500 | LF | $110 | $55,000 |
| 6" - 4B | 650 | LF | $110 | $71,500 |
| 6" - 4C | 500 | LF | $110 | $55,000 |
| 6" - 4D | 3100 | LF | $110 | $341,000 |
| 6" - 4E | 1000 | LF | $110 | $110,000 |
| Decommission Existing Septic Tank | 407 | EA | $500 | $203,500 |
| Sub-total = Construction + Contingency (20%): | | | | $20,548,800 |
| Engineering, Administration, Legal & CM (25%): | | | | $5,137,200 |
| Total: | | | | $25,686,000 |

## Subarea 5

A conventional gravity sewer system is feasible for the entire subarea. Except for an extra depth sewer that is required at Avenida Robles in order to direct the flow of the entire subarea to the Goetz Road trunk, the entire subarea can be served by a normal depth gravity sewer system.

**Table 4-6**
**Subarea 5 System**

| Item | Quantity | Unit | Unit Price | Amount |
|---|---|---|---|---|
| 8-inch Main | | | | |
| Depth < 10 | 14900 | LF | $102 | $1,519,800 |
| 48" Diameter MH | 50 | EA | $8,500 | $425,000 |
| Sewer Lateral | | | | |
| SB-177 (Minimum Depth) | 545 | EA | $3,800 | $2,071,000 |
| Decommission Existing Septic Tank | 220 | EA | $500 | $110,000 |
| Sub-total = Construction + Contingency (20%): | | | | $4,950,960 |
| Engineering, Administration, Legal & CM (25%): | | | | $1,237,740 |
| Total: | | | | $6,188,700 |

## Subarea 8

A conventional gravity sewer system is feasible for the reach in Conejo Drive running parallel to Goetz Road. Twenty-five lots are serviced by this reach of gravity sewer. The reach is aligned to connect with the Goetz Road Trunk at Vista Way.

**Table 4-7**
**Subarea 8 System**

| Item | Quantity | Unit | Unit Price | Amount |
|---|---|---|---|---|
| 8-inch Main | | | | |
| Depth < 10 | 3200 | LF | $109 | $348,800 |
| 48" Diameter MH | 10 | EA | $9,100 | $91,000 |
| Sewer Lateral | | | | |
| SB-177 (Minimum Depth) | 25 | EA | $4,100 | $102,500 |
| Decommission Existing Septic Tank | 25 | EA | $500 | $12,500 |
| Sub-total = Construction + Contingency (20%): | | | | $665,760 |
| Engineering, Administration, Legal & CM (25%): | | | | $166,440 |
| Total: | | | | $832,200 |

## Subarea 9

A conventional gravity sewer system is feasible for the portion of the subarea east of the ridge. Normal depth sewer can be constructed within street right-of-way, although an easement would be required for a reach of gravity sewer in the southwest corner of the subarea.

Two lift stations are required for the western portion of the subarea, one to pump flow from the southwest corner of the subarea, and another to pump flow from the low-lying northwest corner of the subarea. Flows from both lift stations are directed through force mains to a collection manhole at Struble Lane and Vista Way, and there the combined flow is joined with the gravity flow down Vista Way to the Goetz Road Trunk. Constructing these lift stations requires the acquisition of land.

**Table 4-8**
**Subarea 9 System**

| Item | Quantity | Unit | Unit Price | Amount |
|---|---|---|---|---|
| 8-inch Main | | | | |
| Depth < 10 | 25400 | LF | $109 | $2,768,600 |
| 10-inch Main | | | | |
| Depth < 10 | 200 | LF | $109 | $21,800 |
| 48" Diameter MH | 90 | EA | $9,100 | $819,000 |
| Sewer Lateral | | | | |
| SB-177 (Minimum Depth) | 820 | EA | $4,100 | $3,362,000 |
| Pipeline Easements | | | | |
| SFR | 2 | EA | $5,000 | $10,000 |
| Lift Station | | | | |
| Cross Hill Dr<br>Units Served: 137 UNITS | 1 | EA | $500,000 | $500,000 |
| Hecht Rd<br>Units Served: 232 UNITS | 1 | EA | $700,000 | $700,000 |
| Lift Station Eminent Domain | | | | |
| | 2 | EA | $100,000 | $200,000 |
| Force Main | | | | |
| 6" - Cross Hill Dr to Vista Wy | 1200 | LF | $109 | $130,800 |
| 6" - Hecht Rd to Vista Wy | 1700 | LF | $109 | $185,300 |
| Decommission Existing Septic Tank | 270 | EA | $500 | $135,000 |
| Sub-total = Construction + Contingency (20%): | | | | $10,599,000 |
| Engineering, Administration, Legal & CM (25%): | | | | $2,649,750 |
| Total: | | | | $13,248,750 |

### 4.2.4 Concept 2 – Vacuum Sewer System

A vacuum sewer system was considered as an alternative concept for subareas 2, 4 and 9, and a system layout and costs were provided by a leading vendor. However, due to the lift limitations inherent in a vacuum sewer system and the undulating topography of the subareas, extra depth gravity sewers would still be required. For these reasons, this alternative is considered to be not feasible and thus was eliminated from further consideration.

