TO: COMMITTEE MEMBERS

FROM: BRUCE BUEL

DATE: JULY 30, 2008

AGENDA ITEM
2
AUG. 4, 2008

REVIEW FUGRO REPORT ON SOUTHLAND DISPOSAL CAPACITY

ITEM

Review Fugro Report on Southland Disposal Capacity [Forward Recommendation to Board].

BACKGROUND

Attached is a Fugro technical memorandum providing Fugro's estimate of the physical capacity of the existing disposal ponds. As indicated in the technical memorandum, Fugro estimates that the existing ponds can percolate an average of .57 million gallons per day of treated wastewater without increasing the size of the subsurface mound. Given that NCSD's current discharge averages approximately .58 to .63 million gallons per day, our current discharge will slowly increase the size of the mound (assuming that regulatory issues do not interfere). However, as new growth occurs (build out is projected at 1.2 mgd) the mound will grow faster and faster until it is no longer feasible to continue operations. Staff continues to believe that another source of disposal will be needed, but some time is available to select the best option or combination of options.

RECOMMENDATION

Staff recommends that the Committee review the attached technical memorandum and forward a recommendation to the Board to accept the technical Memorandum.

ATTACHMENTS

- Fugro Report on Southland Disposal Capacity

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FUGRO WEST, INC.

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June 30, 2008
Project No. 3596.003

PROJECT MEMORANDUM

To: Mr. Bruce Buel
Nipomo Community Services District

From: Peter Leffler and Paul Sorensen

Subject: Supplemental Groundwater Modeling Analysis

In accordance with our proposal dated May 22, 2008, Fugro has collected additional data and performed additional groundwater modeling runs to evaluate groundwater mounding beneath the Nipomo Community Services District's (NCSD) Southland wastewater treatment facility (WWTF) percolation ponds. Results of our previous modeling work (Fugro, February 2008), showed that the amount of influent coming into the facility and the associated groundwater mound, as measured by water levels in the WWTF monitoring wells, were growing over time. The purpose of the modeling effort documented in this memo was to evaluate the amount of inflow that could be disposed in the percolation ponds over the long term, while not causing any further growth of the mound above present levels.

DATA COLLECTION

The additional data collected for this effort included the amount of monthly influent to the plant from October 2007 to March 2008, and groundwater levels measured in early June 2008. A summary of data collected previously and data collected for the current analysis are included in Tables 1 and 2. Table 1 shows the average daily influent rate to the plant from 1991 to March 2008. The influent data show a consistent gradual increase in flow (0.16 to 0.43 MGD) from 1991 to 1998, followed by a slight decrease in inflow in 1999 and a period of stable flows (ranging from 0.40 to 0.42 MGD) between 2000 and 2003. Influent amounts increased to 0.47 MGD and 0.63 MGD in 2004 and 2005, followed by another slight decline and leveling off to a range of 0.58 to 0.59 MGD.

Table 2 shows available groundwater level data from four monitoring wells near the percolation ponds. Groundwater level data are limited but generally show increasing groundwater elevations from 2000 to 2005. Groundwater levels stabilized in wells MW-1 and MW-2 from 2005 to 2008. Slight increases (less than 2 feet) in groundwater levels occurred at wells PZ-1 and MW-3 from 2005 to 2008.

It should be noted that the District data apparently reflects inflow volumes to the facility. The modeling analysis and expected impacts on the effluent groundwater mound are based on effluent discharge volumes. For purposes of this analysis, we have assumed that the metered inflow to the plant equals the effluent discharge volumes into the percolation ponds.

GROUNDWATER MODELING

A transient groundwater flow model was previously developed for the site to evaluate groundwater mounding beneath and surrounding the percolation ponds. The model was calibrated to historical groundwater levels, and subsequently used to run scenarios to predict groundwater mounding associated with increasing rates of effluent disposal in the percolation ponds between 2007 and 2017 (Fugro, February 2008).

The groundwater modeling effort documented in this memo was designed to determine the amount of effluent disposal that could occur over the long term (e.g., 10 years) without generating additional growth in the groundwater mound beneath and around the percolation ponds. This work involved the use of the transient groundwater flow model previously developed for the site (Fugro, February 2008). The existing model was updated with the additional data for plant influent between October 2007 and March 2008 and the groundwater levels measured in early June 2008. The original model was developed using six-month stress periods (October to March and April to September) to generally match the occurrence of wet and dry seasons. Therefore, the updated model was run for an additional stress period covering October 2007 to March 2008. The predictive scenario model runs described below were simulations of the time period from April 2008 to 2017.

Based upon a review of previous modeling results, annual plant influent for recent years (Table 1), and historical groundwater levels (Table 2), three model scenario runs were conducted with assumed steady influent rates of 0.56, 0.57, and 0.58 MGD from April 2008 to September 2017. The modeling results at the three different rates are summarized in Table 3. The model scenario run of 0.58 MGD indicated that the groundwater elevations in the four monitoring wells would increase over current levels by an amount ranging from 0.34 to 0.60 feet. The 0.57 MGD model scenario run showing relatively stable groundwater levels with a decrease in water levels at three of the monitoring wells ranging up to 0.18 feet to an increase at one well of 0.13 feet. Finally, the 0.56 MGD model scenario run showed declines in groundwater levels at all wells ranging from 0.34 to 0.75 feet. The modeling results indicate that relatively stable groundwater elevations at the monitoring wells are achieved at a discharge rate of 0.57 MGD over the long term (Figures 1 through 4).

CONCLUSIONS/RECOMMENDATIONS

The results obtained from the supplemental groundwater modeling work indicate that the percolation ponds can dispose an average of approximately 0.57 MGD over a long period of time and not cause further increases in groundwater elevations at the existing monitoring wells. If it is determined that the groundwater mound must be maintained at higher or lower elevations than current levels, additional model runs could be conducted to evaluate the percolation pond discharge rate that would result in a specified set of groundwater levels in the monitoring wells.

Our original modeling effort (Fugro, February 2008) also provided an estimate of groundwater discharge to Nipomo Creek as a function of the growth of the groundwater mound. The groundwater flow model could also be used to further evaluate the projected percolation pond discharge rates given certain allowable limits on groundwater discharge to Nipomo Creek.

ATTACHMENTS: TABLES 1, 2, AND 3 AND FIGURES 1, 2, 3, AND 4

Table 1. Summary of Historical Average Annual Influent at Southland WWTF

Water Year	Model Stress Periods	Influent Flow (MGD)
1991	13/14	0.18
1992	15/16	0.23
1993	17/18	0.23
1994	19/20	0.27
1995	21/22	0.31
1996	23/24	0.33
1997	25/26	0.38
1998	27/28	0.43
1999	29/30	0.36
2000	31/32	0.40
2001	33/34	0.42
2002	35/36	0.40
2003	37/38	0.42
2004	39/40	0.47
2005	41/42	0.63
2006	43/44	0.59
2007	45/46	0.59
2008 (1st half)	47	0.58

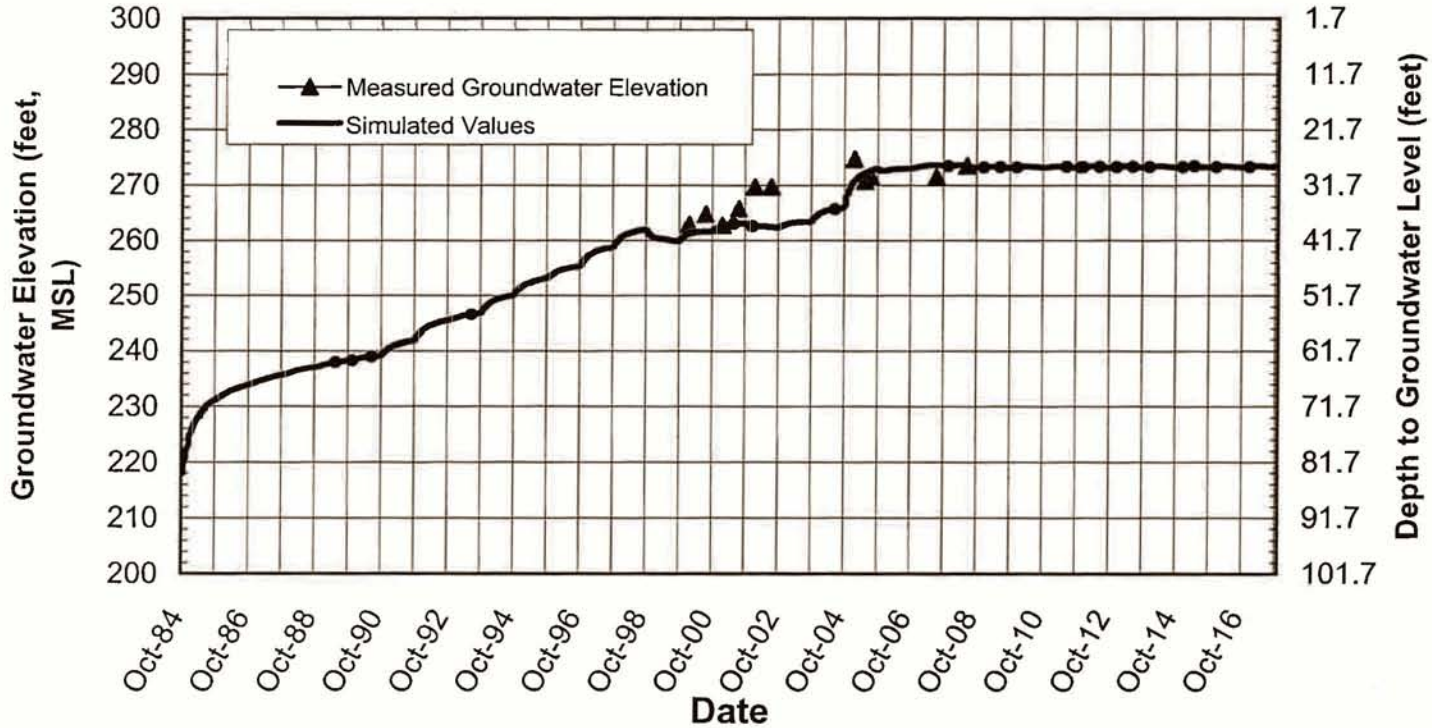
Table 2. Shallow Groundwater Levels for Southland WWTF

