ADDENDUM TO

ENVIRONMENTAL IMPACT REPORT

LION'S GATE RESERVE

LEAD AGENCY: COUNTY OF SANTA CLARA

File #4039-67-28-93
SCH #94043016

January 1997

PREFACE / INTRODUCTION

Overview of Project Modifications

This EIR Addendum has been prepared to address the changes to the Lion's Gate Reserve project that have been proposed since the time that the EIR on the project was certified by the County Board of Supervisors in August 1996. These changes to the project are briefly described below and addressed in detail in the body of this document.

1) Wastewater Treatment: Modification of the proposed wastewater collection and treatment process and treatment plant location such that all of the project-generated wastewater would be conveyed by conventional gravity sewers to a treatment plant located in the southeast portion of the site near Turlock Avenue. (There would be no individual on-site septic tanks as previously proposed.) The wastewater would receive tertiary treatment using the Sequencing Batch Reactor (SBR) process, with final treatment and denitrification provided by an adjacent constructed wetland area. (The principal difference between tertiary treatment and the secondary treatment system previously proposed for the project is that tertiary treatment provides a higher level of filtration for the removal of contaminants, heavy metals and suspended solids, and also provides a higher level of nutrient removal. Under the tertiary treatment process proposed, the treated effluent would contain nitrate concentrations of less than 2 mg/l and a coliform count of less than 2.2/100ml, while secondary treated effluent would contain nitrate levels less than 25 mg/l and a coliform count of less than 23/100ml.) The treated effluent would be stored in a dedicated pond located to the south of the treatment facility, and would be applied as irrigation water on the nearby landscaped areas along the project frontage. The previous proposal involved collection of effluent only (with solids to remain in on-site septic tanks), which would be pumped up-gradient to a conventional treatment plant where it would receive secondary treatment, and then sprayed over the nearby practice range and open space areas.

2) Flood Control: Modification of the proposed on-site flood control facilities such that a substantial portion of stormwater exceeding a flowrate of the 10-year storm would be diverted to the residential lake proposed for the southeast portion of the site, thereby significantly reducing the risk of downstream flooding during major storms including the 100-year event. During the 100-year event, approximately 400 cfs of the 800 cfs that would overspill West Branch Llagas Creek west of Coolidge/Turlock Avenues under existing conditions would be diverted to the lake, thereby reducing downstream flooding by about half. The previous proposal was to provide sufficient on-site attenuation of storm runoff such that the project would not result in any increased potential for downstream flooding relative to existing conditions. Thus, under the previous plan, the lake would have provided detention storage for approximately 65 cfs added by the project during the 100-year event, but would not have provided additional protection for the existing downstream flooding problems.

Format of CEQA Review

This document has been prepared in accordance with the requirements of the California Environmental Quality Act (CEQA) which sets forth specific requirements for the documentation of potential environmental impacts which may result from modifications made to a proposed project after an EIR on the project has been certified. Under these circumstances, Sections 15162 through 15164 of the CEQA Guidelines provide for the preparation of one of three types of documents depending on the situation. The criteria to be met for each type of document are as follows: 1) a 'Subsequent EIR' shall be prepared if the changes to the project are substantial, and will result in major revisions to the EIR, and involve a substantial increase in the severity of previously identified impacts; 2) a 'Supplement to an EIR' shall be prepared if the changes are substantial and the severity of impacts are increased, but only minor changes or revisions to the EIR are necessary; and 3) an 'Addendum to an EIR' shall be prepared if some minor changes and additions are necessary, but the conditions which would necessitate the preparation of a Supplement to an EIR are not present. In the present case, the proposed modifications may or may not be considered substantial, but the overall effect of the changes would be beneficial environmentally, and in no instance would the severity of the impact be increased, as discussed in the body of this document. In addition, the changes to the EIR required to address the proposed project modifications are minor in nature. Thus two of the required criteria for preparing a Subsequent EIR and one of the required criteria for preparing a Supplement to an EIR would not apply. Therefore, according to CEQA criteria noted above, the type of environmental document that should be prepared in this instance is an 'Addendum to an EIR.'

Organization of This Document

As an Addendum to the EIR, this document identifies revisions to the certified EIR which reflect the changes in analysis resulting from the proposed modifications to the project. In order to facilitate the reader's comprehension without having to refer back to the certified project EIR, this document contains the affected impact sections in their entirety. Thus the impact sections from the project EIR on Hydrology and Drainage, and Wastewater Treatment and Disposal, as well as their corresponding summary sections, have been included in this document. Changes to the text are indicated by ~~strikethrough~~ for deletions and <u>underline</u> for additions.

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* The contents only include sections of the Draft EIR that have been revised in this Addendum.

SUMMARY

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SUMMARY OF IMPACTS AND MITIGATIONS

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E. HYDROLOGY AND DRAINAGE

1. The project would potentially result in increased downstream flooding during the 100-year and 10-year storms.
(Potential Significant Impact)

1. The on-site lake proposed for the southern residential cluster subdivision would be designed to provide sufficient detention storage for increased peak runoff resulting from site development. In addition, a diversion structure would be constructed in the creek channel to divert a substantial portion of the flows exceeding the existing 10-year flow rates to the residential lake, which would be sized to accommodate flows from the 100-year event. ~~With this pond~~ With these facilities, the peak flow rates leaving the project site during ~~the 100-year and the 10-year storms~~ significant storm events would be substantially lower than under existing conditions.
(Less-than-Significant Impact with Mitigation)

2. Portions of the residential cluster subdivisions and the wastewater treatment facility ~~would~~ may be subject to shallow flooding (one-foot average depth) during a 100-year event, and the proposed structures could also partially obstruct this sheet flow through the site. However, the total area of the site subject to shallow flooding would be reduced by flood control improvements included in the project.
(Potential Significant Impact)

2. Potential impacts to the residential subdivisions and the wastewater treatment facility from shallow flooding would be mitigated by constructing building pads on fills raised above flood elevations. The partial obstruction of shallow overland sheet flows by the proposed development would be mitigated by balancing fills with cuts within the flood-prone areas.
(Less-than-Significant Impact with Mitigation)

Q. WASTEWATER TREATMENT AND DISPOSAL

1. The proposed project would increase the demand for wastewater treatment and disposal facilities at the site.
(Potential Significant Impact)

1. Increased wastewater from the project would be treated and disposed with new facilities to be constructed in conjunction with the project.
(Less-than-Significant Impact with Mitigation)

Q. WASTEWATER TREATMENT AND DISPOSAL (CONT'D)

2. The proposed wastewater disposal facilities may result in degradation of surface water and groundwater quality. **(Potential Significant Impact)**	2. Groundwater wells would monitor water quality up-gradient and down-gradient of the proposed spray irrigation area and the storage ponds, with corrective action taken as necessary. **(Less-than-Significant Impact with Mitigation)**
3. The use of reclaimed wastewater for ~~golf course~~ landscape irrigation, and storage of the treated effluent near the residential area ~~would~~ could expose humans to possible physical contact with the treated wastewater, resulting in a potential public health hazard. **(Potential Significant Impact)**	3. The wastewater would be treated to ~~levels deemed acceptable for disposal on golf courses,~~ tertiary levels, and would therefore be acceptable for unrestricted landscape irrigation. ~~and the areas affected would be posted to notify golfers and employees where irrigation by treated wastewater is occurring.~~ Signs would be posted within the irrigated landscape areas and at the effluent storage pond to notify residents of the presence of reclaimed water. **(Less-than-Significant Impact with Mitigation)**
4. There is a potential for overflow of the storage reservoir, resulting in a public health hazard. **(Potential Significant Impact)**	4. The wastewater storage reservoir would have sufficient capacity to accommodate high rainfall years. **(Less-than-Significant Impact with Mitigation)**
5. The wastewater treatment and disposal system could generate odors. However, since the SBR process proposed involves no odor-producing anaerobic digestion and would be entirely enclosed, no noticeable odors would be generated. ~~**(Potential Significant Impact)**~~ **(Less-than-Significant Impact)**	5. ~~Odor control would be achieved by mechanisms incorporated into the design of the pump stations and the treatment plant, and by measures to be undertaken at the effluent storage pond. **(Less-than-Significant Impact with Mitigation)**~~ No mitigation required.
6. The existing pond and proposed open water areas of the project, such as the wastewater storage pond and residential lake, have the potential to be sites for breeding of mosquitoes, which could create a nuisance and a potential public health problem. **(Potential Significant Impact)**	6. Mosquito breeding would be controlled by several methods, as appropriate for each type of water body. These methods would include the circulation of water to prevent stagnant conditions, the introduction of mosquito fish, and the application of larvacides. The specific mosquito mitigation measures would be formulated in consultation with the Department of Environmental Health Vector Control District. **(Less-than-Significant Impact with Mitigation)**

Q. WASTEWATER TREATMENT AND DISPOSAL (CONT'D)

7. The location of the treatment plant near Turlock Avenue could result in potential noise impacts to existing and proposed residences in the vicinity. However, the pumps and aerators at this treatment plant would be largely submerged and entirely enclosed within a building, thus minimizing noise. **(Less-than-Significant Impact)**	7. No mitigation required.
8. The location of the treatment plant in proximity to existing and proposed residences could expose residents to potential release of hazardous materials used in the treatment process. However, this treatment plant would not involve the use of hazardous materials. **(Less-than-Significant Impact)**	8. No mitigation required.

I. PROJECT DESCRIPTION

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B. DESCRIPTION OF THE PROPOSED PROJECT

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Wastewater Treatment and Disposal

The proposed method of wastewater treatment and disposal for the project is the use of a centralized collection and treatment operation, with spray irrigation of the treated effluent ~~onto the proposed practice range~~ over specified landscape areas. All of the wastewater from the residential lots and golf course facilities would ~~have septic tanks for the primary treatment (settlement) of solids, with untreated effluent piped to the proposed treatment facility to be located north of the driving range~~ be collected by gravity flow and conveyed to a treatment facility located in the southeastern portion of the site near Turlock Avenue. The treatment plant would provide tertiary treatment and would utilize the Sequencing Batch Reactor (SBR) process, combined with disinfection and final treatment at a constructed wetland nearby. (The principal difference between tertiary treatment and the secondary treatment system previously proposed for the project is that tertiary treatment provides a higher level of filtration for the removal of contaminants, heavy metals and suspended solids, and also provides a higher level of nutrient removal. Under the tertiary treatment process proposed, the treated effluent would contain nitrate concentrations of less than 2 mg/l and a coliform count of less than 2.2/100ml, while secondary treated effluent would contain nitrate levels less than 25 mg/l and a coliform count of less than 23/100ml.)

An effluent storage pond would be excavated ~~to the northwest of the driving range~~ just south of the treatment facility, to provide wet weather storage of the treated effluent. This pond would appear as part of the residential lakes proposed for this area, but in fact would be a separate impoundment. The treated effluent would be disposed of by spray irrigation over ~~the driving range, the chipping green area, and a 3 to 4 acre area in the adjacent permanent open space area to the west~~ the nearby landscaped areas along the site frontage (~~see Section *III. Q. Wastewater Treatment and Disposal*).~~ The treated effluent would be applied at rates matching the evapotranspiration rate of the landscape plants, and spray irrigation would ~~not occur~~ be greatly reduced during the winter months when rainfall would provide for most of the water needs. Thus there would be no leaching or runoff of effluent into the groundwater or on-site drainages. (For a detailed description of the proposed treatment and disposal facilities see Section *III. O. Wastewater Treatment and Disposal.*)

The golf course maintenance facility located at the western end of the golf course would not be connected to the centralized wastewater disposal system, but would have its own individual septic tank and leachfield.

II. CONSISTENCY WITH PLANS, POLICIES AND REGULATIONS

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D. SANTA CLARA COUNTY POLICIES AND REGULATIONS

General Plan

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Health and Safety

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Wastewater Disposal

The following General Plan policies on Wastewater Disposal are applicable to the project:

R-HS 42 All new septic systems shall be located only in areas where:

a. there is reasonable assurance that they will function effectively over a long period;
b. they can be designed to have a minimum negative impact on the environment; and
c. they will not contaminate wells, or surface and groundwater supplies.

R-HS 43 Septic systems shall not be allowed where site characteristics impede their operation, including sites with:

a. high groundwater conditions;
b. highly permeable soils where wastewater will percolate in excess of one minute per inch;
c. limited depth to bedrock; or
d. gradients in excess of 20% without appropriate studies.

R-HS 44 Alternative or specially engineered wastewater systems may be allowed for commercial or industrial uses, providing:

a. the County has approved a program which ensures that the system's long term maintenance, operating, monitoring and liability costs are provided for by the owner of the facility;
b. the proposed system has a track record of safe and effective long term operation under conditions similar to those in Santa Clara County;
c. the proposed system includes adequate measures to prevent environmental damage in the event of system failure;
d. is appropriate to the site for which it is proposed;
e. is in compliance with all the other pertinent County policies and regulations; and
f. with Regional Water Quality Control Board wastewater discharge requirements.

R-HS 45 Alternative wastewater treatment and disposal systems may be allowed for individual residential development only if:

a. a traditional septic system adequate to serve the proposed development could be constructed, if needed;
b. it can be shown that the alternative system will function more effectively than a septic tank system and be beneficial to the environment;
c. the density of the proposed residential development is consistent with the density normally allowed within that property's General Plan land use designation;

d. the proposed system has a tack record of safe and effective long term operation under conditions similar to those in Santa Clara County;
e. the proposed system is in compliance with all other pertinent County policies and regulations;
f. the system is appropriate to the site for which it is proposed;
g. the proposed system includes adequate measures to prevent environmental damage in the event of system failure, such as discharge of inadequately treated effluent to the land (e.g., surface, lakes, streams, etc.);
h. the proposed system will operate in full compliance with Regional Water Quality Control Board waste water discharge requirements; and
i. the County has approved a program which ensures that the system's long term maintenance, operating, monitoring and liability costs are provided for by the owner of the facility. Such a program may include, but is not limited to, recorded contractual obligations, permit fees or insurance policies; special permit conditions; and, performance bonds for system replacement.

R-HS 46 Alternative waste water disposal systems intended to serve two or more residences may be allowed only if:
a. they comply with all provisions of the preceding policy; and
b. there exists an appropriate public entity which has agreed to, and is financially able to, assume full responsibility for the system's long term maintenance, operating, monitoring and liability costs.

Analysis: The proposed wastewater treatment facilities conform with the above policies in all respects. If necessary, a traditional septic system could be constructed to serve the residential development. However, given the historically high nitrate levels in the Llagas Groundwater Basin, it would be beneficial to the environment to utilize the proposed alternative system here instead. The proposed Sequencing Batch Reactor (SBR) process would be particularly beneficial here since it would provide tertiary level treatment resulting in final nitrate concentrations of less than 2 mg/l. (See Section *III. Q. Wastewater Treatment and Disposal* for a detailed discussion of the proposed treatment system.)

The wastewater system proposed for the project would require the approval of the County Department of Environmental Health and the Central Coast Regional Water Quality Control Board, which would in effect implement the above policies. Therefore, the project would be consistent with the Wastewater Disposal policies of the General Plan.

III. ENVIRONMENTAL SETTING, IMPACTS AND MITIGATION MEASURES

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E. HYDROLOGY AND DRAINAGE

This discussion is mainly based on the following reports: Hydrology and Drainage - Lion's Gate Development prepared by Schaaf & Wheeler in November 1995; and the Preliminary Design Report for the Lion's Gate Reserve Master Drainage Plan prepared by Pacific Advanced Civil Engineering in November 1996. ~~The full report is included as~~ Both of these reports are contained in Appendix D of this EIR.

Environmental Setting

Area-Wide Drainage

The project site is located in the Llagas Creek watershed which drains from the eastern slopes of the Santa Cruz Mountains and the western slopes of the Mount Hamilton Range south to the Pajaro River and Monterey Bay near Watsonville. The major tributaries of Llagas Creek are Little Llagas Creek, Madrone Channel, Coralitos Creek, San Martin Creek, Church Creek, and West Branch Llagas Creek. Llagas Creek and its tributaries drain a total of approximately 105 square miles upstream of its confluence with the Pajaro River south of Gilroy.

The climate of the south Santa Clara Valley is similar to that of the San Francisco Bay Area. Summers are warm and dry while winters are mild and moderately wet. Nearly 90 percent of the annual rainfall occurs in the late fall or winter months, with January normally being the wettest. The mean annual precipitation varies within the Llagas Creek watershed from a high of over 50 inches in the Santa Cruz Mountains to a low of 14 inches on the valley floor. The basin-wide average is approximately 20 inches per year.

Stream flows in Llagas Creek are regulated by Chesbro Reservoir, which is owned and operated by the Santa Clara Valley Water District. The reservoir has a total storage capacity of approximately 8,100 acre-feet. The reservoir is operated for water supply purposes, but does provide some incidental flood control benefit due to peak flow attenuation.

The upland areas of the Llagas Creek watershed have soils developed on sedimentary rock, basic igneous rocks and serpentine rocks. The main soils are of the Los Gatos, Gaviota, Vallecitos and Haymen associations. They range in depth from shallow to deep, and are located on steep to very steep slopes. The vegetative cover includes grasses, oak, pine, brush and hardwoods. The infiltration rates of water in the upland areas is generally slow. The upland soils are classified as having a high to very high erosion potential.

The upland portions of the Llagas Creek watershed have very little development at this time, and the County General Plan calls for only limited development in the future with mostly open space. On the valley floor, most of the Llagas Creek channel and its tributaries are leveed or perched channels with channel banks higher than adjacent areas on one side or both sides of the stream channel. Therefore, overflows from the channel tend to flow away from and parallel to the channel.

Based on information from the Federal Emergency Management Agency (FEMA) Flood Insurance Study for Santa Clara County, there are extensive areas of floodplain from Llagas Creek and its tributaries. The most serious of these are within the City of Morgan Hill from West Little Llagas Creek, and in the City of Gilroy from West Branch Llagas Creek.

The Santa Clara Valley Water District and the Soil Conservation Service have completed a flood control project for the Llagas Creek watershed. The downstream reach from Bloomfield Road to the Ronan Channel has been improved to 100-year design standards, and the reach from the Ronan Channel to Route 101 has been improved to 10-year design standards. In addition, 100-year design channels have been provided in the urban areas of Morgan Hill and Gilroy. Improvements in Gilroy included diversion of West Branch Llagas Creek to the Ronan Channel, and channel improvements upstream to Day Road. The project was designed to eliminate most flooding in Gilroy south of Day Road. This project has been completed, and FEMA is in the process of changing the Flood Insurance Rate Maps for this area.

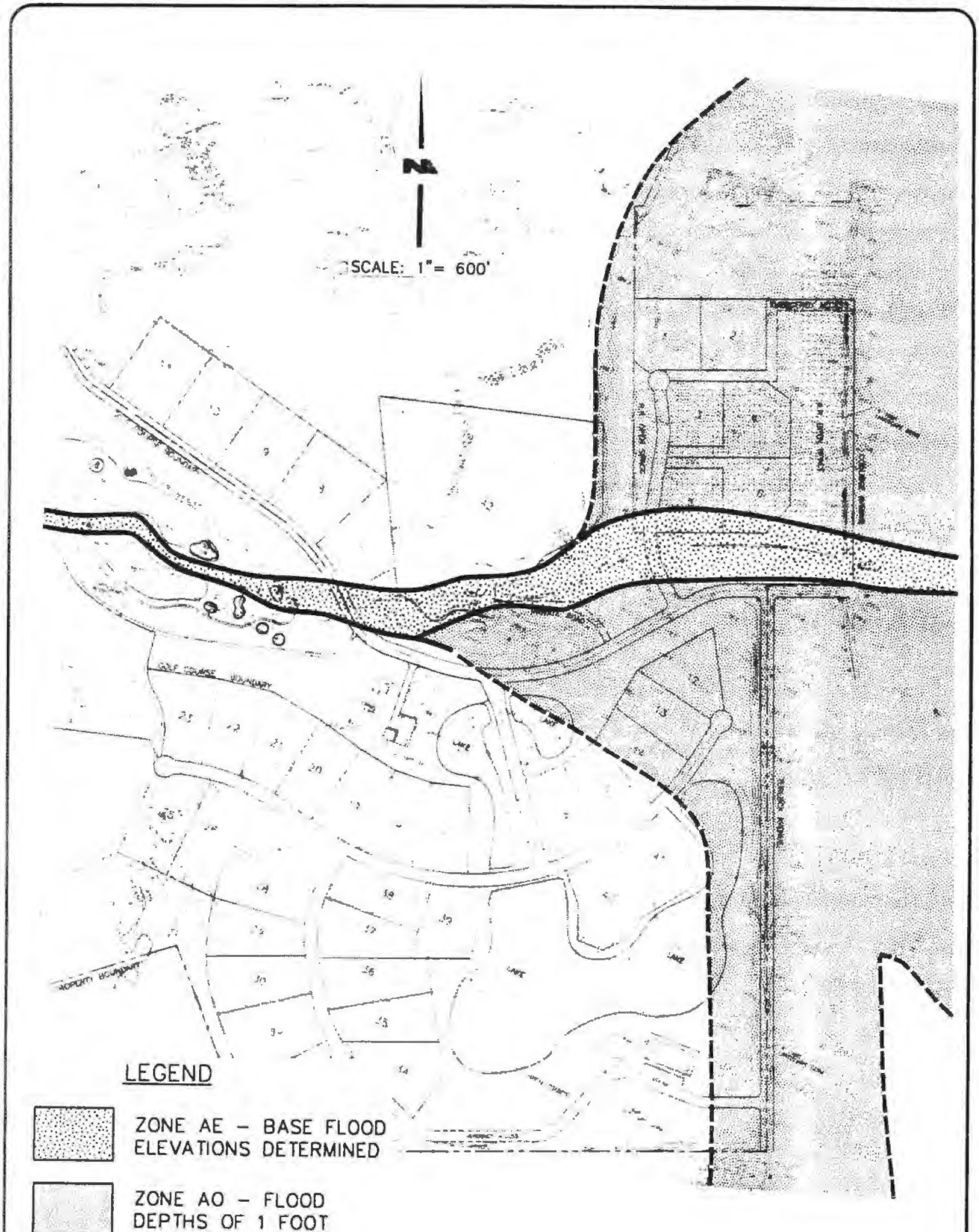
Site Drainage and Flooding Conditions

The project site drains to two separate drainages. The western portion of the site drains to the west to Hayes Creek near Watsonville Road while the majority of the site drains via the east to the West Branch Llagas Creek. A network of intermittent and ephemeral streams flow from the higher elevations on the perimeter of the central valley into the West Branch of Llagas Creek. The Creek has 8 primary tributaries, 4 of which drain the hills north of the valley and with the other 4 originating on the southern ridgeline. These tributary streams flow during winter and spring months for varying periods and are dry the remainder of the year. West Branch Llagas Creek discharges to the Ronan Channel which joins Llagas Creek near Highway 152 east of Gilroy. Hayes Creek drains to Llagas Creek near Watsonville Road, south of Morgan Hill. The are no detailed floodplain studies for Hayes Creek. The area is designated as Zone D on the Flood Insurance Rate Map. Zone D is defined as an area of undetermined flood hazard.

The existing Flood Insurance Rate Maps for West Branch Llagas Creek do not include detailed floodplain studies upstream of Golden Gate Avenue, approximately 2 miles south of Highland Avenue. The stream channel on the project site is designated as Zone A, approximate 100-year floodplain. At Turlock Avenue, the floodplain is shown as approximately 300 feet wide along the channel north of Highland Avenue.

West Branch Llagas Creek has been restudied by FEMA to update the existing Flood Insurance Rate Maps. The draft work maps are currently in the review process and are not expected to be become effective until late 1996. The SCVWD is using the revised maps as the best available information in the interim. The proposed 100-year floodplain for West Branch Llagas Creek near Highland Avenue is significantly larger on the revised maps than on the current maps. The proposed floodplain includes shallow flooding from the channel commencing at the ranch complex on the project site and including the area south of Highland Avenue, west of Turlock Avenue, and the area north of Highland Avenue west of Coolidge Avenue (see Figure 13).

The hydrology for the detailed floodplain study shows an estimated 100-year peak flow rate of 850 cubic feet per second for West Branch Llagas upstream of ~~the on-site overflows upstream of~~ Turlock Avenue. An estimated 400 cfs overflows Highland Avenue toward the south upstream of Turlock Avenue. An additional 355 cfs overflows from the channel toward the north upstream of Coolidge Avenue. The northern overflow crosses Coolidge Avenue north site and flows overland to the east and south to the West Branch Llagas Creek channel at Highland Avenue. The majority of the overflow to the south flows overland to the south and east and crosses Turlock Avenue to rejoin the West Branch Llagas Creek floodplain between Highland Avenue and Golden Gate Avenue. A portion of the overflow continues south along the west side of Turlock Avenue.



100-YEAR FLOODPLAIN

FIGURE 13

Ordinances and Regulations that Address Drainage and Flooding

County Drainage Manual: This manual contains guidelines for design and installation of drainage facilities for projects. Projects must demonstrate that drainage will be handled adequately in order to avoid drainage and flooding problems. These guidelines ensure that there are no on- or off-site drainage problems associated with a project.

Grading Ordinance: The ordinance requires that all drainage structures and devices be consistent with the adopted County Drainage Manual and its standards. It outlines disposal requirements for both on- and off-site drainage; provides for slope protection and erosion control; and the design of dikes, swales and ditches.

Land Development Regulations: The County Land Development Engineer reviews all projects to ensure no on- or off-site drainage impacts would occur as a result of the proposed project.

Zoning Ordinance: For projects requiring a use permit, Section 47-5(d) of the Zoning Ordinance ensures that adequate storm drainage exists or shall be provided as a part of the project; and that no on- or off-site drainage impacts would result from the project.

Special Flood Hazard Area Ordinance: This ordinance applies to all areas of special flood hazard (i.e., within the 100-year flood zone as established by FEMA) within the unincorporated area of Santa Clara County. No new development shall occur, or structure or improvement shall be constructed in a flood zone without compliance with this ordinance.

Significance Criteria

With respect for flooding and drainage impacts, Appendix G of the CEQA Guidelines states that a project will normally have a significant effect on the environmental if it will: "(g) Cause substantial flooding, erosion or siltation."

Impact and Mitigation

Impact 1. **The project would potentially result in increased downstream flooding during the 100-year and 10-year storms. (Potential Significant Impact)**

The proposed residential development on the project site would increase the amount of impervious area on the site and therefore increase the runoff from the site.

The cluster residential development area south of Highland Avenue would be served by storm drains which would discharge to the 20-acre lake proposed for the main subdivision area. The overflows from the lake would discharge via storm drains to West Branch Llagas Creek upstream of Coolidge Avenue. In addition, there are approximately 73 acres of hillside area upstream of this residential development area. Drainage from this area would also be collected by the storm drain system and discharge to the lake. The total area of this drainage area is approximately 240 acres.

The golf course would also be located entirely within the West Branch Llagas Creek watershed which drains to the east. There would be no development in the western portion of the site which drains to the west to Hayes Creek. The West Branch Llagas Creek watershed upstream of Turlock Avenue is approximately 1,060 acres or 1.66 square miles. The golf course development would include approximately 240 acres, the majority of which would be landscaping and turf. The upstream hillside areas would not be affected. The existing creek channel and pond would be largely maintained in their existing configurations. A new pond would be constructed west of the existing pond to serve as an irrigation water reservoir and to detain runoff from the undeveloped area upstream. The new pond would include approximately 9 acre-feet of detention storage.

To analyze potential drainage and flooding impacts, the project site was divided into the following 3 drainage areas: the cluster residential subdivision south of Highland Avenue; the area upstream of the existing pond; the area upstream of the proposed new irrigation reservoir; and the area downstream of the pond golf course reservoir. Discharge rates were estimated for the 10-year and 100-year storms for existing and project conditions.

The results of the flooding analysis show that the proposed golf course would reduce the flow from the site to West Branch Llagas Creek. The golf course would decrease the estimated peak runoff from the watershed because the proposed irrigated turf would maintain a dense layer of thatch which would act as a sponge and reduce runoff, whereas the existing unirrigated range grasses tend to be sparse, with exposed dirt between grass clumps, which does not retain as much runoff. The estimated 100-year peak flow from the golf course area would decrease from 780 cubic feet per second to 765 cubic feet per second, a decrease of 2 percent. The 10-year peak flow rate would decrease from 375 cubic feet per second to 360 cubic feet per second, a decrease of 4 percent.

The proposed golf course irrigation reservoir would also act as a detention facility to reduce the estimated peak flow rate from the western portion of the watershed. For purposes of analysis, the existing pond was assumed to be full at the start of the storm and to have minimal effect on the flood hydrograph. The proposed irrigation reservoir was assumed to be full to spillway elevation at the start of the storm, and to have a 12-foot wide spillway The estimated storage capacity of the pond is 9-acre-feet with 3 feet of flow over the spillway. The detention storage in the irrigation reservoir would reduce the estimated 100-year peak flow at the pond from 59 cubic feet per second to 39 cubic feet per second, a reduction of 20 cubic feet per second. However when routed downstream and combined with the larger watershed downstream, the detention storage reduces the peak by approximately 10 cubic feet per second. This is due to the difference in timing between the peak flow in the upper watershed and the lower portion of the watershed. The peak flow from the upper watershed is delayed by the travel time along the creek channel and arrives after the peak from the lower watershed. Therefore the peaks do not add directly. The detention storage in the upper watershed acts to increase the timing difference of the upper watershed.

The proposed golf course grading would also include local detention areas to contain runoff from the turf areas for water quality purposes. These would also act to reduce runoff from the site, particularly for small storms. The effect of these detention areas on larger storms would depend on the design and placement of each area and whether the upstream hillside areas would drain to the detention areas or directly to the creek. Therefore, the effects of potential detention storage on the golf course other than the larger pond were not considered in the hydrograph analysis.

The flooding analysis indicated that the proposed cluster residential development would result in a potential increase in the peak runoff from the development site. The 100-year peak flow from the entire watershed would increase from 236 cubic feet per second to 301 cubic feet per second, an increase of 28 percent. The 10-year peak flow rate would increase from 120 cubic feet per second to 160 cubic feet per second, an increase of 33 percent. The increase in peak runoff is due to both the increased impervious area in the development, and the more efficient drainage system which collects runoff faster than the existing overland flow conditions.

However, the cluster residential subdivision would include a proposed lake, and runoff would be drained to the lake, then released to West Branch Llagas Creek. Only the proposed equestrian center in the southeastern corner of the site would be below the lake elevation and would drain toward Turlock Avenue. There is no storm drain system along Turlock Avenue, but runoff flows along the road under existing conditions.

The residential cluster subdivision is located in a drainage area of 240 acres, which would drain to the proposed lake. Without the lake, increased peak runoff from the cluster residential subdivision would potentially increase the peak flow in West Branch Llagas Creek downstream of the project.