### 4.2.5 Concept 3 - Low Pressure Sewer System

The possibility of substituting small pump systems at individual dwellings or clusters of dwellings, instead of a conventional gravity system, is a serious consideration in light of the challenges inherent in constructing a conventional gravity system in the Quail Valley area. One of the major cost advantages derives from the relative ease of constructing a small diameter pressure pipe at a uniform depth of 30-36 inches, compared to a deeper engineered gravity collection line.

Low pressure STEP or grinder pump solutions for subareas 2, 4 and 9 would eliminate the need for several lift stations and some extra depth sewer required for a conventional gravity sewer system. However, a conventional gravity system would still be required as the primary backbone system and in part for all of the subareas. Thus, a low pressure system is not feasible as a stand-alone alternative.

### 4.2.6 Concept 4 – Combination Sewer System

The combination sewer system concept is a hybrid alternative combining a low pressure sewer system for low-lying portions of the subarea with a conventional gravity sewer system . Since much of subareas 2 and 9 may be served by a conventional gravity sewer system, a cheaper low pressure sewer system is considered to serve the depressed or low-lying areas instead of more expensive conventional lift stations and force mains. Because of the undulating topography and difficult construction conditions in subarea 4, the flow from the entire subarea is directed via low pressure sewer system to the gravity main along the west boundary of the subarea. From there, the flows from subarea 2, 3 and 4 are collected and pumped to the Goetz Road Trunk.

The leading suppliers of STEP and grinder pump systems, Orenco Systems, Inc., and E/One Sewer Systems, were asked to submit preliminary designs for their respective systems to serve the above-designated portions of the Quail Valley study area.

In order to estimate capital costs for the combination sewer concept, it is necessary to select the preferred low pressure sewer system. Comparison of the STEP to the grinder pump systems for the Quail Valley application included review of the submitted vendor designs as well as available literature. The following pertinent points are noted:

- Both STEP and grinder pump systems allow the installation of shallow, terrain following pressure pipe conveyance systems (HDPE or PVC) to convey the pumped effluent or sewage.
- The STEP system would allow the use of existing septic tanks, but would require installation of septic tanks for all new homes. Existing septic tanks would need to be inspected to determine condition for retrofit of STEP.
- Installation costs of both systems appear to be roughly similar where existing septic tanks can be retrofitted, but STEP costs are somewhat greater where new septic tanks are needed.
- Frequency of maintenance calls and costs of maintenance, rehabilitation, and replacement appear to be greater for STEP, based on data and maintenance reports by owners of both systems.
- Power costs are roughly similar for both systems.
- The E/One grinder pump has superior hydraulic performance characteristics for this application where pumping heads into the pressure pipe grid are highly variable with location and time. The unusually steep pump curve shows only minor variation in discharge even with large variations in total dynamic head (TDH).
- Grinder pump effluent is reportedly more compatible with aerobic treatment plants than is STEP effluent, which features anaerobic BOD with a high percentage of

ammonia which is toxic to microbes. In this application, STEP effluent would be blended with raw sewage in the backbone gravity system.

For the above reasons, the grinder pump system is selected as the preferred low pressure solution for purposes of determination of capital and long term costs.

Both the Orenco STEP and E/One grinder pump systems appear to be well-engineered and supported. However, there is very limited operational history of such systems in California, and the vendor-provided information on performance history of the competing systems needs to be corroborated.

### Subarea 2

Using a low pressure sewer system for the low-lying portion of subarea 2 eliminates the need for the conventional lift station there. Once the flow is joined with the conventional gravity sewer system at Hampshire Drive and La Bertha Lane, the entire flow is pumped to La Bertha Lane and Mountain View Place and joined with the sewer system along the north boundary of subarea 4.