Date	PZ-1	MW-1	MW-2	MW-3
Jan. 2000	262.9	255.3	251.6	232.9
July 2000	264.7			
Jan 2001	262.7			
July 2001	265.7			
Jan. 2002	269.7			
July 2002	269.7			
July 2004		259.9	257.2	256.7
Jan. 2005	274.7	256.9	256.2	257.7
May 2005	270.7	262.9	262.2	258.7
July 2005	271.7	259.9	260.2	259.7
July 2007	271.5	262.0	260.4	259.5
June 2008	273.5	262.3	260.5	261.3

Table 3. Groundwater Model Scenario Results – Maximum Change for Historic High Groundwater Elevations in Feet

Percolation Rate (MGD)	MW-1	MW-2	MW-3	PZ-1
0.58	0.35	0.34	0.60	0.37
0.57	-0.09	-0.10	0.13	-0.18
0.56	-0.54	-0.54	-0.34	-0.75

Figure 1. PZ-1: Measured and Simulated Groundwater Levels



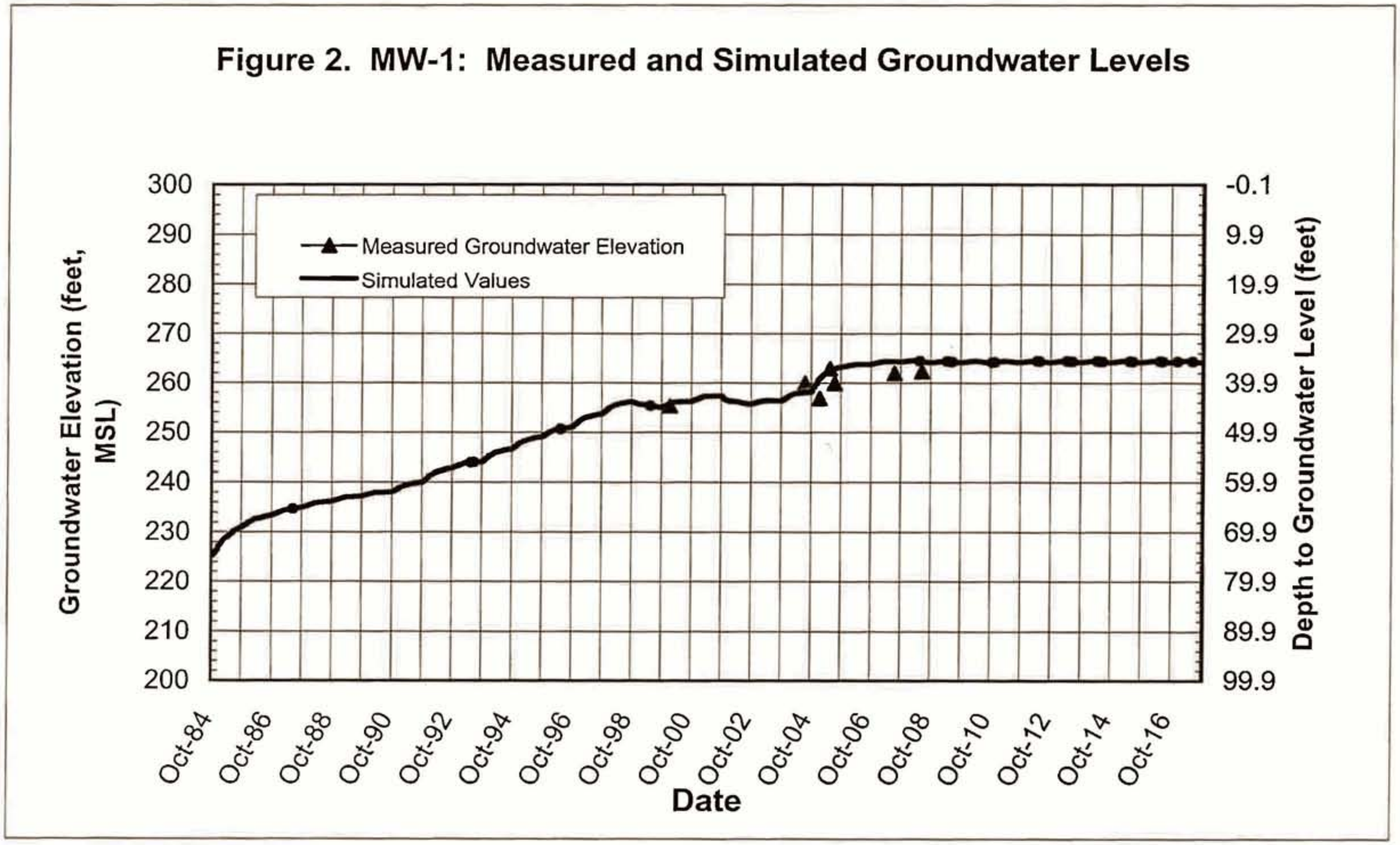


Figure 2. MW-1: Measured and Simulated Groundwater Levels
Measured Groundwater Elevation
Simulated Values
Groundwater Elevation (feet, MSL)
Depth to Groundwater Level (feet)
300
290
280
270
260
250
240
230
220
210
200
-0.1
9.9
19.9
29.9
39.9
49.9
59.9
69.9
79.9
89.9
99.9
Oct-84
Oct-86
Oct-88
Oct-90
Oct-92
Oct-94
Oct-96
Oct-98
Oct-00
Oct-02
Oct-04
Oct-06
Oct-08
Oct-10
Oct-12
Oct-14
Oct-16
Date

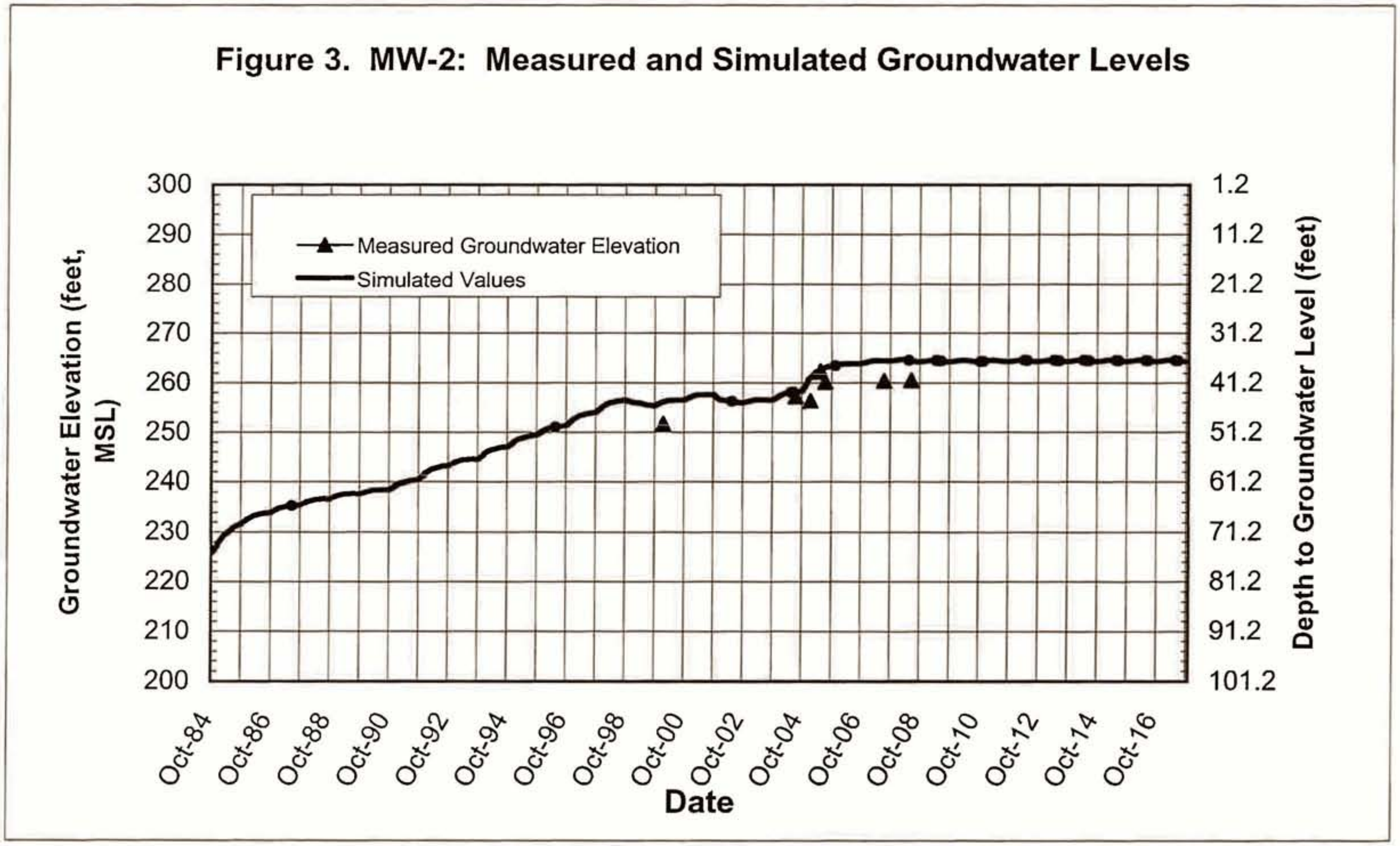


Figure 3. MW-2: Measured and Simulated Groundwater Levels
Measured Groundwater Elevation
Simulated Values
Groundwater Elevation (feet, MSL)
Depth to Groundwater Level (feet)
300
290
280
270
260
250
240
230
220
210
200
1.2
11.2
21.2
31.2
41.2
51.2
61.2
71.2
81.2
91.2
101.2
Oct-84
Oct-86
Oct-88
Oct-90
Oct-92
Oct-94
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Oct-14
Oct-16
Date