Mitigation 1. **The on-site lake proposed for the southern residential cluster subdivision would be designed to provide sufficient detention storage for increased peak runoff resulting from site development. In addition, a diversion structure would be constructed in the creek channel to divert a substantial portion of storm flows exceeding existing 10-year flow rates to the residential lake, which would be sized to accommodate about one-half of the flows from the 100-year event. ~~With this pond~~ With these facilities, the peak flow rates leaving the project site during ~~the 100-year and 10-year storms~~ significant storm events would be substantially lower than under existing conditions.**

The potential increased runoff from the residential area during the 100-year event would be 65 cubic feet per second, without the proposed lake. The proposed lake would have a normal water surface elevation less than the top of bank elevation of West Branch Llagas Creek at the outfall from the pond. ~~The outfall would have a flap gate to prevent high water levels in the creek from discharging back into the pond~~. The diversion structure in the creek would be designed such that a substantial portion of the flows in the creek less than the existing 10-year peak flow would pass under the structure and would not be able to enter the side channel to the lake. Flows exceeding the 10-year peak flow would be blocked by the structure and diverted to the lake for temporary storage (see Figure 13a). This would reduce the 100-year flow rate leaving the site from approximately 800 cfs under existing conditions to approximately 400 cfs. This substantial reduction in flood flows leaving the site would significantly reduce flooding problems along the West Branch of Llagas Creek downstream of the site. However, there still would be overland and downstream flooding during the 100-year event, but the extent and volume of flooding would be reduced as a result of the proposed diversion and storage. Once the storage capacity of the lake is reached, any additional flows would be prevented from entering the lake. Instead, these extreme flood flows would be allowed to overspill the creek, as would

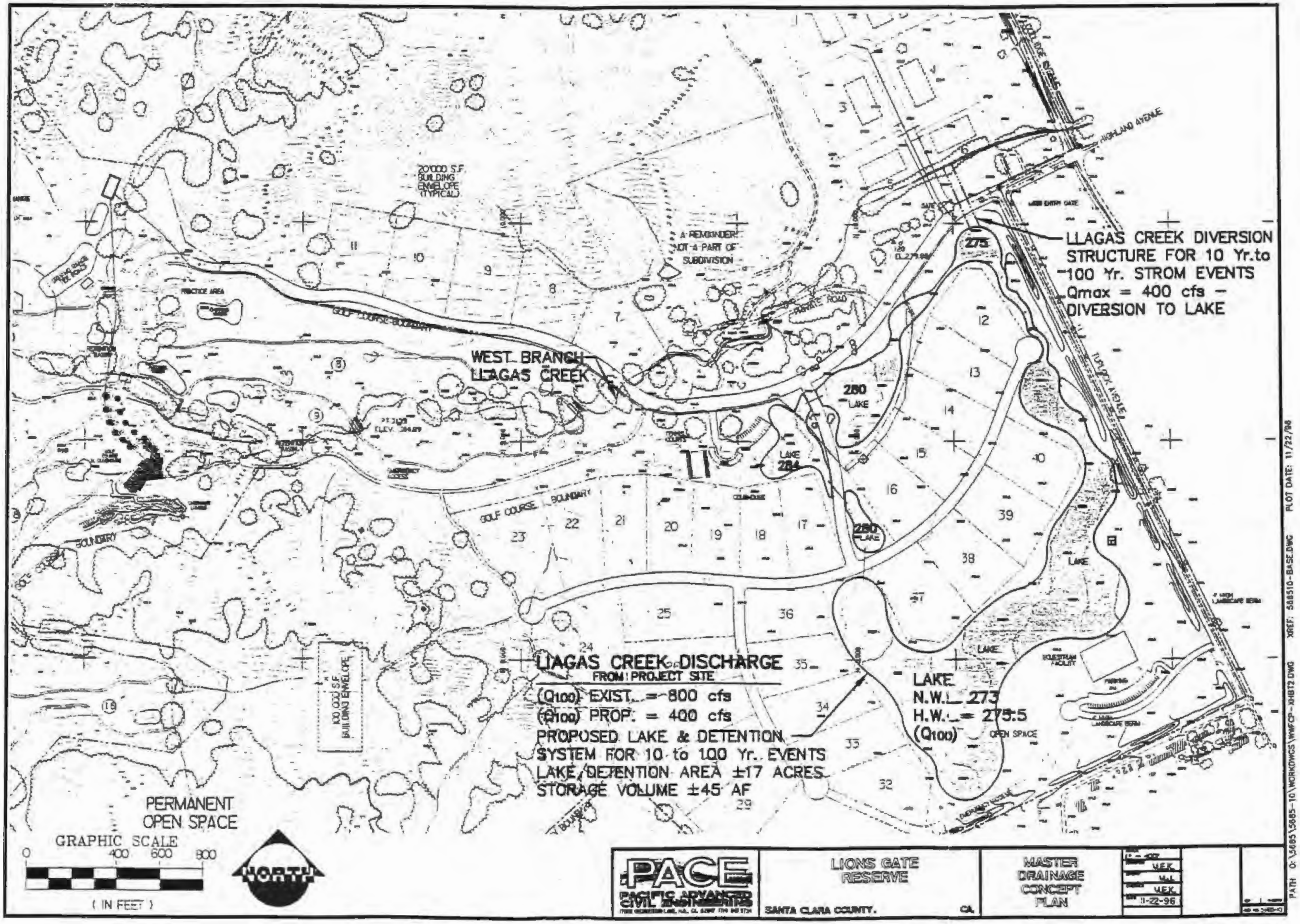

LLAGAS CREEK DIVERSION
STRUCTURE FOR 10 Yr.to
100 Yr. STROM EVENTS
Qmax = 400 cfs -
DIVERSION TO LAKE
HIGHLAND AVENUE
WEST BRANCH
LLAGAS CREEK
20,000 S.F.
BUILDING
ENVELOPE
(TYPICAL)
A REMAINDER
NOT A PART OF
SUBDIVISION
GOLF COURSE BOUNDARY
PRIVATE ROAD
LAKE
LLAGAS CREEK DISCHARGE
FROM PROJECT SITE
(Q100) EXIST. = 800 cfs
(Q100) PROP. = 400 cfs
PROPOSED LAKE & DETENTION
SYSTEM FOR 10 to 100 Yr. EVENTS
LAKE/DETENTION AREA ±17 ACRES
STORAGE VOLUME ±45 AF
LAKE
N.W.L. 273
H.W.L. = 275.5
(Q100)
OPEN SPACE
100,000 S.F.
BUILDING ENVELOPE
PERMANENT
OPEN SPACE
GRAPHIC SCALE
0
400
600
800
(IN FEET)
NORTH
PACE
PACIFIC ADVANCED
CIVIL ENGINEERING
LIONS GATE
RESERVE
SANTA CLARA COUNTY.
CA.
MASTER
DRAINAGE
CONCEPT
PLAN
11-22-96
PATH: O:\5685\5685-10\WORKDWGS\WHFCP-XHBT2.DWG XREF: 568510-BASE.DWG PLOT DATE: 11/22/96

FIGURE 13a

occur under existing conditions. The outflow from the ~~pond~~ lake would only occur when the water level in the creek is low. Therefore, the outflow from the pond would not contribute to the existing flood problems from the creek channel.

~~The proposed pond in the residential development would include an overflow spillway release for larger flood events, and an active detention storage volume between the normal water level and the spillway crest. Based on a preliminary design which includes 2 feet of active detention storage below the spillway crest and one foot of storage above the spillway crest, the proposed pond could contain approximately two thirds of the total runoff from the residential development area and the upstream hillside area during the 10 year 24 hour design storm. The pond would release approximately 30 cfs over the spillway to Turlock Avenue during the 10 year storm. This would be significantly less than the existing condition peak flow rate of 120 cfs. For smaller flood events there generally would be no spill from the pond, and runoff stored in the pond would be released to the creek after the high water levels in the creek have receded. The outlet to the creek would release approximately 20 cfs to drain the active storage volume of the pond in 24 hours after the storm.~~

~~During the 100 year 24 hour flood event, the total runoff to the lake would be approximately 125 acre feet. With no outlet release to the creek during the storm, the pond would overflow to Turlock Avenue once the active storage has filled. The estimated peak overflow would be 140 cfs for the 100 year flood. The existing peak runoff from the site during the 100 year event is estimated to be 236 cfs. Thus, although the shallow flooding along Turlock Avenue that occurs during the 100 year event under current conditions would not be eliminated, it would be substantially reduced by the flood control elements to be incorporated into the project.~~

~~The only potential adverse effect of increased peak runoff from the hillside cluster residential development site would be to increase the peak flow in West Branch Llagas Creek downstream of the project. Due to the operation of the outlet from the pond, this could only occur once the high water levels in the creek have receded and the potential for downstream flooding has passed. Therefore, there would be no increase in downstream flooding. The low flows in the creek would continue for a longer time after a storm due to the releases from the detention pond. This should not be a significant impact.~~

Since the residential lake would be sized to contain a substantial portion of the 100-year peak flow, the shallow flooding that occurs along the Turlock and Coolidge Avenue frontage areas of the site during the 100-year event would be significantly reduced (see discussion under 'Impact 2' below).

The equestrian center area in the southeast portion of the project site would not drain to the pond in the residential development area. Due to the site topography, there would be a berm between the equestrian center and the pond to contain the pond. The maximum height of the berm would be approximately 7 feet. The equestrian center would continue to drain to Turlock Avenue and ultimately to West Branch Llagas Creek. Because of the limited impervious area associated with the equestrian center, there should be no increase in runoff from the area after the project. In addition, the proposed equestrian center would include a detention pond for water quality purposes.

<u>Impact 2.</u> **Portions of the residential cluster subdivisions would be subject to shallow flooding (one foot average depth) during a 100-year event, and the proposed dwellings could also potentially obstruct this sheet flow through the site. <u>However, the total area of the site subject to shallow flooding would be reduced by flood control improvements included in the project.</u> (Potential Significant Impact)**

Based on the revisions to the existing Flood Insurance Rate Map, shown in Figure 13, the West Branch Llagas Creek would overflow to the south upstream of Turlock Avenue (i.e., at the on-site ranch complex). For the 100-year flood, approximately 400 cubic feet per second would cross through the northeastern portion of the cluster residential development, in particular through Lots 12, 13 and 14 at the northeast corner of the subdivision. This mapped overflow crosses the site and Turlock Avenue to rejoin West Branch Llagas Creek 500 to 1,000 feet downstream of Highland Avenue. The overflow is indicated as shallow flooding with an average depth of one foot, indicating that the proposed lots would be prone to flooding. In addition, grading for the residential lots in the overflow area could adversely affect the sheetflow through the area if the flow is obstructed. Similarly, grading for the access road the project and landscaping along Turlock Avenue could affect the sheetflow across the site.

The revised flood maps also show an overflow to the north from West Branch Llagas Creek upstream of Coolidge Avenue. For the 100-year flood, approximately 355 cubic feet per second would cross through proposed the rural residential development north of Highland Avenue and west of Coolidge Avenue. The overflow would flow overland to rejoin West Branch Llagas Creek at the culvert under Highland Avenue. Part of the overflow is designated as shallow flooding with an average depth of one foot, and a small sliver along the north boundary is indicated for flood depths of 0.5 to 2.5 feet. All six of the 5-acre lots are within the mapped 100-year floodplain area and thus would be prone to flooding. Also, grading for the residential lots and cul-de-sac in the floodplain could have an adverse affect on the sheetflow if flow is obstructed.

<u>Both the area subject to potential sheet flooding and the volume of flood water spilled would be substantially reduced by the flood diversion and storage facilities described under 'Mitigation 1' above. The residential lake would detain the increment of runoff generated by the project in addition to approximately 400 cfs of the peak flow during the 100-year event, which would represent approximately one-half of the overland flows overspilling the creek west of Coolidge/Turlock Avenues on the project site during the 100-year event. The precise reduction in flood plain area would be calculated in conjunction with the preparation of the Final Master Drainage Plan for the project.</u>

Mitigation 2. **Potential impacts to the residential subdivisions from shallow flooding would be mitigated by constructing building pads on fills raised above flood elevations. The potential obstruction of sheetflows by the proposed development would be mitigated by balancing fills with cuts within the flood-prone areas.**

The potential impact of placing a portion of the proposed residential development within the 100-year floodplain areas would be mitigated by balancing the grading within the 100-year floodplain. This would mean that fills required to elevate building pads above flood elevations would need to be balanced by cut areas to allow flood flows between the buildings. This procedure is generally most effective in shallow flooding areas with limited building coverage as in the proposed project. If the buildings cover a large percentage of the floodplain and are in deeper flood area, and effective balance between cut and fill would be problematic. For instance, if a building obstructs 50 percent of the floodplain in 3 feet of flood depth, the building pads would have to be elevated 3 feet, and the remainder of the floodplain would have to be excavated 3 feet to balance the cut and fill. This would lead to an elevation difference of 6 feet between the building pads and the adjacent ground. In the proposed project, the building densities would be very low with 2 to 3 acre residential lots. Thus, building elevations of 1 to 2 feet above existing grade would become 2 to 3 feet or less above the new ground elevations because of the larger area available to balance the fill.

Conclusion. **With implementation of the above mitigations as proposed in the project, the potential flooding impacts of the project would be reduced to less-than-significant levels.**

Q. WASTEWATER TREATMENT AND DISPOSAL

The following discussion is ~~largely~~ based on the following reports: Wastewater Feasibility Study for Lion's Gate Reserve prepared by Questa Engineering in December 1995; and Preliminary Design Report for the Lion's Gate Reserve Project Wastewater System prepared by Pacific Advanced Civil Engineering in December 1996. These reports are contained in Appendix N.

Environmental Setting

No public sanitary sewer system exists on the project site or in adjacent areas. The nearest public sanitary sewer system is located in the City of Morgan Hill, approximately one mile north of the project site.

The existing wastewater facilities for the on-site residences located on Highland Avenue consist of individual septic systems, which appear to be functioning normally.

Ordinances and Regulations that Address Wastewater

Sewage Disposal Ordinance: This ordinance establishes standards for the approval, installation and operation of individual, on-site sewage disposal systems (septic tank and leachfields) consistent with the appropriate California Regional Water Quality Control Board standards and basin plans. These standards are adopted so as to preclude the creation of health hazards and nuisance conditions and to protect surface and groundwater quality. Systems generating more than 2,500 gallons per day of effluent must be reviewed by the appropriate Regional Water Quality Control Board. Percolation tests are required to determine the suitability of a site for leachfields and to determine the amount of leachfields required. The systems are required to be set back a minimum distance from wells, creeks, reservoirs, springs, etc. The County Department of Environmental Health implements this Ordinance and issues the required septic tank permits.

~~County Ordinance Code - Chapter II, Article 3, Private Sewage Disposal in Lexington Basin: This ordinance sets additional requirements for the establishment of sewage disposal systems in the Lexington Basin. All lands within the basin have been mapped according to septic suitability, with varying design criteria, including minimum lot sizes, stipulated for each zone. In areas with poor septic suitability ratings, the ordinance requires installation of a second drainfield in the event of failure of the first leachfield. The ordinance requires 10 feet of separation between the leachlines and underlying groundwater table or bedrock.~~

County Zoning Ordinance: Section 47-(d) stipulates use permit findings that waste and sanitation facilities shall satisfy applicable County, state and federal requirements and that the use shall not adversely affect water quality.

Significance Criteria

With respect to wastewater, Appendix G of the CEQA Guidelines states that a project will normally have a significant effect on the environment if it will:

> "(f) Substantially degrade water quality;
> (g) Contaminate a public water supply; or
> (h) Substantially degrade or deplete groundwater resources.
> (s) Extend a sewer trunk line with capacity to serve new development."

Impacts and Mitigation

<u>**Impact 1.**</u> **The proposed project would increase the demand for wastewater treatment and disposal facilities at the site. (Potential Significant Impact)**

The proposed residences, golf course clubhouse, overnight units, swim and tennis center, and equestrian center would significantly increase the wastewater disposal requirements for the property. Although use of the golf facilities would vary seasonally and between weekdays and weekends, wastewater facilities should be designed on the basis of maximum expected daily flows, i.e., assuming 100-percent facility use. In order to calculate overall flows, the maximum wastewater treatment requirements were estimated for each project component, as described below.

<u>Single-Family Residential Units</u>: The project includes 41 custom residential lots. For central wastewater facilities, average flows from single-family residential units are typically estimated to be in the range of about 200 to 250 gallons per day (gpd) per connection. The actual flows will vary depending upon the size, occupancy and character of the residences, and the degree to which water conserving plumbing devices and practices are incorporated in the homes. The recent laws in California requiring low-flow plumbing devices (e.g., 1.6-gallon flush toilets) in new construction have had a measurable effect on wastewater flows; typical flows from new residential areas tend to average less than 200 gpd/house. (A similar project in Monterey County has experienced average daily flows of 150 to 175 gallons per dwelling over a six year period of operation.) To be conservative in planning wastewater facilities for the proposed project, an average daily unit flow estimate of 250 gpd/residence was assumed; this would adequately account for wastewater from a 4 to 5 bedroom (or more) residence on each parcel. On this basis, the total estimated flow contribution from the proposed 41 single-family residences would be 10,250 gpd (average dry weather flow).

<u>Clubhouse</u>: The clubhouse would generate wastewater from the restaurant, the employees and golfers. The flow estimates for each are as follows:

<u>Restaurant</u>: Based on a unit flow of 10 gallons per meal, the total daily flow for a maximum 200 meals would be 2,000 gpd.

<u>Golfers</u>: At a unit flow for restrooms of 5 gpd, 200 golfers would generate a total of 1,000 gpd. Assuming 10 percent of golfers would take showers, at 25 gpd, this would result in an additional 2,000 gpd for showers.

<u>Employees</u>: Up to 30 employees would work in and around the clubhouse on any given day. Based on a unit flow of 15 gpd per employee, the maximum flow would be 450 gpd.

<u>Overnight Lodging</u>: The maximum flows for the 45 overnight units were estimated on the basis of 150 gpd/unit, yielding total flows of 6,750 gpd.

<u>Swim and Tennis Center</u>: These facilities would be available for use by residents, corporate members and their guests. The facilities would include restrooms, showers and, perhaps, a small kitchen. Use of these facilities would be greatest in the summer and on weekends, and smallest in the winter and during the week. Accordingly, daily wastewater flows would fluctuate greatly. For planning purposes, the maximum daily flow is estimated to be 500 gpd.

based on 50 visitors/employees per day and a unit flow of 10 gpd/person. In addition, backwash water from the swimming pool filter and occasional draining of the spa at the proposed recreation center would go to the wastewater system and add small volumes to the overall flow (i.e., not more than a few hundred gallons per week; and it would be greater in the warm summer months than in the winter). The spa would likely be drained once or twice per year, contributing about 1,000 to 1,500 gallons of flow to the system at each draining. These flows constitute minor miscellaneous additions that are accounted for by the 1,000 gpd "contingency" contained in the preliminary wastewater flow projections (see Table 17).

Equestrian Center: This facility would have restrooms for employees and visitors. The wastewater flows from the equestrian facility are estimated to be approximately 400 gpd, based on 25 visitors/employees per day at a unit flow of 10 gpd/person, and 150 gpd for the caretaker's residence.

TABLE 17

ESTIMATED WASTEWATER FLOWS*

Activity	Number of Units	Daily Flows	Total (gpd)
Residences	41 houses	250 gpd	10,250
Golf Course Clubhouse			
• Restaurant	200 meals	10 gal/meal	2,000
• Golfers			
• Restroom	200	5 gpd	1,000
• Showers	20	25 gpd	500
• Employees	30	15 gpd	450
Overnight Units	45 rooms	150 gpd	6,750
Practice Range	50 golfers	3 gpd	150
Equestrian Center	25 visitors	10 gpd	250
Subtotal			22,000
Contingency			1,000
Total Project			**23,000**

*This does not include the wastewater flows for the golf course maintenance building (approximately 300 gpd) which would be served by an individual septic system.

The total estimated wastewater flows are summarized in Table 17. Based on the above generation rates, the total wastewater flow for the Lion's Gate project is estimated to be approximately 23,000 gpd. This includes a contingency of approximately 5 percent to account for uncertainties about the specific details of project facilities that would not be determined until the design stage. Final wastewater facility design would also need to anticipate and provide for peak flow conditions which, on a daily basis, may be in order of 25 to 30 percent higher than the average daily flow. For the proposed project this translates to a peak system flow estimate of about 30,000 gpd.

Mitigation 1. Increased wastewater from the project would be treated and disposed of with new facilities to be constructed in conjunction with the project.

The proposed method of wastewater treatment and disposal for Lion's Gate project involves a central collection, treatment and disposal system for the golf course facilities (except the maintenance facility) and all of the residential development. The various elements of this system are described below and shown in Figure 23 (Revised). <u>For a more detailed description of the treatment system and process, see Preliminary Design Report for the Lion's Gate Reserve Project Wastewater System prepared by Pacific Advanced Civil Engineering in December 1996, which is contained in Appendix N.</u>

~~Septic Tanks: Each residential lot, the clubhouse/overnight complex and the equestrian center would be provided with septic tanks where primary effluent treatment (i.e., sedimentation) would occur. The effluent from the tank would then be piped to centralized treatment and disposal facilities (described below) instead of individual leachfields.~~

~~Collection System: The collection system would consist of a network of small diameter plastic pipes. The flow from the septic tanks to the collection system would be generally by gravity, although some pumping units would be required where septic tanks are at lower elevations.~~

~~Transmission Line/Pump Stations: The collection system would consist of 2 to 4 inch diameter PVC pipe which would convey all septic tank effluent to a central treatment plant, to be located north of the practice range. The collection system would have two major branches: one branch to serve the residential units, equestrian facility and the swim and tennis center; and a second branch to serve the golf course clubhouse and overnight lodging units. Both branches would require a central pump station, located approximately as shown in Figure 23.~~

~~Treatment Facility: As noted above, primary sedimentation is to be provided by the individual on-lot septic tanks. The remaining treatment would be provided by a central treatment plant, to be located adjacent to the practice range. The treatment plant would occupy an area of about 3,000 to 4,000 square feet. The plant would consist of a fully enclosed proprietary "package" system that would produce secondary level effluent quality. The plant would include the following elements: (a) below ground, built-in-place concrete vaults for sedimentation and clarification; (b) oxidation process for secondary treatment; and (c) liquid chlorination system for disinfection.~~

~~Storage Facilities: The wastewater facilities would include short-term emergency storage and long-term wet weather storage, as described below.~~

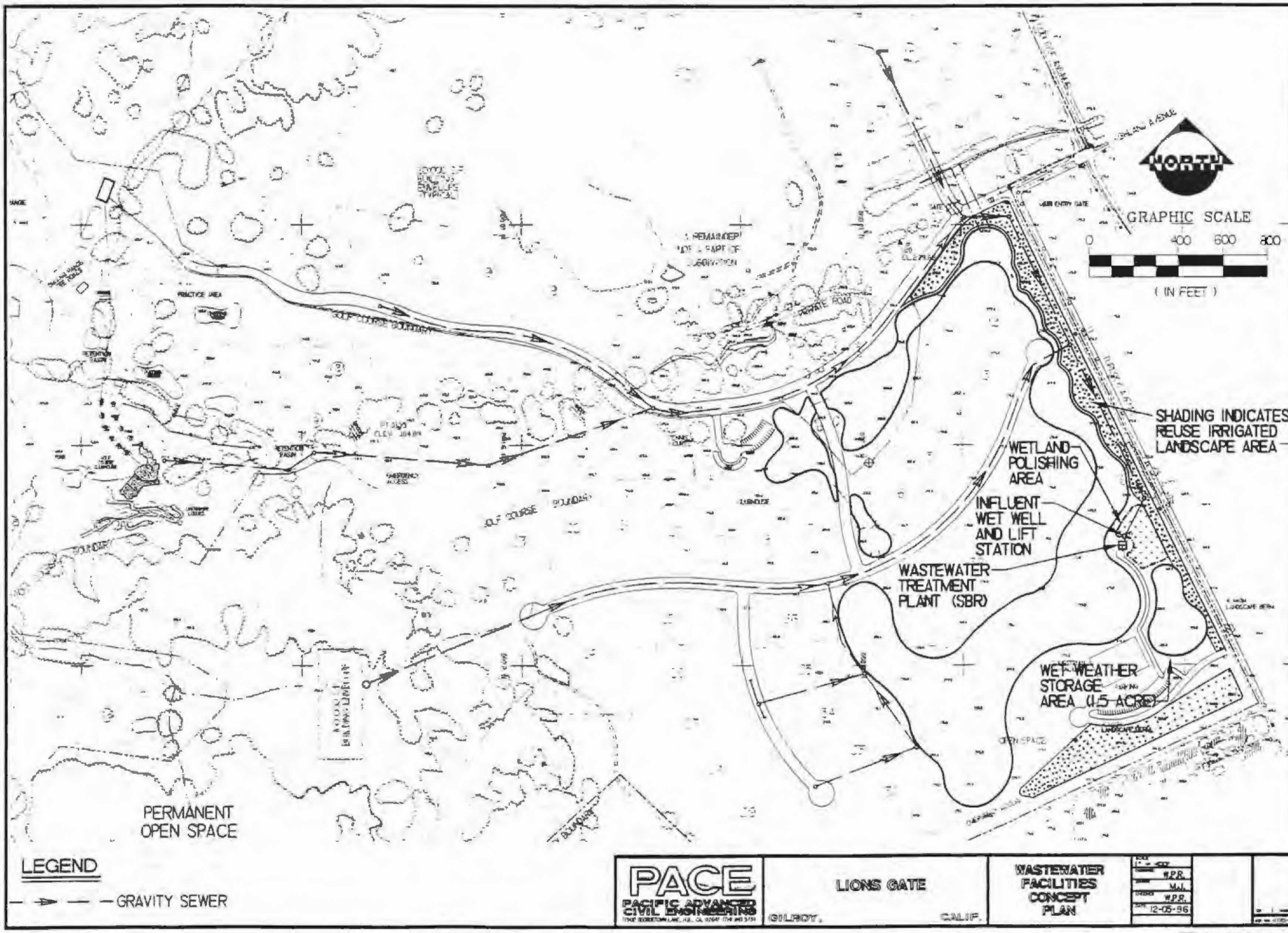

GRAPHIC SCALE
0
400
600
800
(IN FEET)
SHADING INDICATES REUSE IRRIGATED LANDSCAPE AREA
WETLAND POLISHING AREA
INFLUENT WET WELL AND LIFT STATION
WASTEWATER TREATMENT PLANT (SBR)
WET WEATHER STORAGE AREA (1.5 ACRE)
PERMANENT OPEN SPACE
LEGEND
GRAVITY SEWER
PACE
PACIFIC ADVANCED CIVIL ENGINEERING
LIONS GATE
GILROY, CALIF.
WASTEWATER FACILITIES CONCEPT PLAN
W.P.R.
M.J.
W.P.R.
12-05-96
PLOT DATE: 12/04/96

FIGURE 23

~~Short term Emergency Storage: Short term emergency storage for one day of peak flow would be provided by underground tanks located alongside the treatment plant, and would have a capacity of 30,000 gallons. Each of the pump stations in the collection system would also have emergency storage capacity, roughly equal to one day of sewage flow from the respective service area, bringing the total emergency storage in the system to about two days of flow. The sewer pump stations would include alarm systems with auto dialers and standby generator(s) for emergency power. This would ensure continuous pump station operation during power outages or mechanical breakdown of an individual pump. Emergency power would be provided by a dedicated unit at each pump station.~~

~~Long term Wet Weather Storage: Long term (90 day) storage of treated wastewater during the wet season would be provided by a storage pond to be located in the "saddle" area immediately upslope and to the northwest of the practice range. The storage pond would be roughly 16 feet deep (at capacity), with an additional two feet of freeboard and an overall maximum water surface area of about 30,000 square feet. The storage volume of the pond at capacity would be approximately 8 acre feet. The pond would be lined with a clay, plastic or gunite liner to prevent leakage.~~

~~Disposal Facilities: Treated wastewater would be disposed of entirely by spray irrigation of restricted access turf grass and open space portions of the project. The areas planned for irrigation include the golf course practice range and chipping area, plus about 3 to 4 acres of open space grassland knolls on the west side of the storage pond (see Figure 23). The overall land area required for irrigation is estimated to be about 12 acres. This is based on the assumption of an 8-month irrigation season (roughly March through November). The calculations are based solely on the evapotranspiration requirements for irrigated pasture; they assume negligible loss of water to percolation. The total volume of reclaimed water to be disposed of during the irrigation season includes the daily wastewater flow during the irrigation season, plus all wastewater and rainfall collected in the storage reservoir during the winter months. The total volume is estimated to be about 28.2 acre feet in a wet rainfall year.~~

Collection System: The wastewater generated by the residential area and golf course facilities would be collected in 8-inch gravity flow sewers and conveyed to an advanced treatment facility located near the eastern site boundary approximately 200 feet west of Turlock Avenue. This system would collect all of the sewage generated, unlike the system previously proposed where only the effluent was to be collected for treatment with the solids to be settled out in individual septic tanks. Since the wastewater would be collected by gravity flow, there would be no need for individual step pumps, lift stations or force mains as required under the previously proposed system. (Under the proposed system there would be three small pumps, which would all be located at the treatment plant.) This would result in greater system reliability, with less potential for pump failure, and would also represent a substantial savings in both capital costs and ongoing power and maintenance costs. This configuration would also be preferable to the Regional Water Quality Control Board, which had expressed concern with the potential for failure of the numerous pumps previously proposed.

Treatment Facility: The proposed treatment method would involve tertiary treatment utilizing the Sequencing Batch Reactor (SBR) process, combined with final treatment at a constructed wetland area nearby. (The principal difference between tertiary treatment and the secondary treatment system previously proposed for the project is that tertiary treatment provides a higher level of filtration for the removal of contaminants, heavy metals and suspended solids, and also provides a higher level of nutrient removal. Under the tertiary treatment process proposed, the treated effluent would contain nitrate concentrations of less than 2 mg/l and a coliform count of less than 2.2/100ml, while secondary treated effluent would contain nitrate levels less than 25 mg/l and a coliform count of less than 23/100ml.) The wetland area would consist of a lined pond two feet deep and planted with wetland species which would provide bio-filtration and biological denitrification. With the tertiary treatment provided by the system, the effluent would meet or exceed Title 22 Reclaimed Water Class II standards for restricted access recreational impoundments.

Since the proposed treatment process would handle all of the sewage generated, the solids would settle out as sludge. (Under the previous proposal, the solids would be retained in individual septic tanks and periodically pumped out and hauled away by tanker trucks.) In the proposed SBR system, the sludge remaining from the treatment process would be periodically removed by tanker truck for disposal and treatment at a municipal wastewater treatment plant, as occurs with septic tank sludge. Sludge removal would occur every three months, when approximately 3,000 gallons of sludge would be removed. Since both the treatment process and sludge storage would occur underwater within a totally enclosed building, and since the treatment process involves a significant amount of aeration, the potential for odor generation is minimal. (See discussion under 'Impact 5' below.)

As mentioned, the treatment plant would be located near the southeast corner of the site to take advantage of gravity flow and reduce pumping requirements. The entire treatment facility, including disinfection tanks, sludge ponds, and controls, would be housed in a one-story building with a low-profile barn-like or residential appearance. The treatment facility would occupy an area measuring 40 feet by 40 feet, and the adjacent constructed wetland would be approximately .75 acres in area (see Figure 2 in the report by Pacific Advanced Civil Engineering, contained in Appendix N). An SBR treatment plant similar to the one proposed has been in operation at the Ciello Vista Estates project in Hollister since 1989. That facility currently serves 76 residences and treats approximately 50,000 gallons of wastewater daily, about double the volume of the proposed treatment facility for the Lion's Gate project. That facility appears as a dwelling located within a residential neighborhood, with the nearest house located 100 feet away. Photographs of this facility are provided in Figure 23a.