**Table 4-9**
**Subarea 2 - Combination Gravity and Low Pressure System**

| Item | Quantity | Unit | Unit Price | Amount |
|---|---|---|---|---|
| 8-inch Main | | | | |
| Depth < 10 | 12500 | LF | $102 | $1,275,000 |
| 48" Diameter MH | 51 | EA | $8,550 | $436,050 |
| Sewer Lateral | | | | |
| SB-177 (Minimum Depth) | 393 | EA | $4,850 | $1,906,050 |
| Lift Station | | | | |
| La Bertha Lane<br>Units Served: 462 UNITS | 1 | EA | $600,000 | $600,000 |
| Lift Station Eminent Domain | | | | |
| Lift Station Site | 1 | EA | $70,150 | $70,150 |
| Force Main | | | | |
| 6" - La Bertha Lane to Newsport Drive | 1500 | LF | $103 | $154,500 |
| Low Pressure Sewer System | | | | |
| Ginder Pump Station | 69 | EA | $2,900 | $200,100 |
| Pump/Panel Installation | 69 | EA | $1,000 | $69,000 |
| 1.25" HDPE Pipe Lateral | 3,450 | LF | $18 | $62,100 |
| Lateral + Kit | 69 | EA | $1,200 | $82,800 |
| 2.00" HDPE Pipe | 12000 | LF | $44 | $528,000 |
| Air Release Valve | 1 | EA | $600 | $600 |
| Clean Out | 1 | EA | $300 | $300 |
| Decommission Existing Septic Tank | 213 | EA | $500 | $106,500 |
| Sub-total = Construction + Contingency (20%): | | | | $5,491,150 |
| Engineering, Administration, Legal & CM (25%): | | | | $1,372,788 |
| Total: | | | | $6,863,938 |

## Subarea 4

A combination conventional gravity sewer system and low pressure sewer system is feasible for subarea 4, as all the flow is directed to a conventional pump station at the southwest corner of subarea 4. From there it is pumped to a collection manhole at Avenida Robles, where it is connected to the Goetz Road Trunk.

**Table 4-10**
**Subarea 4 - Combination Gravity and Low Pressure System**

| Item | Quantity | Unit | Unit Price | Amount |
|---|---|---|---|---|
| 8-inch Main | | | | |
| Depth < 10 | 2350 | LF | $110 | $258,500 |
| 10-inch Main | | | | |
| Depth < 10 | 250 | LF | $110 | $27,500 |
| Depth 10-15 | 250 | LF | $152 | $38,000 |
| Depth 15-20 | 250 | LF | $318 | $79,500 |
| 12-inch Main | | | | |
| Depth < 10 | 800 | LF | $110 | $88,000 |
| 15-inch Main | | | | |
| Depth < 10 | 1000 | LF | $110 | $110,000 |
| 48" Diameter MH | | | | |
| Depth < 10 | 1 | EA | $9,200 | $9,200 |
| Depth 10-20 | 1 | EA | $14,650 | $14,650 |
| Depth 20+ | 18 | EA | $20,050 | $360,900 |
| Lift Station | | | | |
| Newport Drive & Mountain View Place<br>Units Served: 1814 UNITS | 1 | EA | $500,000 | $500,000 |
| Goetz Drive & Mountain View Place<br>Units Served: 628 UNITS (SA4) & 232 UNITS (SA3) | 1 | EA | $250,000 | $250,000 |
| Lift Station Eminent Domain | | | | |
| Lift Station Site | 2 | EA | $57,500 | $115,000 |
| Force Main | | | | |
| 6" - Goetz Drive to La Bertha Ln | 700 | LF | $110 | $77,000 |
| 8" - Newport Drive to Goetz Road | 3000 | LF | $147 | $441,000 |
| Low Pressure Sewer System | | | | |
| Ginder Pump Station | 1120 | EA | $2,900 | $3,248,000 |
| Pump/Panel Installation | 1120 | EA | $1,000 | $1,120,000 |
| 1.25" HDPE Pipe Lateral | 47,450 | LF | $18 | $854,100 |
| Lateral + Kit | 1120 | EA | $1,200 | $1,344,000 |
| 1.50" HDPE Pipe | 125 | LF | $44 | $5,500 |
| 2.00" HDPE Pipe | 33103 | LF | $44 | $1,456,532 |
| 3.00" HDPE Pipe | 3255 | LF | $45 | $146,475 |
| 4.00" HDPE Pipe | 1055 | LF | $48 | $50,640 |
| Air Release Valve | 12 | EA | $600 | $7,200 |
| Clean Out | 35 | EA | $300 | $10,500 |
| Decommission Existing Septic Tank | 407 | EA | $500 | $203,500 |
| Sub-total = Construction + Contingency (20%): | | | | $10,815,697 |
| Engineering, Administration, Legal & CM (25%): | | | | $2,703,924 |
| Total: | | | | $13,519,621 |

## Subarea 9

For the western portion of subarea 9, a low pressure sewer system eliminates the need for two conventional lift stations. The flow from the western portion is directed via low pressure sewer system to connect with a conventional gravity sewer system on the eastern portion of subarea 9. The flow from the entire subarea is connected to the Goetz Road Trunk at Vista Way.