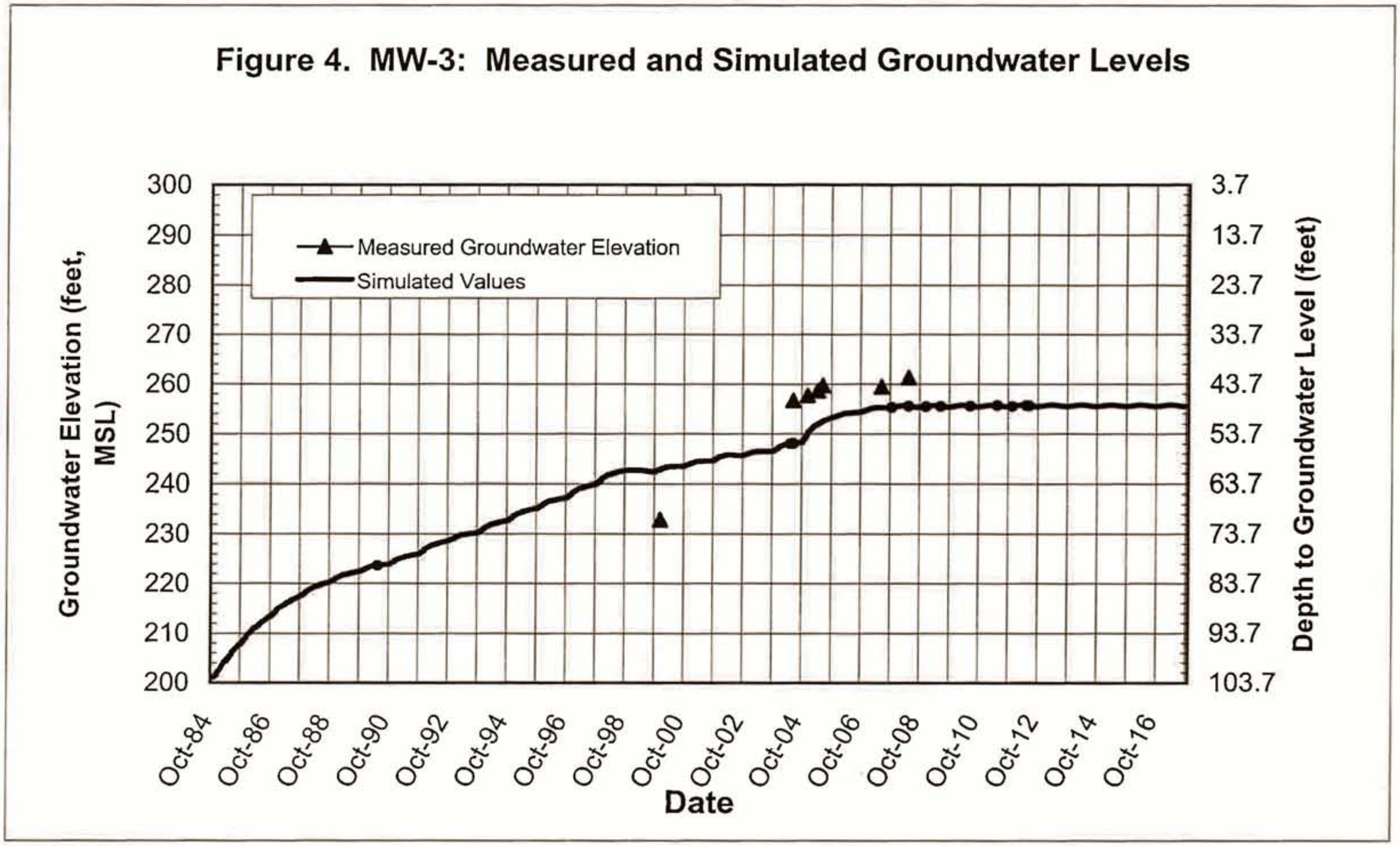
Figure 4. MW-3: Measured and Simulated Groundwater Levels
Measured Groundwater Elevation
Simulated Values
Groundwater Elevation (feet, MSL)
Depth to Groundwater Level (feet)
Date
300
290
280
270
260
250
240
230
220
210
200
3.7
13.7
23.7
33.7
43.7
53.7
63.7
73.7
83.7
93.7
103.7
Oct-84
Oct-86
Oct-88
Oct-90
Oct-92
Oct-94
Oct-96
Oct-98
Oct-00
Oct-02
Oct-04
Oct-06
Oct-08
Oct-10
Oct-12
Oct-14
Oct-16

TO: COMMITTEE MEMBERS

FROM: BRUCE BUEL BJB

DATE: JULY 30, 2008

AGENDA ITEM
3
AUG. 4, 2008

REVIEW FUGRO REPORT ON OFF-SITE DISPOSAL RESEARCH

ITEM

Review Fugro Report on Off-Site Disposal Research [Forward Recommendation to Board].

BACKGROUND

Attached is a Fugro technical memorandum documenting the results of the preliminary geotechnical research on the Pasquini property located on the West side of Orchard Road South of the intersection of Southland and Orchard. As detailed in the technical memorandum, Fugro's initial research indicates that there are no fatal flaws with limited disposal of treated wastewater on the Pasquini property, however, more research is recommended regarding the potential for disposal to destabilize the bluff. Additionally, if the Pasquini property were to be selected as an additional disposal site, special consideration would need to be made to ensure that the treated wastewater was introduced below the top ten feet of the soil column.

Staff believes that the site deserves further consideration and it should be evaluated along with other options when the Master Plan is finalized and the Draft EIR is prepared (See agenda item 4).

RECOMMENDATION

Staff recommends that the Committee review the attached technical memorandum and forward a recommendation to the Board to accept the technical Memorandum.

ATTACHMENTS

- Fugro Report on Offsite Disposal Research

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July 30, 2008
Project No. 3596.002

Nipomo Community Services District
Post Office Box 326
148 S. Wilson Street
Nipomo, California 93444

Attention: *Mr. Bruce Buel*
General Manager

Subject: **Hydrogeologic and Geotechnical Assessment of APN 090-311-001, Nipomo, California**

Dear Mr. Buel:

Fugro West Inc. is pleased to submit this initial feasibility analysis and hydrogeologic and geotechnical assessment of the 192-acre parcel southwest of Orchard Road. The objective of the study is to provide a preliminary assessment of the parcel as a potential new site for expansion of the percolation pond component of the Nipomo Community Services District's Southland Wastewater Treatment Facility. It is important to understand that this report is a compilation of our current understanding of the hydrogeology of the site and immediately surrounding area. Further field investigation will be required to more completely characterize the site. Recommendations for such field work are outlined in this report.

Sincerely,

FUGRO WEST, INC.

Timothy A. Nicely, P.G., C.Hg.
Project Hydrogeologist

Paul Sorensen, P.G., C.Hg.
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Project Manager

David A. Gardner, P.G., C.Hg.
Principal Hydrogeologist
Senior Vice President

Gresham Eckrich
Staff Engineer

Jonathan D. Blanchard, P.E.
Principal Geotechnical Engineer

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PLATES

APPENDICES

HYDROGEOLOGIC AND GEOTECHNICAL ASSESSMENT OF APN 090-311-001, NIPOMO, CALIFORNIA

SITE DESCRIPTION AND BACKGROUND

The Nipomo Community Services District (District) is planning for the expansion of the District's Southland Wastewater Treatment Facilities (WWTF). One site being investigated is a 192-acre parcel southwest of Orchard Road (APN 090-311-001) in Nipomo, California, known as the Pasquini property. The intent of this investigation was to provide a preliminary assessment of the parcel as a potential new site for expansion of the percolation pond and effluent disposal component of the District's Southland WWTF.

The Pasquini property is currently fallow land that is used for cattle grazing. The property extends approximately 3,500 feet southwest of Orchard Road to Riverside Road. The southern edge of the parcel is formed by the floodplain of the Santa Maria River, and is characterized by a steep bluff that is approximately 80 to 130 feet high. The northwestern edge of the parcel is bounded by houses. Natural slope inclinations along the face of the bluff range from approximately 2h:1v to 1.2h:1v (horizontal to vertical). The slope appears to have been eroded by past meanders of the Santa Maria River. The existing ground surface above the bluff is mainly gently sloping, stabilized sand dunes. The existing site grade ranges from approximately elevation (el.) 320 feet near Orchard Road to approximately el. 290 feet along the top of the bluff. The base of the bluff along Riverside Drive is at about el. 170 feet. The Central Coast Aqueduct Extension (state water pipeline) passes through the southeastern quarter of the site along a northeast to southwest alignment. Immediately northwest of the state water pipeline alignment, a relatively deeply eroded drainage canyon intersects the face of the bluff, and occupies approximately six acres along the bluff slope. The slopes within the upper approximately 5 to 40 feet of the drainage canyon have vertical faces.

Development of percolation ponds at the site would likely consist of grading an earth pond that would generally be excavated to provide the needed storage capacity and surface area to allow for percolation of the effluent. Relatively small earthen berms would likely be used in combination with graded cut slopes to create the ponds. The suitability of the site for percolation requires that the soils beneath the site have sufficient permeability to allow for percolation of the treated effluent, and that aquitards (clay layers) are not present that would prevent vertical percolation or cause infiltrated water to daylight at the ground surface, such as along the face of the bluff or drainage canyon.

Characterization of the Southland WWTF (located on Southland Drive as shown on Plate 1) in Fugro's Phase 1 investigation led to a better understanding of a dual aquifer system beneath the WWTF. The shallow aquifer, which ranges from 60- to 140-feet below ground surface (bgs) at the Southland facility, is separated from a deep aquifer by a thick, relatively impermeable aquitard (clay layer) that precludes vertical migration of treated wastewater discharged to percolation basins from the surface to the deep aquifer. In the subsurface, the aquitard layer is inclined to the southwest. If the inclination of the aquitard continues without change to the west, it would be more than 250 feet bgs beneath the parcel under investigation. With regional groundwater levels apparently approximately 170 to 200 feet below ground surface, the aquitard would not present a constraint to effluent disposal.

WORK PERFORMED

PURPOSE

The purpose of this investigation was to assess the presence and thickness of possible aquitards under the parcel, as well as to evaluate the suitability of the parcel, or portions of the parcel, for development of percolation ponds. In addition to investigating the gross suitability of the parcel for percolation ponds, the potential for the percolated water to daylight on the bluff that borders the site was also considered.

SCOPE OF WORK

This report presents the results of a preliminary assessment that incorporates hydrogeological characterization of the site based on field exploration. Work conducted as part of this study included the following tasks:

- Conduct an initial site visit;
- Evaluate the site geologic and hydrogeologic setting and potential geologic hazards;
- Grossly characterize the presence of low permeability layers or lenses within the dune sand, geologic contacts, and local unconformities;
- Conduct Cone Penetrometer soundings (CPT);
- Drill three hollow stem auger (HSA) borings to depths of approximately 150 to 200 feet;
- Prepare gross geologic and hydrogeologic cross sections across the site;
- Analyze the gross suitability of the site for potential percolation ponds;
- Assess the potential for the site relative to geologic hazards such as seismicity, liquefaction of the subsurface with additional saturation, and slope stability;
- Prepare this summary report; and
- Present conclusions and recommendations for future work.