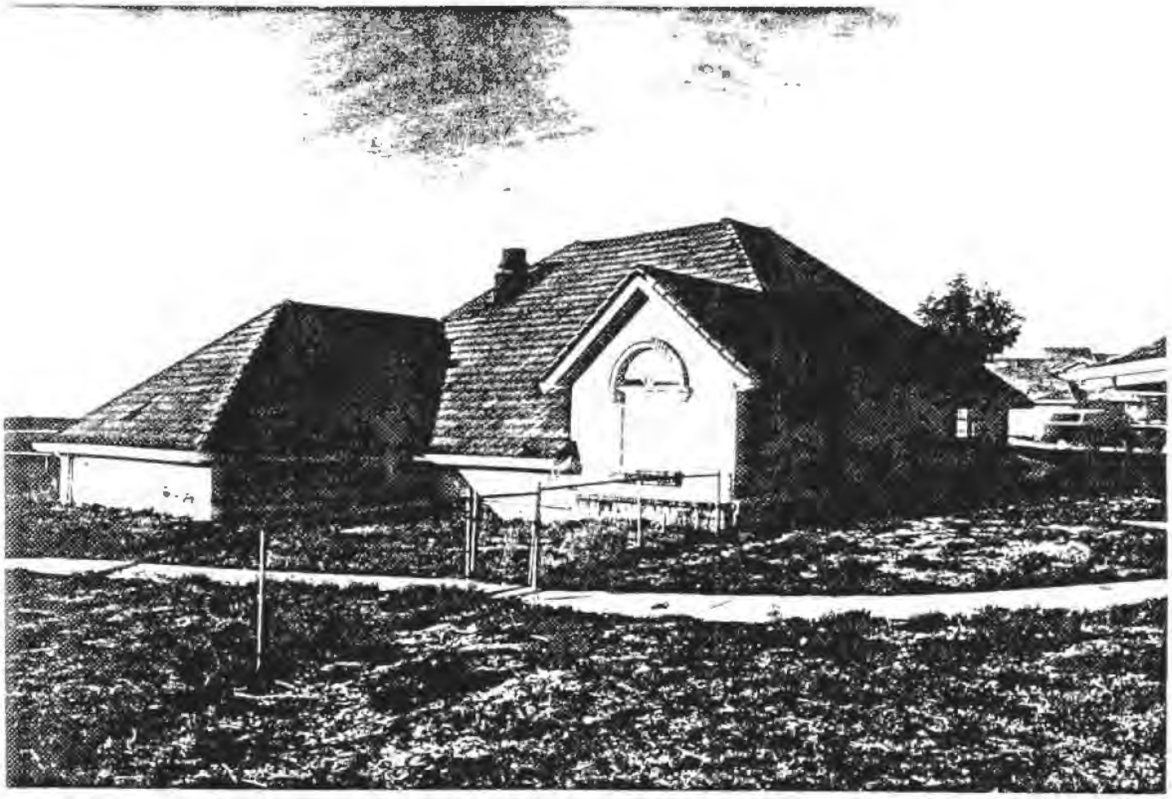
Emergency Power Supply: To provide for uninterrupted power supply in the event of an extended power failure, back-up power source would be provided to pump influent from the wet well to the containment/sludge pond within the treatment plant.

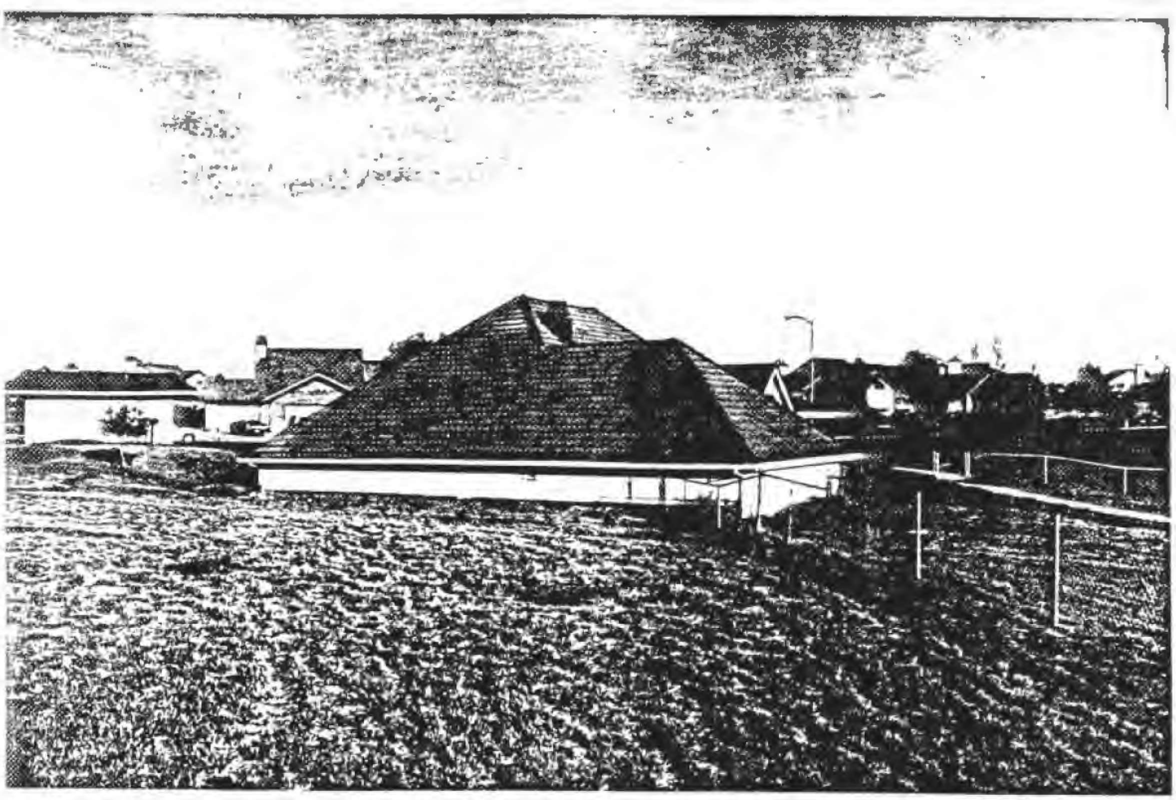
Short-term Emergency Storage: Short-term emergency storage for 24 hours of peak flow required by Title 22 would be provided by the sludge containment pond. In an extreme emergency, an additional 20 days of emergency storage could be provided by the lined constructed wetlands.

Long-term Wet Weather Storage: Long-term (120-day) storage of treated wastewater during the wet season would be provided by a dedicated effluent storage pond to be excavated to the west of the treatment plant. The pond would appear as part of the residential lakes proposed for this area, but in fact would be a separate impoundment. The pond would have a capacity of 6.4 acre-feet and would occupy 1.75 acres. The pond would be lined with either clay or PVC depending on soil suitability; the liner would be backfilled with a minimum of 18 inches of soil and landscaped to blend in with the surrounding area.

<u>SEQUENTIAL BATCH REACTOR TREATMENT PLANT</u>

Hollister, California





100,000 gallon per day facility in residential neighborhood, enclosed in house structure
Fluidyne, Inc. SBR Equipment

FIGURE 23a

Since the pond would contain tertiary treated effluent, it would meet or exceed the Title 22 standards for restricted access recreational impoundments. This means that the level of treatment would be sufficient to allow incidental body contact but not total body contact. The lake and irrigated areas would be posted with the required signage for usage of reclaimed water. (See discussion under 'Impact 3' below.)

Effluent Disposal: The treated effluent would exceed the required level of disinfection for unrestricted landscape irrigation, and would be applied over the frontage landscaped area. It is estimated that enough reclaimed water would be generated to provide irrigation water for 8 acres of landscaped area.

Solids Disposal: Every three months, approximately 3,000 gallons of liquid sludge would be transported by tanker truck to a nearby large scale municipal treatment facility for sludge processing and disposal. Sludge processing is an ongoing process at large-scale facilities with belt presses and/or sludge drying beds. The transported sludge is highly aerated and easily introduced into the processing system. The previously proposed system of septic tanks would also require hauling of septage to nearby treatment facilities. However, septage is in an anaerobic condition and is not compatible for easy disposal in most activated oxygen type treatment facilities. The facilities have to introduce the septage slowly, so as not to upset the balance in the treatment system biomass. Therefore, disposal of sludge is generally less problematic than disposal of septic tank septage. Preliminary discussions with representatives of the South County Regional Wastewater Authority indicate that the nearest municipal wastewater treatment plant at Gilroy should have no technical difficulty accepting the relatively small quantity of sludge generated by the project. However, acceptance of the project's sludge for treatment and disposal is subject to approval by the Board of Directors for the Authority (Jay Baksa, Authority Manager, personal communication).

Facility Operation and Maintenance: The proposed community wastewater system would be owned and operated by the Community Services District (CSD) established for the project. Since the system would generate more than 2,500 gallons of effluent per day, it would be under the jurisdiction of the Central Coast Regional Water Quality Control Board; as such, the system would require a waste discharge permit from the Regional Board. The CSD would be the responsibility party (i.e., "discharger") named in the Waste Discharge Requirements (i.e., permit) issued by the Regional Board for the facility. Actual day-to-day operations could be performed by employees of the CSD or by contractors. However, the CSD would have ultimate responsibility for compliance with the Waste Discharge Requirements and the submittal of monitoring reports to the Regional Board.

With respect to day-to-day operations, Title 22 of the California Administrative Code contains specific requirements for monitoring, record keeping and treatment plant maintenance to assure public health protection. A certified wastewater treatment plant operator would be required for the treatment plant. It is anticipated that testing and regularly scheduled maintenance would require less than 20 hours per week for a well-trained individual with maintenance help as required. The SBR equipment manufacturer would provide a detailed operation and maintenance manual including regularly scheduled maintenance items such as dissolved oxygen sensor calibration. Additionally, the Santa Clara County Sewage Disposal Ordinance requires that community wastewater systems be monitored by the designer for one year, and that the operator execute a maintenance contract with a sanitary engineering firm for the first 5 years of system operation.

Maintenance Facility

The maintenance facility would not be connected to the centralized wastewater system, but would have its own septic tank and leachfield system. Based on a generation rate of 15 gpd for 15 employees, maximum flows would be 225 gpd. Preliminary soils and groundwater studies indicate that there is adequate depth to groundwater, and that the soils in the vicinity have acceptable percolation rates for the planned leachfield.

Alternative Wastewater Treatment Configurations

Several alternative methods of wastewater treatment and disposal were studied for the Lion's Gate project, as described below.

Individual Residential Septic Systems: The main alternative to the proposed wastewater system would include: a) the use of individual septic systems for each residential lot (and the equestrian center, and the swim and tennis center); and b) a separate package treatment plant, storage pond and spray irrigation system solely for the golf course clubhouse and lodging units. This alternative is feasible as studies to date have verified adequate soil depth/groundwater conditions to support individual septic systems at the residential building sites. The layout of the residential sites has been planned to match the septic system options and limitations. A package treatment plant system for the golf course facilities is also feasible. It would be about one-half the size and capacity of the proposed wastewater system to serve the entire development. The advantages of the proposed wastewater plan over this option of utilizing residential septic systems are as follows:

- All wastewater treatment and disposal would come under the maintenance and management authority of a public district and certified wastewater personnel;

- A greater percentage of the wastewater would be made available for reclamation and reuse for irrigation of a portion of the golf course (the practice areas), reducing the demand on other irrigation water sources; and,

- The overall nitrate loading from the project would be reduced, since the secondary treatment followed by irrigation removes a substantially greater amount of nitrate than do individual septic tank-leachfield systems. The use of package treatment plants with spray irrigation is identified as a nitrate control management objective in the Santa Clara Valley Water District's draft plan for the Llagas Groundwater Basin.

~~The one advantage of the individual residential septic system option would be the elimination of the effluent collection system (and its associated pump stations and piping) in favor of a simple, on-site gravity flow system at each house.~~

~~Conventional Gravity Sewers: Conventional gravity sewers, as opposed to effluent only sewers, were considered as a system design option. Conventional sewers would eliminate the need for a septic tank at each house/building, but the construction costs and excavation~~

~~requirement for larger diameter gravity sewers, manholes and lift stations spread over the development area would offset the savings. The on-site treatment plant could be designed to accommodate either effluent or raw sewage from a conventional sewer system. If conventional sewers were to be used, an additional screening and sludge handling process would be included at the treatment plant. Ultimately, disposal of the sludge would be by hauling to an approved landfill site. An advantage of the system design proposed for the project is the ability to build-in surplus storage or emergency disposal capacity at the stations or individual building sites with the use of subsurface leachfield trenches. This is possible because of the inclusion of septic tanks for primary treatment at each house/building. Septic tank effluent can be disposed in appropriately sited leaching trenches, but raw sewage cannot.~~

Effluent-Only Sewers and Secondary Treatment: Effluent-only sewers were previously proposed for the project. Instead of collecting all sewage for treatment at a central treatment plant, this would entail the installation of individual septic tanks, but not leachfields, at each homesite, and also at the clubhouse and overnight accommodations complex. The septic tanks would provide primary effluent treatment (i.e., sedimentation) with the effluent from each tank piped to the treatment plant for further treatment. One benefit of this collection system is that it reduces construction costs due to the smaller diameter pipes, and it provides additional emergency storage capacity in the individual septic tanks. Effluent-only sewers are less attractive when the collection system operates entirely by gravity, as is currently proposed, where reduction of pumping costs is not a consideration. In addition, effluent-only sewers require regular pumping of septic tanks at individual homesites and golf facilities, which involves some inconvenience to homeowners and the golf course operator.

From an environmental standpoint, this alternative would achieve far less removal of nitrates than the system proposed. Since the wastewater would receive only secondary treatment, the total nitrogen concentration in effluent from the treatment plant would be approximately 25 mg/l, although natural denitrification at the storage pond would be expected to reduce this to 3 to 4 mg/l at the time of final discharge to the irrigation system. In the proposed system, the wastewater would receive tertiary treatment, resulting in the removal of total nitrogen to less than 2 mg/l.

Municipal Sewerage: The possibility of extending sewer service from the City of Morgan Hill to the project site was considered in connection with prior development plans for the project site. The project site is not within the sewer service area for the Morgan Hill/Gilroy Wastewater Treatment Plant and would require annexation and several miles of sewer pipeline construction. Due to the relatively small wastewater flows from the Lion's Gate project, and the substantial distance to the Morgan Hill/Gilroy system, sewer connection to the system would not be a practical alternative.

Impact 2. **The proposed wastewater disposal facilities may result in degradation of surface and groundwater quality. (Potential Significant Impact)**

Under proper operation, the proposed disposal of wastewater to land should not result in any noticeable impacts on surface water quality in local drainages or the West Branch of Llagas Creek. This is because the system would be subject to the Regional Board's standard requirement that there be no runoff of wastewater from any spray disposal area into streams or drainages; and the spray disposal operations are planned to be confined to the irrigation season only. To further minimize the risks of reclaimed water runoff into streams, the proposed spray areas are to be set back 100 feet or more from local drainages. (Note: Treated effluent would be applied to the spray irrigation area at rates matching the evapotranspiration rate of the practice range turfgrass. Also spray irrigation would not occur during the winter months when the turfed areas are likely to be saturated. Thus, there is no potential for treated effluent to leach or run off into on-site drainages.)

A critical water quality concern in the Llagas Groundwater Basin area, where the Lion's Gate project is located, is the concentration of nitrate in groundwater. The Llagas Groundwater Basin has documented high levels of nitrate attributable to agricultural wastes and fertilizer, wastewater disposal and other land use activities. Sources of nitrate loading from the Lion's Gate project would include golf course fertilizers and on-site wastewater disposal. The nitrate analysis for golf course fertilizers prepared by Audubon Conservation Services (see Appendix E), estimated an annual nitrogen loading ranging from 262 lbs to 1,965 lbs of nitrogen, with a resultant nitrate-nitrogen concentration ranging from 0.6 mg/l to 4.5 mg/l reaching the groundwater. The mass nitrate-nitrogen loading from wastewater disposal is estimated conservatively to be about ~~263~~ 21 lbs per year. The combined total nitrogen loading for golf course fertilizers and wastewater disposal is estimated to be ~~525~~ 283 to ~~2,228~~ 1986 lbs per year, which equates to projected groundwater concentration of ~~1.2~~ 0.7 mg/l to ~~5.1~~ 4.6 mg/l. (The equivalent concentration as NO3 would be from ~~5~~ 3 to ~~23~~ 20 mg/l.) These nitrate loading calculations are a prediction of long-term cumulative nitrate levels resulting from the project, based on average annual conditions.

The nitrate loading analysis is based on very conservative (i.e., worst case) assumptions for the nitrogen content of treatment plant effluent (25 mg/l), nitrogen removal rate in the storage pond (40%), and uptake by the soils and vegetation (75%). Higher nitrogen removal rates are attainable with plant design (e.g., Sequencing Batch Reactor or SBR) or through an operating mode specifically selected to optimize nitrogen removal. A good example of the latter is the Las Palmas Ranch Wastewater Reclamation plant in Monterey County, which has a waste discharge limit of 10 mg/l nitrate-nitrogen set by the Central Coast Regional Water Quality Control Board. The total nitrogen concentration in effluent from the treatment plant ranges from 18 to 24 mg/l (as compared with our estimate of ~~25~~ 2 mg/l); ~~but, the final discharge from the storage pond is typically in the range of 3 to 4 mg/l, due to denitrification in the pond. Uptake by turf grass and soils in the irrigation area further reduces the concentration of nitrate-nitrogen reaching groundwater (probably to 1 to 2 mg/l, or less). Based on the demonstrated performance of the Las Palmas Ranch facility, reduction of nitrate concentrations to very low levels, e.g., a few mg/l, is feasible; however, a "zero nitrate discharge" is not an achievable or realistic standard~~. given the very low nitrate levels in the treated effluent, any additional denitrification in the effluent storage pond would be negligible.

The existing groundwater nitrate concentrations in the vicinity of the project site (at San Martin), as reported in the SCVWD Llagas Groundwater Basin Nitrate Study (November 1995), are indicated to be in the range of about 7 to 43 mg/l (as NO3). Historic sampling of a water well on the project site is also reported to fall within this range. The Lion's Gate project site is currently used for cattle grazing; and nitrogen associated with cow manure and urine represents the main current source of nitrate loading to groundwater and surface water runoff. Generally, in pasture and rangeland situations the majority of nitrogen in animal wastes is readily assimilated into the soil and vegetation. However, where soils are damp, where animals congregate and where they have direct access to streams and other drainages, a portion of the nitrogen will be carried by runoff or percolate into the groundwater. These are likely the current routes of nitrogen input to the Llagas Groundwater Basin from the project site.

Under the proposed project, the cattle grazing is planned to be entirely eliminated in favor of the golf course and residential development. From a nitrogen loading standpoint, the turf fertilizer and reclaimed wastewater would essentially replace animal wastes as the principal source of nitrate on the project site. Because of the slow rate of groundwater movement, it is likely to take several years for any changes in water quality to be noticeable. Moreover, as indicated by the water-chemical mass balance analysis in the wastewater feasibility study, the nitrate loading (in terms of resultant concentration) from the project is estimated to be roughly comparable to existing background groundwater conditions (i.e., ~~5~~ 3 to ~~23~~ 20 mg/l under project conditions, versus 7 to 43 mg/l under existing conditions). Thus, any long-term change in groundwater nitrate concentration is likely to be very slight and difficult to discern.

There is a slight possibility of leakage or spill of wastewater during a major earthquake. However, since the package wastewater treatment plant facilities would consist largely of below ground tankage, the potential consequence of failure or release of wastewater during an earthquake would likely be insignificant. In the unlikely event of a spill, wastewater would be directed to the lined wetland area nearby which has many times the storage capacity of the treatment plant. However, this is a valid issue which would be covered in the "Contingency Plan," which is a standard element of the Waste Discharge Requirements that would be adopted for the wastewater facilities by the Regional Water Board.

Mitigation 2. Groundwater wells would monitor groundwater quality up-gradient and down-gradient of the proposed spray irrigation area and the storage ponds, with corrective action taken as necessary.

Groundwater at the project site would be monitored as a precautionary measure in connection with the wastewater disposal systems and the golf course maintenance activities. All of the existing water wells on the property and the new proposed irrigation well would be periodically monitored for nitrate. Additionally, ~~a dedicated monitoring well~~ groundwater quality monitoring would be performed within and ~~immediately down-gradient (east) of the wastewater spray field areas (practice range and chipping areas)~~ reuse irrigation areas and the storage pond and constructed wetland.would be added to distinguish possible localized effects from the wastewater systems. The Regional Board may also require that additional monitoring wells be installed. This would provide a basis for detecting any changes over time and for making adjustments in fertilizer application rates or wastewater operations. ~~In the unlikely event that evidence of contamination is found, corrective action could include incorporating additional treatment processes to further reduce nitrate levels prior to disposal.~~ (The specific measures to be taken would be stipulated in the "Contingency Plan" for the treatment operation, which is a standard element of the Waste Discharge Requirements contained in the "permit" from the Regional Board.) In addition, surface water upstream and downstream of the spray irrigation area would also be monitored for water quality.

Impact 3. **The use of reclaimed wastewater for ~~golf course~~ landscape irrigation and storage of the treated effluent near the residential area ~~would~~ could expose humans to possible physical contact with the treated wastewater, resulting in a potential public health hazard. (Potential Significant Impact)**

~~The areas planned for spray disposal of treated effluent include the golf course practice and chipping area and the grassy hillside knolls adjacent to the proposed wastewater storage pond.~~

Unlike secondary treatment previously proposed, the tertiary treated water in the current proposal can be used for unrestricted landscape irrigation. Since incidental body contact with this level of treated effluent is permissible under Title 22, the public health risk to residents making casual contact with the irrigation water would be virtually nil.

Unlike the storage pond for secondary treated effluent previously proposed, the storage pond for tertiary treated effluent currently proposed would be 'a restricted access recreational impoundment' under Title 22, where incidental body contact would be permitted but total body contact would not. Thus there would be a some public health risk in the unlikely event that individuals were to engage in total body contact activity in the lake, such as swimming.

~~**Mitigation 3. The wastewater would be treated to levels deemed acceptable for disposal on golf courses, and the areas affected would be posted to notify golfers and employees where irrigation by treated wastewater is occurring.**~~

~~State wastewater reclamation criteria recognize golf course irrigation as a suitable use for treated wastewater, and contain standards to protect against unacceptable risks to public health. The areas to be irrigated with treated wastewater would have restricted access and activities, and limited opportunity for human contact with the treated wastewater. The areas of the golf course proposed for irrigation are the practice range, which would be accessible primarily to maintenance staff, and the chipping area, which would have more general accessibility to the golfers. Both areas should be posted with appropriate signs indicating the irrigation with reclaimed water; and irrigation of these areas would need to be limited to times when people are not present, i.e., evenings. The other areas planned for irrigation are grassland knolls that are well removed from general public access. These sites would be part of the permanent open space area and would be accessible to an occasional hiker or horseback rider. Evening spray disposal in those areas is also recommended.~~

~~With diligent compliance with waste discharge requirements, the risks to public health would be minimal. However, if desired, the wastewater system could be upgraded and operated to meet the treatment standards for unrestricted landscape irrigation, as defined in Title 22 of the California Administrative Code.~~

Mitigation 3. The wastewater would be treated to tertiary levels which is acceptable for unrestricted landscape irrigation. Signs would be posted at the irrigated landscape area and at the effluent storage pond to notify residents of the presence of reclaimed water.

Since the tertiary level of treatment proposed would result in coliform counts of less than 2.2/100ml, compared with 23/100ml for secondary treatment, there is far less concern with incidental contact with contaminants. Since the reclaimed water would be quite clean, there is no State requirement to fence-off irrigation areas to prevent human incursion. However, signs would be posted within the irrigated landscape area to inform residents that reclaimed effluent is being used. In addition, signs would be posted around the effluent storage pond indicating its use for reclaimed water storage and warning that swimming is not permitted.

<u>Impact 4.</u> **There is a potential for overflow of the storage reservoir, resulting in a public health hazard. (Potential Significant Impact)**

There is the possibility of an overflow from a wastewater storage reservoir during high rainfall years, if the reservoir capacity is exceeded.

Mitigation 4. The wastewater storage reservoir would have sufficient capacity to accommodate high rainfall years.

To minimize or eliminate the possibility of overflow, the reservoir would be sized to include: (a) surplus storage capacity to account for extreme wet weather effects; and (b) two-feet of freeboard in the pond above the projected maximum water depth <u>(which is substantially greater than the amount of rainfall expected in the 100-year/24-hour storm)</u>. The calculated winter storage requirement is based on ~~90~~ <u>120</u> days with no irrigation. An additional contingency available for a wet winter would be selective spray disposal during the rainy season. In particular, the grassland knolls near the reservoir site would provide suitable winter spray disposal capacity for emergency use without posing a threat of runoff to streams or ponding of treated wastewater in public use areas. In the future, should the wastewater flows exceed the system design, the capacity of the wastewater storage pond could be expanded. Additionally, a reserve leachfield area could be constructed near the treatment plant or pump stations for emergency use.

<u>Impact 5.</u> **The wastewater treatment and disposal system could generate odors.<u> However, since the SBR process proposed involves no odor-producing anaerobic digestion and would be entirely enclosed, no noticeable odors would be generated.</u>**
~~(Potential Significant Impact)~~ <u>(Less-than-Significant Impact)</u>

~~Odors could be generated within the immediate vicinity of the two main pump stations and at the treatment plant. At the effluent storage pond, odors could be created by algae which could grow in the nutrient laden water.~~

<u>Since the SBR treatment process occurs entirely under water and involves a significant amount of aeration, the potential for odor generation is minimal. In conventional treatment processes, odors are created because the process relies on digestion by methane-producing anaerobic bacteria which exist under conditions where oxygen is absent. The SBR process does not include anaerobic bacteria but relies an digestion by aerobic and anoxic bacteria which do not produce odor-generating methane. Additionally, the constant aeration involved in the SBR process prevents the creation and proliferation of anaerobic bacteria. Also, the sludge would be in an aerated liquid state while on-site and when removed for disposal, thus further reducing the potential for odor problems. No drying or composting of sludge would occur on-site. Instead, the stored sludge would be transferred directly from underwater storage to tanker trucks for disposal at an approved wastewater treatment facility. The entire treatment facility would be completely enclosed in a structure to further eliminate the potential for odor dispersion. As an example, the SBR treatment plant at the Ciello Vista Estates project in Hollister has received no odor complaints since it began operating in 1989 (Ed Lantz, Water Technologies Inc., personal communication). As mentioned, that facility treats approximately</u>

50,000 gallons of wastewater daily, about double the volume of the proposed Lion's Gate facility, and is located 100 feet from the nearest residence, while the Lion's Gate facility would be approximately 400 feet from the nearest existing or proposed dwellings.

The potential for odors to be generated by algae that might form in the constructed wetland area is minimal. The nutrient levels of the effluent entering the wetland would be low and the wetland plants would compete for the sunlight that the algae need. In addition, stagnant water conditions would be avoided by the continuous circulation of water and periodic variations in water levels. There is virtually no potential for algae formation in the effluent storage pond since the nitrate levels in the lake would be below 2 mg/l after final treatment.

Since the proposed system contains no pump stations outside the treatment plant site, the potential for odor generation in the sewage collection system is minimal.

~~**Mitigation 5.** **Odor control would be achieved by mechanisms incorporated into the design of the pump stations and the treatment plant, and by measures to be undertaken at the effluent storage pond.**~~

~~Odor control at the pump stations would be achieved by venting through subsurface soil "scrubber" trenches, or above ground activated carbon canister-type filters. If properly maintained, these measures can be expected to reduce pump station odors to a level of insignificance.~~

~~To eliminate odors at the treatment plant, the plant would be designed to capture and eliminate methane and hydrogen sulfide odors with a vacuum system, and with soil filtration.~~

~~Control measures for algae include: (a) aeration of the wastewater pond; (b) addition of chemicals such as non-toxic dyes; and (c) promotion of duck weed to block light penetration. With proper maintenance attention, these measures can be effective in reducing algae problems to less than significant levels.~~

Mitigation 5. **No mitigation required.**

Impact 6. **The existing pond and the proposed open water areas of the project, such as the wastewater storage pond and the residential lake, have the potential to be sites for breeding of mosquitoes, which could create a nuisance and a potential public health problem. (Potential Significant Impact)**

Mitigation 6. **Mosquito breeding would be controlled by several methods, as appropriate for each type of water body. These methods would include the circulation of water to prevent stagnant conditions, the introduction of mosquito fish, and the application of larvacides. The specific mosquito mitigation measures would be formulated in consultation with the Department of Environmental Health Vector Control District.**

~~At the wastewater storage pond, the water would be circulated through the pond, with a portion removed each day for irrigation.~~ Both the constructed wetland and the effluent storage pond would be prevented from becoming breeding areas for mosquitoes and other insects by keeping the water circulating. The turnover and movement of water would interfere with the mosquito breeding cycle during the warm months. ~~The potential mosquito problem would also be minimized by the remote location of the storage pond which is well away from any residences or golf activity areas.~~ At both the effluent storage pond and the constructed wetland, mosquito fish would be appropriate since they would be within a closed system with no potential for the fish to escape. At the existing pond in the central area of the site, and at the proposed lakes for the residential area, the introduction of mosquito fish and the circulation of water would not be appropriate measures for mosquito abatement. Since both of these water bodies would have outlets to West Branch Llagas Creek, the introduction of mosquito fish would risk the escape of the fish resulting in potential disruption of native species. For these ponds, mosquito abatement may require the use of one or more of the following three larvacides: lightweight oil, BTI and methoprene, which can be applied by air or with ground equipment. The oil, which contains surfactants, forms a very thin film on the water surface and essentially suffocates both the larval and pupal stages of the mosquito. The oil tends to dissipate within three or four days, depending on weather conditions. BTI *(Bacillus thuringlensis israelensis)* is a naturally occurring bacterial pathogen of mosquitoes. It is most effective against the larval stages and is approved for use in sensitive habitats by the U.S. Fish and Wildlife Service. Methoprene is an insect growth regulator which prevents the mosquito from developing from the pupal to the adult stage. Extensive research has demonstrated that methoprene has very little impact on non-target organisms, and the U.S. Fish and Wildlife Service has approved its use in sensitive habitats, such as the habitat of the endangered Santa Cruz long-toed salamander.

Prior to design and construction of the new ponds, the Department of Environmental Health Vector Control District would be consulted to ensure a design that will inhibit the development of mosquito breeding.

Impact 7. **The location of the treatment plant near Turlock Avenue could result in potential noise impacts to existing and proposed residences in the vicinity. However, the pumps and aerators at this treatment plant would be largely submerged and entirely enclosed within a building, thus minimizing noise.**
(Less-than-Significant Impact)

In addition to being submerged and enclosed, the aerators would also have mufflers and would be located at least 400 feet from the nearest existing or proposed dwellings. (At the Ciello Vista project in Hollister, noise from the SBR treatment plant is inaudible at the nearest dwellings located 100 feet away.)

Mitigation 7. **No mitigation required.**

Impact 8. **The location of the treatment plant in proximity to existing and proposed dwellings could expose residents to potential release of hazardous materials used in the treatment process. However, this treatment plant would not involve the use or generation of hazardous materials. (Less-than-Significant Impact)**

Although chlorine is often used in the disinfection stage of wastewater treatment, chlorination will not be utilized here. Instead, disinfection will be accomplished by the use of ozone and/or ultraviolet which are not hazardous materials. By not using chlorine there also would be no need for the chemicals used in dechlorination. In addition, there would be no creation of toxic trihalomethanes (THMs) or other chlorine by-products. The treatment process would not involve the use or generation of any hazardous materials.

Mitigation 8. **No mitigation required.**

Conclusion. **With the installation of the proposed wastewater facilities in accordance with applicable standards, and with the implementation of the mitigation measures set forth above, the potential wastewater and related impacts resulting from the project would be non-significant or would be reduced to less-than-significant levels.**

V. ALTERNATIVES TO THE PROPOSED PROJECT

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G. ALTERNATIVES FOR WASTEWATER TREATMENT AND DISPOSAL

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~~**Conventional Gravity Sewers**~~

~~This alternative would involve placing the entire project on conventional gravity sewers. Thus there would be no individual septic tanks at the residential lots of the golf course facilities. The treatment plant would therefore include an additional screening and sludge handling process, with disposal of sludge at an approved landfill site. Also, the treatment plant would require additional emergency storage, to make up for the extra storage provided by the septic tanks and lift stations under the proposed system.~~

~~Since this system would not utilize individual leachfields, the potential nitrate loading would be about the same as for the proposed treatment system. However, the centralized handling and screening of solids at the treatment plant site would result in a greater potential to generate unpleasant odors than the proposed system.~~

~~In other respects, there would be no significant difference in environmental effect between the conventional gravity sewer alternative and the proposed wastewater system.~~

Effluent-Only Sewers and Secondary Treatment

Effluent-only sewers and secondary treatment were previously proposed for the project. Instead of collecting all sewage for treatment at a central treatment plant, this would entail the installation of individual septic tanks, but not leachfields, at each homesite, and also at the clubhouse and overnight accommodations complex. The septic tanks would provide primary treatment (i.e., sedimentation) with the effluent from each tank piped to the treatment plant for further treatment. The process would include secondary treatment, and the effluent would be disinfected to remove most of the pathogens, but the tertiary steps of filtration and denitrification would not be included.