Alternatives A and B differ only in the treatment of subarea 4, with Alternative A featuring extra-depth sewers in some areas and Alternative B featuring normal depth sewers but more lift stations.

**Table 4-11**
**Subarea 9 - Combination Gravity and Low Pressure System**

| Item | Quantity | Unit | Unit Price | Amount |
|---|---|---|---|---|
| 8-inch Main | | | | |
| Depth < 10 | 15000 | LF | $109 | $1,635,000 |
| 10-inch Main | | | | |
| Depth < 10 | 200 | LF | $109 | $21,800 |
| 48" Diameter MH | 55 | EA | $9,100 | $500,500 |
| Sewer Lateral | | | | |
| SB-177 (Minimum Depth) | 397 | EA | $4,100 | $1,627,700 |
| Low Pressure Sewer System | | | | |
| Ginder Pump Station | 369 | EA | $2,900 | $1,070,100 |
| Pump/Panel Installation | 369 | EA | $1,000 | $369,000 |
| 1.25" HDPE Pipe Lateral | 18,450 | LF | $18 | $332,100 |
| Lateral + Kit | 369 | EA | $1,200 | $442,800 |
| 1.50" HDPE Pipe | 125 | LF | $44 | $5,500 |
| 2.00" HDPE Pipe | 7500 | LF | $44 | $330,000 |
| 3.00" HDPE Pipe | 5500 | LF | $45 | $247,500 |
| 4.00" HDPE Pipe | 2000 | LF | $48 | $96,000 |
| Air Release Valve | 5 | EA | $600 | $3,000 |
| Clean Out | 10 | EA | $300 | $3,000 |
| Decommission Existing Septic Tank | 270 | EA | $500 | $135,000 |
| Sub-total = Construction + Contingency (20%): | | | | $8,182,800 |
| Engineering, Administration, Legal & CM (25%): | | | | $2,045,700 |
| Total: | | | | $10,228,500 |

### 4.2.7 Alternatives Selected

Of the four concepts considered, the vacuum system option and stand-alone pressure system are not feasible for the Quail Valley application. Remaining for consideration are two conventional gravity alternatives and the combination gravity/low pressure system alternative.

## 4.3 COMPARATIVE CAPITAL COSTS

For the three major alternatives, the capital costs are summarized as one measure of comparison of the sewer system options. Capital costs represent estimated construction costs (including a 20% contingency factor), plus an additional 25% to cover engineering, legal, administration, unaccounted-for right-of-way acquisitions and construction management.

Table 4-11 provides a summary of the estimated costs for the conventional gravity alternative, including subalternatives A and B for Subarea 4, and the combination alternative, hereafter referred to as Alternative C..

**Table 4-12**
**Summary of Estimated Capital Costs**
**For Sewer Alternatives**

| Study Area | Alternative A | Alternative B | Alternative C |
|---|---|---|---|
| Goetz Road Trunk | $3,272,361 | $3,272,361 | $3,272,361 |
| Subarea 2 | $7,915,425 | $7,915,425 | $6,863,938 |
| Subarea 3 | $3,237,900 | $3,237,900 | $3,237,900 |
| Subarea 4 | $24,731,640 | $25,686,000 | $13,519,621 |
| Subarea 5 | $6,188,700 | $6,188,700 | $6,188,700 |
| Subarea 8 | $832,200 | $832,200 | $832,200 |
| Subarea 9 | $13,248,750 | $13,248,750 | $10,228,500 |
| Total Capital Cost | $59,426,976 | $60,381,336 | $44,143,220 |

As previously described, the only difference between Gravity System Alternatives A and B is the service of Subarea 4 with some extra-depth pipes and fewer lift stations, or a normal-depth system with more lift stations.

It is noted from the above table that there is little difference in the capital costs between Alternatives A and B, since the added cost of constructing deep sewers approximates the cost of the additional lift stations.

However, the combination option (Alternative C) shows significantly lower capital costs in subareas 2, 3, and 4 which are all or partially served by the low pressure systems. The overall capital cost savings amount to $15 to $16 million when compared to the two gravity options.