FIELD EXPLORATION

The field exploration program for this study consisted of advancing a total of 13 cone penetration test (CPT) soundings and three borings. A summary of the CPT and Hollow Stem Auger exploration is presented as Table 1 – Summary of CPT and Hollow Stem Auger Exploration. The locations of the hollow stem auger holes are presented on Plate 2. Logs of the CPT and hollow stem auger exploration are included in Appendix A.

Table 1 – Summary of CPT and Hollow Stem Auger Exploration

Exploration Number	Total Depth	Elevation
B-101	160	303
B-102	126	303
B-103	126	313
C-101	74	305
C-102	87	302
C-103	123	314
C-104	104	295
C-105	62	324
C-106	49	298
C-107	51	313
C-108	125	287
C-109	90	318
C-110	98	306
C-111	46	291
C-112	57	306
C-113	82	328

Drilling

The drilling subcontractor for the project was S/G Drilling Company of Lompoc, California. S/G used a truck mounted CME 85 drill rig to advance three (3) 8-inch diameter hollow stem auger borings at the site. The drilling was performed during the period of Wednesday, May 21 to Friday, May 23, 2008. The borings were advanced to depths between approximately 98 and 157 feet below the existing ground surface. The approximate locations of the borings are shown on Plate 2, Site Map and Cross Section Locations.

The borings were sampled using a 2-inch outside diameter standard penetration test (SPT) split-spoon sampler and a 3-inch outside diameter modified California split-spoon sampler. The modified California sampler was equipped with 1-inch high brass rings. The SPT sampler was used without liners. The samplers were driven into the materials at the bottom of the drill hole using a 140-pound automatic trip hammer with a 30-inch drop. The blow count (N-value) shown on the boring logs is the number of blows from the hammer that were needed to drive the sampler 1 foot, after the sampler had been seated at least 6 inches into the material at the bottom of the hole.

Groundwater was encountered at depths of between 98 and 157 feet below ground surface. The borings were backfilled with the soil cuttings and tamped after drilling. The sample intervals, N-values, a description of the subsurface conditions encountered, and other field and laboratory data are presented on the logs of the borings in Appendix A.

Cone Penetrometer Testing

Fugro Geosciences of Santa Fe Springs, California performed the CPT soundings for field exploration program during the period of May 19 to May 21, 2008. Two CPT soundings (CPT-101 and CPT-102) were advanced at the Southland WWTF adjacent to two existing percolation ponds and 11 CPTs at the subject parcel. At the Southland WWTF, the two CPTs were advanced to obtain CPT signatures at the existing ponds. The CPTs at the Southland WWTF were advanced to 74 and 86 feet below ground surface, respectively for C-101 and C-102. The 11 CPTs at the subject parcel were advanced to depths of between 46 feet (C-111) and 124 feet (C-108). The locations of the CPT soundings on the Pasquini property are shown on Plate 2.

The CPT soundings were performed using an electric cone penetrometer. The penetrometers were advanced into the ground using a hydraulic ram mounted in a truck having a weight of approximately 20 tons. The cone penetrometer has a diameter of approximately 1.4 inches. Cone tip resistance (q_c) and sleeve friction (f_s) were recorded on the penetrometer during all CPT soundings. Data was recorded at approximately 2 cm intervals using an on-board computer to provide a near-continuous profile of the soil conditions encountered during penetration. The friction ratio (FR) was computed for each value of q_c and f_s recorded. The data was retrieved electronically for use in subsequent geotechnical analyses. CPT data and soil behavior type classifications were used to evaluate the subsurface conditions encountered at the site. Plots of each CPT sounding are presented with the boring log data in Appendix A.

LABORATORY TESTING

Laboratory testing was performed on selected samples obtained during the field exploration. Laboratory tests for moisture content, unit weight, fines content (percent of soil, by dry weight, passing the No. 200 sieve), grain size analysis, direct shear strength and permeability were performed as part of this program. The tests were performed in general accordance with the applicable standards of ASTM. Direct shear samples were performed on samples saturated with water prior to testing and samples having the in situ moisture content. Results of laboratory testing are presented in Appendix B.

GENERAL CONDITIONS

Fugro prepared the conclusions, recommendations, and professional opinions of this report in accordance with the generally accepted geotechnical principles and practices at this time and location. This warranty is in lieu of all other warranties, either expressed or implied. This report was prepared for the exclusive use of Nipomo Community Services District and their authorized agents only. It is not intended to address issues or conditions pertinent to other parties, projects or for other uses. The report and the drawings contained herein are not intended to act as construction drawings or specifications. Explorations and services have not been requested nor performed to assess the presence or absence of hazardous or toxic materials.

The scope of services did not include any environmental assessments for the presence or absence of hazardous/toxic materials in the soil, surface water, groundwater, or atmosphere. Any statements, or absence of statements, in this report or data presented herein regarding odors, unusual or suspicious items, or conditions observed are strictly for descriptive purposes and are not intended to convey engineering judgment regarding potential hazardous/toxic assessment.

Soil and rock deposits can vary in type, strength, and other geotechnical properties between points of observations and exploration. Additionally, groundwater and soil moisture conditions also can vary seasonally or for other reasons. Therefore, we do not and cannot have a complete knowledge of the subsurface conditions underlying the site. The conclusions and recommendations presented in this report are based upon the findings at the points of exploration, and interpolation and extrapolation of information between and beyond the points of observation, and are subject to confirmation based on the conditions revealed by construction.

SITE CONDITIONS

GEOLOGIC SETTING

The project vicinity is within the Santa Maria basin, a transitional area between the Coast Ranges geomorphic province to the north and the Transverse Ranges to the south. The Santa Maria basin is underlain by a structural depression, with Tertiary-age rocks forming a series of broad folds (synclines and anticlines) with westward trending axes (Worts, 1951; Dibblee, 1994). The northernmost synclinal fold forms the basin beneath the Santa Maria Valley. The basin originated during the Miocene and is filled with up to 15,000 feet of marine and non-marine sediments overlying Cretaceous-age ultramafic and sedimentary rocks (Tennyson, 1992).

A map showing the a compilation of the regional geology as mapped by Lettis et al. (1994), Dibblee (1994) and Hall (1978) is shown on Plate 7, Regional Geologic Map. The Southland WWTF and the Pasquini property are located in the southeastern edge of the Nipomo Mesa. Surficial sediments on the Nipomo Mesa typically consist of Pleistocene Older Dune Sand Deposits, as mapped by Hall and Corbató (1967). The wind blown sediment has been stabilized by vegetation, and is present over most of the Nipomo Mesa. The deposits form a triangular lobe approximately four miles wide that extends inland 12 miles to just beyond Highway 101 and typically range from about 150 to 250 feet in thickness. The southern end of Nipomo Mesa is the steep bluff above Riverside Road, and is entirely composed of units of dune sand as exposed at the site. The sediments are typically highly permeable, which precludes appreciable runoff (DWR, 2002). Groundwater production from the sediments is relatively insignificant (Papadopulos, 2004). Relatively fine-grained layers of variable thickness and continuity are interlayered throughout the dune sand deposits, which locally restricts the vertical movement of groundwater (or applied wastewater from percolation ponds) within the unsaturated zone.

Sandy colluvial deposits are present at the base of the bluff and as thinly overlying soil along portions of the slope. The flood plain of the Santa Maria River borders the base of the bluff and is mapped by Hall and Corbató as being underlain by alluvial terrace deposits (Qt), but is likely younger alluvium (Qa). Throughout the Nipomo Mesa, the dune sand is generally underlain by the Paso Robles and Careaga Formations (DWR, 2002). The Paso Robles Formation is typically composed of unconsolidated to poorly consolidated sediments. The Careaga Formation is composed of unconsolidated to well consolidated sediments.

HYDROGEOLOGIC SETTING

The hydrogeology of the Nipomo Mesa has been described by numerous investigators. Papadopulos (2004) reports that the older dune sand deposits (Qds) of the Nipomo Mesa contain limited amounts of groundwater, with the primary aquifer being the underlying Paso Robles Formation. This primary aquifer is in direct hydraulic connection with the Santa Maria groundwater basin. Papadopulos (2004) also reports that the older dune sand deposits locally contain clay layers and that some of the shallow groundwater of the Nipomo Mesa is diverted laterally along these low permeability layers, creating such geographic features as Black Lake and Little Oso Flaco Lake.

For several years the firm of SAIC has worked for Nipomo CSD to create various maps that depict annual changes of groundwater in storage in the Nipomo Mesa and surrounding area. The maps use a GIS based contouring program that integrates water level data from key wells. Two such monitoring wells exist in the immediate vicinity of the Pasquini parcel (Appendix E) which are inferred to represent current and basin water level high (1982) and basin low (1992) water level conditions (personal communication with Mr. Brad Newton, SAIC).

The data used by SAIC were reviewed within the context of the CPT soundings and borings drilled as part of this study of the Pasquini parcel. SAIC recognizes that the data used in their analysis are "less than optimal" for the area simply because there is a general lack of water level data for the deeper aquifer (i.e., the Paso Robles formation). Water level data do not exist for the overlying older dune sand deposits. In general, data from SAIC would support a southwesterly flow of groundwater in the Paso Robles formation toward the Santa Maria River. Depth to groundwater is in the range of 200 to 250 feet below ground surface, depending on seasonal conditions and longer term (i.e. wet cycle v. dry cycle) conditions. The data are not sufficient to determine either the presence or absence of shallow, perched groundwater.

Similarly, both CPT and boring log data from this study were inconclusive with respect to the presence or absence of shallow (perched) groundwater within the older dune sand deposits. The hollow-stem auger borings encountered groundwater (wet cuttings) at depths as shallow as 157 feet (B-101), 116 feet (B-102), and 98 feet (B-103). The groundwater encountered in the borings may be associated with a perched groundwater table encountered within the dune sand, and above the groundwater level associated with the regional deeper aquifer described by SAIC. Inspection of Plate 2 and Plate 4 -- Cross Section B-B' suggests that the depths to inferred perched groundwater relative to the deep aquifer regional water level surface developed by SAIC is significant. The continuity of such perched groundwater is unknown but would appear to be at depths below the layers of dense older dune sand deposits obtained from the CPT soundings. While it is tempting to interpret a uniformly sloping perched groundwater surface flowing to the southwest, more data are needed to convincingly support such an interpretation. It is equally possible that the perched water surfaces encountered in the borings are isolated and not connected.