One benefit of this collection system is that it reduces construction costs due to the smaller diameter pipes, and it provides additional emergency storage capacity in the individual septic tanks. Effluent-only sewers are less attractive when the collection system operates entirely by gravity where reduction of pumping costs is not a consideration. In addition, effluent-only sewers require regular pumping of septic tanks at individual homesites and golf facilities, which involves some inconvenience to homeowners and the golf course operator.

From an environmental standpoint, this alternative would achieve far less removal of nitrates and coliform bacteria than the system proposed. Since the wastewater would receive only secondary treatment, the total nitrogen concentration in effluent from the treatment plant would be approximately 25 mg/l, although natural denitrification at the storage pond would be expected to reduce this to 3 to 4 mg/l at the time of final discharge to the irrigation system. In the proposed tertiary treatment system, the total nitrogen contained in the effluent would be less than 2 mg/l. In addition, the coliform count in the tertiary treated effluent would be less than 2.2/100ml, while secondary treated effluent would contain a coliform count of approximately 23/100ml.

In other respects, there would be no significant difference in environmental effect between the effluent-only sewer and secondary treatment alternative and the proposed wastewater system.

Appendix D

Hydrology and Drainage Report

Prepared By

Pacific Advanced Civil Engineering

November 1996

PRELIMINARY DESIGN REPORT

for the

LION'S GATE RESERVE

MASTER DRAINAGE PLAN

Prepared By:

Pacific Advanced Civil Engineering
17902 Georgetown Lane
Huntington Beach, CA 92647

November 22, 1996

#6785

Date: 11/20/96
Job #: 6785E
By: PACE

1. Project Purpose and Need

The enclosed proposed modifications to the project drainage plan are submitted to enhance the previously submitted EIR drainage plan. Similar to the previous plan, the proposed modifications include runoff detention facilities to ensure no increased potential for downstream flooding as a result of the project. In fact, the proposed plan includes more aggressive flood control measures in response to the County's request that the project do more to help alleviate the significant flooding problems that currently exist downstream of the project site.

2. Off-site Drainage

The West Branch of Llagas Creek tributary drainage area (up stream of Coolidge Avenue) includes a majority of the Lion's Gate Reserve Project site. The West Branch of Llagas Creek exits the project site in the easterly boundary just north of the intersection of Coolidge Avenue and Highland Avenue. According to the Santa Clara County drainage engineering section, the West Branch of the Llagas Creek causes significant flooding of areas downstream of the Lion's Gate Reserve project. Therefore, it is critical that on-site developed conditions do not increase the downstream drainage flooding.

In an effort to not only mitigate on-site drainage runoff, but to substantially reduce the downstream flooding, the Lion's Gate Reserve project is proposing the following regional drainage solutions:

A. Provide storm runoff detention via proposed on-site lake/detention system for 10 to 100 year rainfall events.

   1. Construct West Branch Llagas Creek stream diversion structure to divert flows above the 10-year event into the lake/detention basin. The proposed diversion structure will consist of a concrete "L" Section in plain view with an open channel conveyance for flows in Llagas Creek for up to 400 cfs. Flows in excess of 400 cfs will pass over side spillway weir to the south and be conveyed to the lake/detention basin. At a high water level of elevation 275, the detention basin will not accept additional run-off and flows in excess of the 100 year storm will overtop the Llagas Creek diversion structure and continue on in the historic flow path.

   2. Store ± 45 acre feet of runoff from the West Branch of Llagas Creek in ± 2.5 foot freeboard of the proposed ± 17 acre Lion's Gate Reserve lake/detention system.

3. The result of the proposed detention will be to reduce the existing 100-year peak flow rate as it exits the Lion's Gate Reserve site from ± 800 cfs to ± 400 cfs. This will reduce 100-year runoff peak flows to approximately the 10-year runoff condition which is a substantial reduction and significantly reduce downstream flooding problems.
4. The lake/detention area will be excavated and a normal lake water surface maintained at elevation 273.0. The flood waters will be conveyed by the proposed lake/stream/channel system along Turlock Avenue. The lake/detention system will store the runoff up to elevations 275.5 at which point the inlet stream/channel will back-up and not allow additional runoff to enter the lake system; thus forcing flows in excess of the 100-year event down the Llagas Creek.

2. On-site Drainage

The on-site drainage improvements include the following elements:

1. Routing of urban runoff to detention/retention or lake areas prior to any discharge to the West Branch of Llagas Creek. The on-site drainage system will include roadway catch basin collection system and discharge to drywells at the lake perimeter prior to overflow to the lake system.

2. Individual lot drainage, as part of the master drainage plan, will be prepared to minimize any cross lot drainage to adjacent lots and to determine detailed on-site drainage system requirements.

3. Riparian Area Avoidance and Enhancement

The proposed drainage plan will minimize impacts to the existing waters of the U.S. surrounding Riparian Areas. The proposed lake/detention basin will be utilized to provide additional riparian and open water areas.

4. Master Drainage Plan

A master drainage plan for the Lion's Gate project will be prepared in accordance with the proposed project plans. The master drainage plan will include the following elements:

A. Hydrologic modeling, for pre and post developed conditions, (HEC-1 for the off-site drainage area and rational method for the on-site drainage areas) for determination of rainfall runoff peak flow rates, runoff volumes and time of concentrations for various storm frequencies (2, 10, 50 and 100 year events).

Date: 11/22/96
Job #: 6785E
By: PACE

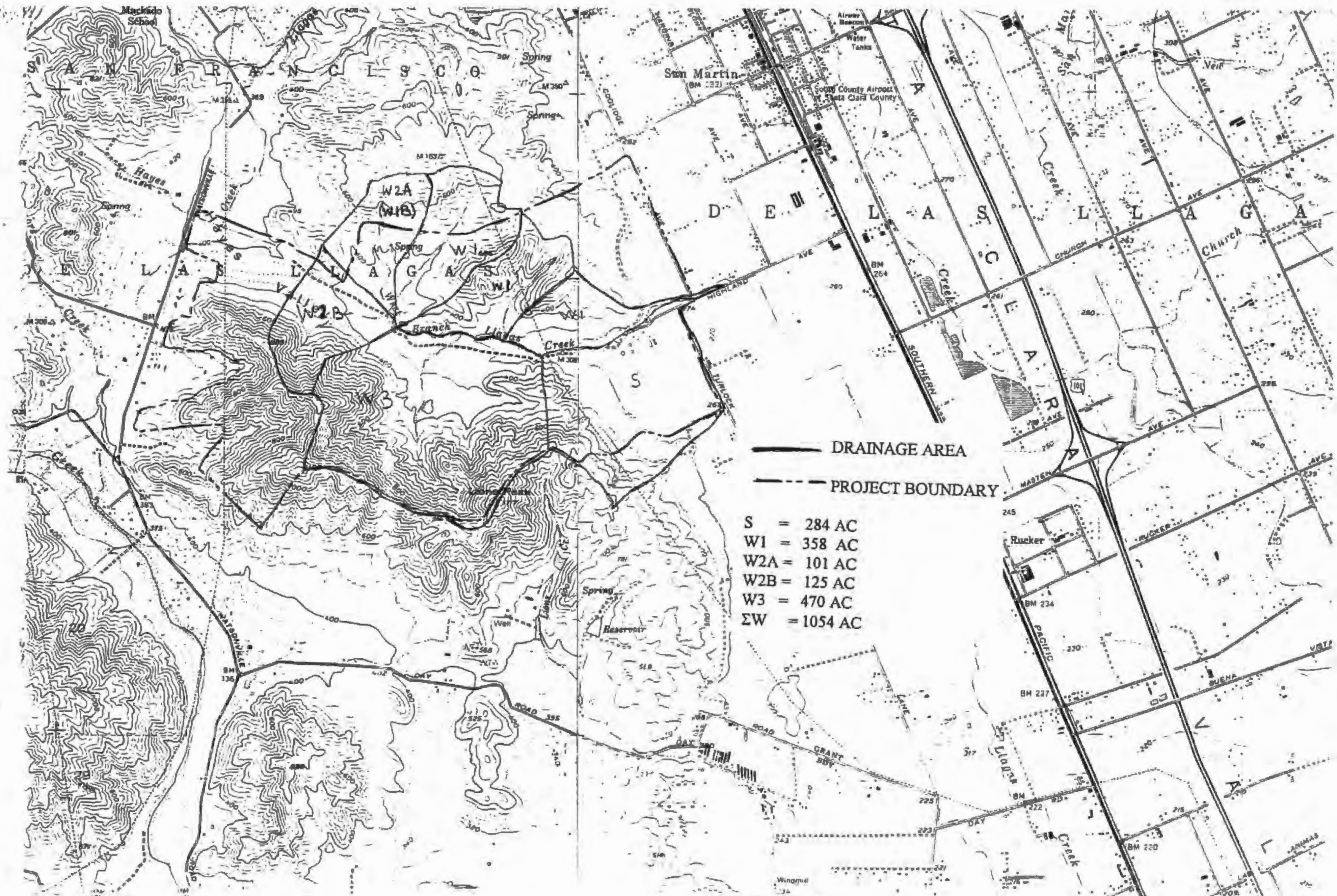
B. West Branch Llagas Creek hydraulic modeling HEC-1 and HEC-2 for determination of existing and proposed condition creek water surface profiles and proposed detention basin routings.

C. On-site and off-site drainage plan which coordinates with the project site plan regarding runoff routing and sizing of storm culverts and other hydraulic features.

The master drainage plan will be submitted to Santa Clara County and the Santa Clara Water District for review and approval.

The preliminary hydrologic analysis prepared in this report is based upon HEC-1 model obtained from the Santa Clara Country Water District and is included in Appendix.

APPENDIX


DRAINAGE AREA
PROJECT BOUNDARY
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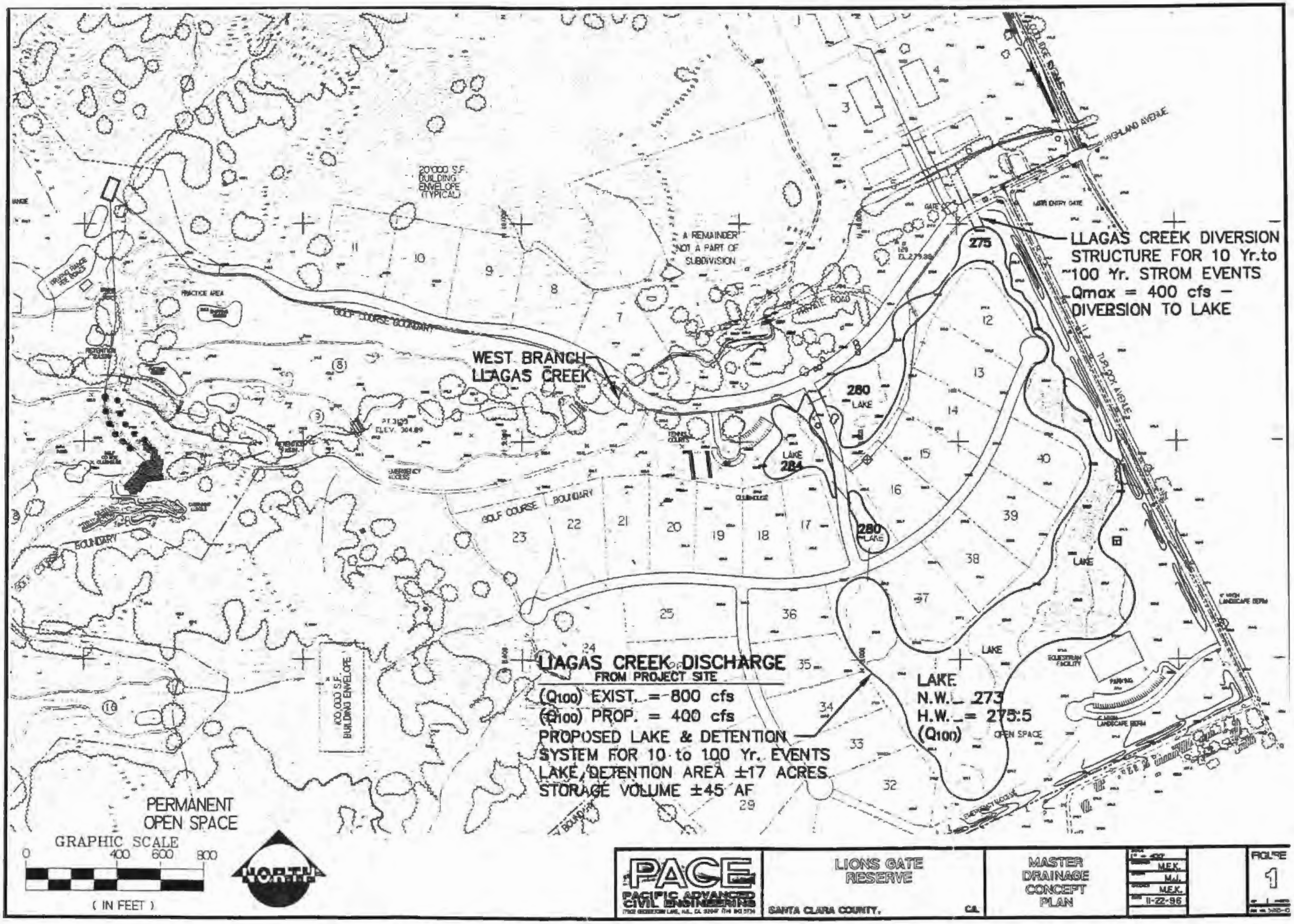
W1 = 358 AC
W2A = 101 AC
W2B = 125 AC
W3 = 470 AC
ΣW = 1054 AC
W2A
(W1B)
W1
W2B
W3
S
San Martin
South County Airport of Santa Clara County
Machado School
Rucker
SOUTHERN PACIFIC
HIGHLAND AVE
TURLOCK
COOLIDGE
CHURCH AVE
MASTEN
RUCKER
BUENA VISTA
SANTA CLARA
LLAGAS
SAN FRANCISCO
Branch Llagas Creek
Reservoir
Windmill



LLAGAS CREEK DIVERSION STRUCTURE FOR 10 Yr.to 100 Yr. STROM EVENTS Qmax = 400 cfs – DIVERSION TO LAKE
WEST BRANCH LLAGAS CREEK
HIGHLAND AVENUE
TURLOCK AVENUE
GOLF COURSE BOUNDARY
PRIVATE ROAD
A REMAINDER NOT A PART OF SUBDIVISION
20,000 S.F. BUILDING ENVELOPE (TYPICAL)
100,000 S.F. BUILDING ENVELOPE
PRACTICE AREA
TENNIS COURTS
CLUBHOUSE
EMERGENCY ACCESS
MAIN ENTRY GATE
EQUESTRIAN FACILITY
PARKING
OPEN SPACE
280 LAKE
LAKE 284
280 LAKE
275
LAKE
LLAGAS CREEK DISCHARGE
FROM PROJECT SITE
(Q100) EXIST. = 800 cfs
(Q100) PROP. = 400 cfs
PROPOSED LAKE & DETENTION SYSTEM FOR 10 to 100 Yr. EVENTS
LAKE/DETENTION AREA ±17 ACRES
STORAGE VOLUME ±45 AF
LAKE
N.W.L. 273
H.W.L. = 275.5
(Q100)
PERMANENT OPEN SPACE
GRAPHIC SCALE
0
400
600
800
(IN FEET)
NORTH
PACE
PACIFIC ADVANCED CIVIL ENGINEERING
LIONS GATE RESERVE
SANTA CLARA COUNTY, CA
MASTER DRAINAGE CONCEPT PLAN
M.E.K.
M.J.
M.E.K.
11-22-96
FIGURE
1

HEC1 S/N: 1343001791   HMVersion: 6.33   Data File: wbl.dat

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*            MAY   1991                 *                    *   HYDROLOGIC ENGINEERING CENTER      *
*          VERSION 4.0.1E               *                    *        609 SECOND STREET             *
                                        *                    *     DAVIS, CALIFORNIA 95616          *
  RUN DATE  11/22/1996  TIME  14:13:22  *                    *         (916) 756-1104               *
*                                       *                    *                                      *
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X      X  XXXXXXX   XXXXX         XXX
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:::  Full Microcomputer Implementation  :::
:::               by                 :::
:::       Haestad Methods, Inc.      :::
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37 Brookside Road * Waterbury, Connecticut 06708 * (203) 755-1666

THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL   LOSS RATE:GREEN AND AMPT INFILTRATION
KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

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  2            ID  RAINFALL DISTRIBUTION IS BASED ON C.O.E. STANDARD STORM.
  3            ID  LOSS RATES OF RURAL PARTS ARE BASED ON MATCHING REGIONAL PEAKS & VOLUMES.
  4            IT      30       0       0     101
  5            IO       5       0

  6            KK     201
  7            KM  UPPER W.B.LLAGAS @ HIGHLAND (LGN1 @ 201).
  8            BA    3.62    0.00    0.00
  9            PB    8.00
 10            PI   0.780  0.8300  1.1900  1.2300  1.0200  1.0800  1.7700  1.6200  2.2300  1.8600
 11            PI   1.440  2.2700  1.6800  1.5500  2.0500  1.6600  1.7400  2.1300  1.7700  1.6200
 12            PI   1.600  1.7900  1.8400  1.8700  1.8800  1.9900  2.6600  2.9800  1.9000  2.2900
 13            PI   3.380  3.2300  2.8700  3.4500  6.0000  5.7700  2.2100  2.4600  2.3600  2.0000
 14            PI   1.930  1.7800  2.0700  1.3200  1.8300  1.7200  1.6300  1.6000  0.0000  0.0000
 15            LU   0.133   0.067   5.000
 16            UC   0.890   1.400
 17            BF   -2.00   -0.01  1.3797

 18            KK     202
 19            KM  ROUTE UPPER W.B.L.  TO  FITZGERALD (202)
 20            RL      0.    0.00
 21            RM       1    0.44    0.00

 22            KK     202
 23            KM  MIDDLE W.B.L. @ FITZGERALD (LGN2 @ 202)
 24            BA    1.77    0.00    0.00
 25            PB    7.74
 26            LU   0.129   0.069   7.000
 27            UC   1.240   0.280
 28            BF   -2.00   -0.01  1.3797

 29            KK     202
 30            KM  TOTAL W.B.LLAGAS @ FITZGERALD (LGN1+2 @ 202).
 31            HC       2

 32            KK      20
 33            KM  ROUTE (LGN1+2)TO MOREY CHAN. CONF. @ 20
 34            RL      0.    0.00
 35            RM       1    0.61    0.00

 36            KK      20
 37            KM  LOWER W.B.L. @   MOREY (LGN3 @ 20)
 38            BA    1.48    0.00    0.00
 39            PB    7.52
 40            LU   0.125   0.063  41.000
 41            UC   1.240   0.170
 42            BF   -2.00   -0.01  1.3797

 43            KK      20
 44            KM  TOTAL W.B.LLAGAS CREEK U/S LIONS (LGN1+2+3 @ 20)
 45            HC       2
```

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10


 46           KK      19
 47           KM  LIONS CR. U/S MOREY (LG01 @ 19)
 48           BA    2.09    0.00    0.00
 49           PB    8.04
 50           LU   0.134   0.067   9.000
 51           UC   0.870   0.640
 52           BF   -2.00   -0.01  1.3797

 53           KK      19
 54           KM  NORTH MOREY U/S MOREY(LG02@ 19)
 55           BA    1.02    0.00    0.00
 56           PB    7.68
 57           LU   0.128   0.064  41.000
 58           UC   1.150   0.180
 59           BF   -2.00   -0.01  1.3797

 60           KK      19
 61           KM  MOREY CHANNEL U/S N MOREY CONFL. (LG03 @ 19)
 62           BA    0.76    0.00    0.00
 63           PB    7.46
 64           LU   0.124   0.062  50.000
 65           UC   0.500   0.160
 66           BF   -2.00   -0.01  1.3797

 67           KK      19
 68           KM  MOREY CHANNE D/S N.MOREY (LG02+3 @ 19)
 69           HC       2

 70           KK      19
 71           KM  LIONS CREEK D/S MOREY CHANNEL. (LG01+2+3)@ 19.
 72           HC       2

 73           KK      20
 74           KM  ROUTE LIONS TO W.B.L. CONFL.
 75           RL      0.    0.00
 76           RM       1    0.17    0.00

 77           KK      20
 78           KM  LG04
 79           BA    0.25    0.00    0.00
 80           PB    7.35
 81           LU   0.123   0.061  52.000
 82           UC   0.470   0.160
 83           BF   -2.00   -0.01  1.3797

 84           KK      20
 85           KM  LIONS CR. U/S W.B.LLAGAS.(LG01+2+3+4)@ 20
 86           HC       2
```

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10


  87          KK      20
  88          KM  W.B.LLAGAS D/S LIONS CR @ 20.
  89          HC       2

  90          KK      21
  91          KM  ROUTE W.B.LLAGAS TO MILLER SLOUGH CONF. @ 21
  92          RL      0.    0.00
  93          RM       1    0.75    0.00

  94          KK      21
  95          KM  AREA TRIB. TO RONAN CHANNEL (LGO1 @ 21)
  96          BA    2.14    0.00    0.00
  97          PB    7.01
  98          LU   0.117   0.058  31.000
  99          UC   1.730   0.180
 100          BF   -2.00   -0.01  1.3797

 101          KK      21
 102          KM  W.B.LLAGAS U/S MILLER SLOUGH @  21
 103          HC       2

 104          KK      21
 105          KM  MILLER SLOUGH U/S W.B.LLAGAS(LGP1 @ 21)
 106          BA    1.83    0.00    0.00
 107          PB    7.12
 108          LU   0.119   0.059  62.000
 109          UC   2.020   0.180
 110          BF   -2.00   -0.01  1.3797

 111          KK      21
 112          KM  W.B.LLAGAS D/S MILLER SLOUGH
 113          HC       2

 114          KK      18
 115          KM  ROUTE W.B.LLAGAS TO MAIN LLAGAS CR.
 116          RL      0.    0.00
 117          RM       1    0.31    0.00

 118          KK      18
 119          KM  AREA TRIB. TO W.B.LLAGAS(LGO2 @ 18)
 120          BA    2.84    0.00    0.00
 121          PB    6.66
 122          LU   0.111   0.056   9.000
 123          UC   2.350   0.200
 124          BF   -2.00   -0.01  1.3797

 125          KK      18
 126          KM  TOTAL W.B.LLAGAS U/S LLAGAS CR. @ 18 (INCLUDES LGO2).
 127          HC       2
 128          ZZ
```

```
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*              MAY   1991               *                          *   HYDROLOGIC ENGINEERING CENTER     *
*           VERSION 4.0.1E              *                          *         609 SECOND STREET           *
*                                       *                          *      DAVIS, CALIFORNIA 95616        *
* RUN DATE  11/22/1996  TIME  14:13:22  *                          *          (916) 756-1104             *
*                                       *                          *                                     *
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                    RAINFALL DISTRIBUTION IS BASED ON C.O.E. STANDARD STORM.
                    LOSS RATES OF RURAL PARTS ARE BASED ON MATCHING REGIONAL PEAKS & VOLUMES.

  5 IO         OUTPUT CONTROL VARIABLES
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                    IPLOT           0  PLOT CONTROL
                    QSCAL          0.  HYDROGRAPH PLOT SCALE

   IT          HYDROGRAPH TIME DATA
                     NMIN          30  MINUTES IN COMPUTATION INTERVAL
                    IDATE      1    0  STARTING DATE
                    ITIME        0000  STARTING TIME
                       NQ         101  NUMBER OF HYDROGRAPH ORDINATES
                   NDDATE      3    0  ENDING DATE
                   NDTIME        0200  ENDING TIME
                    ICENT          19  CENTURY MARK