For the overall Quail Valley Community, the capital cost for the various options ranges from about $11,600 to $15,600 per parcel to install the community system (total capital cost ÷ total ultimate parcels). As previously discussed, this does not include offsite facilities. Note that if financial considerations dictate the need to precisely disaggregate costs among the subareas, refinement would be necessary to take into account shared facilities. This would best be accomplished at the Preliminary Design Report (PDR) phase.

## 4.4 LIFE-CYCLE COST ANALYSIS

The long term costs of the various alternatives must take into account ongoing or recurrent operation, maintenance, rehabilitation and replacement of facilities. Because there is considerable difference between the ongoing costs of lift stations and gravity sewers, or between conventional gravity/force main systems and low pressure systems, an estimate of long term cost is important in assessing the comparative fiscal viability of the alternatives.

An accepted method of estimating the comparative value of expenditures of differing amounts made at differing times is termed "life-cycle costing." Future expenditures are given a "present worth" value which is the equivalent amount which would need to be initially invested in an escrow account at a stipulated interest rate, to cover anticipated future or recurring expenditures. In this way, both the initial capital cost and the anticipated future cost of a project can be fairly considered in the comparison of alternative projects.

### 4.4.1 Time-Value of Money

Life-cycle costing is particularly sensitive to the discount rate assumed. The cost of money related to time is a function of both the cost of borrowing (interest rate) and inflation rate. Unsubsidized interest rates are assumed to 6% to 7%, while inflation in recent years has generally run at 3% to 4% per year. The effective time value of money is the difference between interest and inflation over the long term. Future costs discounted at this rate realistically reflect the implications of interest and inflation on deferred expenditures.

For purposes of this comparative analysis, it is assumed that interest rates average 6% and inflation 3%; thus, a 3% discount rate or compound interest factor is used to compute present worth of deferred or annual expenditures.

### 4.4.2 Methodology

Anticipated future costs include on-going O&M, power, and recurrent replacement of facilities. Future costs for the alternatives are approximated based on the following information sources and assumptions:

- Long term O&M costs for sewer lines and lift stations are estimated from unit values obtained from the District, as compiled from historical data.
- Long term O&M costs for low pressure facilities are estimated from information obtained from leading vendors based on existing systems.
- Power costs for lift stations are estimated from connected HP and load factors and an assumed energy rate per KWH.
- Power costs for low-pressure system grinder pumps are estimated from vendor information factored to reflect prevailing energy rates.
- Replacement frequency (expected life) of facilities is assumed as follows:
  Pipelines (pressure or gravity) ........................... 60 years
  Lift stations equipment ...................................... 15 years
  Structure ........................................ 40 years
  Low Pressure system equipment........................ 15 years
  Structure ........................................ 40 years
- Replacement cost for facilities is less than the initial capital cost, since right-of-way acquisition and some of the engineering and construction management costs would be eliminated. Replacement costs are thus based on the estimated initial construction cost plus an added 15% for a reduced level of engineering, construction management, and contingencies.

The following tables summarize the annual and future recurring costs of the key facilities for each of the three alternatives, along with the compiled present worth of future expenditures.

### 4.4.3 Operation and Maintenance Costs

Operation and maintenance of a sewer system includes ongoing inspection, cleaning, repair and rehabilitation of sewer lines, lift stations and force mains. Ongoing maintenance of a well designed gravity sewer system is minimal. Lift station operation and maintenance involves periodic inspection and trouble shooting to ensure proper and reliable functioning of equipment. Low pressure systems require monitoring and periodic although infrequent maintenance of the numerous individual grinder pump stations.

Costs for operation and maintenance of facilities in each of the three alternatives are expressed in terms of average annual costs, and converted to present worth by applying "uniform series" present worth factors.

The following tables summarize operation and maintenance costs, including power, for each alternative, and the present worth value of long term costs.

**Table 4-13**
**Annual Operation and Maintenance Costs**

| Subarea | Alternative A | Alternative B | Alternative C |
|---|---:|---:|---:|
| Goetz Road Trunk | $431 | $431 | $431 |
| Subarea 2 | $7,837 | $7,837 | $7,837 |
| Subarea 3 | $446 | $446 | $446 |
| Subarea 4 | $25,774 | $28,668 | $93,414 |
| Subarea 5 | $782 | $782 | $782 |
| Subarea 8 | $168 | $168 | $168 |
| Subarea 9 | $13,480 | $13,480 | $50,982 |
| Total O&M Cost | $48,919 | $51,812 | $154,060 |