SUBSURFACE CONDITIONS

The subsurface conditions encountered beneath the parcel consist of dune sand deposits (Qds). The locations of the three (3) borings drilled and eleven (11) CPT soundings are shown on Plate 2. The logs for the explorations are presented in Appendix A. A discussion of the soil conditions encountered is provided below.

Dune Sand Deposits (Qds)

Dune sand deposits beneath the parcel consist of poorly graded sand (SP), sand with silt (SP-SM) and silty sand (SM) and were encountered in each of the explorations performed. The dune sand deposits were encountered to the maximum depths explored. Field blow counts recorded in the dune sand deposits ranged from 12 blows per foot (bpf) to refusal using standard penetration samplers and 17 bpf to refusal using modified California samplers. The subsurface profile is characterized as a dense dune sand unit overlying a very dense dune sand unit. Explorations within about 400 feet of the top of the bluff encountered an upper dense dune sand unit to depths ranging from approximately 25 to 40 feet. Explorations further back from the top of the bluff encountered the upper dense dune sand unit to depths of up to approximately 15 to 50 feet below ground surface.

Driven ring samples of the dune sand deposits tested in the laboratory had unit weights ranging from 91 to 128 pounds per cubic foot (pcf) and moisture contents ranging from 3 to 21 percent. Friction angles ranging from 35 to 38 degrees were estimated from direct shear tests performed on driven ring samples of the dune sand. The approximate cohesion values estimated from the direct shear tests ranged from approximately 300 pounds per square foot (psf) for samples that were saturated with water prior to testing, up to approximately 700 psf for samples tested at their in situ moisture content.

Six undisturbed samples collected from the hollow stem flight auger borings were tested in the laboratory for permeability determination (vertical direction) in accordance with ASTM method D-5084 (falling head method) or D-2434 (constant head method). The results of these tests are summarized on Table 2 that also includes the borings from which the samples were collected, the depths of the samples, soil classification per ASTM D2487 (based on the Unified Soil Classification System), and the percent passing the number 200 sieve (fines percentage). All of the undisturbed samples were representative of older dune sand deposits but reflect variable permeability values that correlate directly to the percentage of fines. Not surprisingly, samples of silty sand (SM) with a greater amount of fines display a lower permeability value. The slower permeability values, about 10 gallons per day per square foot, are representative of older dune sand deposits that were subject to some degree of weathering and soil development, likely resulting from extended periods when the dune sand deposits were exposed to surficial weathering, oxidation, and erosion. The higher permeability values estimated from the laboratory tests, in the 200 gallon per day per square foot range, are representative of clean, poorly graded sand (SP, SP-SM) having lesser amounts of silt.

Table 2 – Laboratory Testing Summary

Boring No.	Depth (feet)	Classification	Laboratory Determined Permeability		Passing No. 200 Sieve
			cm/sec	gpd/ft2	
B-102	10	Silty Sand (SM)	4.4E-4	9.3	29
B-102	50	Sand (SP), poorly graded	1.0E-2	212.0	4
B-102	70	Sand with Silt (SP-SM)	9.4E-3	199.3	9
B-103	10	Silty Sand (SM)	3.6E-3	7.6	15
B-103	20	Sand (SP), poorly graded	1.2E-2	254.5	3
B-103	70	Silty Sand (SM)	5.2E-4	11.0	15

A general comparison of the CPT profiles (see Plate 4 -- Cross Section B–B', boring B-102 and CPT C-107) to the laboratory determined permeability values suggest the higher values can be assigned to the zones of lower friction ratios (light yellow) while the lower permeability values are representative of the higher friction ratios and the zones delineated by the olive green color. The lower permeability values may reflect the previously mentioned fines content and weathering process as well as increases in density of the dune sand deposits with increasing depth. Importantly, the lower permeability values and inferred CPT correlations represent potential restrictions to the vertical flow of groundwater in the unsaturated zone to the regional water table.

GEOLOGIC HAZARDS

SEISMICITY

The parcel is within a seismically active region of Central California that will likely be subjected to strong ground motion resulting from moderate to large earthquakes in the future. We understand no structures are planned for the development of the site. Impacts to the new percolation ponds and site could consist of instability of the bluff or percolation pond slopes in response to seismic loads, liquefaction of saturated soil below the pond, and settlement of the ground surface and berms used to confine the ponds, if needed. Earthquake (pseudostatic) forces were considered in our slope stability analyses of the existing bluff slope, discussed in the following section of this report.

FAULTING

Three faults are mapped or are inferred in the site vicinity: the inferred trace of the Wilmar Avenue fault mapped approximately 4,500 northeast of the site, the inferred trace of the Oceano fault, mapped approximately 6,000 southwest of the site, and the inferred trace of the Santa Maria River fault, tentatively mapped just east of Orchard Road. The Santa Maria River fault adopts the west-northwest trend and displays a vertical offset of about 180 to 250 feet, according to interpretations by DWR (2002) and Luhdorff and Scalmanini (2000). The fault may act as a partial hydraulic barrier to groundwater flow in the area. The Wilmar Avenue fault is mapped by Anderson and LaForge (1985) as merging with the Santa Maria River fault upstream near the Santa Maria River Bridge on Highway 101. The fault locations are interpreted from inferred offsets in well logs and steps in the Franciscan bedrock from geophysical data. The California Geologic Survey (CGS, 2002) groups the Oceano, Wilmar Avenue and several other faults as the San Luis Range fault system, which they consider to be potentially active. No known active faults cross the site and the site is not located within an Alquist-Priolo zone.

The mapped fault locations are poorly constrained and lack clear evidence of displacement of Holocene dune sands or Quaternary alluvium (Asquith 1997, Manson 1985) in the project vicinity. Within the Santa Maria Valley, the inferred locations of the faults are concealed by relatively deep alluvium. It is our opinion that the presence of the faults does not pose a significant fault rupture hazard to the project. However, significant ground motion could impact the site if an earthquake were to occur on the San Luis Range fault system within the life of the project.

LIQUEFACTION

The potential for liquefaction to impact the site was evaluated using the CPT data and estimated ground motion parameters for the design earthquake. Liquefaction is a loss of soil strength due to a rapid increase in soil pore water pressures due to cyclic loading during a seismic event. Liquefaction commonly occurs in loose to medium dense sandy soil that is below the groundwater table at the time of an earthquake. The potential and severity of liquefaction will depend on the intensity and duration of the strong ground motion.

A ground motion of approximately 0.52g was considered for the liquefaction analysis in accordance with the California Building Code (CBC, 2007). An earthquake magnitude of 7.2 was used in the evaluation, corresponding to the maximum magnitude of the San Luis Range (S. Margin) fault (CGS, 2002), located approximately 1.1 miles from the site (Blake, 2000). The analysis was performed using procedures described in Moss et al. (2003), 1997 NCEER guidelines (Youd and Idriss, 2001) and Boulanger and Idriss (2004) for performing liquefaction analyses using CPT results. The results of the liquefaction analysis are presented in Appendix D.

The soil encountered at the site generally consisted of loose to very dense dune sand. Based on the analysis, near-surface loose to medium dense sand encountered to depths ranging from about 2 to 15 feet below the ground surface are potentially liquefiable under the design earthquake. Liquefaction would only occur if the soil was saturated by the pond during an earthquake. The potentially liquefiable soil is near the surface, and could be removed by grading, if needed. The design of the ponds may need to consider the presence of the potentially liquefiable near-surface soil below the pond slopes so that proper site preparation and grading can be performed to maintain slope stability.

Interbedded layers of medium dense dune sand were encountered below the near-surface unit at various depths between about 15 and 40 feet in CPT-106. The interbedded layers of soil had estimated factors of safety near 1, and although theoretically prone to liquefaction, are likely dense enough that the deeper soils would not be prone to significant strength loss or settlement based on the preliminary analysis.

Typically, liquefaction hazards are mitigated by removal-and-replacement or in situ densification of liquefiable soils. However, note that these mitigation measures may affect the percolation capacity of the site. The grading could likely be limited to supporting the perimeter berm or slopes of the pond, if needed, to maintain slope stability of the slopes considering liquefaction.

SLOPE STABILITY

The stability of the existing bluff that forms the southern boundary was evaluated as part of this study. The slope of the existing bluff is relatively steep, with inclinations ranging from approximately 2h:1v to 1.2h:1v. The purpose of the analyses was to evaluate the stability of the bluff under existing slope conditions, and to evaluate whether or not the stability of the slope could be impacted if the site is developed with the new percolation ponds. The analyses also considered the stability of an existing near-vertical slope face within the eroded drainage canyon to back-calculate strength parameters for comparison to strength parameters estimated from laboratory tests. Output and results from the slope stability analyses are presented in Appendix C.

Slope Geometry

The location of the profile selected for the analyses is about 600 feet northwest of the eroded drainage canyon, as shown on Plate 2. We estimated the bluff slope profile with an approximately 1.2:1 (horizontal to vertical) inclination based on topographic mapping by the United States Geological Survey (USGS, 1965) and our own field measurements at the site. We estimated the drainage canyon slope profile based on field observations, because the geometry of the existing cut slope includes: inclinations varying from near-vertical to 1.5h:1v; colluvium deposited along the base of the slope face; undermined and eroded features; and a near-vertical head of the slope. Due to the scale of the topographic map, these features were not accurately identified on the map. The slopes were evaluated with respect to the stability criteria discussed below.

Slope Stability Criteria

Slope stability results were evaluated relative to criteria presented in the State's *Guidelines for Evaluating and Mitigating Seismic Hazards* (California Division of Mines and Geology, 1997) and San Luis Obispo County (2005). For the purpose of evaluating analytical results, a slope is considered stable when the estimated factor of safety is at least 1.5 under static loading conditions, and at least 1.1 under pseudostatic (earthquake) loading conditions when using a horizontal pseudostatic coefficient of 0.15. A factor of safety 1.0 represents the theoretical boundary below which a slope is no longer stable and experiences failure. Factors of safety greater than 1.0 are theoretically stable; however, a factor of safety of at least 1.5 are typically used to define stable slope conditions in practice to help account for uncertainties associated with characterizing subsurface conditions and limitations associated with the geotechnical analyses used to evaluate slope stability.