              COMPUTATION INTERVAL    0.50 HOURS
                   TOTAL TIME BASE   50.00 HOURS

        ENGLISH UNITS
            DRAINAGE AREA          SQUARE MILES
            PRECIPITATION DEPTH    INCHES
            LENGTH, ELEVATION      FEET
            FLOW                   CUBIC FEET PER SECOND
            STORAGE VOLUME         ACRE-FEET
            SURFACE AREA           ACRES
            TEMPERATURE            DEGREES FAHRENHEIT
 *** WARNING *****  POSSIBLE INSTABILITIES IN THE MUSKINGUM ROUTING FOR REACH      20.
    REDUCE NSTPS OR DECREASE YOUR COMPUTATION INTERVAL (FIRST FIELD OF THE IT RECORD).
```

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
HYDROGRAPH AT	201	1404.	18.50	1011.	611.	299.	3.62		
ROUTED TO	202	1326.	19.00	1006.	611.	299.	3.62		
HYDROGRAPH AT	202	818.	18.50	498.	291.	140.	1.77		
2 COMBINED AT	202	2108.	18.50	1485.	898.	440.	5.39		
ROUTED TO	20	1931.	19.00	1475.	896.	440.	5.39		
HYDROGRAPH AT	20	692.	18.50	430.	261.	127.	1.48		
2 COMBINED AT	20	2518.	18.50	1884.	1152.	566.	6.87		
HYDROGRAPH AT	19	936.	18.50	611.	363.	176.	2.09		
HYDROGRAPH AT	19	490.	18.00	303.	184.	89.	1.02		
HYDROGRAPH AT	19	416.	18.00	223.	136.	66.	0.76		
2 COMBINED AT	19	906.	18.00	525.	320.	155.	1.78		
2 COMBINED AT	19	1832.	18.00	1135.	683.	331.	3.87		
ROUTED TO	20	1781.	18.50	1133.	683.	331.	3.87		
HYDROGRAPH AT	20	135.	18.00	72.	44.	21.	0.25		
2 COMBINED AT	20	1871.	18.50	1205.	727.	353.	4.12		
2 COMBINED AT	20	4389.	18.50	3061.	1873.	919.	10.99		
ROUTED TO	21	3928.	19.00	3033.	1865.	919.	10.99		
HYDROGRAPH AT	21	896.	18.50	571.	344.	167.	2.14		
2 COMBINED AT	21	4703.	19.00	3578.	2204.	1086.	13.13		
HYDROGRAPH AT	21	760.	18.50	516.	320.	156.	1.83		
2 COMBINED AT	21	5424.	19.00	4081.	2522.	1242.	14.96		
ROUTED TO	18	5360.	19.00	4076.	2519.	1242.	14.96		
HYDROGRAPH AT	18	988.	19.00	693.	410.	198.	2.84		
2 COMBINED AT	18	6348.	19.00	4748.	2927.	1440.	17.80		

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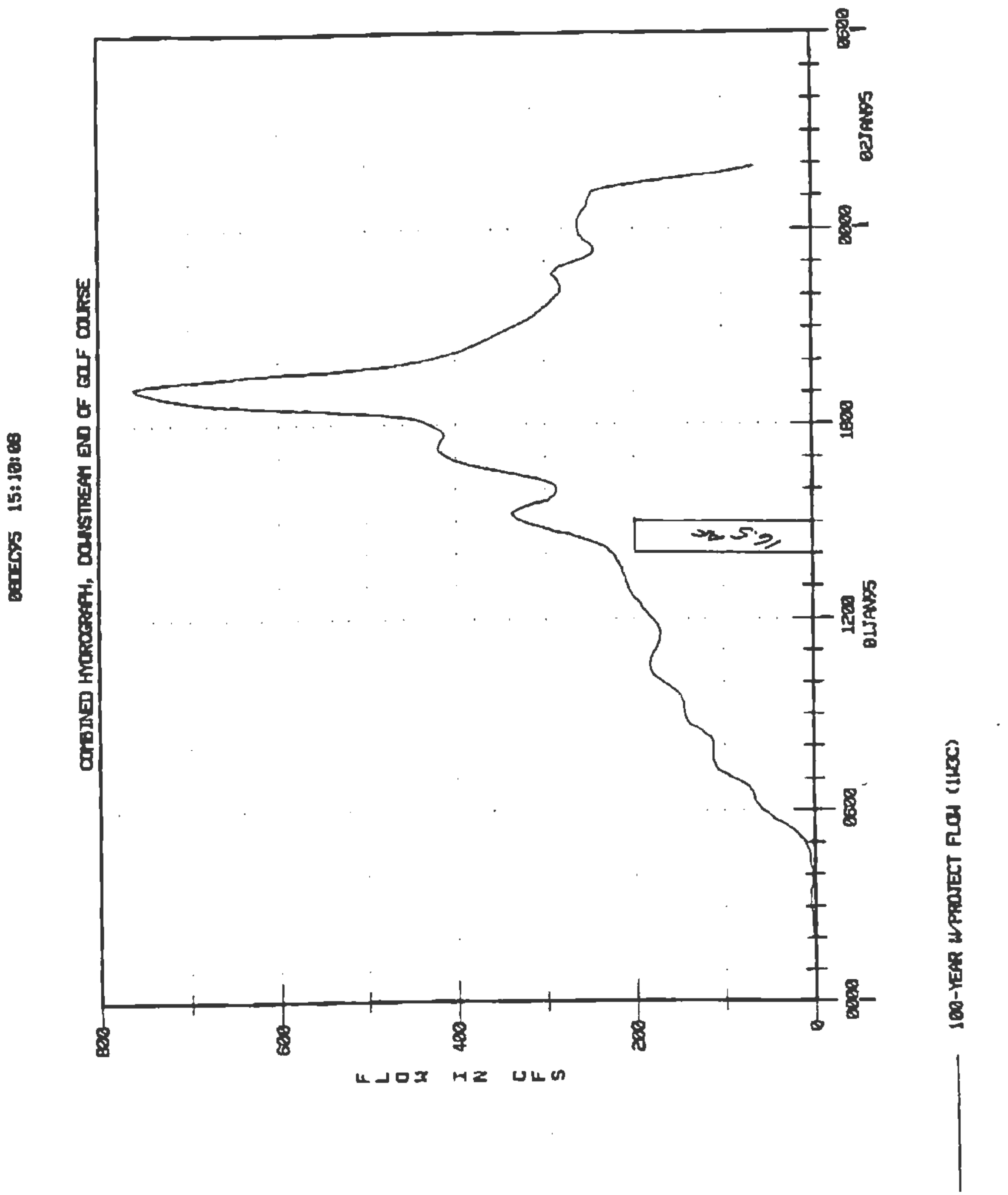
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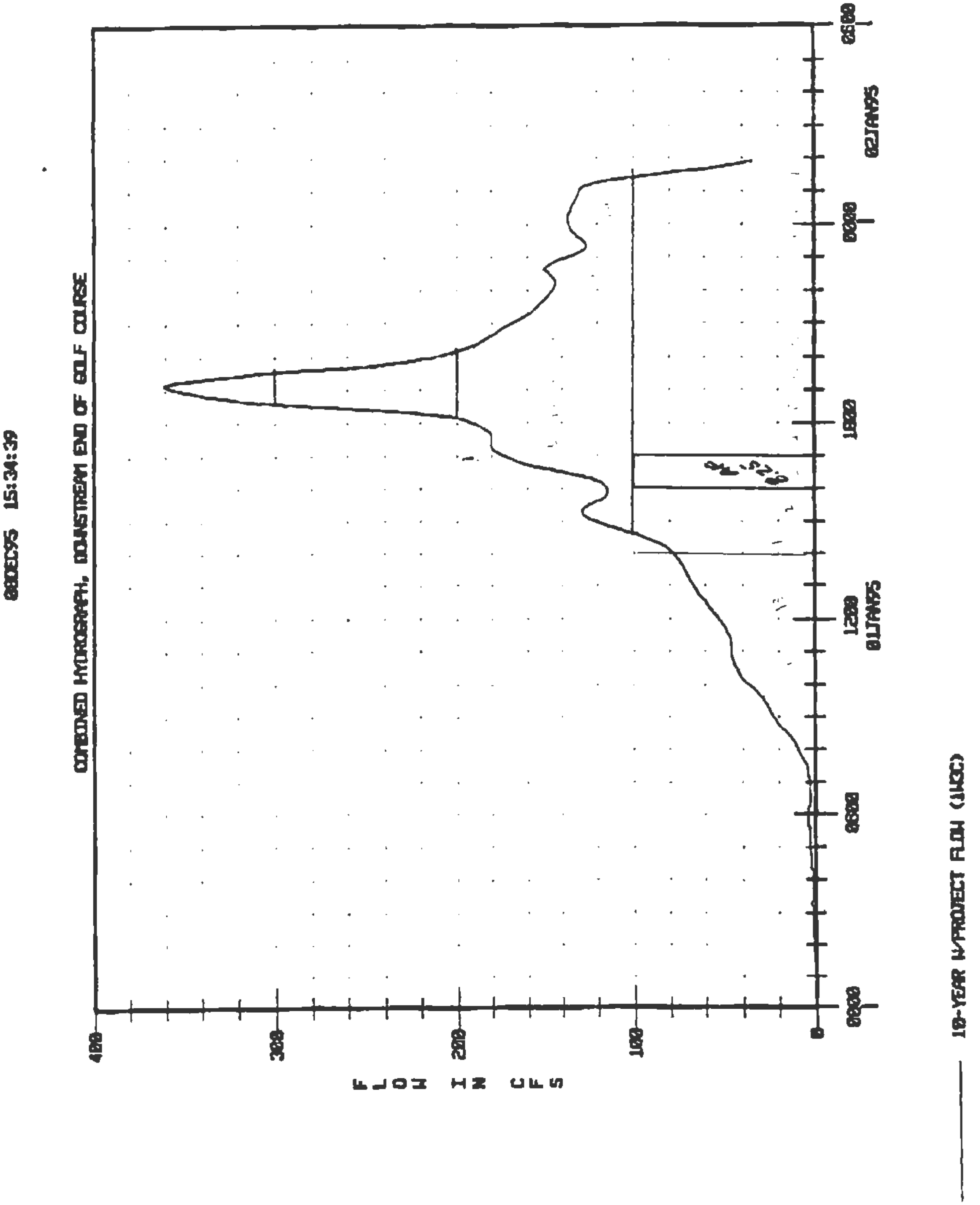
16.5 hr
0000
0600
1200
01JAN95
1800
0000
0600
02JAN95
0
200
400
600
800
FLOW IN CFS
100-YEAR W/PROJECT FLOW (1H3C)



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COMBINED HYDROGRAPH, DOWNSTREAM END OF GOLF COURSE
FLOW IN CFS
0
100
200
300
400
0600
1200
01JAN95
1800
0000
02JAN95
0600
10-YEAR W/PROJECT FLOW (1H3C)

Appendix N

Wastewater System Preliminary Design Report

Prepared By

Pacific Advanced Civil Engineering

December 1996

PRELIMINARY DESIGN REPORT

for the

LION'S GATE RESERVE

WASTEWATER SYSTEM

REGISTERED PROFESSIONAL ENGINEER
No. 46808
Exp. 6-30-99
CIVIL
STATE OF CALIFORNIA

Prepared By:

Pacific Advanced Civil Engineering
17902 Georgetown Lane
Huntington Beach, CA 92647

December 2, 1996

#6785

PRELIMINARY DESIGN REPORT
for the
LION'S GATE RESERVE PROJECT

WASTEWATER SYSTEM

I. INTRODUCTION

The proposed Lion's Gate Reserve, San Martin, CA consists of 1676 acres in the Hayes Valley, approximately 1 mile west of the rural community of San Martin. The project development concept consists of a golf course and lakes, clubhouse, lodges, 41 estate homesites, and other open space. The proposed method of sewage treatment is by gravity collection to an onsite wastewater treatment/reclamation plant.

II. WASTEWATER COLLECTION, TREATMENT AND DISPOSAL

System Description

The entire project can be sewered by gravity flow of all sewage (not just effluent as previously proposed) to an advanced treatment plant (see attached Figure 1). The sewer collection system will pass through a grit screen and empty into a wet well at the treatment plant. Lift pumps will be used to lift the influent to the SBR tank. The proposed treatment method will utilize the Sequential Batch Reactor (SBR) process, combined with disinfection and final (tertiary) treatment occurring at a constructed wetland area. Discharge of treated effluent will pass from the SBR tank, through a disinfection tank, and flow by gravity out of the disinfection tank through the wetlands polishing cells (please refer to Figure 2). As an alternative to the wetlands polishing system, a rapid sand filtration system will be considered. Finally, treated (oxidized, clarified, disinfected, and polished) effluent will be pumped from the wetlands for use in project landscape buffer irrigation. A storage basin will be provided near the irrigation facilities for storage of reuse water during winter wet weather periods when irrigation reuse is not acceptable.

With the tertiary treatment provided by the system, the effluent will meet Title 22 Reclaimed Water Class II standards (i.e., median 7-day total coliform count less than 2.2/100 ml). This level of treatment **exceeds** the required level of disinfection for its intended use as irrigation water for limited access landscaping (please see Appendix, Table 3.0 - California Code Summary of Title 22 Treatment and Water Quality Requirements). Irrigated areas will be posted with required signage for usage of reclaimed water.

The plant will be owned, operated, and maintained by the Community Services District (CSD).

Plant Site and Building Requirements

The treatment plant site will be located near the southeast corner of the site to take advantage of gravity flow and reduce pumping requirements. Gravity collection to the plant represents substantial savings in both capital costs and ongoing power and maintenance costs over individual pumped septic systems. Regulating agencies, including the Regional Water Quality Control Board, were concerned about the potential for the possible failure of the many individual pumps previously proposed. As shown in the Design Data Section of this report, the effluent water quality produced by the proposed SBR facility substantially exceeds all treatment requirements and specifically reduces nitrate levels well below existing ground water levels (see Appendix for nitrate loading calculations).

The SBR facility, including the disinfection tanks, sludge ponds, and controls, will be housed in a low profile barn-like or residential building. The building will be one story and will only occupy an approximately 40' by 40' footprint. The minimal land coverage, adjacent wetland area, low building profile, and screening provided by a frontage berm along Turlock Avenue all will combine to make the facility inconspicuous.

Treatment Process Description

Basically, the proposed SBR is a one-tank batch treatment process which uses jet aeration and an arrangement of baffles to carry the wastewater through all the processes: biological oxidation, sedimentation, nitrification, and dentrification. These processes occur in a timed sequence during five basic operational modes or periods: (1) fill, (2) react, (3) settle, (4) decant, and (5) idle. Sludge is pumped from the SBR tank to a sludge holding pond, where it is further treated and reduced in volume. Sludge removal from the sludge holding pond will be required infrequently; approximately 3,000 gallons of sludge will be removed every 3 months by tanker truck and taken to an approved municipal treatment facility with sludge processing capabilities. For a detailed description of the SBR Treatment Process, please refer to the information in the Appendix provided by Fluidyne Corp., a leader in the sewage treatment industry.

SBR Conceptual Design Data

Based upon our review of the proposed Lion's Gate Reserve development, and the previously prepared wastewater generation summary table (Table 1.0). We propose a single cell Sequential Batch Reactor (SBR) system with an design flow treatment capacity of 30,000 gallons per day (gpd).

The projected SBR system includes the following elements and design flow rates:

• Bar Screen	50 gpm
• Wetwell Lift Station (2)- 2 hp pumps	50 gpm
• 16' x 48 x 17' SBR Treatment Tank (1) 5 hp jet pump, (2) 5 hp blowers	30,000 gpd
• 16' x 24' x 12' Disinfection Tank	30,000 gal
• Sludge Digester/Emergency Storage Reservoir	40,000 gal
• Effluent Discharge Pump station	275 gpm
• Wet Weather Effluent Storage	6.4 ac-ft

The proposed SBR treatment system will provide an advanced level of treatment to provide high quality of effluent suitable for reuse for all types of irrigation. The preliminary design parameters for the SBR as listed below will exceed the established reuse requirements.

Criteria	Influent Design Data	SBR Treated Effluent Quality	% Removed	Typical Treatment Requirements
Flow Average Day gpd)	25,000	25,000	-	-
Flow Max. Day (gpd)	50,000	50,000	-	-
BOD (mg/l)	300	<5	>95	<30
TSS (mg/l)	250	<5	>95	<30
TN (mg/l)	40	<2	>90	<10

It is evident that the SBR treatment process exceeds typical treatment quality requirements. The high level of nitrate removal is notable and especially important to this site because of the existing groundwater contamination. And, with proper calibration, operation, and maintenance of the SBR system, the above treatment performance can be exceeded.

Disinfection

Effluent discharged from the SBR during the decant cycle will pass through the disinfection tank. The disinfection tank will provide approximately 6 hours of contact time prior to discharge to the wetlands treatment cell. Disinfection will be accomplished by either UV, or ozone methods. Preliminary feasibility analysis suggests that the disinfection method may be a combination of ozone (O2-O3 aeration) and ultraviolet disinfection as required. Disinfection goals are to meet the requirements for total coliform count < 2.2/100 ml. With the use of ozone and/or UV disinfection systems there will be no creation of toxic THM's or other chlorine by/products, thus eliminating any need for dechlorination.

Effluent Polishing - Freewater Surface Wetland Treatment System

The effluent from the SBR disinfection system will flow by gravity through a polishing cell, where bio-filtration and wetlands biological dentrification occurs. The system will consist of a lined area with freewater surface treatment wetlands and irrigation storage. The wetlands treatment cell will be approximately 0.5 to 0.75 acres in size and approximately 2 feet deep. The wetlands will provide a five day treatment retention time (at average effluent discharge rates) prior to discharge into the storage reservoir portion of the wetlands.

The wetlands are for polishing of the effluent only, and are not relied upon to meet the SBR treatment goals. The constructed wetlands will be planted with effective wetland plants to polish, filter, and treat the water through a variety of biological, chemical, and physical processes. Wetlands have proven especially effective for the reduction of nutrient levels (Gerald Moshiri, Ph.D. et al., 1993). The wetland plants will be selected based on indigence, local availability, treatment system functionality, and aesthetics. Thus, the wetlands will have a natural, aesthetically pleasing appearance and will appear to be part of the natural treatment system.

Title 22 Compliance for Effluent Reuse:

The treated effluent from the SBR in the wetlands area will be monitored to meet Title 22 requirements for irrigation reuse (Appendix. Table 3.0 - "California Code Summary of Title 22 Treatment and Water Quality Requirements"). The water will be disinfected to the coliform count of < 2.2/100 ml (Class 2), which exceeds the requirement for limited access landscape irrigation.

The project effluent will be used for irrigation of the project landscape buffer and equestrian grazing area along the east and south-east of the project. The effluent irrigation area requires a maximum area of 8 acres based upon winter irrigation rates.

III. OPERATIONAL ISSUES

Reliability: Extensive reliability measures have been incorporated into the treatment plant design. The wet well will provide a safety margin of storage volume for primary effluent storage.

Emergency storage will be provided by the sludge/containment pond and the lined wetland pond. California Title 22 Code, Division 4 requires that "where short term storage retention or disposal provisions are used as a reliability feature, these shall consist of facilities reserved for... storing or disposing of ...wastewater for at least a 24-hour period." The sludge pond, with 40,000 gallon capacity, will provide 24-hour emergency storage for untreated wastewater and act as a standby primary and sedimentation unit process facility. As an additional reliability measure for an extreme emergency, the treatment facility will have the ability to store untreated wastewater in the lined wetlands area, thus providing a 20-day emergency storage volume.

The treatment plant effluent disposal reliability, in addition to the site irrigation, is further provided by the ability to store effluent for over 120 days during wet weather months in an adjacent storage area. The equivalent 120-day winter effluent volume of approximately 6.4 acre-feet can be held in an approximately 1.75 acre containment area adjacent to the landscape buffer which will utilize the effluent. The normally dry storage area shall be lined with either clay or PVC depending upon soil suitability; the liner will be backfilled with a minimum of 18 inches of soil and landscaped to blend with the surrounding area.

Potential Flooding: Neither the SBR facility/building or the wetlands will be susceptible to flooding during a major storm event, since the entire lined wetland area and the SBR facility will be elevated above the 100-year storm event. The adjacent lake will be constructed with sufficient berming to prevent inundation outside the lake during the 100-year storm event. Current proposed flood control improvements and site grading will significantly reduce this flooding (please refer to Lion's Gate Master Drainage Report). However, in the absence of such improvements, the facilities will all be constructed on pads above the 100-year flood elevation.

Back-up Power Supply: A back-up power supply in the form of a portable or in-place diesel/propane (respectively) generator will be provided in the event of an extended power failure. The back-up generator shall be sized to provide a minimum of 480 VAC, 60 kW.

Solids Disposal: Plant headworks screenings shall be collected from the bar screen and stored in rubbish containers and disposed of properly in a sanitary landfill. The sludge removed from the SBR cell will be processed in the sludge digester basin and thickened. It is estimated that approximately 3,000 gallons of liquid sludge will be removed from the sludge digester every 3 months of operation. The liquid sludge will be transported in tanker trucks to a nearby large scale municipal treatment facility for sludge processing and disposal. Sludge processing is an ongoing process at large scale facilities with belt presses and/or sludge drying beds. The transported sludge is highly aerated and easily introduced into the processing system. In contrast to the sludge hauling, the previously proposed system of septic tanks would require hauling of septage to nearby treatment facilities. Septage is in an anaerobic condition and is not compatible for easy disposal in most activated oxygen type treatment facilities. The facilities have to introduce the septage slowly, so as not to upset the balance in the treatment system bio-mass. Overall, disposal of sludge is preferable to septic tank's solid waste.

Earthquake Safety: The treatment facility will be designed and constructed so that, in the event of a major catastrophe such as an earthquake, spill of untreated sewage would only occur into lined, contained areas (e.g., the wetlands). In addition, the treatment system tanks are mostly below ground level, thus minimizing the risk of a spill.

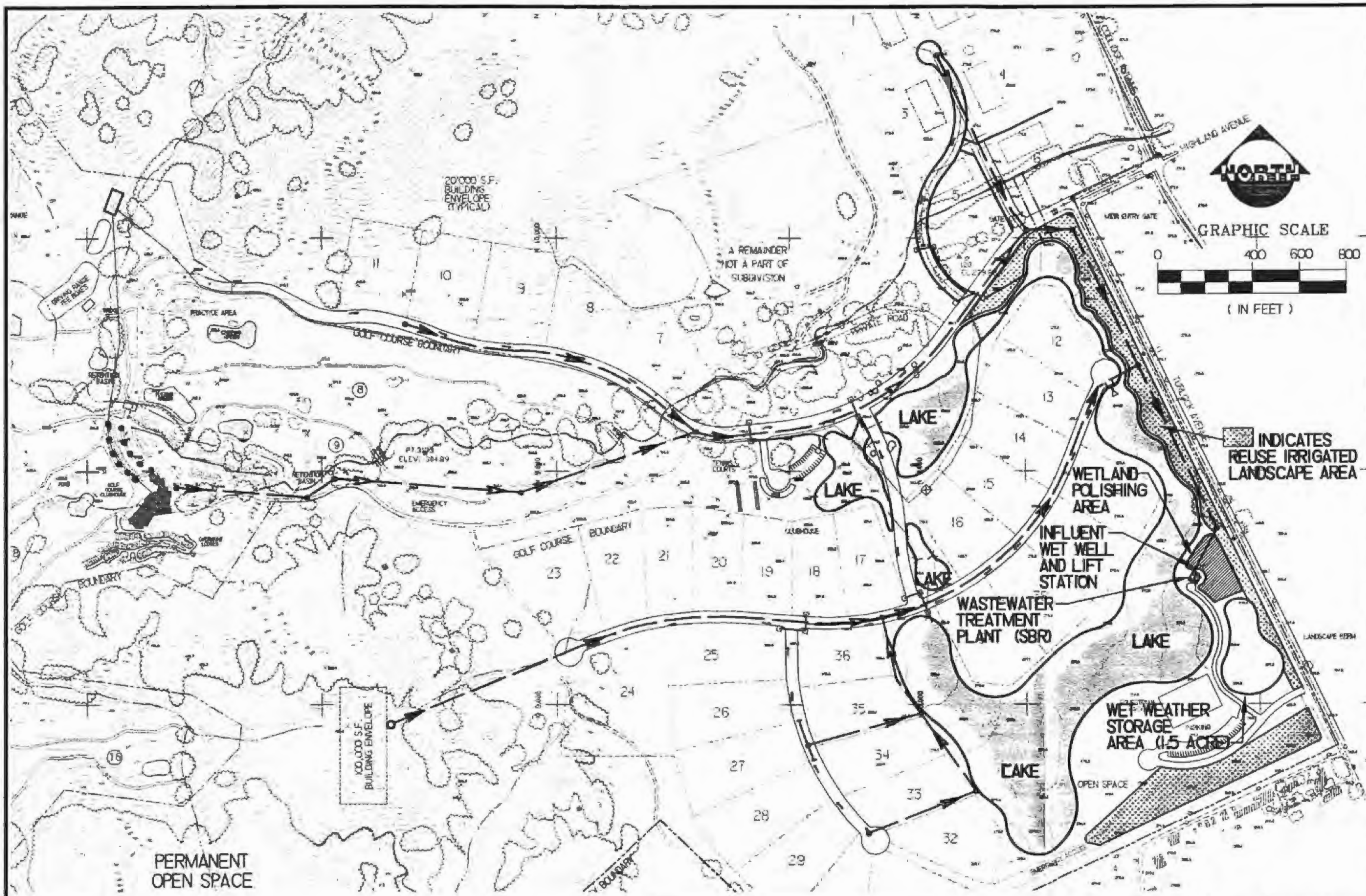
Operations and Maintenance: The plant will be operated by a certified operator as required by the Regional Water Quality Board. It is assumed that the CSD will contract with an operations individual or company to operate and maintain the facility. Testing and regularly scheduled maintenance should require less than 20 hours per week for a well trained individual with maintenance help as required. The SBR equipment manufacturer will provide a detailed operation and maintenance manual including regularly scheduled maintenance items such as dissolved oxygen sensor calibration, etc.

Testing and Water Quality: The licensed plant operator will provide an approved laboratory with water samples for testing as required by the Title 22 standards.

Environmental Issues:

Nitrates: The groundwater nitrate levels is a significant environmental issue. The SBR treatment combined with wetlands polishing will optimize nitrate removal levels. As previously stated, all previous EIR recommendations for groundwater quality assurance should be followed. As previously required, a provision for a downstream groundwater monitoring well should be included.

Odors: The SBR treatment process utilizes a significant amount of aeration and the treatment process occurs below water level the potential for odors is minimal. Also, the sludge is in an aerated liquid state while on site and when removed from sludge disposal, thereby reducing the potential for odor concerns. The entire treatment facility will be enclosed in a structure to further eliminate the potential for odor dispersion. This method has been used successfully at the Hollister, California wastewater treatment plant (see enclosed photo in Appendix). The Hollister facility is completely enclosed and located in a residential neighborhood and has no mechanical air scrubber system or odor problem.



LEGEND

— → — — GRAVITY SEWER

PACE PACIFIC ADVANCED CIVIL ENGINEERING	LIONS GATE GILROY, CALIF.	WASTEWATER FACILITIES CONCEPT PLAN	W.P.R. M.J. W.P.R. 12-09-96	FIGURE 1

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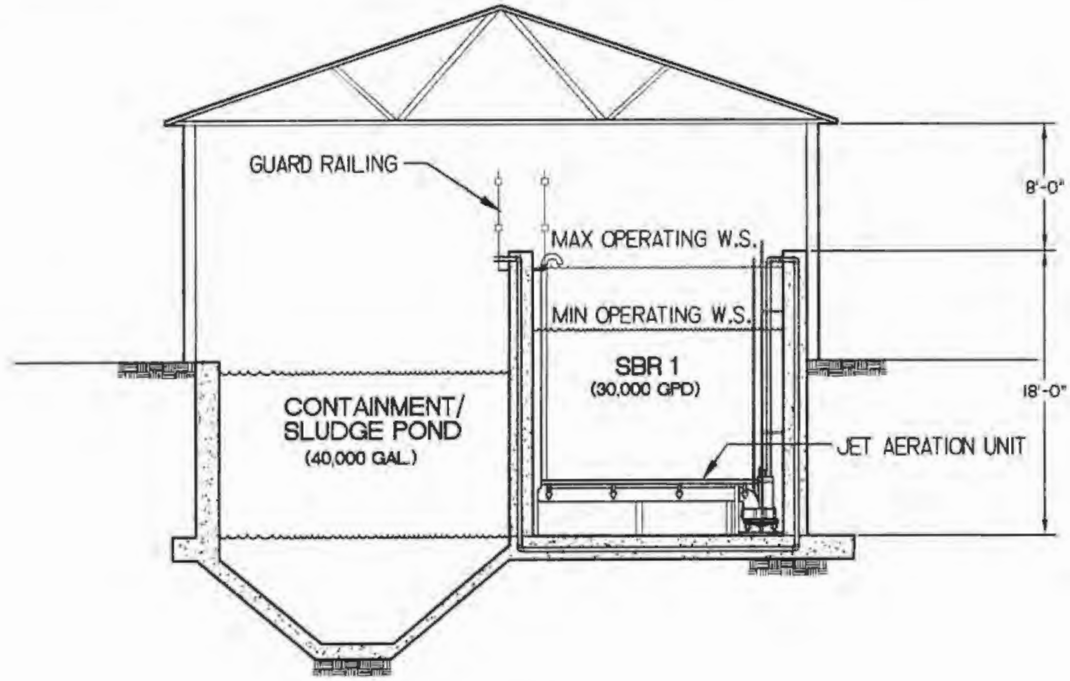

SECTION A

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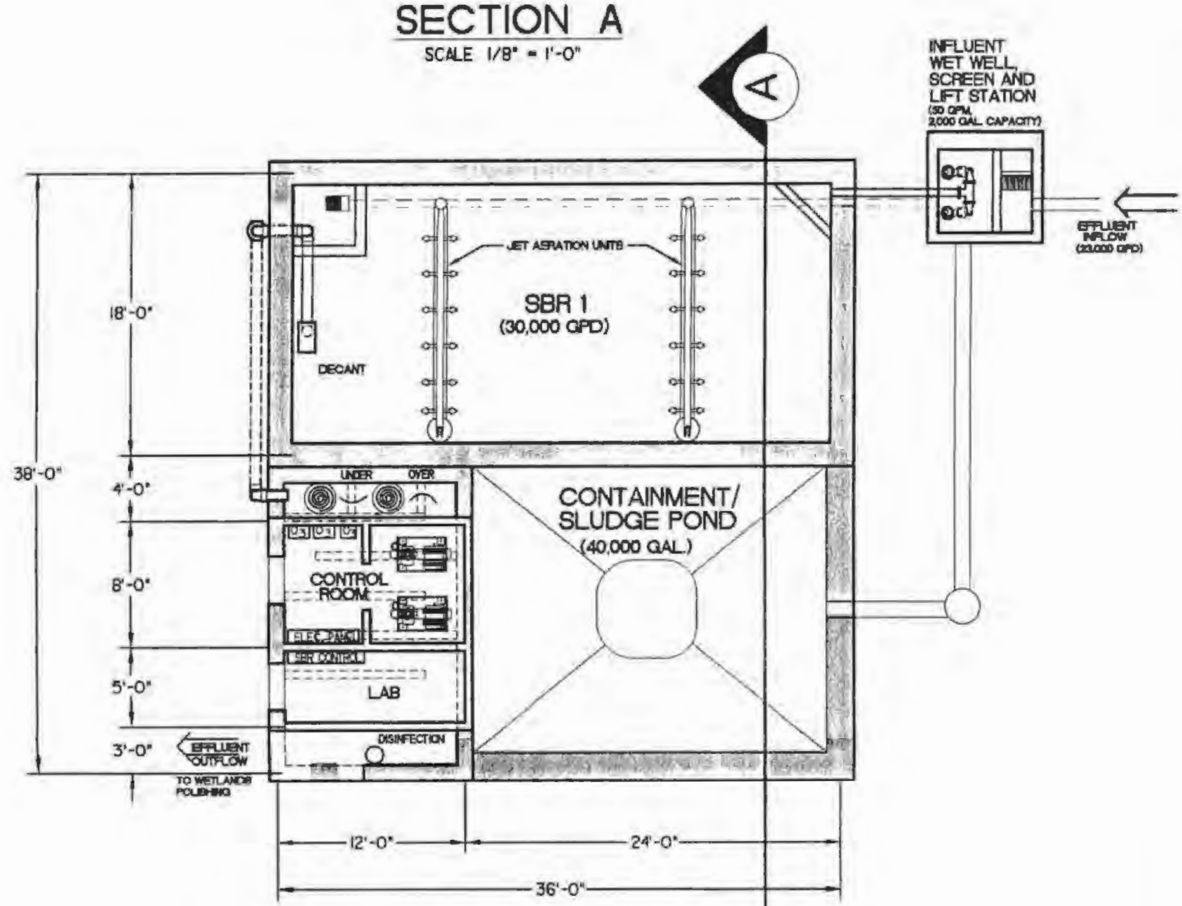

PLAN

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SANTA CLARA COUNTY CALIF.		DRAWN: M.J.	17902 GEORGETOWN LANE H.B., CA. 92647 (714) 843-5734 FAX 848-4820	FIGURE
		CHKD: W.P.R.		
		DATE: 12-05-96		
		JOB No. 6785-10		

PATH = O:\6785\6785-10\WORKDWGS\678510-SBR.DWG PLOT DATE 12/05/96

APPENDIX

- Table 1.0 - Wastewater Generation Data
- Table 2.0 - Advantages of SBR Wastewater Treatment Systems
- Table 3.0 - California Title 22 Code Summary
- Nitrate Loading Calculations
- Fluidyne SBR Treatment System Information

Table 1.0
Wastewater Generation Data

NOTE: TEXT AND TABLE TAKEN FROM LION'S GATE RESERVE EIR APPENDIX M "WASTEWATER FEASIBILITY STUDY FOR LION'S GATE RESERVE SANTA CLARA COUNTY, CALIFORNIA" BY QUESTA ENGINEERING CORPORATION DECEMBER 1995.

The total estimated wastewater flows are summarized in Table 17. Based on the above generation rates, the total wastewater flow for the Lion's Gate project is estimated to be approximately 23,000 gpd. This includes a contingency of approximately 5 percent to account for uncertainties about the specific details of project facilities that would not be determined until the design stage. Final wastewater facility design would also need to anticipate and provide for peak flow conditions which, on a daily basis, may be in order of 25 to 30 percent higher than the average daily flow. For the proposed project this translates to a peak system flow estimate of about 30,000 gpd.**

TABLE 17
ESTIMATED WASTEWATER FLOWS*

Residences	41 houses	250 gpd	10,250
Golf Course			
Clubhouse	200 meals	10 gal/meal	2,000
• Restaurant			
• Golfers	200	5 gpd	1,000
• Restroom	20	25 gpd	500
• Showers	30	15 gpd	450
• Employees			
Overnight Units	45 rooms	150 gpd	6,750
Practice Range	50 golfers	3 gpd	150
Equestrian Center	25 visitors	10 gpd	250
Subtotal			22,000
Contingency			1,000
Total Project			23.000
*This does not include the wastewater flows for the golf course maintenance building (approximately 300 gpd) which would be served by an individual septic system.			

**Note: System design hydraulic capacity of 2 x average day.

Table 2.0
Advantages of Fluidyne SBR Wastewater Treatment System

The U.S. Environmental Protection Agency (EPA) has published reports Regarding Sequencing Batch Reactions (SBR's) stating the following treatment system highlights.

1. SBR's provide advanced level treatment and can meet varied and stringent water quality objectives (i.e. peak shaving, nitrate and phosphorous removal, etc.) by simply changing operational strategies or reprogramming the plant software. This is in contrast to conventional plants which would require major expenditures of capital to build larger facilities for advanced treatment.

2. Inherent to the SBR design is it's ability to provide equalization of both flow and quality, and SBR's are generally free from surges, short circuiting and other problems typically seen in conventional plants.

3. SBR plants are reported simpler to operate than conventional plants by a ratio of about 2:1.
   - SBR's require less equipment
   - SBR's require less capital cost
   - SBR's have lower maintenance, labor and material cost.
   - SBR's use less power to operate
   - SBR's total operating cost is lower
   - SBR's are fully automated
   - SBR's seldom require repairs. If necessary, however, repairs can usually be accomplished without any plant down time.

4. In several cases SBR's were constructed instead of continuous flow plants because of the large savings in capitol costs. Savings were important since several plants were 100% privately funded. The cost of a SBR system is about one-half of the cost of a conventional system of similar treatment ability and capacity.

5. Minimal operation complexity along with minimal maintenance time is required for SBR system operation (the 1.0 MGD EPA funded plant in Idaho Springs, Colorado, requires an operator only for about 2 days per week).

6. The total area space required for a SBR is significantly less than for a conventional system.

7. With the SBR design odor is virtually non-existent and plant effluent water quality can be maintained at drinking water standards, including very low nutrient levels which may be the most important factor for discharge and reuse/recharge.

8. The SBR design includes minimal open water areas, thus minimizing effluent evaporation and other losses and maximizes the available effluent for reuse. All water is a resource and the SBR technology conserves it and provides the highest quality treatment available.

9. SBR's produce higher quality effluent without addition of chemicals.
   - SBR's have easier settling floc without the addition of chemicals.
   - SBR's water effluent is so solids-free that it is much easier to filter the effluent if required.

10. SBR's can be programmed to deal with varying degrees of high BOD and suspended solids. SBR's are much less susceptible to system upsets cased by uneven strengths in the influent flow cycles.

11. SBR's are easily expandable to handle additional capacity.

Table 3.0

CALIFORNIA CODE SUMMARY OF TITLE 22 TREATMENT AND WATER QUALITY REQUIREMENTS

Reclamation Alternative	Treatment and Effluent Quality Requirement*	Reclaimed Water Class
Golf course (with contiguous homes), parks, playgrounds and schoolyard irrigation	Tertiary treatment (oxidation, coagulation, clarification, filtration and disinfection); 7-day median # of coliforms ≤ 2.2 per 100 mil, plus maximum of 23/100 ml. in any one sample.	I
Recreation impoundment (non-restricted access)	Tertiary treatment (oxidation, coagulation, clarification, filtration and disinfection); 7-day median # of coliforms ≤ 2.2 per 100 mil, plus maximum of 23/100 mil in no more than 1 sample in a 30 day period.	I
Agricultural food crops for human consumption #	Secondary to tertiary treatment, (extent of treatment varies depending on type of crop and application)	II or I
Recreation impoundment (restricted access)	Secondary treatment (oxidation and disinfection); total effluent coliform <2.2/100 ml, median 7 day.	II
Landscape impoundment	Secondary treatment (oxidation and disinfection); total effluent coliform <23/100 ml, median 7 day.	III
Pasture for milking animals	Secondary treatment (oxidation and disinfection); total effluent coliform <23/100 ml, median 7 day.	III
Golf course, (without contiguous homes), cemetery, freeway, median, and limited access landscape irrigation	Secondary treatment (oxidation and disinfection); total effluent coliform <23/100 ml, median 7 day, plus maximum of 240/100 ml in any 2 samples.	III
Fodder, fiber and seed crops, orchards and vineyards	Primary treatment (screened).	**

* Total effluent coliform requirements refers to a 7 day median value.

** Title 22, in its current form, allows primary effluent for this type of reuse, but in practice, secondary effluent (Class II) is typically required.

\# Reclaimed water not allowed for some crops, such as rice.

Lion's Gate Project
Nitrate Loading Calculations
for
Wastewater Plus Golf Course Fertilizer

Assumptions

- Golf Course Fertilizer Leached (F): 262 lbs to 1,965 lbs (per Audobon Cooperative Sanctuary System)
- Total Annual Recharge Volume (R): 51.9 million gallons (per Audobon Cooperative Sanctuary System)
- Total Nitrogen (N_2) in Secondary Treated Effluent: 2 mg/l
- Wastewater Nitrogen Reduction Through Pond Storage (P): 40%
- Wastewater Nitrogen Reduction Through Plant Uptake and Soil Dentrification (I): 75%
- Average Wastewater Flow = 23,000 gpd = 8.4 million gallons/year.

Calculations

1. Wastewater Nitrogen Leached (W)

$W = 8.34\ ((N_2)\ (1 - P)\ (1 - I)\ (8.4 \text{ million gallons})$
$W = (8.34)\ (2 \text{ mg/l})\ (1 - 0.4)\ (I - 0.75)\ (8.4)$
$W = 21 \text{ lbs/year}$

2. Total Combined NO_3 - N Concentration in Recharge Water:

$$N_c = \frac{W + F}{(8.34)\ (R)}$$

$$N_c = \frac{27 + 262}{(8.34)\ (51.9)}$$

$N_c =$ 0.65 mg/l NO_3 - N Low Estimate

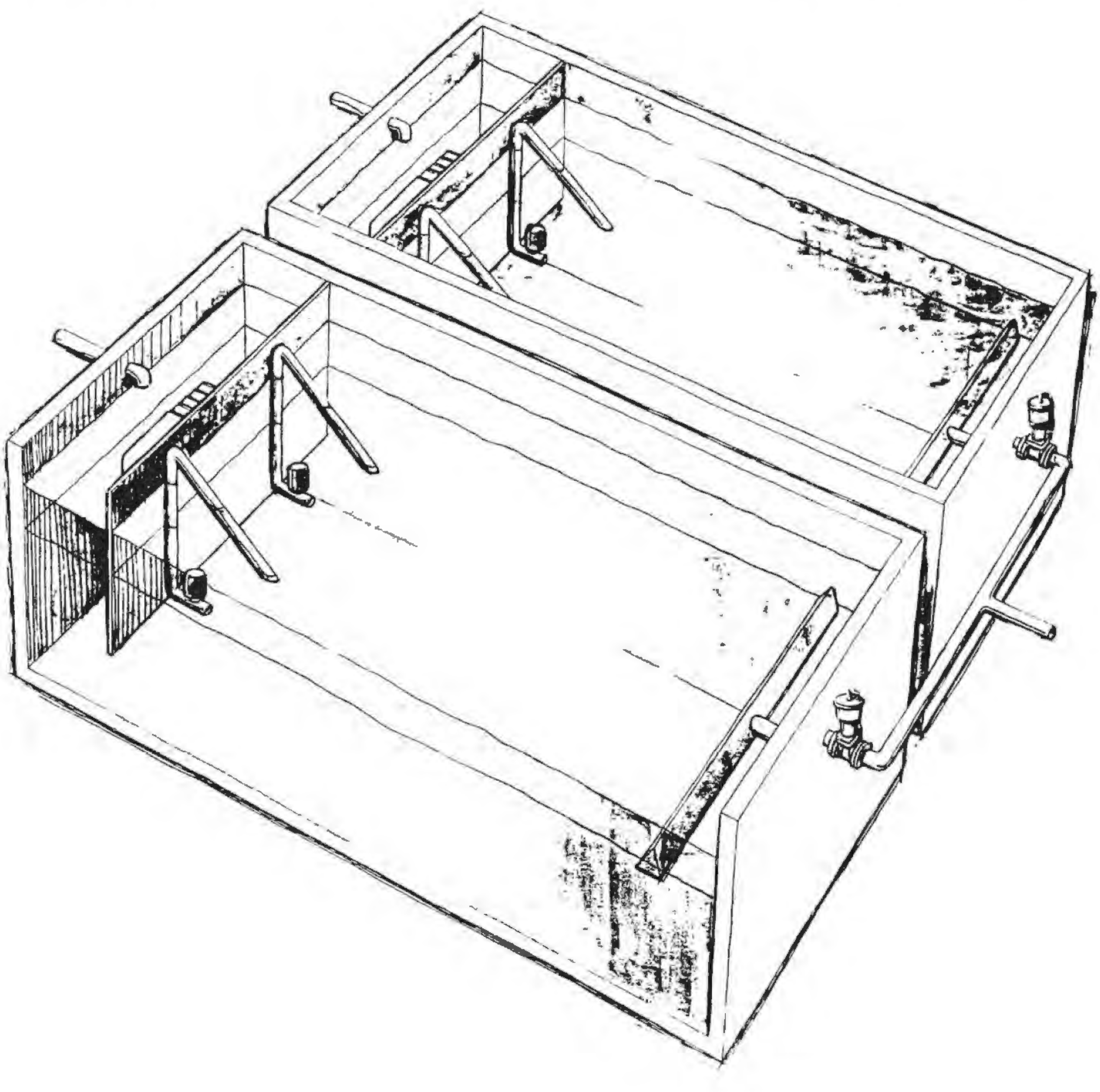
to

$$N_c = \frac{21 + 1{,}965}{(8.34)\ (51.9)}$$

$N_c =$ 4.59 mg/l NO_3 - N High Estimate

FLUIDYNE

Sequencing Batch Reactor for economical, reliable, advanced wastewater treatment



FLUIDYNE

A low cost, easily controlled system

Fluidyne's unique Sequencing Batch Reactor (SBR) System answers the need for a reliable yet easily controlled waste water treatment system that fits within limited budgets.

The SBR is particularly suited for systems:

1) with a wide range of inflow and/or organic loadings;
2) requiring minimal operator attention;
3) requiring extremely close control of effluent quality, such as for removal of specific components; and
4) in small to medium size communities and industries such as food processing.

Innovation rooted in proven concepts