**Table 4-14**
**Present Worth of Operation and Maintenance Costs**

| Subarea | Alternative A | Alternative B | Alternative C |
|---|---|---|---|
| Goetz Road Trunk | $14,350 | $14,350 | $14,350 |
| Subarea 2 | $261,237 | $261,237 | $261,237 |
| Subarea 3 | $14,875 | $14,875 | $14,875 |
| Subarea 4 | $859,143 | $955,602 | $3,113,797 |
| Subarea 5 | $26,075 | $26,075 | $26,075 |
| Subarea 8 | $5,600 | $5,600 | $5,600 |
| Subarea 9 | $449,342 | $449,342 | $1,699,400 |
| Total Present Worth of O&M | $1,630,621 | $1,727,080 | $5,135,334 |

### Replacement Costs

Table 4-15 is a summary of the replacement components for each alternative by subarea, showing replacement frequency and present worth value of future expenditures.

Note that for recurring expenditures such as lift stations or low pressure system mechanical equipment, several replacements would be required over the time frame of the evaluation. For this evaluation, a 75 year life cycle time frame is assumed, meaning that four replacements of mechanical equipment would occur, (at 15, 30, 45, and 60 years). One replacement of structures would be required at 40 years, and a single replacement of pipelines is assumed to take place at 60 years.

The present worth value of the sequence of future replacements, in the case of mechanical equipment at a 15 year frequency, is greater than the initial construction cost.

**Table 4-15**
**Replacement Costs and Present Worth**

| Alternative A (Gravity - Extra Depth) | | | |
|---|---|---|---|
| Subarea | Replacement Cost | Frequency | Present Worth |
| Goetz Road Trunk | $3,010,572 | 60 | $510,894 |
| SA2 - Pipelines | $4,726,673 | 60 | $802,116 |
| SA2 - Structures | $634,800 | 40 | $194,630 |
| SA2 - Equipment | $423,200 | 15 | $629,722 |
| SA3 | $2,436,965 | 60 | $413,553 |
| SA4 - Pipelines | $4,726,673 | 60 | $802,116 |
| SA4 - Structures | $1,393,800 | 40 | $427,339 |
| SA4 - Equipment | $929,200 | 15 | $1,382,650 |
| SA5 | $4,618,170 | 60 | $783,703 |
| SA8 | $623,645 | 60 | $105,833 |
| SA9 - Pipelines | $8,017,110 | 60 | $1,360,504 |
| SA9 - Structures | $828,000 | 40 | $253,865 |
| SA9 - Equipment | $552,000 | 15 | $821,376 |
| Total Present Worth of Replacement Cost: | | | $8,488,300 |

**Table 4-15**
**Replacement Costs and Present Worth (Continued)**

| Alternative B (Gravity - Normal Depth) | | | |
|---|---|---|---|
| Subarea | Replacement Cost | Frequency | Present Worth |
| Goetz Road Trunk | $3,010,572 | 60 | $510,894 |
| SA2 - Pipelines | $4,726,673 | 60 | $802,116 |
| SA2 - Structures | $634,800 | 40 | $194,630 |
| SA2 - Equipment | $423,200 | 15 | $629,722 |
| SA3 | $2,436,965 | 60 | $413,553 |
| SA4 - Pipelines | $7,162,603 | 60 | $1,215,494 |
| SA4 - Structures | $2,014,800 | 40 | $617,738 |
| SA4 - Equipment | $1,343,200 | 15 | $1,998,682 |
| SA5 | $4,618,170 | 60 | $783,703 |
| SA8 | $623,645 | 60 | $105,833 |
| SA9 - Pipelines | $8,017,110 | 60 | $1,360,504 |
| SA9 - Structures | $828,000 | 40 | $253,865 |
| SA9 - Equipment | $552,000 | 15 | $821,376 |
| Total Present Worth of Replacement Cost: | | | $9,708,108 |

| Alternative C (Combined Sewer System) | | | |
|---|---|---|---|
| Subarea | Replacement Cost | Frequency | Present Worth |
| Goetz Road Trunk | $3,010,572 | 60 | $510,894 |
| SA2 - Pipelines | $4,726,673 | 60 | $802,116 |
| SA2 - Structures | $634,800 | 40 | $194,630 |
| SA2 - Equipment | $423,200 | 15 | $629,722 |
| SA3 | $2,436,965 | 60 | $413,553 |
| SA4 - Pipelines | $5,604,835 | 60 | $951,140 |
| SA4 - Structures | $2,931,120 | 40 | $898,681 |
| SA4 - Equipment | $4,047,080 | 15 | $6,022,055 |
| SA5 | $4,618,170 | 60 | $783,703 |
| SA8 | $623,645 | 60 | $105,833 |
| SA9 - Pipelines | $5,812,650 | 60 | $986,407 |
| SA9 - Structures | $579,238 | 40 | $177,594 |
| SA9 - Equipment | $1,075,727 | 15 | $1,600,682 |
| Total Present Worth of Replacement Cost: | | | $14,077,011 |

### 4.4.4 Life-Cycle Cost of Alternative Systems

The following table summarizes the life-cycle costs of the two gravity system alternatives (A and B) and the combination alternative (C), by combining the initial capital cost with the present worth of the long term costs of O&M and replacement.