Analysis Methods

The slope stability analyses were performed using the computer program GSTABL7 (Gregory, 2001). GSTABL7 was used with STEDwin Version 3.07 (Van Aller 2002) to estimate factors of safety for slope stability under the static and pseudostatic loading conditions being evaluated. GSTABL7 requires the user to input the surface and subsurface profile boundaries; soil properties including unit weight (γ), friction angle (ϕ) and cohesion (c); groundwater levels; and the analysis method to be used. The soil properties and conditions used for our analyses are presented in Appendix C. Slope stability analyses were performed using the modified Bishop method to estimate factors of safety for circular failure surfaces. A key to the results of our slope stability analyses is presented on Plate C-1 in Appendix C.

Selection of Shear Strength Parameters

Effective shear strength parameters (ϕ and c) were selected for slope stability analyses based on laboratory direct shear tests and back-calculation analysis. Laboratory tests were performed on driven ring samples obtained from the field exploration program. The strength parameters used for the analyses are presented on the slope stability plots included in Appendix C. The shear strength of dune sand deposits was estimated for the peak strength condition obtained from direct shear tests results (Blake et al 2002, CDMG 1997). As part of our analysis, we back-calculated the strength parameters necessary to maintain the stability (factor of safety equal to 1.0) of the upper sand unit exposed in an approximately 40-foot high, near-vertical slope face within the drainage canyon. The purpose of evaluating the near-vertical slope was to provide a comparison of strength parameters (mainly cohesion) with those estimated from laboratory tests.

Based on direct shear laboratory tests, a friction angle of 35 and 36 degrees was used to characterize the strength of the upper and lower dune sand units, respectively. To further characterize the strength of the upper dune sand unit, cohesion values of 150 and 400 psf were selected using half the estimated cohesion obtained from direct shear test results and back-calculation, respectively. A cohesion value of 300 psf for the lower dune sand unit was selected based on half the estimated cohesion obtained from direct shear test results.

Groundwater Considerations

Groundwater was encountered in B-101, B-102 and B-103 at el. 143, 190 and 212 feet, respectively. A groundwater elevation of 143 feet, about 30 feet below the base of the bluff, was used in the slope stability analyses. The location of the groundwater table is shown on the plots of the slope stability results in Appendix C.

Summary of Slope Stability Results

Based on visual observation of the bluff slope, the bluff face appears to have weathered to its angle of repose as the Santa Maria River eroded away the sandy material at the toe of the slope. The slope is locally rilled and eroded. We observed evidence of relatively shallow (less than 2 to 3 feet) slumps and eroded areas within the upper approximately 20 feet of the bluff. Some of the features appear relatively recent, and some have been stabilized by vegetation. These geomorphic features are characteristic of past surficial instability. Sandy materials on the subject slope may become prone to surficial instability due to weathering and exposure of the near-surface soils that can destroy or weaken the cohesive properties of these materials. However, we did not observe evidence of deep-seated or global slope instability of the existing bluff that would suggest that there is instability that extends significantly beyond the tops of the existing slopes.

For the initial stability analysis of the bluff slope, strength parameters from direct shear tests were used to estimate factors of safety for slope stability. The estimated factor of safety from the initial analyses ranges from about 1.2 and 0.94 for static and pseudostatic conditions, respectively. The factors of safety are below the minimum 1.5 and 1.1 factors of safety used to define stable slope conditions for static and pseudostatic loading conditions. The existing bluff slope, at an inclination of about 1.2:1, is therefore considered potentially unstable relative to the minimum factors of safety used to define stable slope conditions. While evidence of deep-seated slope instability was not observed along the slope, the slope would still be considered potentially unstable or to have its stability compromised more easily than a slope that is considered stable. .

Further slope stability analysis of the bluff slope was performed to check the shear strength parameters used in the analysis compared to strength parameters estimated from the back-calculation analysis of the existing near-vertical slope within the drainage canyon. The cohesion estimated from the back calculation was about 400 psf, more than double that estimated from the direct shear tests. The estimated factors of safety for the existing bluff slope using the back-calculated cohesion is about 1.3 and 1.0 for static and pseudostatic conditions, respectively, suggesting that the stability of the slope may be better than initially estimated under static and pseudostatic loading conditions. The laboratory tests do suggest however, that the cohesive strength of the soil may be vulnerable to weakening when the soil is saturated.

The estimated factors of safety for the existing slope are independent of whether or not the percolation ponds are constructed on the site. The estimated factors of safety for static loading is below the minimum 1.5 used to define slope stability; however as discussed above, the visual reconnaissance of the bluff did not reveal that there is active slope instability of the overall bluff. Pseudostatic analysis suggests that the existing slope maybe prone to displacement in response to strong ground motion, such as may occur during a relatively strong earthquake on one of the local faults.

Table 3 – Summary of Slope Stability Analyses

Slope Conditions	Estimated Factor of Safety: Static Condition	Estimated Factor of Safety: Pseudostatic Condition
Bluff Slope, cohesion value of upper unit based on direct shear laboratory tests	1.18	0.94
Bluff Slope, cohesion value of upper unit based on back-calculation analysis	1.27	1.00
Near-Vertical Drainage Canyon Slope (back-calculation used to estimate cohesion of dune sand corresponding to a static factor of safety of 1).	1.00	–

The stability of the bluff, and slopes in general, will be influenced by the potential for groundwater to daylight on the existing slopes. Springs or elevated groundwater daylighting on the slope can result in piping and erosion of the slope, loss of cohesion, and further instability of the slope. Seepage forces from groundwater in association with surface runoff and erosion likely contributed to the formation of the existing drainage canyon that intersects the buff face. The stability of the existing bluff slope is likely to not be impacted by the construction of new percolation ponds provided that groundwater does not daylight at the base of the bluff or on the slope surface. The ponds would need to be setback sufficiently to prevent daylighting of groundwater at the base of the bluff or on the slope surface.

CONCLUSIONS

HYDROGEOLOGY

The CPT and boring log data compiled as part of this study indicate the presence of low permeability layers at variable depths in the unsaturated zone, older dune sand deposits. The low permeability layers are most pronounced at depths below about 75 feet and within the southerly parts of the Pasquini parcel. The continuity of these low permeability layers is not exactly known but are sufficiently represented in the CPT soundings to create a concern relative to the ultimate fate of wastewater discharged in percolation ponds on the parcel. Water level data obtained from the three hollow stem auger borings drilled on the parcel suggest that discharge of wastewater within the northerly third of the Pasquini parcel (i.e., adjacent to and immediately south of Orchard Road within an approximate 35-acre area) would be at a sufficient distance from the bluff of the floodplain of the Santa Maria River, and would not daylight on the slope face. This conclusion is however subject to performing supplemental field work on the northerly portion of the parcel.

We understand that the Nipomo CSD has an ultimate need to dispose of up to an additional 0.63 million gallons per day (daily average) of treated wastewater in supplemental percolation basins. This assumption is based on the ability of the existing Southland WWTF percolation ponds to accommodate about 0.57 MGD (Fugro Project Memorandum dated June

30, 2008) and various assumptions of future Nipomo CSD build out wastewater flow volumes. The roughly square shaped northerly third of the Pasquini parcel is about 35 acres in size. Assuming 80 percent of this area could be developed to percolation basins and that the soils (subject to confirmation percolation testing) could be expected to percolate up to 10 gallons per day per square foot (gpd/ft2; considered a reasonable assumption given the percolation characteristics of unweathered dune sand deposits as evident on the CPT soundings). Given an infiltration rate of up to 10 gpd/ft2, the 35-acre gross area should be able to accommodate about 1.2 MGD of treated wastewater. Further field work however is recommended to support this conclusion as in the following sections of this report.

SLOPE INSTABILITY

The existing bluff slope that borders the project site to the south is potentially unstable relative to the minimum factors of safety used to define stable slope conditions. The potentially unstable state of the existing bluff slope is unlikely to be adversely impacted by the proposed project, provided groundwater elevations remain below the base of the bluff and the proposed percolation ponds are adequately set back from the top of the bluff. The bluff slope is generally prone to surficial instability due to surface run-off and weathering associated with its steep inclination, lack of vegetation, and grazing livestock. To help maintain stability of the bluff slope, the percolation ponds should be designed to maintain groundwater levels below the base of the existing bluff, and such that grading of the site does not modify the drainage conditions that would allow uncontrolled surface water to run over the slope.

DRAINAGE AND EROSION CONSIDERATIONS

It is our opinion that the stability of the existing bluff slope is predominantly influenced by erosion that has resulted from groundwater daylighting on the slope during high groundwater periods and storm events. In addition, subsurface seepage and piping at the toe of the slope may erode and destabilize the bluff. Percolation ponds are likely to be setback from the top of the bluff, and should not require grading near the top of the bluff. Surface drainage should generally be controlled such that surface water does not run toward or over the bluff slope. If drainage must be directed towards the bluff, drainage water should be collected in lined swales or ditches that will direct surface water to controlled drainage structures. Concentrated flows and runoff should not be permitted to discharge onto the bluff slope. Down drains, solid pipes, or lined ditches should be provided to carry water to the base of the slope. Energy dissipation and erosion control devices should be provided at the outlet of drainage pipes and in areas of concentrated flow and runoff to reduce the potential for erosion.

OPERATIONS AND MAINTENANCE

The dune sand deposits at the site will be vulnerable to erosion where exposed or disturbed by grading. Site conditions, particularly on sloping ground, are dynamic and should be considered in the operation and maintenance of the facility. Ongoing erosion, changes in drainage, and landsliding are some of the factors that should be reviewed on an ongoing basis.

The top of the bluff slope is comprised of dune sand and colluvial deposits. In our opinion, localized instability could occur as a result of periods of storm runoff or precipitation, ongoing weathering of the slope, earthquakes or other factors. Ongoing maintenance of slopes should be provided to help maintain the slope, reduce the potential for raveling or erosion along the face of the slope, and evaluate whether or not additional grading of the slope is needed to maintain the stability of the slope.

RECOMMENDATIONS

Based on the findings and conclusions of this report, should the District desire to move forward with additional investigation of the Pasquini parcel for potential percolation pond development, we recommend the following field work be performed within the northerly one-third, roughly square-shaped 35 acres adjacent to Orchard Road.