Fluidyne's Sequencing Batch Reactor represents an innovation in the field — but the concept of treating wastewater by the batch goes way back. In fact, the original (1914) activated sludge plants **were** batch operations. The switch to the now-conventional continuous flow methodology was largely made to solve mechanical difficulties (diffuser plugging) and reduce the supervision required by the then inadequate batch control systems.

The Fluidyne SBR System gives you the benefits of high quality, low cost batch treatment without the original disadvantages. Aeration is by large-orifice jet mixers (also used in hundreds of conventional plants) which resist clogging as well as create an extremely high rate of oxidation. Supervision is simplified by use of a pre-programmed panel which controls all functions.

No clarifier, sludge recycle pump stations, sludge return pumps or bridgework are involved, so construction costs are minimized. Tank walls can be reinforced concrete or steel. No rotating shafts, gear drives or submerged bearings are used, so maintenance costs are low, too. Energy needs are also very low.

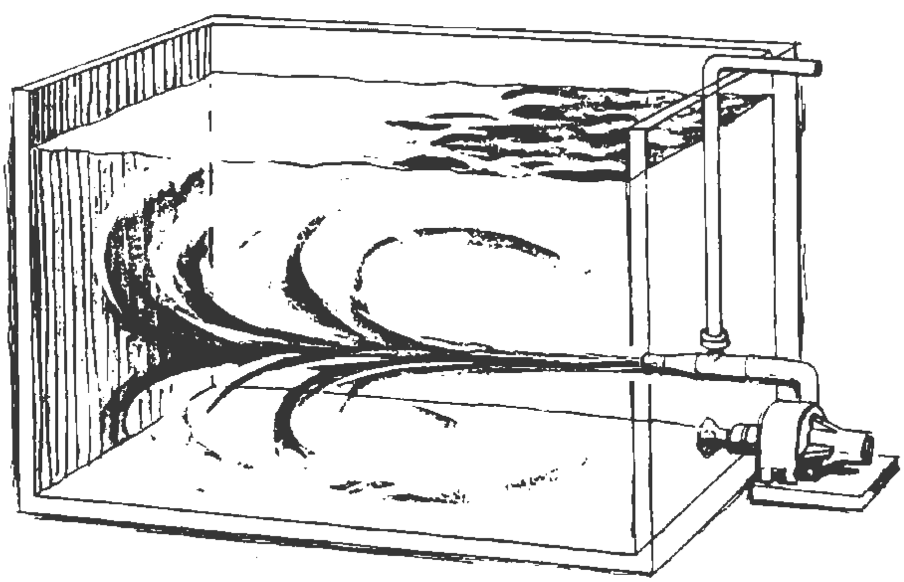
Basically, it's a one-tank system

Conventional continuous-flow treatment systems employ separate staged tanks arranged in a series to process wastewater.

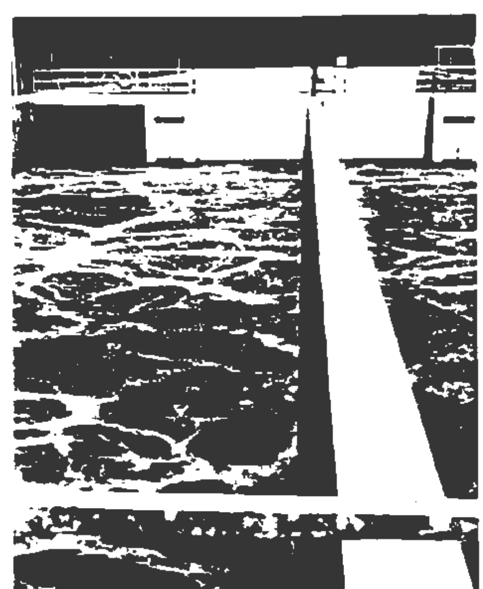
A Fluidyne SBR System does it all in just one tank. You may put several SBR tanks in operation, but that's modular adjustment to capacity needs.

Each SBR tank is equipped with a jet aerator and an arrangement of baffles to carry wastewater through all processes — biological oxidation, sedimentation, nitrification and denitrification. These processes occur in a timed sequence during five basic operating modes or periods: (1) fill, (2) react, (3) settle, (4) draw and (5) idle (anoxic fill).

According to control panel programming, the fill period includes contact with micro-organisms, mixing and — for at least part of the period — aeration. (Aeration may be stopped sometime during the fill to promote settling and/or

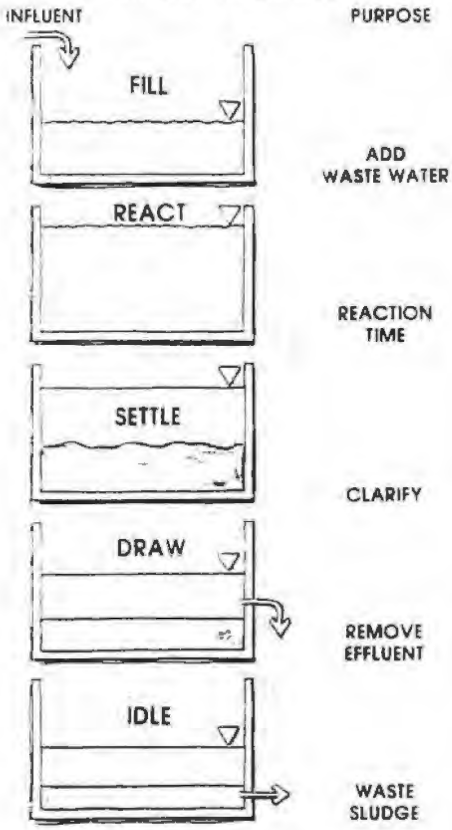


In aerating modes, tank contents are pumped through the jet's inner nozzle into a suction chamber, drawing and mixing with air from an air line, and are then ejected from the larger nozzle into the main tank volume. The resulting homogeneous **fine bubble** entrainment produces a high oxygen-liquid transfer while imparting movement to the tank. The air is stopped during mix-only modes.



Fluidyne's Sequencing Batch Reactor System consists essentially of jet mixer and pump assemblies (one on standby), collecting decanter, a control panel and an arrangement of baffles within a tank. Low-pressure blowers are supplied as part of the jet aeration system for larger plants.

SINGLE TANK SBR



denitrification.) The air supply and mixing are adjusted during the react period. Then the tank is allowed to settle, leaving clarified water to be decanted during the draw. Mixing and aeration of the remaining sludge can be resumed during the idle period, while waiting for new influent.

In a multiple SBR system, different tanks will be in different modes, with incoming wastewater directed to the first idling unit. A single-tank SBR system can be adapted for either a continuous or non-continuous inflow.

Sludge wasting needs range from the infrequent in low-yield single tank systems: to once each cycle in high-yield multiple tank systems.

You get a system to suit

We can adapt a Fluidyne SBR System to a wide variety of plant sizes, wastewater characteristics and effluent requirements — in rectangular tanks, circular basins or oxidation ditches.

We custom design the larger installations and can help you with everything from initial design through start-up. However, we also offer SBR package plants (including tankage) in modules for inflows of 5,000 to 30,000 GPD and SBR pre-engineered plants for in-flows of 20,000 to 100,000 GPD. (See back of this brochure for details and sizing information.)

Components common to all Fluidyne SBR Systems include:

Mixers and aerators — jet nozzle, operating with and without air, providing aerobic oxidation or anoxic mixing. Two jets typically supplied per module; one operates while the other serves as 100% in-place standby.

Decant system — designed to decant clear liquid without scum or disturbance of settled sludge. Handles peak hydraulic flows occuring during storm cycle. Solids excluding design eliminates solids accumulation during react period.

Discharge control system — an innovative, reliable system to meet a variety of discharge requirements. Automatic operation — may be siphon, pump or valve to meet the individual application need.

Process control panel — directs sequential operation of aerators and discharge control valve according to the selected program. A proper sequence will be set during start-up; however, the operator can easily reset for a new sequence.

Influent bar screen — oversized to prevent clogging of jet nozzle and pumps.

Advantages of the SBR System process:

• Jets improve process stability through more effective mixing. The superior process kinetics of the SBR increase biomass activity, providing reaction enhancement. The inherent equalization capability buffers organic or toxic shock loads. The ability to hold without discharging offers the possibility of treatment to a desired level prior to discharge.

• Baffled or sequencing tank design eliminates short circuiting of influent, promotes a fast settling biological floc (low SVI) and enhances substrate utilization.

• Sequencing operation adds to control of shock loads and greatly increases surface settling area for liquid-solids separation.

• Automatic control provides a flexible response to varying load conditions or production schedules, while reducing the operator attention.

• The design eliminates the conventional overflow clarifier and gives simpler and more positive biological solids control. It also eliminates sludge return pumping stations and difficult-to-control "common baffle" sludge return systems.

continued

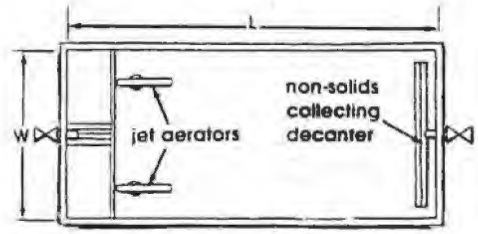
SBR System Advantages *(continued)*

- Fogging, splashing and icing problems associated with surface entrainment aeration are avoided.
- All operating equipment is easily accessible and serviceable. No extended shafts or high maintenance gear drives are used. Retrievable submersible pumps can be serviced locally.
- The system is safer than conventional plants. No personnel work above the tank liquids, no exposed rotating devices are used.
- Jet mixing is highly energy efficient since almost all pumping energy converts to mixing energy. Less horsepower is needed to do the same work than with other systems.

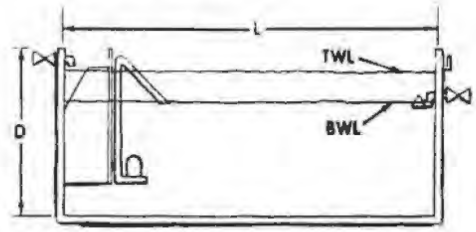
Sizing a standard Fluidyne SBR System

Standard SBR Package Plants and SBR Pre-engineered Plants are available from Fluidyne. The difference between the two types is that Package Plants are furnished with FRP or epoxy-coated steel tankage while Pre-engineered Plants for the larger inflows are supplied less the required concrete tankage. Otherwise both types come complete with all needed mechanical and control components plus any requested design and start-up assistance.

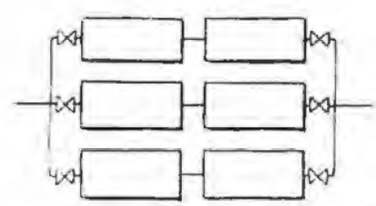
Remember, the plants listed in the charts are modules. You can build larger systems by applying two or more modules.



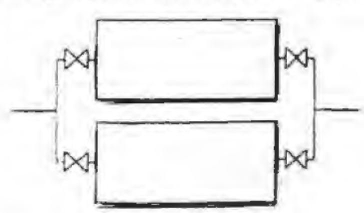
SBR Pre-engineered Plant modules fit in rectangular open concrete tanks, new or existing, provided by others.



SBR Package Plant module featuring horizontal enclosed tank.



Arrange package modules to fit capacity needs — such as three trains of SBR-25's in two stages to build a 150,000 GPD plant.



SBR-1000's in tandem tanks create a 200,000 GPD plant.

STANDARD FLUIDYNE SBR PACKAGE PLANTS
Horizontal enclosed tank, unless otherwise indicated

Model no.	Pop. Equiv.	Flow, GPD @ 100 G/C/D	BOD_5 lbs/D @ 200 mg/l	Tank D x L, ft	Tank vol. usable gal	Pump/aerator HP
SBR-5V*	50	5,000	8.3	11 x 11	7,000	2
SBR-10V*	100	10,000	16.7	11 x 17	11,400	2
SBR-10	100	10,000	16.7	11 x 17	11,400	2
SBR-15	150	15,000	25.0	11 x 27	18,300	3
SBR-20	200	20,000	33.3	11 x 36	24,400	5
SBR-25	250	25,000	41.7	11 x 43	29,100	5
SBR-30	300*	30,000	50.0	12 x 43	34,500	7.5

*Vertical open-top tank

Design is based on influent containing 200 mg/l BOD_5 and 40 mg/l TKN, assuming 100% nitrification and 40% denitrification. Peak sustained flow capability is 2.8 x design flow, peak biosorption flow capability is 4.3 x design. If plantsite is over 2000 ft. elevation, use next size larger aeration system.

STANDARD FLUIDYNE SBR PRE-ENGINEERED PLANTS
Operating equipment — concrete tank by owner

Model no.	Pop. Equiv.	Flow, GPD @ 100 G/C/D	BOD_5 lbs/D @ 200 mg/l	Tank I.D. W x L x H, ft	Tank vol. usable gal	Pump/aerator HP
SBR-200	200	20,000	33	10 x 21 x 17	26,300	5
SBR-300	300	30,000	50	12 x 27 x 17	40,500	7.5
SBR-400	400	40,000	67	14 x 30 x 17	52,500	10
SBR-500	500	50,000	83	14 x 36 x 17	63,000	15
SBR-750	750	75,000	125	14 x 52 x 17	91,000	20
SBR-1000	1000	100,000	167	16 x 60 x 17	120,000	30

Design basis is the same as for Fluidyne SBR Package Plants.
Auxiliary aeration system available for BOD_5 and TKN concentrations of more than 200 and 40 mg/l, respectively.
Auxiliary aeration can increase BOD_5 handling capability of any model by up to a factor of five.

FLUIDYNE
CORPORATION
Corporate and National Sales Offices
2140 South Ivanhoe St., Denver, CO 80222
(303) 758-4015 Telex 45-636

Manufacturing and Engineering Facilities located in:
Cedar Falls, IA • Davenport, IA (319) 266-9967
Export Department: Denver, CO Telex 45-636

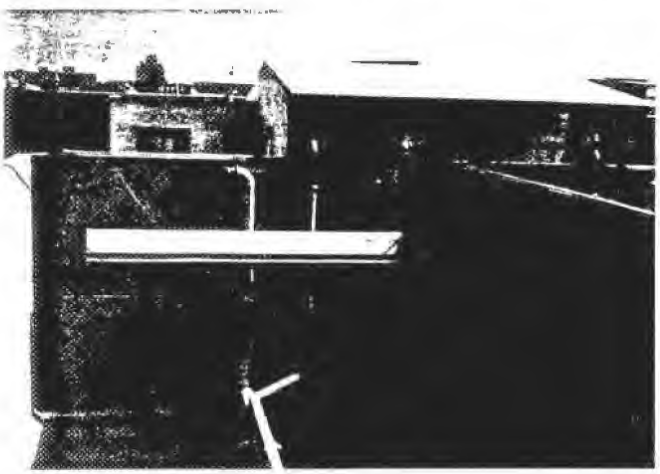
FLUIDYNE

WEF
'93 Issue

FLUIDYNE FORUM

PRIVATE GOLF COURSE IN MEXICO RECLAIMS WATER FOR IRRIGATION USE

Fluidyne Corporation has provided Campestre Torreon the first Sequencing Batch Reactor in Mexico that turns municipal wastewater into irrigation water.



The Decision:

Mexico's National Water Commission has recently implemented a series of measures intending to preserve the precious water in the area. The agency increased taxes on aquifer removal rights, encouraged utilization of treated wastewater especially for crops and gardens and promoted well water use for only human consumption in areas of most need.

The administrative body at Campestre Torreon determined that it was not possible to continue to irrigate the golf course with well water, realizing that there was a greater need for potable water in other sectors of the city. The engineers decided that they would reclaim wastewater from the city sewer and use that as their irrigation source.

After much deliberation, it was determined that the Fluidyne Sequencing Batch Reactor would be the ideal treatment system. Campestre Torreon based the decision on several key factors: (1) Ability to maintain the ecosystem in their man-made lakes due to high quality effluent, (2) Lower capital costs over other processes, (3) Minimal operator attention and time, and (4) Ability to surpass the necessary levels of BOD, TSS, and greases/oils needed for irrigation.

The Design:

Fluidyn's SBR was designed to treat 3200 m3/day (864,000 gpd) from the city's sewer line. Influent BOD, TSS, and greases/oils levels were based on 250mg/l, 300 mg/l and 100 mg/l respectively. The NWC has set standards for treated effluent used for irrigation. These are 30 mg/l BOD, 50 mg/l TSS and 20 mg/l greases and oils. Disinfection after treatment was required to control algae and bacteria growth in the lake and to eliminate the high levels of fecal coliforms.

The Process:

From the city line the raw sewage is directed to a primary basin where solids can settle before treatment. Then the liquid travels to a dual tank SBR system. As one tank fills, the other tank proceeds through the different cycles of the SBR. The contents of the tank are mixed and aerated using Fluidyne's high efficiency FRP jet headers. When a tank reaches top water level, inflow is diverted to the other tank so that biological reactions can be completed in the full tank. Then the biological solids settle and the clear liquid is decanted through a Fluidyne FRP Solids Excluding Decanter. (See plan view below). From there the decanted liquid travels through a Fluidyne FRP disinfection system where chlorine is added and mixed by a jet nozzle into a reactor tube. All the above functions are regulated by a programmable logic controller. After disinfection, the effluent flows to a storage tank and then it is pumped to a lake on the golf course where the water can be reused.

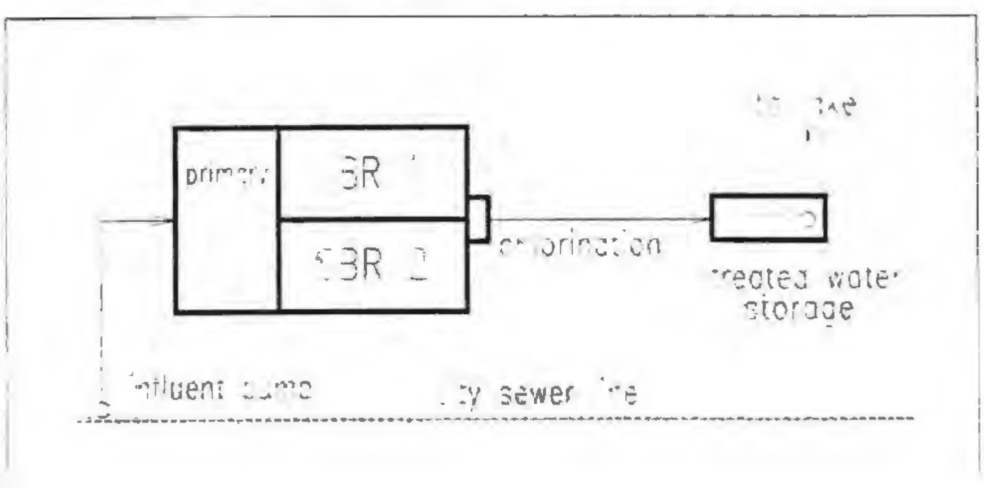
The Campestre Torreon wastewater treatment plant has now been in operation for almost a year and a half. The SBR has easily surpassed all NWC standards. Even the higher than expected grease and oil influent levels are being reduced by over 97%. In February of 1993, the Industrial Metallurgic Laboratory in Mexico tested the effluent quality. The results can be seen in the Table below.

Sample	Influent	Effluent to lake
Greases & Oils	174 mg/l	4 mg/l
TSS	290 mg/l	19 mg/l
BOD	200 mg/l	1.4 mg/l

The Conclusion:

In a country such as Mexico where water is considered so valuable, the Fluidyne SBR now allows a city to take well water that was once used for irrigation and provide it to 2500 additional families. Probably the best way to show the treated wastewater is of high quality is the presence of 3000 to 4000 migratory ducks on the irrigation lake and a thriving fish population in the lake.

The Fluidyne SBR is also beneficial to Campestre Torreon in an economical sense. The golf course now does not have to pay high fees for well water rights. Campestre Torreon expects to recover their investment with the Fluidyne SBR in four years.



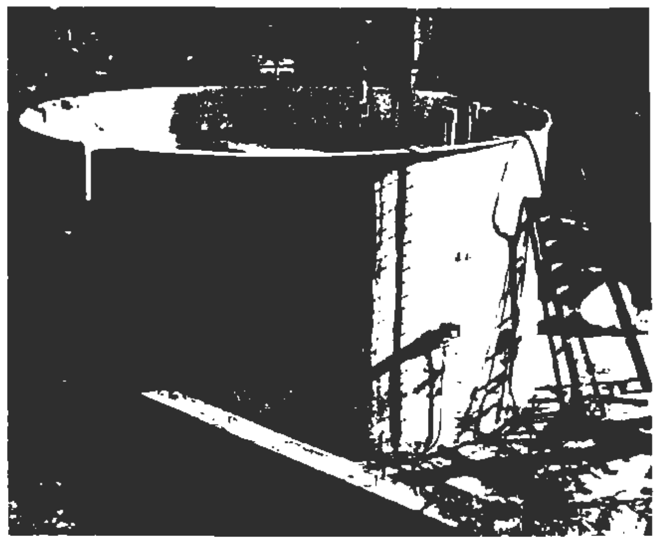
FLUIDYNE SOLVES CAMP PROBLEMS BY SWITCHING TO A SBR

Woodleaf, a Young Life camp for teenagers located in Challenge, California, had a problem with their existing treatment system. Their twenty-year old plant consisted of a septic tank followed by aerobic treatment. From there, the treated effluent was pumped to leach fields via a dosing tank. The problem was that the effluent still contained high levels of BOD, TSS and ammonia which were quickly deteriorating the leach field. Plus, during the peak months of summer terrible odors were annoying the campers.

Young Life wanted to continue on site disposal to safeguard the environment and to insure the camp as the best possible neighbor, above any reproach from downstream water users. Based on this, Fluidyne designed a SBR package that allowed for secondary effluent disposal to the existing leach fields. Woodleaf chose the Fluidyne SBR because of its reputation for high quality effluent, ability to handle variable flow conditions, and capability of removing ammonia and nitrates.

The plant was designed to remove better than 90% of BOD and Total suspended solids and to treat an ultimate flow of 40,000 gpd. However, built into the control mechanisms was a turndown capability to treat lesser flows during periods of low camp population. DO controls were included to provide the greatest oxygen-transfer efficiency.

Photo below: Woodleaf's SBR tank consists of 8 panels constructed by Fluidyne out of fiberglass reinforced polyester and installed by a Fluidyne technician on the job site. The DO controls are mounted on the exterior wall.



MINE ACCIDENT DOESN'T SLOW FLUIDYNE HYDRO-GRIT ™

Connellsville, Pennsylvania wanted a system that would successfully remove large amounts of grit from raw sewage before treatment in its 7 MGD plant. So in 1990, the city selected the Fluidyne Hydro-Grit™ based on the systems ability to separate and remove grit particles including fine grit, handle variable feed stream flow rates, and have low energy requirements. The fact that the Hydro-Grit™ was all-hydraulic, non-mechanical, and non-clogging also attracted the city.

Three years later, Fluidyne's Hydro-Grit™ System has worked above and beyond the expectations of the city of Connellsville, Pennsylvania. Early in 1993, a local contractor was grouting an underground mine and accidentally drilled through the city sewer line. As a result, several tons of fine grained coal refuse grout were carried to the wastewater treatment plant. John Tomaro of Widmer Engineering, the engineer for Connellsville, took a photograph of the Hydro-Grit™ after the coal had been removed from the influent.

In a letter to Fluidyne's sales representative, John Tomaro writes "As witnessed by the photo, the Fluidyne "hydro-grit" chamber performed better than expected in removing this fine grained material from the raw wastewater. As they say "a picture is worth a thousand words" and I would certainly specify this unit on future projects."

Photo below: Fluidyne Hydro-grit classifier after removal of the fine grained coal from Connellsville, PA sewer line.

Information on the Hydro-grit™ is available from Fluidyne or its sales representatives.

TESTS DEMONSTRATE STRENGTH OF FRP

Continued research and testing into the Fluidyne composite materials show the superior strength qualities of Fluidyne fiberglass reinforced polyester. Fluidyne has developed special composites and fabricating techniques which far exceed industry standards. These techniques are used in much of the equipment and tanks Fluidyne supplies to wastewater treatment plants.

Recent linear stress tests conducted by an independent laboratory show the superior strength of Fluidyne products. Two fiberglass samples were tested with one sample withstanding 14,300 psi and the other sample withstanding 14,100 psi. With the majority of Fluidyne's products based on a 1500 psi requirement, the tests demonstrate the durability and sturdiness of Fluidyne products.

Contact Fluidyne for detailed design information and recommendations to meet your requirements in the following areas:

- **Jet Aeration**
- **Sequencing Batch Reactors**
- **Package Treatment Plants**
- **SBR Pilot Plants**
- **Jet Mixing**
- **Jet Disinfection**
- **Grit Separation and Removal**
- **Fiberglass pipe and tanks**

FLUIDYNE

2816 West First Street
Cedar Falls, IA 50613
Phone: (319) 266-9967
Fax: (319) 277-6034

SECOND ADDENDUM TO

ENVIRONMENTAL IMPACT REPORT

LION'S GATE RESERVE

(CordeValle)

LEAD AGENCY: COUNTY OF SANTA CLARA

File #4039-67-28-93
SCH #94043016

June 1998

TABLE OF CONTENTS*

* The contents only include sections of the EIR that have been revised in this Addendum.

INTRODUCTION

Description of Project Modifications

This Second EIR Addendum has been prepared to address the changes to the Lion's Gate Reserve (Cordevalle) project that have been proposed since the time that the EIR on the project was certified by the County Board of Supervisors in August 1996 and the first EIR Addendum was prepared in January 1997.

The main changes to the project addressed in this EIR Addendum include the following: 1) relocation and redesign of the clubhouse/overnight complex; 2) modifications to the golf course plan to accommodate the relocation of the clubhouse complex; 3) elimination of the previously proposed equestrian center and its replacement with a much smaller stable near the northeastern portion of the site; 4) changes to the boundaries of the golf course parcel and the cluster subdivision/permanent open space parcels resulting from the above project modifications, and 5) modification of the proposed on-site flood control facilities such that there would be a reduction in flood flows leaving the site during frequent storm events such as the 2-year event. These project changes are described in detail below, followed by a summary evaluation of potential impacts resulting from these modifications. The changes to the EIR resulting from these project modifications are addressed in the body of this addendum. It should be noted that there are two additional new project elements which are expected to be added in the future and which are not covered in this addendum. These include a future winery/grape processing facility and a water storage tank. These future facilities are briefly described below under 'Future Project Modifications'.

Clubhouse/Overnight Complex

The clubhouse facilities, overnight guest units, and associated parking area are now proposed to be located on the northern side of the West Branch of Llagas Creek instead of the south side as previously proposed. The size of the clubhouse facility has also increased somewhat and the layout and design of the complex has also been altered to be more low profile in character with greater separation among buildings. (The site plan and elevations for the redesigned complex are included in the EIR text portion of this Addendum.) The increase in floor area for the clubhouse has been necessitated largely because the original concept plan underestimated the space requirements for the various clubhouse functions. (A detailed floor area breakdown for clubhouses functions is provided in the text of this EIR addendum.) The number of overnight units remains the same at 45; however, the total floor area of guest units is actually slightly less than originally proposed due to a reduction in meeting room space. The parking area and planned drainage improvements for the complex and parking area are also to be modified, and the total number of parking spaces has increased.

The larger overall land area required for the complex has increased for several reasons including: the clubhouse facilities are now largely planned for one main floor instead several stories as originally proposed; the guest units are now planned to consist entirely of single story units instead of the two-story buildings as originally planned; the separation among buildings has increased to create a campus-like setting; the overall square footage of the clubhouse has increased, and; the increase in parking spaces has resulted in a larger area devoted to parking.

The main changes resulting from the relocation and reconfiguration of the clubhouse/overnight complex are summarized in the table below. This table shows figures from the certified EIR (July 1996), as well as figures reflecting the design first approved by the Architectural and Site Approval Committee (ASA) in June 1997, in addition to the currently proposed changes to be considered by ASA on June 11, 1998.

	EIR (7/96)	1st ASA (6/97) (approved)	2nd ASA (6/98) (proposed changes)
Clubhouse/Overnight and Parking Acreage	6.3 acres*	15.6 acres	19.1 acres
Clubhouse Complex Floor Area	29,170 sf	±45,000 sf	55,100 sf**
Overnight Complex Floor Area	34,000 sf	±41,000 sf	32,500 sf
Parking Spaces	250	320	350

* Did not include parking area at driving range.
** Includes 3,200 sf freestanding pro shop.

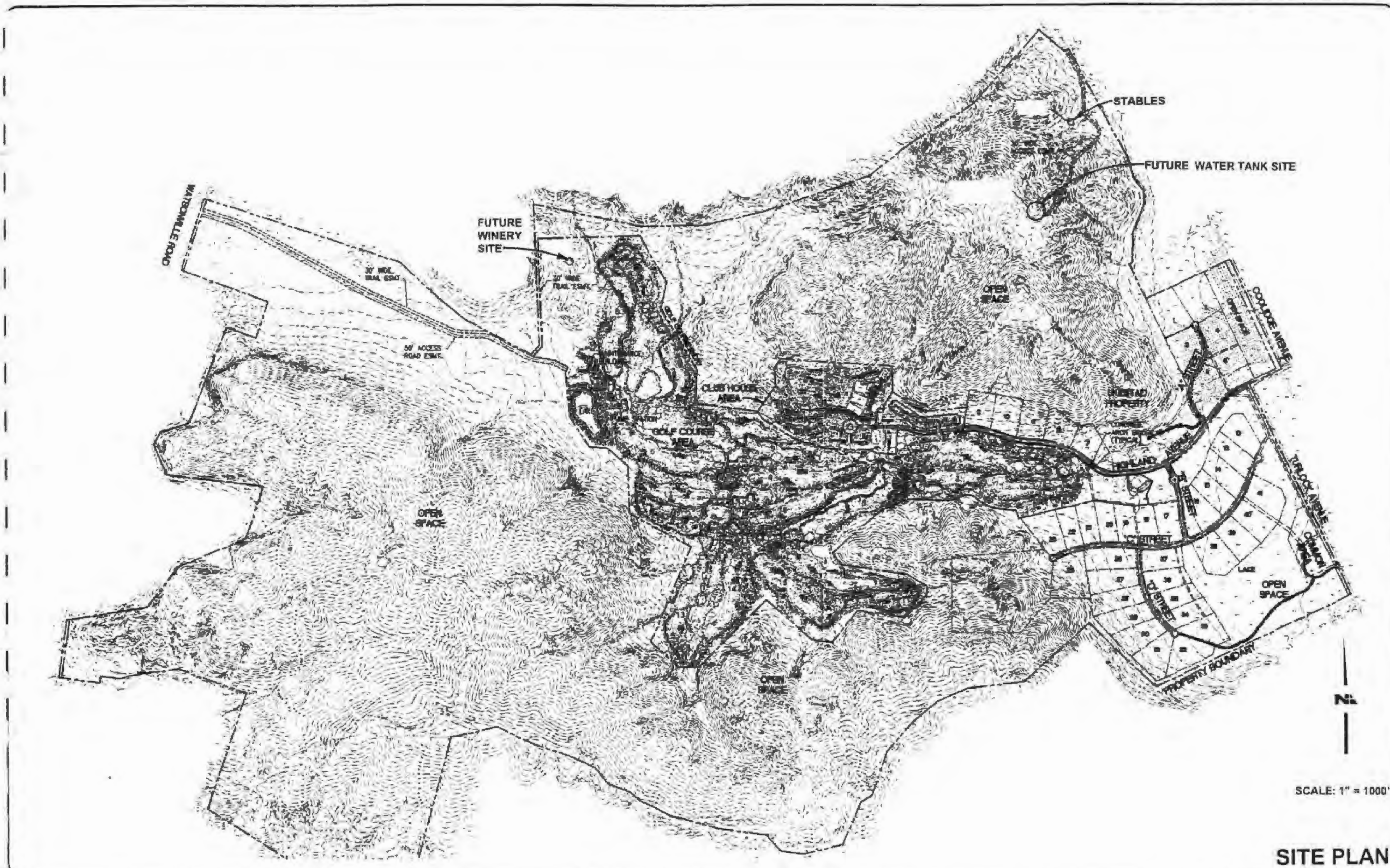
The new clubhouse location is preferred by the applicant because it provides more land area for the facilities, thus allowing for a less intense building pattern. The new site has better sun exposure with its southward orientation, and it also offers better views of the golf course and Lion's Peak, as well as better protection from the wind. The new clubhouse/overnight complex site is located to the south of a series of low ridges and hills where it is completely screened from view from off-site locations. The new site avoids the use of retaining walls, and also avoids the landslide on the adjacent hillside to the south, which required a geotechnical engineering solution for the previous clubhouse location. The clubhouse/overnight complex will be sited a minimum of 75 feet from the main creek channel and the tributary channel to the east. The new location allows the parking area to be consolidated into a single location north of the main access road, and allows siting of the parking area closer to the clubhouse area. The new parking lot location is several hundred feet from the main creek channel and 75 feet from a tributary channel at its nearest edge. No tree removal is required at the new clubhouse location.

The proposed location of the complex on the north side of the creek also eliminates the need for a vehicle bridge across the creek, as well as crossings by sanitary sewer and utility lines. The County Fire Marshal's office has indicated that the new location and configuration for the clubhouse/overnight complex is preferable to the previous plan because the shorter length of the access road would improve response times, the less steep slope of the fire access route to the overnight units improves accessibility, and the generally better accessibility of single-story structures compared to multi-story buildings proposed previously.

The new location of the clubhouse complex is partially in an area that was previously planned for golf course fairways. The necessary adjustments to the golf course plan resulting from the clubhouse relocation are described below.

<u>Golf Course Modifications</u>

Several changes to the proposed golf course layout have been made to accommodate the relocated clubhouse and overnight complex. The original hole #7 was eliminated to make way for the clubhouse which resulted in several adjustments to the layout and routing of the golf course, including some changes in golf hole numbering. To replace hole #7, a new hole (#2) is planned near the eastern end of the golf course along the south side of the main access road. The new hole #2 does not cross the main creek channel as the old hole #7 did, and thus results in fewer potential impacts to the creek. In addition, hole #18 was lengthened by extending it into the former clubhouse site. The old 18th hole drainage swale and lake were replaced by a broader and deeper swale that runs down the length of the left side of the hole and discharges over a weir into the creek. In the revised plan the retention basin has been moved westward to the north side of the 11th hole. The external boundaries of the golf course parcel were also moved inward in several places including the former site of the overnight units,



SITE PLAN

Source: PACE Engineering

the area west of the 13th hole and south of the irrigation reservoir, and the north edge of driving range (which has been substantially reduced in area).

As a result of the above modifications, the overall acreage of the golf course parcel (which includes the clubhouse/overnight complex) increases from 270 acres to 277 acres. The areas of boundary expansion occur at the currently proposed location of the overnight complex and at the site for a new winery/grape processing facility proposed near the northwestern edge of the golf course (see 'Future Project Modifications' below. These expansions are largely compensated by the golf course boundary contractions noted above, such that the net increase in acreage for the golf course parcel is 7 acres (270+7=277). The modifications to the golf course plan result in environmentally beneficial changes such as reduction of number of holes crossing the main creek channel from 3 to 2, and a reduction in overall tree removal from 18 to 16.