**Table 4-16**
**Life Cycle Costs (75 Years)**

| Costs | Alternative A | Alternative B | Alternative C |
|---|---|---|---|
| Capital Construction | $59,426,976 | $60,381,336 | $44,143,220 |
| O&M (Present Worth) | $1,630,621 | $1,727,080 | $5,135,334 |
| Replacement (Present Worth) | $8,488,300 | $9,708,108 | $14,077,011 |
| Life Cycle | $69,545,897 | $71,816,524 | $63,355,565 |

It is noted from the above tables that although capital costs are much less for the combination system (Alternative C), the present worth of future costs for Alternative C is much greater. The difference in capital costs between the lowest cost gravity system (Alternative A) and Alternative C is over $15 million, while the life cycle cost differential is only about $6 million (about 5% of the total life cycle cost). A sensitivity analysis was done by assuming that the life expectancy of the pipelines exceeded the life-cycle time frame, so that no pipeline replacement costs were incurred. The results did not change the relative life cycle costs of the three alternatives.

## 4.5 ADDITIONAL CONSIDERATIONS

Although capital or life-cycle costs are likely the dominant consideration in the feasibility of a Quail Valley community sewer system, there are other important considerations, including constructability and level of community disruption during construction, as well as long term O&M burden to be incurred. Following is a brief discussion of considerations and their implications for the three alternatives. Note that the District will need to decide on the relative significance of the non-quantifiable considerations.

### 4.5.1 Constructability

The primary considerations in constructability are the geotechnical issues pointed out in the "Limited Geotechnical Study" by Inland Foundation - difficult trenching and groundwater. Because of the limited nature of the subject study, only generalized conclusions can be drawn about how the rock excavation will affect the type of equipment needed and production rates. From on-site observation and additional discussions with contractors familiar with the study area, the following is concluded regarding constructability.

**Excavatability:** Although rocky soils are predominant, shallow alluvial soils and fill materials are readily excavated, and even the deeper partially weathered rock is assumed to be "rippable" and can likely be trenched with proper equipment. Reduced production rates are accounted for in the construction cost estimates for the various alternatives.

**Dewatering:** High groundwater and the need for dewatering can be largely remediated by limiting construction to the summer and fall months. Even so, groundwater will likely be encountered along drainage courses and in low-lying areas. It is also likely that groundwater will contain domestic sewage and as such, will have high levels of bacterial contamination. This will dictate the need for special procedures to protect the health of construction crews. The SARWQCB regulates all discharges and as such will need to approve of disposal of the dewatering effluent.

The SARWQCB has been contacted, and has provided information on the quality of groundwater from samples in the area. Officials have indicated that any discharge to stream courses will be prohibited, but that it may be possible to discharge into evaporation ponds constructed nearby if the effluent does not contain volatile organic contaminants. Another possibility is pumping through an "invasion" pipeline to the temporary lift station being constructed to serve the Canyon Heights (subarea 7) development. A third, more costly option is pumping and hauling to the EMWD PVRWRF or other nearby WWTP. Indications are that the quantities of dewatering water resulting from construction in the Quail Valley area will be relatively small. Dewatering issues would be reduced in the combination alternative which includes shallow pressure pipe in many of the areas where groundwater is most likely to be encountered.

**Right-of-Way Acquisition:** Right-of-way will need to be acquired for lift station sites and where sewer lines traverse private property. The conventional gravity alternatives for subarea 4 require traversing a significant number of parcels, necessary to avoid constructing additional lift stations. The preliminary sewer alignments avoid developed parcels, and it is noted from the parcel overlay of the recent aerial photo (Figure 2) that the lots along the southwest-trending drainage courses are mostly undeveloped.

The cost of right-of-way acquisition is based on recent lot sales, converted to cost per square foot (see Section 4.2.2). Where it is necessary to traverse a residential parcel in subarea 4, a 20-foot easement might render a 4,000-5,000 square foot lot unbuildable. However, since the lots within the drainage courses are apparently already at marginally buildable best, the unit value is believed to represent a fair market value. If new septic systems were prohibited, the pre-project value would obviously be much less.