- To assess the percolation capacity of surficial soils, a series of conventional percolation tests should be performed in accordance with Uniform Plumbing Code standards or County of San Luis Obispo Health Department accepted methods. Given the gently rolling topography of the area, the percolation tests should be performed at the anticipated grade (elevation) of the base of the percolation basins. It will be necessary to develop a conceptual grading plan for percolation basins in the area which will provide a rough estimate of the anticipated elevation of the base of the percolation basins. Based on the approximate 35-acre gross area under consideration, we recommend a percolation test for every 2 acres of actual percolation basin area, or about 12 such tests.

- Consideration should also be given to constructing a prototype percolation pond to allow for larger scale testing of the percolation capacity of the soil. A small percolation basin, perhaps 10- to 20-foot square could be installed at the site. A metered supply of water, possibly from a local hydrant, would be needed to charge the basin and estimate the percolation capacity of the soil. The basin would be flooded with water to maintain a constant head above the bottom of the basin, and the test would be continued until a stabilized infiltration capacity for the basin could be obtained (typically up to about 30 days). Casings would be installed in drilled holes, backfilled with native soils, to allow for hydro-probe monitoring during testing. The hydro-probe is a nuclear device that can be used to estimate the degree of saturation in the soil versus depth. The hydro-probe could be particularly useful to evaluate whether or not the siltier soils encountered at various depth will cause any horizontal deflection of the infiltrated water.

- Critical to the success of the supplemental percolation basin facility is the ability of the wastewater to percolate and flow more or less vertically through the relatively deep unsaturated zone and merge with the water table of the deeper aquifer at an elevation below the base of the bluff (some 2000 feet to the southwest). The success of this is dependent on a better definition of the depth and continuity of any low permeability layers under the suggested 35-acre portion of the Pasquini parcel.

We recommend the drilling and construction of four groundwater monitoring wells (possibly completed in two different depth zones) in the proposed 35-acre area. The monitoring wells would be drilled under permit with the County of San Luis Obispo using the rotary wash method, geophysically electric logged, and appropriately completed in either an upper and/or deep aquifer zone depending on interpretation of the geophysical survey. The completed monitoring wells would be used to obtain water level data and background water quality data for the area. Ultimately, the monitoring wells could be used as part of Regional Water Quality Control Board points of compliance associated with Waste Discharge Requirements (WDRs) that would be developed for use of the parcel.

- Based on the data obtained from the above field work, a numerical groundwater flow model could be constructed for the area to better predict the fate and transport of wastewater discharged into percolation basins. The model would essentially be an expansion of the numerical model developed to assess the percolation capacity of the Southland WWTF basins recently performed by Fugro. The need for and attributes of the numerical model would depend on the data obtained from the previously described work actions.

TO: COMMITTEE MEMBERS

FROM: BRUCE BUEL

DATE: JULY 30, 2008

AGENDA ITEM
4
AUG. 4, 2008

DISCUSS PROCESS TO FINALIZE MASTER PLAN & PHASE UPGRADES

ITEM

Discuss process to finalize Master Plan & Phase Upgrades [Forward Recommendation to Board].

BACKGROUND

Boyle Engineering is under contract with NCSD to finalize the Southland Wastewater Treatment Facility Master Plan. Since publication of the Draft Master Plan in February 2007, the District has attempted to find a suitable disposal option to augment the capacity available at the existing percolation ponds south of the WWTF and to define the regulatory/legal constraints related to development of these options. The RWQCB is anxious for NCSD to complete the upgrades to the collection system and the treatment works so that the District's discharge satisfies the requirements of our Discharge Order. Likewise, NCSD staff believes that the collection system and treatment upgrades are necessary for operations and to avoid fines. The Board has instituted the necessary rate increases to pay for the debt service of the projected $12 million cost of the collection system and treatment works upgrades. However, the 2007 Draft Master Plan did not propose or cost out a disposal solution; the Town Sewer rates imposed by the Board do not fund a disposal solution; and it is not clear that there is a preferred disposal option or combination of disposal options available without further study.

Staff's proposal would be to phase the project so that the funded collection system and treatment upgrades proceed immediately and the disposal options follow in a second phase.

In regards to disposal, attached is a listing of ten disposal options that could be considered along with a proposal from Boyle Engineering to evaluate these options for inclusion into the Final Master Plan. Staff believes that the research proposed by Boyle should be done now so that the information can be presented in the Phase I EIR as co-equal alternatives. Staff believes that disposal can be addressed at a programmatic level in the EIR so that all the options are addressed and the District can avoid allegations of "Piece-mealing".

RECOMMENDATION

Staff recommends that the Committee discuss staff's proposed phasing concept; the listing of disposal options; and the Boyle Proposal.

ATTACHMENTS

- Listing of potential disposal options
- Boyle Proposal

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Nipomo CSD Southland Wastewater Treatment Facility
Program Support – Percolation Site Planning

Percolation Suitability and Disposal Options

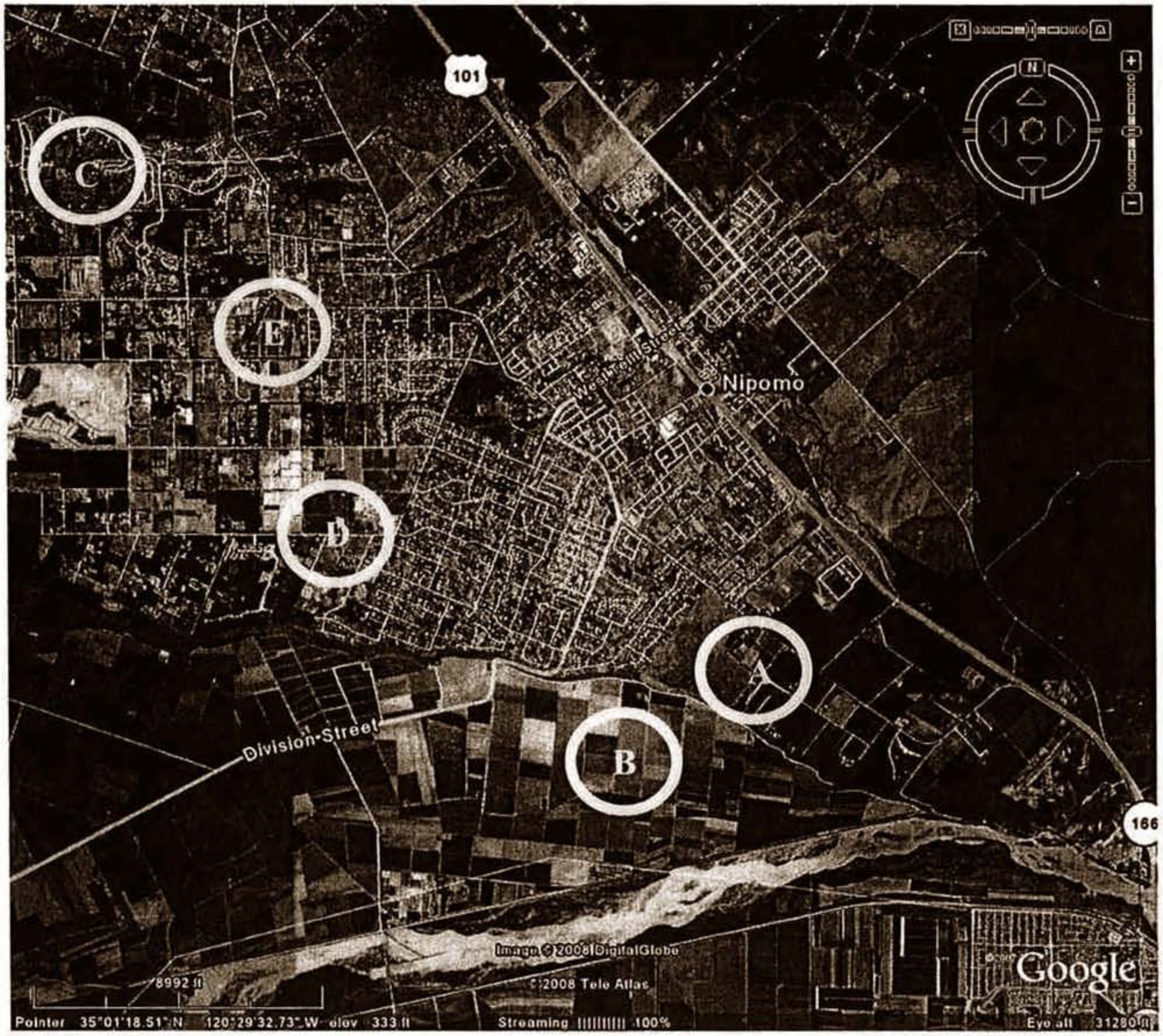
Option	Suitability for Percolation	Expected Treatment	Critical Information Needs	Relative Cost
Existing	Poor. • Top of mound is rising. • Flows laterally to Nipomo Creek. • Impermeable layer.	• Upgrade to remove nitrates. • Pump mound down to reduce flow to creek.	• Where to perc excess? • What pumping rate and schedule will stabilize mound and reduce flow to Nipomo Creek to acceptable level?	
Option1A: Pump mound and perc at Site A* with ponds	unknown	Upgrade to remove nitrates.	• Suitability for percolation? • Willing owner? • Land cost?	Lowest = 1
Option 2A: Pump mound and perc at Site A* with subsurface system	unknown	Upgrade to remove nitrates.	• Suitability for percolation? • Willing owner? • Land cost?	2
Option1B: Pump mound and perc at Site B* with ponds	unknown	Upgrade to remove nitrates.	• Suitability for percolation? • Willing owner? • Land cost? • Environmental Concerns? • Compatible with Santa Maria Groundwater Decision?	3
Option 4: Recycle water to agricultural users south of Southland	Unknown	Meet Title 22 requirements, tertiary with disinfection if sites are uncontrolled	• User survey • Wet weather strategy • Possible hybrid approach?	4
Option 5: Pump mound and perc at Mesa Road site (Percolation ponds or subsurface infiltration)	Unknown	Upgrade to remove nitrates	• Suitability for percolation? • Willing owner? • Public outreach strategy? • Land cost?	5

Nipomo CSD Southland Wastewater Treatment Facility
Program Support – Percolation Site Planning

Option	Suitability for Percolation	Expected Treatment	Critical Information Needs	Relative Cost
Option 3C: Recycle water to landscape users, such as Woodlands, Nipomo Park, Blacklake (Site C) and other developments	Presumed good	Meet Title 22 requirements, tertiary with disinfection if sites are uncontrolled	• User survey • Wet weather strategy • Possible hybrid approach?	Highest = 6
Option 6: Perforate the Aquitard below Southland	Underlying material assumed good. Perforating aquitard may present challenges.	Meet Title 22 requirements and draft criteria for groundwater injection	• Water quality and movement in lower aquifer. • Aquifer hydraulic characteristics. • Technical feasibility.	Unknown. Depends on perforation technique.
Option 7: Irrigate on Highway 101 ROW	n/a	Meet Disinfected Secondary-23 Recycled Water requirements.	• CalTrans agreement	Unknown. Depends on cost sharing of vegetation and irrigation maintenance.
Option 8: Pump mound and perc at Site D* with ponds	unknown	Upgrade to remove nitrates.	• Suitability for percolation? • Willing owner? • Land cost?	3
Option 9: Pump mound and perc at Site E* with ponds	unknown	Upgrade to remove nitrates.	• Suitability for percolation? • Willing owner? • Land cost?	3

* See next page for general location of potential disposal sites.