The refinements to the golf course design have also resulted in an increase in overall earthwork quantities. The total volume of cut has increased from 344,390 cubic yards (cy) in the EIR to 414,650 cy under the current grading plan, and the total volume of fill has increased from 269,900 cy in the EIR to 387,900 cy under the current plan. These grading increases have been necessitated by the following design changes: additional fills needed to elevate the tee and green sites; additional grading at the practice facility/driving range to provide flatter grades at the tee boxes and smoother slope transitions throughout; changes to the drainage plan along the 18th hole to provide a more naturalistic surface drainage pattern instead of underground pipes, and; deepening of the irrigation lake to provide additional storage capacity. As originally proposed, cuts and fills would be balanced on-site.

<u>Elimination of Equestrian Center and Replacement with a Small-Scale Stable</u>

The original project proposal evaluated in the EIR included a full equestrian center on 12.8 acres in the southeast corner of the project site. As described in detail in the EIR, this was to have been a 40,000 square facility with space for up to 30 horses, a covered riding arena, living quarters for a caretaker/manager, an outdoor riding ring, a training area/paddock and pasture, a paved access road and parking area, and an on-site retention basin to capture runoff from the site. The applicant proposes to eliminate the equestrian center from the project. In its place, a small stable large enough for up to 10 horses is planned for the northeast corner of the site, where it would be removed from the residential subdivisions and yet provide convenient access to the on-site riding trails. The stable would have a floor area of up to 4,000 square feet and would occupy 1 to 2 acres, which includes the stalls plus a small storage area for hay, and an adjoining area for corrals. The stable would have an informal rustic design to fit in with the rural surroundings. The stable is intended solely as a place for homeowners of the project to keep their horses and would not include the other facilities previously proposed for the equestrian center.

<u>Boundary Modifications to Golf Course and Permanent Open Space Area</u>

The land use modifications discussed above result in changes to the boundaries of the golf course parcel and the permanent open space area. As discussed above, the expansion of the clubhouse/overnight complex and the future addition of the winery/processing center would result in a net increase of 7 acres in the westerly portion of the golf course parcel. In addition, minor modifications made to the cluster subdivision plan since the EIR was certified in July 1996 has resulted in an expansion of the residential cluster subdivision by 11.2 acres. Also, the wastewater treatment plant added in 1997 (see EIR Addendum of 1/97) occupies a 5.3-acre common area that was originally within the permanent open space area. The net effect of these modifications is a 24-acre reduction of the permanent open space area (from 1,265.7 acres to 1,241.7 acres). As shown below, this reduced open space area still comprises sufficient land area to comply with the 90 percent open space requirement applicable to the hillside cluster subdivision.

	Acreage per EIR (7/96)	Current Acreage
Golf Course	269.5	277.0
Rural Residential	31.5	31.5
Common Area (WW facility)	--	5.3
Hillside Cluster		
Residential	102.8	114.0
Main Access Road	6.5	6.5
Permanent Open Space	1,265.7	1,241.7
Total Site	1,676.0	1,676.0

Hillside Cluster Parcel	
Total Acreage	1,362.2 acres
Permanent Open Space Required (@ 90%)	1,225.9
Permanent Open Space Provided	1,241.7
Excess Permanent Open Space	15.8 acres

Flood Control Improvements

The changes proposed to the project plans include modification of the proposed on-site flood control facilities. In general, these flood control modifications would provide for a substantial reduction in flood flows leaving the site during more frequent storm events such as the 2-year storm. These improvements would also result in significant reductions in the 100-year and 10-year flows compared to the previously proposed flood control improvements.

The main features of the modified flood control plan are the creation of a diversion channel to parallel the existing West Branch of Llagas Creek at the east end of the project, and the diversion of flood flows carried by the creek and the diversion channel to a 5-acre detention basin alongside Coolidge Avenue north of Highland Avenue. The residential lake south of Highland Avenue would provide detention storage for the adjacent residential area and tributary uplands only. Under the previous plan, a substantial portion of the flood flows carried by the West Branch of Llagas Creek during the 100-year and 10-year events were to have been diverted to the residential lake. This would have provided a significant improvement over existing conditions for these events, but would not have provided reductions in downstream flooding during the more frequent storm events like the 2-year storm, as proposed under the current plan. (The proposed flood control improvements are described in detail in Section *IV. E. Hydrology and Drainage*.)

Future Project Modifications

In addition to the project modifications described above, two new components are expected to be added to the project in the future, which are not described above. These include the following:

1) A 400,000 gallon water storage tank proposed for the northeastern portion of the property which would provide for adequate fire flows to the project and to the neighboring residential areas to the east;

2) A winery/grape processing center which would provide on-site processing for grapes grown in the western portion of the site in accordance with County agricultural mitigation requirements for the project.

The winery/processing center would be located in the northwest portion of the site north of the golf course maintenance facility and would not be open to the public. The winery site consists of land currently allocated to permanent open space which would be removed from permanent open space and included in an expanded golf course parcel. However, as shown above there is sufficient 'surplus' permanent open space area in the project plan that this reduction would not result in the ratio of permanent open space falling below the 90 required for the hillside cluster subdivision.

This EIR Addendum is not intended to provide environmental clearance for the water tank or the winery/processing center. Since these project elements will require individual use permit applications which have not yet been submitted, it is premature to conduct environmental review for these facilities at this time. However, an informal environmental review indicated that these facilities would not result in potentially significant impacts. Therefore, a subsequent EIR addendum will be prepared on these new project components in conjunction with the use permit application process.

Summary Evaluation of Potential Impacts Resulting from Project Modifications

The proposed modifications to the Lion's Gate/CordeValle project would not result in any new significant environmental impacts and in some instances would result in beneficial environmental effects compared with the project evaluated in the EIR. The environmental effects of the project modifications are briefly evaluated below.

Land Use: The increased floor area and land coverage of the clubhouse/overnight complex results in a slight increase in the project's land use intensity. The revised complex would result in an approximately 5 percent increase in impervious surface coverage relative to the project evaluated in the 1996 EIR. However, the overall building intensity is still extremely low, with built and paved surfaces occupying approximately 6 percent of proposed development area and 1.5 percent of the entire project site. Therefore, the proposed increase in building area does not represent a significant impact. No changes are required to EIR Section *III. A. Land Use.*

Parks, Recreation and Open Space: The project modifications result in a reduction of permanent open space from 1,265.7 to 1,241.7 acres. This 2 percent reduction does not represent a significant impact, and the total open space allocation still exceeds the 1,226 acres required to fulfill the 90 percent open space requirement for the Hillside cluster subdivision. EIR Section *III. C. Parks, Recreation and Open Space* has been amended accordingly.

Geology and Soils: The relocation of the clubhouse/overnight complex to the north side of the creek removes it from the potential landslide hazard that exists at the originally proposed site. The currently proposed site is not subject to landslide hazard. The new site is traversed by an inactive fault trace; however, any potential hazard associated with the trace can be mitigated by overexcavation and recompaction of foundation soils over the fault trace, or by deep foundations such as drilled shafts or driven piles, or by modifying the location of structures away from the fault trace. (This is addressed in detail in the geologic report prepared by Twining Labs in May 1998, which is contained in Appendix C of this EIR Addendum.) EIR Section *III. D. Geology and Soils* has been amended accordingly. All other geologic and soils conditions at the new clubhouse site are essentially the same as those at the previously proposed clubhouse site.

Hydrology and Drainage: The proposed flood control modifications would provide for a substantial reduction in flood flows leaving the site during more frequent storm events such as the 2-year storm. These improvements would also result in significant reductions in the 100-year and 10-year flows compared to the previously proposed flood control improvements. The environmental effect would be beneficial relative to the improvements evaluated in the 1996 EIR. Section *III. E. Hydrology and Drainage* has been amended accordingly. The Master Drainage Plan prepared by PACE Engineering which describes and evaluates the flood control modifications is contained in Appendix D of this EIR Appendix.

Water Quality: The removal of the equestrian center from the plan would avoid the creation of potentially contaminated runoff from the center. Although the equestrian center plan provided for isolation of the center from the surrounding drainage area and included an exclusive retention basin to capture runoff, the elimination of the center is environmentally beneficial in terms of potential water quality impacts. The smaller stable now proposed for the northeastern portion of the site would be managed in accordance with County and state requirements to prevent water quality impacts from this facility.

Surface drainage from the relocated and redesigned clubhouse parking lot will be conveyed to underground drains in the adjacent golf course and passed through a biofilter prior to discharge into West Branch Llagas Creek. The previous proposal was to convey discharge to adjacent retention basins. The net effect on water quality would be about the same under the previous and current proposals. EIR Section *III. E. Water Quality* has been amended to reflect the above.

Biological Resources: The revised site plan has been evaluated by H.T. Harvey and Associates. The biologists surveyed the new site for the clubhouse complex and the new stable site and found no sensitive species or habitats that would be affected by these project modifications. Therefore, the proposed modifications would result in no new potential impacts to biological resources. No changes are required to EIR Section *III. F. Biological Resources*. The letter report prepared by Harvey and Associates which addresses the project modifications is contained in Appendix F of this EIR Addendum.

The revised golf course routing plan results in a reduction of fairways crossing the main creek channel from 3 to 2. This will tend to reduce the incidence of golfers entering the creek channel (against course rules) to retrieve errant golf balls, and as such would reduce impacts to riparian habitat.

The revised golf course plan results in a reduction of overall tree loss from 18 to 16 trees, which represents a beneficial effect of the revised plan.

Archaeology: The new location for the clubhouse complex and the new stable site are not within areas of archaeological sensitivity and there are no known archaeological resources in the vicinity of these sites. The western end of the bypass channel along Highland Avenue at the project entrance is in close proximity to recorded archaeological site CA-SCl-76. As such, work at the western end of the bypass channel would be subject to monitoring provisions specified in the EIR. None of these changes necessitate modification of EIR Section *III. E. Archaeology*. A letter report on the project modifications prepared by Basin Research Associates is contained in Appendix G of this EIR Addendum.

Aesthetics: At the new location north of West Branch Llagas Creek, the nearby hills completely shield the clubhouse and overnight complex from view from off-site locations, including the residence overlooking the site from the off-site ridge to the north. If anything the clubhouse complex would be better shielded from view by the intervening hills. The new stable in the northeast portion of the site may be visible from existing residences to the east, but it would be small in scale and have an informal rustic appearance that would blend in with its rural surroundings. The potential visual effects of the proposed flood detention basin adjacent to Coolidge Avenue would be mitigated by the landscaped berm planned along the roadway frontage. Therefore, the project modifications would not result in new or increased visual impacts. EIR Section *III. J. Visual and Aesthetics* has been modified to reflect the above.

Traffic: The larger clubhouse proposed would generate additional traffic since the restaurant component increases in size from 4,000 square feet to approximately 5,800 square feet. An evaluation of the project changes by TJKM Transportation Consultants estimated that total p.m. trip generation from the project would increase by 15 trips as a result of the larger restaurant component. The other modifications would not result in increased trip generation. It was calculated that this additional trip generation would have no effect on levels of service or average vehicle delay at any of the potentially affected intersections. Therefore, the project modifications would have no traffic impacts. No changes are required to EIR Section *III. K. Traffic and Circulation*. The letter report by TJKM that addresses the project changes is contained in Appendix H of this EIR Addendum.

Noise: The relocation of the clubhouse to the north would bring this facility closer to the existing residence on the northern ridge overlooking the site. The new clubhouse location is 3,000 feet from this residence while the original clubhouse location was 3,600 feet away. The analysis in the 1996 EIR concluded that loud music played at the clubhouse during weddings or similar events may be audible at the existing residence under certain conditions but would not result in significant noise impacts. The new clubhouse location was evaluated by Illingworth & Rodkin who concluded that the new location would result in noise levels 2 decibels louder than at the previous site, but that the resulting noise levels would be within the range indicated in the EIR. The new clubhouse location would not result in noise impacts to the existing residence. The letter report by Illingworth & Rodkin that addresses the noise impacts of the project changes in contained in Appendix I of this EIR Addendum.

One of the golf course modifications involves the siting of a new hole (#1) along the south side of the main access road, just south of several planned lots for rural residential dwellings. The new hole would result in fairway mowing at a distances as close as 120 feet from these future residences, compared with a minimum distance of 200 feet under the previous plan. This will result in mower noise being louder at the residences than under the previous plan. However, the County noise ordinance allows for noise sources to exceed County standards if the duration of the noise is limited as prescribed in the ordinance. There is not expected to be any difficulty in meeting these time restrictions. Therefore, this project modification would not result in a significant noise impact. EIR Section *III. L. Noise* has been amended to reflect the above.

Air Quality: The slight increase in traffic generated as a result of the larger restaurant component proposed for the clubhouse would also increase the generation of vehicle emissions. However, according to air quality consultant M'OC Physics Applied, this increase would not be significant in terms of either local carbon monoxide concentrations or in term of pollutants of regional concern. No changes are required to EIR Section *III. M. Air Quality.*

Hazards: The removal of the equestrian center from the plan reduces the concern for potential vector and odor impacts. Although similar issues arise for the new stable, the potential for impacts is much reduced due to the smaller scale of the stable. EIR Section *III. N. Hazardous Materials, Public Health and Safety* has been amended to reflect the above.

Rationale for Preparation of an EIR Addendum

This document has been prepared in accordance with the requirements of the California Environmental Quality Act (CEQA) which sets forth specific requirements for the documentation of potential environmental impacts which may result from modifications made to a proposed project after an EIR on the project has been certified. Under these circumstances, Sections 15162 through 15164 of the CEQA Guidelines provide for the preparation of one of three types of documents depending on the situation. The criteria to be met for each type of document are as follows: 1) a 'Subsequent EIR' shall be prepared if the changes to the project are substantial, and will result in major revisions to the EIR due to the involvement of new significant environmental effects or a substantial increase in the severity of previously identified significant effects; 2) a 'Supplement to an EIR' shall be prepared if the conditions described in #1 above apply but only minor changes or revisions to the EIR are necessary; and 3) an 'Addendum to an EIR' shall be prepared if some minor changes and additions are necessary, but the conditions which would necessitate the preparation of a Supplement to an EIR are not present. In the present case, the proposed modifications may or may not be considered substantial, but the overall effect of the changes would be beneficial environmentally, and in no instance would new significant environmental effects be involved or the severity of a significant effect be increased substantially, as discussed above and in the body of this document. In addition, the changes to the EIR required to address the proposed project modifications are minor in nature. Thus two of the required criteria for preparing a Subsequent EIR and one of the required criteria for preparing a Supplement to an EIR would not apply. Therefore, according to CEQA criteria noted above, the type of environmental document that should be prepared in this instance is an 'Addendum to an EIR.'

Organization of This Document

Since this is the Second Addendum to the EIR, this document identifies revisions to the certified EIR, as modified by the first Addendum, which reflect the changes in project description and environmental analysis resulting from the proposed modifications to the project. In order to facilitate the reader's comprehension without having to refer back to the certified EIR and the first Addendum, this document contains the affected portion of the EIR to provide a context for the text changes. Revisions to the text are indicated by ~~strikethrough~~ for deletions and <u>underline</u> for additions.

SUMMARY

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SUMMARY OF IMPACTS AND MITIGATIONS

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IMPACT	MITIGATION
D. GEOLOGY AND SOILS	
1. Potential secondary ground rupture or sympathetic movement along inactive faults crossing the site may result in minor damage to structures, roadways and utility lines. **(Potential Significant Impact)**	1. Where proposed structures for human occupancy are determined to be underlain by an inactive fault trace, mitigation could consist of modification of the soil foundation, using deep foundations, or modifying the location of the structure away from the shear zone. ~~Appropriate setback distances from those structures may be required.~~ **(Less-than-Significant Impact with Mitigation)**

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IMPACT	MITIGATION
E. HYDROLOGY AND DRAINAGE	
1. The project would potentially result in increased downstream flooding during the 100-year and 10-year storms. **(Potential Significant Impact)**	1. ~~The on-site lake proposed for the southern residential cluster subdivision would be designed to provide sufficient detention storage for increased peak runoff resulting from site development. In addition, a diversion structure would be constructed in the creek channel to divert a substantial portion of the flows exceeding the existing 10-year flow rates to the residential lake, which would be sized to accommodate flows from the 100-year event. With these facilities, the peak flow rates leaving the project site during significant storm events would be substantially lower than under existing conditions.~~ In order to control and detain flood flows generated at the site, a diversion channel, a detention basin and a lake are proposed. These structures would be designed to minimize the extent of flooding within the project boundaries, and would reduce peak flood flows leaving the site during the 100-year, 10-year, and 2-year events relative to existing conditions. **(Less-than-Significant Impact with Mitigation)**

IMPACT	MITIGATION
E. HYDROLOGY AND DRAINAGE (CONT'D)	
2. Portions of the residential cluster subdivisions and the wastewater treatment facility ~~would~~ may be subject to shallow flooding (one-foot average depth) during a 100-year event, and the proposed structures could also partially obstruct this sheet flow through the site. However, the total area of the site that may be subject to shallow flooding would be reduced by flood control improvements included in the project. **(Potential Significant Impact)**	2. Potential impacts to the residential subdivisions and the wastewater treatment facility from shallow flooding would be mitigated by constructing building pads on fills raised above flood elevations. The partial obstruction of shallow overland sheet flows by the proposed development would be mitigated by balancing fills with cuts within the flood-prone areas. **(Less-than-Significant Impact with Mitigation)**

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*

L. NOISE

IMPACT	MITIGATION
5. Noise levels would be temporarily elevated during grading and construction. **(Potential Significant Impact)**	5. Short-term construction noise impacts would be reduced through compliance with the County's Noise Ordinance with respect to hours of operation and maximum noise levels at adjacent property lines. At the eastern edge of the project, the berms proposed along the project boundary would be constructed during the early phases of grading to provide a noise barrier for existing residences nearby. **(Less-than-Significant Impact with Mitigation)**

I. PROJECT DESCRIPTION

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B. DESCRIPTION OF THE PROPOSED PROJECT

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Cluster Residential Subdivisions

The project would include two main residential clusters and related open space areas, as described below.

Rural Residential Cluster Subdivision

The 31.5-acre Rural Residential parcel is located at the eastern edge of the site, adjacent to Coolidge Avenue, north of Highland Avenue. The proposal is to cluster the 6 permitted lots in the western portion of this parcel, with lots ranging in size from 1.7 to 2.5 acres. The eastern and southern edges of the site would remain in permanent open space. The old plum and walnut orchard would be removed and replaced with a 4 foot high

TABLE 1

PROPOSED LAND USES

Land Use	Acreage
Golf Course	
• Open Area	~~263.2~~ 239.9
• Clubhouse, Overnight Facilities & Parking	~~6.3~~ 19.1
• Winery Site	18.0
Residential	
• Hillside Cluster	~~102.8~~ 114.0
• Rural Residential Cluster	31.5
Permanent Open Spacc	~~1,265.7~~ 1241.7
Main Access Road	6.5
Common Area (Wastewater Treatment Facility)	5.3
TOTAL	1,676.0

landscaped berm along the roadway.~~, and a vineyard of approximately 10 acres would be planted behind the berm. This buffer area would range in depth from 250 to 400 feet, comprising a total of approximately 12 acres.~~ Just west of the berm, a detention basin would be excavated to provide flood storage during major storm events and reduce downstream flooding. The southern portion of this site would contain the channel of West Branch Llagas Creek, which flows from west to east alongside Highland Avenue. To prevent flooding along the banks of the creek through this area, a diversion channel would be created which would run parallel and to the south of the creek channel (see 'Drainage' below).

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Permanent Open Space

The Hillside cluster subdivision would include a ~~1,265~~ 1,242-acre permanent open space area which would constitute over 90 percent of the Hillside zone on the site. (This ~~assumes that~~ includes the 259 acres ~~currently~~ formerly designated "Agriculture - Medium Scale" in the County General Plan ~~would be~~ that were redesignated to "Hillsides.") Most of this permanent open space area comprises the hillside areas which flank Hayes Valley on the north and south, and also includes the level pasture land in the western portion of the site near Watsonville Road. This area would include a system of informal trails for hiking and horseback riding.