The combination system alternative would significantly reduce the need to purchase right-of-way, since most of the eminent domain is appurtenant to the pipelines required to serve subarea 4 by gravity. However, the grinder pump units would have to be installed on private property at each residence.

**Access and Community Disruption:** The Quail Valley study area is characterized by relatively light, internally-generated traffic. The roads are abnormally narrow in some areas, particularly throughout subarea 4. Although large trenching equipment might render these streets impassable during construction, subarea 4 has no dead end streets and thus alternate routes would be available. Streets in the other subareas are considerably wider.

Noise and dust generation will be a factor, particularly in the more crowded environment of subarea 4. However, these impacts can be partially mitigated by normal measures.

Gravity Alternative A requires some reaches of deep trenching. Should the deep trenching in the narrow streets of subarea 4 require rock breakers or blasting, another alternative should be selected.

### 4.5.2 Maintenance of On-Lot Facilities

The combination system alternative, if selected by the District, carries additional staff burden considerations inherent in the monitoring and maintenance of about 1,560 small grinder pump station units at individual dwellings. Costs notwithstanding, the need to access and perform maintenance for such a large number of on-lot facilities is a serious consideration. Even assuming, as represented by E/One for its GP 2000 grinder pump, an average of 10 years between service calls, the number of site visits as the system ages could ramp up to an average of 2 to 3 per working day, and could be greater if a large number of units reach their normal service life simultaneously. However, it should be noted that the latest generation E/One units are modular and are fairly easily replaced or substituted for maintenance. Contract maintenance of these systems should be considered, as it would relieve the possibility highly variable demands on District staff and might be a more cost effective solution.

## 4.6 CONCLUSIONS

Following is a summary of the principal conclusions of this investigation.

1. Construction of a community sewer system in Quail Valley is technically feasible, but would encounter significant challenges due to terrain, geotechnical, and development conditions.

2. A conventional gravity system with several lift stations would be the backbone component of all feasible alternatives, with the main collection artery a major trunk line along Goetz Road.

3. A feasible "hybrid" concept is a combination gravity/low pressure system which features select areas being served by small grinder-pump units at individual dwellings, pumping sewage through a pressure pipe network into the gravity system.

4. Estimated capital costs to sewer the Quail Valley service area range from $59,427,000 for the lowest cost gravity alternative to $44,143,000 for the combination alternative.

5. Estimated life cycle costs, including the present worth of long term O&M and replacement, also favor by a smaller margin the combination alternative, at $63,356,000 compared to $69,546,000 for the gravity alternative.

6. The additional consideration of constructability in this challenging environment tends to favor the combination alternative due to the relative ease of installing small diameter pressure pipe at a uniform depth compared to an engineered gravity system. However,

this advantage is offset to some degree by the necessity of having to install grinder pump units at individual dwellings.

7. The additional consideration of long term maintenance and replacement favors the gravity system, since the combination system would require monitoring, maintenance and replacement of hundreds of individual pumping units.

8. The final selection of an alternative is dependant upon which considerations are given the greatest weight, summarized as follows:

| | |
|---|---|
| • Capital Cost | Combination system is less costly by about $15 million, or 25% |
| • Life Cycle Cost | Combination system is slightly less costly, by about $6 million, or 9% |
| • Constructability | Combination system is less challenging in most respects |
| • Operation and Maintenance | Gravity system is less burdensome. |

9. Facilities phasing has not been addressed in this concept study. While the original intent was to find ways to remediate the current pollution problems, the study necessarily encompassed ultimate development. The timing of future development is difficult to predict. However, a phased solution to install a partial system to deal with the more immediate problem areas could significantly reduce the required initial capital outlay. The initial phase might be a low pressure system serving subarea 4 which could initially be pumped to an upsized Canyon Heights lift station, and which would ultimately become part of the combination system (Alternative C) developed herein.

10. Should the District decide to proceed with a Quail Valley sewer system project, and assuming funding/financial issues are solvable, the next step would be a preliminary design report (PDR) which would refine the selected project concept and develop a plan for phased construction. It would also further define geotechnical conditions at specific sites, and detail the design parameters for the pipeline, lift stations and low pressure systems (if selected). The PDR would result in a refinement of costs and specific identification of required easements and acquisitions.

# Appendix A
# Sewer Complaints History

## Appendix B
# Geotechnical Feasibility Invest

# Appendix C
# Utilities Survey

# Appendix D
# Pressure System Description

# Appendix E
# Construction Cost Worksheets