Potential Disposal Sites



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Bruce Buel
General Manager
NIPOMO COMMUNITY SERVICES DISTRICT
148 S. Wilson
Nipomo, CA 93444

June 10, 2008

Southland Disposal Planning Assistance – PROPOSAL

Dear Bruce,

Recent guidance from the Regional Board (4/29/08) regarding probable discharge requirements for the Southland Wastewater Treatment Facility indicates that alternative disposal options will need to be investigated. It is anticipated that the District will move ahead with a programmatic EIR for developing a new disposal site (or sites) combined with a project-level EIR for the upgrades described in the Southland WWTF Master Plan. Before the programmatic EIR can be developed, potential disposal options must be characterized to a level where impacts can be evaluated.

Determining the disposal sites and additional treatment processes is beyond the scope of our existing work order (#011-07) for *Engineering Support for Southland WWTF Management Program*. The Engineering Support project was initiated in June, 2007 and was expected to continue only six months,

> "…until the District procures an engineering design firm (anticipated to begin design in Fall, 2007)." – Scope Letter 5/25/2007

As part of this work Boyle agreed to:

> "… revise the draft Southland Wastewater Facility Master Plan, based on results from the hydrogeologic investigation and determination of a wastewater recharge or reuse strategy." – Scope Letter 5/25/2007

Note that in developing a budget for this work we assumed that a single wastewater reuse or recharge strategy would be selected by the Board sometime in Fall, 2007. In the meantime, the time involved and the scope of investigations has increased beyond the original assumptions:

- After an initial screening, the proposed disposal sites were rejected by the NCSD Board of Directors on January 23, 2008.
- The number and scope of hydrologic investigations has increased. The Phase 1 investigations were completed in July 2007. Since then, a Phase 2 investigation has been initiated, and continues to the present time.

Therefore, at your request, Boyle Engineering is pleased to submit a proposal to assist the District in developing a disposal plan for the Southland Wastewater Treatment Facility. The proposed work is described below:

Task 1 – Board Meeting Participation and Support

Boyle will participate in Board and Committee meetings (2 total) and provide brief, written status reports to the General Manager prior to the Board meetings.

Task 2 – Prediction of Water Quality from beneath Southland Percolation Basins

Boyle will review existing water quality information, recommend additional water quality sampling and analysis by others, compile pertinent water quality data, estimate likely ranges of key water quality parameters (total dissolved solids, nitrate, total nitrogen, pathogens) in the perched effluent mound under the Southland Wastewater Treatment Facility under existing condition and the following future scenarios:

a) Treatment process upgraded as described in the Wastewater Master Plan.

b) Treatment process upgraded as in (b) above plus supplemental water from the City of Santa Maria.

c) Treatment process upgraded as in (b) above plus supplemental water from the City of Santa Maria plus full implementation of a proposed salts management program. (This salts management program is currently under development by Boyle Engineering.)

Task 3 – User Survey for Properties South of Southland WWTF

Boyle will contact up to ten (10) owners of agricultural production land south of the Southland WWTF. Boyle will query these owners regarding the willingness of using treated effluent as an irrigation source, after providing them with information regarding the range of water quality that can be expected, a summary of pertinent regulations, and a range of costs. Boyle will identify the most important issues for these growers which may prevent them from using this resource. Potential obstacles are expected to be high salts content, pH, health concerns, and public perception concerns.

Task 4 –Planning Level Description of Disposal Options

Boyle will review hydro-geologic information and models provided by District consultants, regulatory guidance from the Regional Board, wastewater quantity and quality data provided by the District, and other pertinent information. Boyle will describe at a planning level up to ten (10) disposal options for the Southland WWTF for use in a programmatic EIR. These descriptions will include treatment processes, improvements needed, capital and operational cost projections, preliminary alignments, and general locations.

Task 5 – Coordination with District Team Members

Boyle will assist with reviewing scopes of work, and deliverables, for the environmental permitting analysis and for the hydrogeologic evaluation (to be performed by others). It is assumed two (2) meetings will be conducted with each team member. In addition, Boyle will contact RWQCB staff and request their review comments.

Deliverables

The deliverable will be a letter report which summarizes the information noted above, and one presentation to the District Board or Wastewater Subcommittee.

Budget

Boyle's budget is attached. Payment will be requested on a time and materials basis, with a budget not to exceed $49,400 unless requested in writing. Payment will be based on the attached fee schedule.

We hope this proposal meets your expectations. Feel free to call either of us at 542-9840 if you have any questions or comments.

We look forward to working with you on this project.

Boyle Engineering Corporation

Michael K. Nunley, PE
Managing Engineer

Malcolm McEwen, PE
Project Manager

Encl.: Budget
Project Status Summary

Project Budget

Southland Disposal Planning Assistance | **Nipomo Community Services District**

Task Description	Personnel Hours					Budget		
	Principal	Senior I	Assistant	Clerical	Total Hours	Labor	Non-Labor Fee	Total
Task 1 - Board Meeting Participation and Support								
Board meetings (2)	2	4	2	-	8	$ 1,150	$ 92	$ 1,242
Committee meetings (2)	3	6	3	-	12	$ 1,725	$ 138	$ 1,863
Status reports (2)		4	-	1	5	$ 627	$ 50	$ 677
QC	2				2	$ 380	$ 30	$ 410
Subtotal	**7**	**14**	**5**	**1**	**27**	**$ 3,882**	**$ 311**	**$ 4,193**
Task 2 - Water Quality under Southland								
Review data and recommend additional analyses		4	8		12	$ 1,400	$ 112	$ 1,512
Summarize data and estimate water quality		8	16		24	$ 2,800	$ 224	$ 3,024
QC	2				2	$ 380	$ 30	$ 410
Subtotal	**2**	**12**	**24**	**-**	**38**	**$ 4,580**	**$ 366**	**$ 4,946**
Task 3 - User Survey								
Develop Information Packet	2	8	16	8	34	$ 3,716	$ 297	$ 4,013
Identify and Contact Owners		16	8		24	$ 3,080	$ 246	$ 3,326
Summarize concerns		4	4	2	10	$ 1,114	$ 89	$ 1,203
QC	2				2	$ 380	$ 30	$ 410
Subtotal	**4**	**28**	**28**	**10**	**70**	**$ 8,290**	**$ 663**	**$ 8,953**
Task 4 - Planning Level Description of Disposal Options								
Review pertinent regulations and site-specific information	6	10	40	-	56	$ 6,740	$ 539	$ 7,279
Plan needed improvements	6	10	40	-	56	$ 6,740	$ 539	$ 7,279
Prepare letter report-	3	10	20	5	38	$ 4,405	$ 352	$ 4,757
QC	7	-	-	-	7	$ 1,330	$ 106	$ 1,436
Subtotal	**22**	**30**	**100**	**5**	**157**	**$ 19,215**	**$ 1,537**	**$ 20,752**

Project Budget

Southland Disposal Planning Assistance **Nipomo Community Services District**

Task Description	Personnel Hours					Budget		
	Principal	Senior I	Assistant	Clerical	Total Hours	Labor	Non-Labor Fee	Total
Task 5 - Coordination with District Team Members								
Environmental / Permitting Analysis	10	20		3	33	$ 4,901	$ 392	$ 5,293
Hydrogeologic Evaluation	10	20		3	33	$ 4,901	$ 392	$ 5,293
Subtotal	**20**	**40**	**-**	**6**	**66**	**$ 9,802**	**$ 784**	**$ 10,586**
Total	**55**	**124**	**157**	**22**	**358**	**$ 45,769**	**$ 3,662**	**$ 49,431**

Amounts shown are fee.

Personnel Category	$/HR
Principal	$190.00
Senior I	$140.00
Assistant	$105.00
Clerical	$67.00

Existing work order (#011-07) for Engineering Support for Southland WWTF Management Program

Status 6-10-2008:

Task	**Work Product or Meeting**
Task 1 - Board Meeting Participation and Support	
Board meetings (4)	7/25/2007 – Update to Board 9/12/2007 - Update to Board 12/12/2007 - 1/23/2008 – Meeting where initial percolation sites rejected. 5/14/2008 – attend Board meeting
Committee meetings (4) and status reports (4)	7/23/2007 - Permit Processes 12/3/2007 – Solids Disposal Options 2/4/2008 – attend Committee meeting 5/5/2008 - RWQCB Response
Task 2 - Review of Solids Management Options	Draft memo November 30, 2007
Task 3 - Regulatory Comparison	Draft memo 7/23/2007
Task 4 - Irrigation Opportunities	Draft memo June 10, 2008
Task 5 - Consultation with RWQCB and DHS	
RWQCB meetings (2)	8/3/2007 Meet w/RWQCB 8/13/2007 – Provide description to RWQCB 2/26/2008 - Meet w/RWQCB
DHS meeting (1)	2/26/2008 - Meet w/DHS
Task 6 - Coordination with District Team Members	
Environmental / Permitting Analysis	5/19/08 – meet w/DWA
Hydrogeologic Evaluation	8/31/07 - Review 8/20/07 response from RWQCB to 8/13/07 description 12/3/2007 - attend presentation by Fugro 2/4/2008 - attend presentation by Fugro 5/19/08 – meet w/Fugro
Task 7 - Amendment of Southland WWTF Master Plan	
Amend report	
Board presentation	