~~A small portion (less than one acre) of the northern hillside area adjacent to the golf course driving range would provide the site for winter storage of treated effluent prior to spray irrigation on the driving range.~~

The permanent open space area would include a public trail easement for the proposed San Martin Cross-Valley Trail, which would follow the northern boundary of the site. The trail would be constructed by the County of Santa Clara Department of Parks and Recreation.

The permanent open space area would also include 100 acres of vineyard to be planted ~~in two areas. A 10-acre vineyard would be planted along Coolidge Avenue, within the 250-foot setback area for the proposed Rural Residential subdivision. A 100-acre vineyard would be planted~~ at the western end of the project, in the open field fronting onto Watsonville Road.

The permanent open space area also includes an area of approximately ~~40~~ 35 acres in the southeastern corner of the site. This area would include: buffer areas around the residential lots, a 4-foot landscaped berm along Turlock Avenue, and a 20-acre lake. ~~and a 20-acre equestrian center (see 'Drainage' and 'Equestrian Center' below).~~

The permanent open space areas of the site would be placed in the ownership of the Homeowners Association for the project, and would not be open to the general public, except for the public trail easement described above. The grazing of cattle on the Lion's Gate site (which currently reaches a peak of 250 head) would be discontinued upon construction of the project.

*

*

Golf Course

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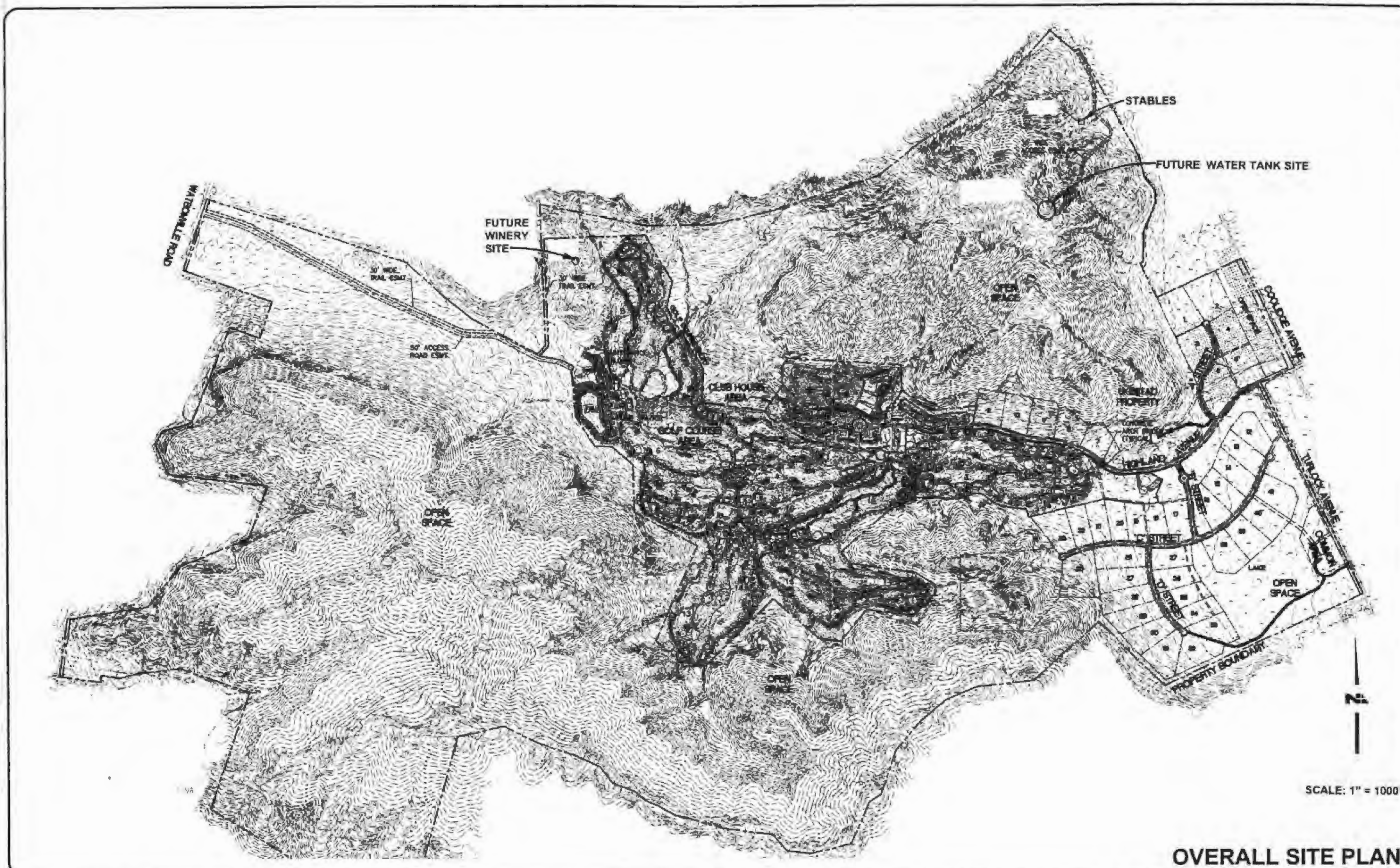
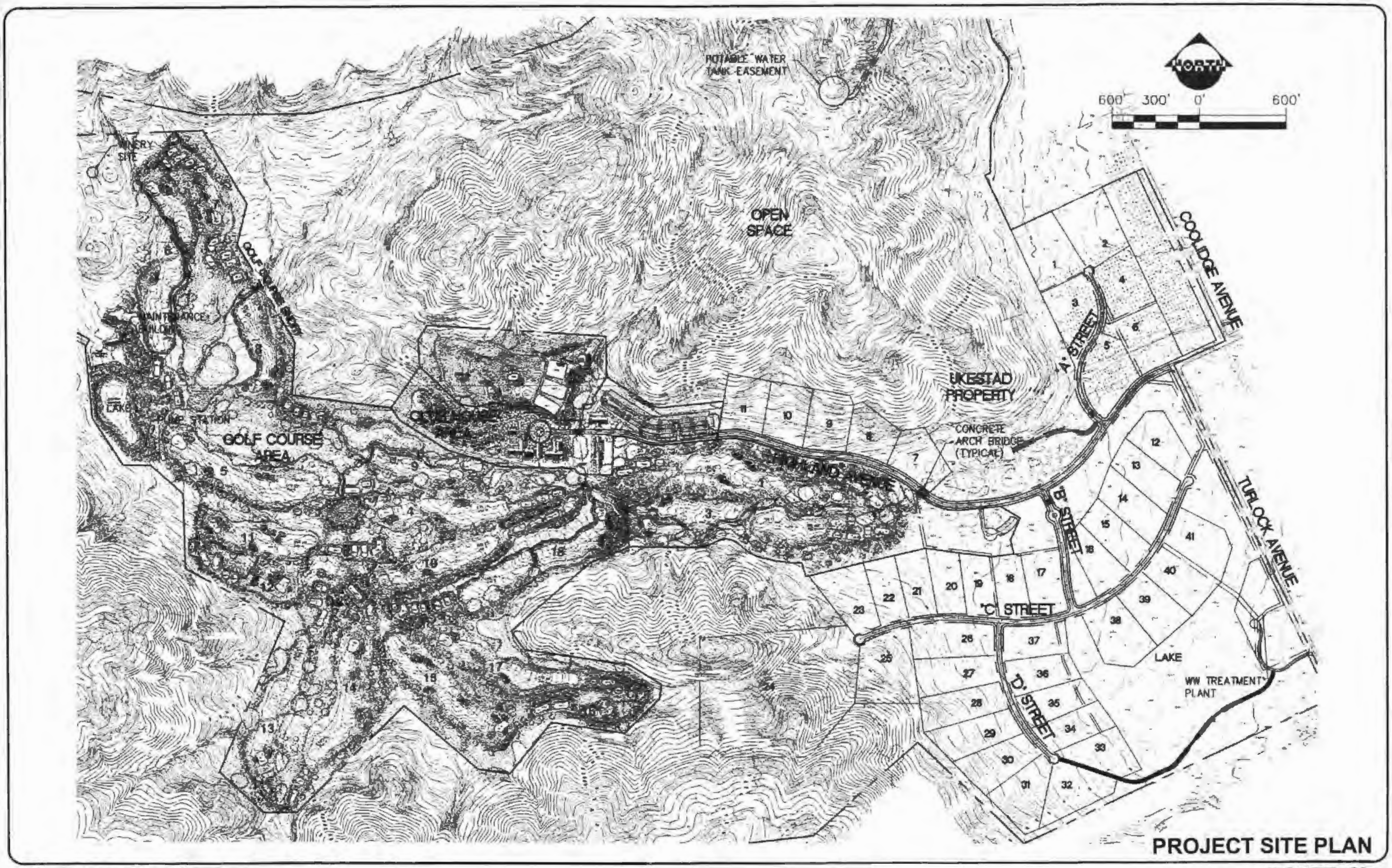
The total quantity of earth to be moved during grading for the golf course and related facilities is estimated to be approximately ~~one million~~ 760,000 cubic yards. A total of ~~18~~ 16 trees would require removal to accommodate the golf course. These would be replaced by over 2,500 native trees to be planted throughout the golf course and the residential areas of the project. (These are trees that have been specifically grown for the project from acorns and seeds collected from the site in 1989.)



Source: PACE Engineering

FIGURE 9a



PROJECT SITE PLAN

Source: PACE Engineering

FIGURE 9b

TABLE 2

PROJECT SUMMARY DATA

RESIDENTIAL	
• Rural Residential Cluster Subdivision (lots)	6
• Hillside Residential Cluster Subdivision (lots)	35
GOLF COURSE	
• Holes	18
• Clubhouse (square feet - includes pro shop)	~~29,000~~ 55,100
• Overnight Accommodations (units)	45
• Parking Spaces (Clubhouse, Overnight, Practice Facilities)	~~250~~ 350
• Maintenance Facility (square feet)	6,000
Grading (cubic yards - cut/fill)	~~500,000/500,000~~ 575,000/527,000
Tree Removal (total)	~~18~~ 16
Tree Planting	2,500+
WATER CONSUMPTION (gallons/day) - (average/peak)	
Golf Course Irrigation (non-potable)	334,000/677,000
Domestic/Landscape/Washdown	57,000/114,000
WASTEWATER FLOWS (gallons/day) - (average/peak)	23,000/30,000

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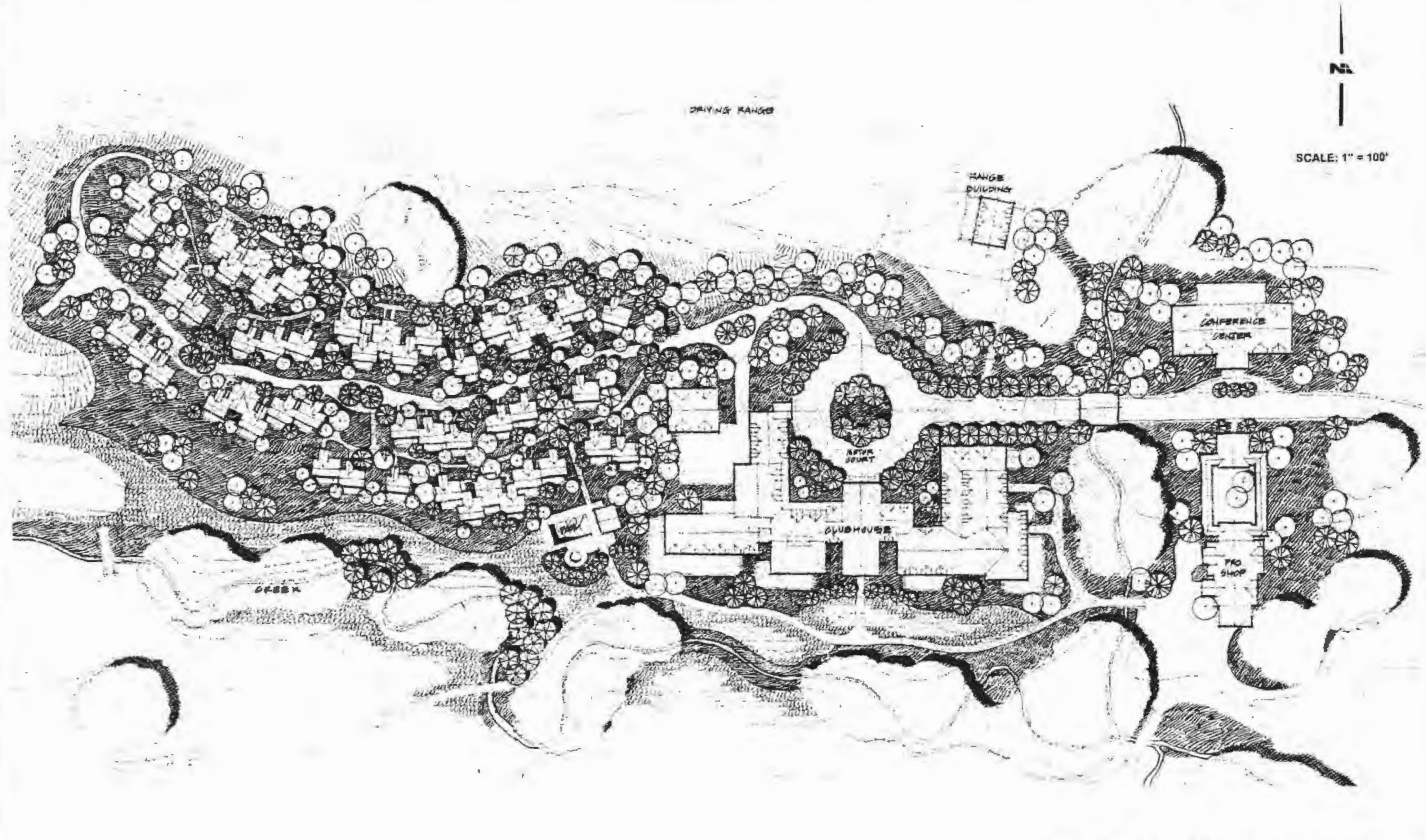
Clubhouse

The focal point of the golf course would be a ~~3-level 29,000~~ 55,122 square-foot clubhouse (inclusive of the ~~golf card barn~~ pro shop) and 45 units of overnight accommodation. This complex is proposed for the ~~foot of the southern hillsides~~ north side of West Branch Llagas Creek in the east-central area of the site (see Figures 10a and 10b). The floor area breakdown for the clubhouse is provided in Table 3.

TABLE 3 (Revised)

CLUBHOUSE FLOOR AREA BREAKDOWN

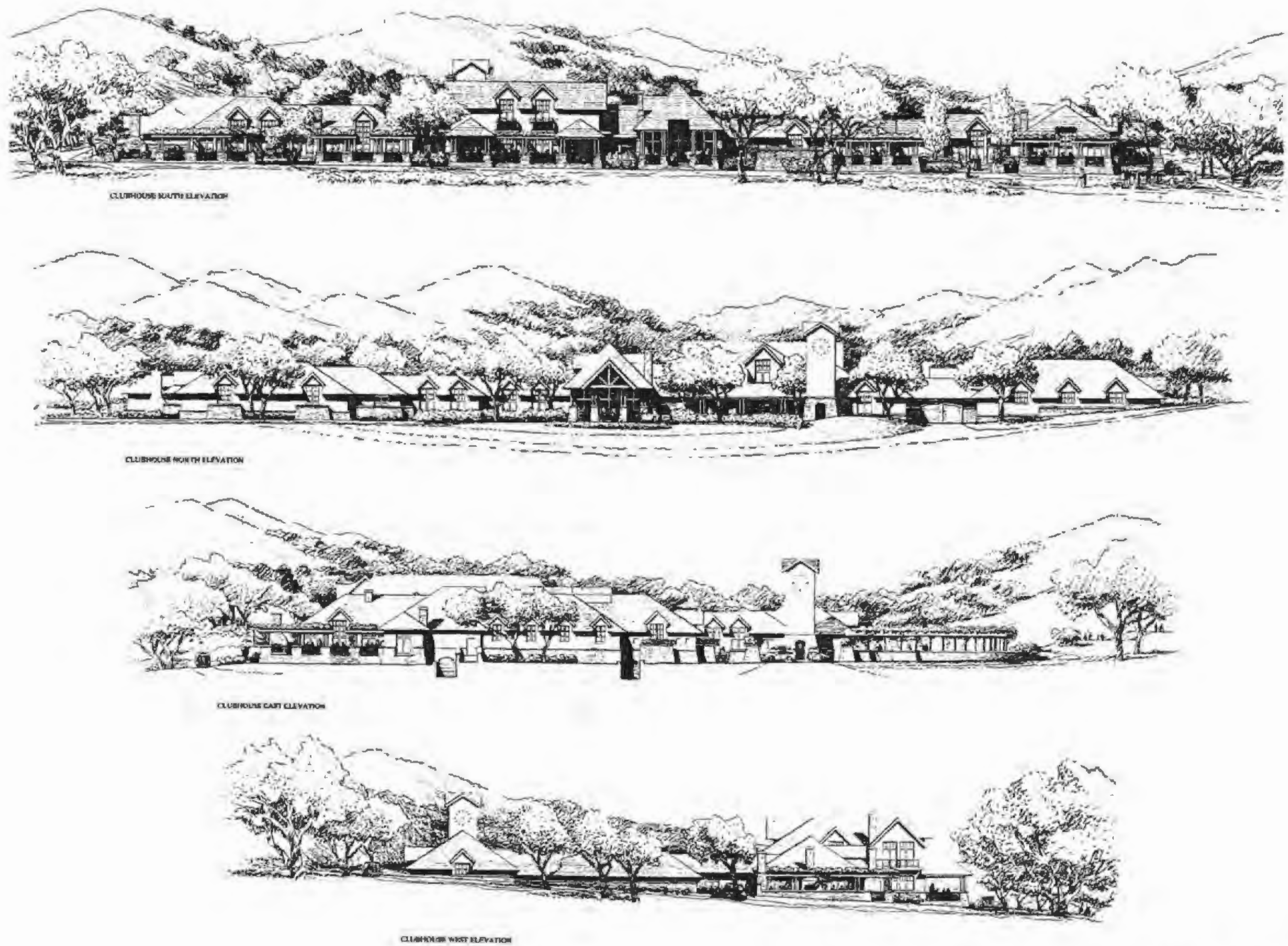
Function	Floor Area (Square Feet)	Function	Floor Area (Square Feet)
Main Floor		Upper Level	
Dining Rooms & Mixed Grill	5,840	Staff Locker Rooms	1,600
Bar	640	Staff Lounge	850
Kitchen	2,336	Meeting Rooms	2,520
Main Hall	2,496	Pre-function	1,000
Front Desk & Reception	1,040	Storage	320
Living Room	720	Circulation	740
Office and Administration	2,416		
Business Center	512	Subtotal	7,030
Boutique	1,040		
Fitness Room	1,296		
Maintenance/Housekeeping	2,784	Lower Level	
Restrooms & Circulation	1,040	Wine Cellar	2,472
Tower & Stairs	476	Cart & Bag Storage	7,432
Men's Facilities		Subtotal	9,904
Wet Area & Lockers	5,232		
Bar & Lounge & Cigar	2,400		
Attendant	432	Pro Shop	3,200
Treatment Rooms	1,012		
Storage, Hall, Entry, Phones	800		
Women's Facilities			
Wet Area & Lockers	1,252		
Lounge	968		
Attendant	256		
Subtotal	34,988	**TOTAL**	**55,122**



SITE PLAN - CLUBHOUSE VICINITY

Source: Backen & Gillam

FIGURE 10a



CLUBHOUSE ELEVATIONS

Source: Backen & Gillam

FIGURE 10b

The clubhouse would be built on three levels, with most functions contained on the main floor at the middle level. The main floor would include a ~~pro shop~~ bar and restaurant, banquet facilities, ~~and a separate members' lounge for corporate members~~, the main hall and reception area, a boutique, fitness center, business and administration offices, and men's and women's locker rooms and lounges. ~~The members' area would include locker rooms, card rooms, a spa and members grill.~~ The lower level would contain the wine cellar and the storage area for bags and golf carts, and the upper level would include the staff locker rooms and lounge, meeting rooms and storage rooms. The pro shop would occupy a stand-alone structure to the east of the main clubhouse. In addition, a small conference center of approximately 6,000 square-feet may be added in the future if demand for meeting space warrants.

The clubhouse would be designed ~~in the style of an Italian hilltown,~~ in the California Regional style, and would take advantage of view opportunities from the base of the hillside. ~~The building would have an adobe type appearance and would be constructed with a building technique known as PISE (Pneumatically Compacted Stabilized Earth) instead of conventional frame construction. By this method the structure of the wall is created by spraying an earth mixture horizontally against a rigid, single thickness form to create walls 24 inches thick. Such massive walls provide excellent insulation for passive heating and cooling~~. The buildings follow the topography of the sloping foothills, and the complex has been arranged in a campus fashion as a series of low, interconnected structures and spaces that break down the perceived mass of the complex. The buildings are intended to harmoniously blend with the environment, and the layout and materials reflect this design objective. A series of natural gardens and terraces soften the appearance of the buildings and merge the interior with the exterior. The materials - stone, plaster, heavy timber, and slate complement the indigenous materials on the site, with the intent of further linking the buildings with their natural context.

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Overnight Accommodations

Adjacent to the clubhouse on the ~~hillside~~ west would be the 45 overnight guest units which would also be constructed in the ~~adobe~~ California Regional style, and would be laid out and designed as an integral part of the overall clubhouse complex (see Figures 10c and 10d). These units would not be typical hotel rooms and would only be available as overnight accommodations for golf course users.

The individual guest units would be approximately 500 to 600 square feet, and would be designed as suites. Some of the guest units would be arranged in clusters surrounding ~~several~~ five small conference rooms of approximately 500 square feet. ~~would be included in the overnight complex. These conference rooms would be located between the two units so they would be accessible from one or both of the adjacent units as needed.~~

Vehicular access to the overnight complex would only be by means of golf carts from the clubhouse ~~parking~~ area. The parking area for the clubhouse and overnight complex would have capacity for ~~188~~ 350 vehicles, with valet parking available from the clubhouse entrance. ~~parking for an additional 61 vehicles to be provided to the north of the clubhouse area adjacent to the practice facility.~~

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~~Equestrian Center~~ Horse Stables

The project would include a small stable where only residents of the project could keep their horses. The stable would be located in the northeastern corner of the site at the base of the easterly facing hillside (see Figure 9a) and would occupy a 1 to 2-acre site. The stable would have a floor area of up to 4,000 square feet and would provide space for up to 10 horses, a small area for hay storage, plus an adjacent corral. The stable would not include any other facilities such as caretakers quarters, riding ring or paddock. The stable would have a simple rustic design to blend in with the rural surroundings. The stable would have driveway access off San Martin Avenue or would be accessible by foot along a path following the toe of the hillside southward to the on-site residential areas.

~~The project would include an equestrian center located on approximately 20 acres at the southeastern corner of the site with driveway access directly off Turlock Avenue. The equestrian center would be open only to project residents for the boarding of privately owned horses. No public horse rental is proposed. The focus of the equestrian center would be a covered riding arena structure measuring 100 by 200 feet. The arena would be surrounded on 3 sides by 20 to 30 indoor/outdoor stalls. The center would also include a hay storage area and living quarters for a caretaker/manager. Other features would include an outdoor riding ring, a training area/paddock and pasture. The access drive and 20-space parking area would be surfaced with all-weather crushed gravel~~. The ~~center~~ stable would have direct access to over 8 miles of private riding trails proposed for the permanent open space areas of the Lion's Gate site. These riding trails would consist of a network of existing trails and vehicle tracks that occur throughout the site. Some minor improvements may be needed to these existing trails, but it is not expected that new trails would be created. ~~Access from the equestrian center to these trails would be via the narrow strips of permanent open space extending west and north of the equestrian center along the project boundary (see Figure 9c).~~

In order to prevent horse manure from entering downstream water courses or groundwater, the ~~equestrian facility~~ stable would be operated in accordance with a manure management plan, as required under Title 23, Chapter 15 of the California Code of Regulations (which pertain to the protection of water quality). Under the manure management plan, debris boxes would be used to store daily stall sweepings and manure. ~~The outdoor riding and pasture areas would have manure picked up daily with a special vacuum vehicle.~~ Disposal of wastes at a local landfill, one which is permitted to accept manure, would occur on ~~a daily basis or every other day~~ on an as-needed basis. ~~Alternatively, on-site composting of manure may be considered instead of off-site disposal. Any proposal to compost manure would require approval from the Department of Environmental Health Solid Waste Unit.)~~ The perimeter of the ~~equestrian center~~ stable site would be fenced to prevent animals from entering nearby drainages ~~and ponds~~ and contaminating the water.

~~The equestrian center would be contoured to direct on-site drainage to a grass swale or swales which would convey runoff to a lined retention pond or basin. This pond would be located at the eastern end of the site, just west of the landscaped berm proposed along Turlock Avenue. The pond would be equipped with a sump pump to remove any floating material, and would be cleaned out regularly to remove accumulated sediments. The pond would be sized for the 10-year storm to prevent overflow of accumulated drainage in all but the most significant flood events (the pond would be fenced to prevent entry, and signs would be posted warning people to keep out.) Any drainage from areas upslope of the equestrian center to the west would be directed around the facility to the proposed residential lake to the north.~~

The ~~equestrian center~~ stable would employ vector control measures as needed, such as baiting for flies, and rodent trapping. As discussed above, manure would be cleaned up daily and placed in debris boxes which would be emptied ~~daily or every other day~~ on an as needed basis and taken to a local landfill ~~or composted on-site.~~

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TYPICAL UNITS SOUTH ELEVATION

TYPICAL UNITS NORTH ELEVATION

TYPICAL UNITS EAST ELEVATION

OVERNIGHT UNITS - ELEVATIONS

Source: Backen & Gillam

FIGURE 10c

CONFERENCE SUITE SOUTH ELEVATION

CONFERENCE SUITE NORTH ELEVATION

CONFERENCE SUITE WEST ELEVATION

OVERNIGHT UNITS - ELEVATIONS

Source: Backen & Gillam

FIGURE 10d

Grading and Excavation

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Cuts and Fills

The total estimated earthwork for the project is approximately ~~one~~ 1.1 million cubic yards of cut and fill, of which approximately 760,000 cubic yards would be for the golf course. All earthwork would be balanced on the site.

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Excavations for Lakes

There are ~~four~~ five lakes, ponds and major detention basins proposed as part of the project. These include the following: the irrigation storage reservoir located at the west end of the golf course, which would involve excavation of ~~68,700~~ 63,350 cubic yards of earth; the runoff detention pond ~~near the 18th green~~ adjacent to the 11th hole, which would require removal of ~~7,950~~ 19,500 cubic yards; the wastewater storage pond ~~north of the driving range~~ adjacent to Turlock Avenue, which would entail the excavation of ~~69,000~~ 15,500 cubic yards of material; the 20-acre lake to be located at the main residential subdivision in the southeastern portion of the site, which would involve the removal of 70,200 cubic yards of earth; and the flood detention basin along Coolidge Avenue, which would involve the excavation of 30,000 cubic yards of material. The excess material generated by these excavations would be used in golf course contouring, constructing the berms along Turlock and Coolidge Avenues, and for building pads in the residential subdivisions.

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Drainage

The project largely incorporates the existing natural drainage system into the design of the golf course and residential areas. In the golf course plan there are several instances where short reaches of tributary drainages would be rerouted or piped to accommodate the fairway layout. Along the West Branch of Llagas Creek there are two locations upstream of the clubhouse site where small existing meanders would be removed in the golf plan. The natural drainage channels in the residential areas would be largely unaltered. The existing flow characteristics of West Branch Llagas Creek are not proposed to be altered in the proposed project plans. However, several flood control improvements are proposed which would reduce flooding potential on the project site as well as downstream. These include a diversion along the West Branch of Llagas Creek, a detention basin along Coolidge Avenue, and a lake/detention basin in the residential area south of Highland Avenue. These flood control features are described in Section *III. E. Hydrology and Drainage*.

Golf Course Drainage

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Some underground storm drains would be installed for the clubhouse and overnight complex. Surface runoff from the parking areas would be conveyed to ~~nearby retention basins~~ underground storm drains and conveyed to the main creek channel. The parking lot runoff would pass through an underground biofilter prior to discharge into the creek channel. ~~Stormwater collected in the basins would not be released to the creek channel but would percolate into the soil or evaporate. The retention basins would be cleaned of accumulated sediments as needed.~~

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~~Equestrian Center~~

~~As noted previously, drainage within the equestrian center would be directed to a retention basin to be located at the eastern end of the site near Turlock Avenue. To the extent feasible, natural drainage originating upslope of the equestrian center would be diverted around the equestrian area and directed to the proposed lake to the north.~~

III. ENVIRONMENTAL SETTING, IMPACTS AND MITIGATION MEASURES

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C. PARKS, RECREATION AND OPEN SPACE

Impacts and Mitigation

Impact 1. **The proposed golf course and residential uses would result in the loss of approximately ~~410~~ 434 acres of semi-natural open space. (Potential Significant Impact)**

The development of the golf course and its related facilities would involve ~~269~~ 277 acres of land in the central valley area of the site, while the residential subdivisions, ~~and~~ roadways, and the wastewater treatment facility would occupy approximately ~~141~~ 157 acres. Approximately 16 percent of the total site area would be converted to golf course uses, and ~~8~~ 9 percent would be converted to residential uses and public facility uses. This acreage consists primarily of fields, an abandoned orchard, grazing land and approximately 20 acres of partially wooded hillsides (although the proposed building envelopes for the two proposed woodland lots are located in areas with little or no tree cover.) The Hayes Valley site was identified as a low priority (rated #26 out of 42) for open space preservation by the County's Open Space 2020 Task Force. The report cited the property's value as watershed, viewshed, and ability to buffer urbanization as primary resources to be protected. The remaining ~~1,265~~ 1,242 acres of property would remain in permanent open space, as required under the Hillside clustering provisions of the zoning district.

Mitigation 1a. **The project would provide approximately ~~263~~ 258 acres of managed recreational open space in the form of a public golf course. The golf course would provide an added recreational opportunity in the County.**

The proposed project would provide additional recreational opportunities which would be open to members of the public. The project would help alleviate the well-documented shortage of golf courses in the County.

Mitigation 1b. **The remaining ~~1,265~~ 1,242 acres of natural and semi-natural area of the site would be preserved as permanent open space as a condition of the cluster development permit.**

Approximately ~~1,265~~ 1,242 acres of oak woodland and grassland on the site would be preserved as permanent open space. This open space would be managed and maintained by the Homeowners Association for the project, and would not be open to the general public.

D. GEOLOGY AND SOILS

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Impacts and Mitigation

Impact 1. **Potential secondary ground rupture or sympathetic movement along inactive faults crossing the site may result in minor damage to structures, roadways and utility lines. (Potential Significant Impact)**

The previous limited fault investigation on the Hayes Valley site by Wahler Associates in 1990 concluded that both of the on-site fault traces are inactive. This was confirmed through exploratory borings and trenching performed by Twining Laboratories in May 1998. Therefore, the potential hazard due to primary ground rupture (as might occur along an active fault trace) is considered minimal at the project site. Secondary ground rupture or *sympathetic movement* along one of the inactive faults on-site could conceivably occur as the result of the strong groundshaking caused by the occurrence of a large earthquake originating on one of the nearby active faults (e.g., Sargent or San Andreas faults). In the event of a large earthquake nearby, sympathetic movement of a fault within bedrock materials at depth might propagate through the overlying sediments to the break the ground surface; but where bedrock covers significant thickness (more than 5 feet) of alluvium or colluvium, displacement at the ground surface would be considered unlikely, although broad tilting and deformation are possible. Any displacements at the surface of the bedrock from such sympathetic fault movement would likely be small, up to a maximum of several inches. The risk of minor damage to structures, roadways and utility lines crossing the on-site fault traces as a result of secondary ground displacement is negligible, but remotely possible. However, dissimilar earth materials may be juxtaposed across the fault, or structurally weak zones of sheared rock may occur coincident with the faults. Such variable foundation properties can result in excessive differential settlement, and damage may occur to buildings constructed across such zones. The areas of the project that could be potentially affected by on-site fault traces include the site of the clubhouse/overnight complex and proposed Lots 7 through 11, 20 through 24, and 30 and 31 (see Figure 11). ~~The clubhouse and overnight accommodations complex would not be affected.~~

Mitigation 1a. **Where proposed structures for human occupancy are determined to be underlain by an inactive fault trace, mitigation could consist of modification of the soil foundation, using deep foundations, or modifying the location of the structure away from the shear zone. ~~Appropriate setback distances for those structures may be required.~~**

~~Detailed fault investigation would be undertaken where structures for human occupancy are planned for areas suspected of being underlain by faults. These studies would determine the potential for surface displacement along the on-site fault traces, with implementation of recommendations as to appropriate measures for site planning, building design, and utilities engineering. The previous fault study (Wahler, 1990) was relatively general and did not address specific proposed building sites.~~

Potential differential settlement in the vicinity of the fault traces may be mitigated by overexcavation and recompaction of foundation soils across the fault, or by deep foundations such as drilled shafts or driven piles. In addition, mitigation may include modifying the location of the structure away from the shear zone. Specific foundation recommendations will be made by Twining Laboratories in the design level geotechnical engineering report.

~~Based on the findings of such explorations, the project geologist could recommend that habitable structures be located off the faults, or in the event of potential sympathetic movement, that a setback zone be established. An appropriate setback distance would be established in discussions between the County Geologist and the project geologist. There is adequate space on all of the proposed lots to accommodate any changes in building locations. Alternatively, the project geologist may conclude that there is no risk of offset along the fault contacts due to the thickness of alluvium, indicating no need for mitigation or avoidance.~~

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Impact 5. **The presence of unstable slopes and existing landslide deposits on the project site may pose a hazard to proposed structures, and may be affected by project grading. (Potential Significant Impact)**

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~~In addition, the spray irrigation of the practice range with treated effluent, which is proposed as an alternative wastewater disposal method, could destabilize existing slide deposits in this area by increasing pore pressures within the slide masses.~~

~~Due to concerns about potential landsliding affecting the feasibility of the proposed overnight units, and Lots 24, 25 and 26, feasibility level geotechnical evaluations of these areas were conducted by Pacific Geotechnical Engineering in December 1995. With respect to the overnight units, it was found that two landslide deposits located upslope of the complex could become reactivated and impact the proposed structures. It was concluded that, while further design level geotechnical studies would be required, it appears that this landslide hazard "can be mitigated or repaired in conventional fashion without exorbitant cost" (see mitigation measures below). (The feasibility report on the clubhouse and overnight complex is contained in Appendix C.)~~

There are two landslide features to the north of the clubhouse/overnight complex which appear to comprise relatively shallow rotational block slides and slumps. These slides are separated from the complex by a ravine which would preclude impact to the complex if the landslide masses were remobilized. As such, the slide masses do not present a hazard to the complex.

E. HYDROLOGY AND DRAINAGE

This discussion is mainly based on the following reports: Hydrology and Drainage - Lion's Gate Development prepared by Schaaf & Wheeler in November 1995; and the ~~Preliminary Design Report for the~~ Lion's Gate Reserve Master Drainage Plan prepared by Pacific Advanced Civil Engineering in ~~November 1996~~ January 1998 (with an addendum dated April 1998). ~~Both of~~ These reports are contained in Appendix D of this EIR.

Environmental Setting

Area-Wide Drainage

The project site is located in the Llagas Creek watershed which drains from the eastern slopes of the Santa Cruz Mountains and the western slopes of the Mount Hamilton Range south to the Pajaro River and Monterey Bay near Watsonville. The major tributaries of Llagas Creek are Little Llagas Creek, Madrone Channel, Coralitos Creek, San Martin Creek, Church Creek, and West Branch Llagas Creek. Llagas Creek and its tributaries drain a total of approximately 105 square miles upstream of its confluence with the Pajaro River south of Gilroy

The climate of the south Santa Clara Valley is similar to that of the San Francisco Bay Area. Summers are warm and dry while winters are mild and moderately wet. Nearly 90 percent of the annual rainfall occurs in the late fall or winter months, with January normally being the wettest. The mean annual precipitation varies within the Llagas Creek watershed from a high of over 50 inches in the Santa Cruz Mountains to a low of 14 inches on the valley floor. The basin-wide average is approximately 20 inches per year.

Stream flows in Llagas Creek are regulated by Chesbro Reservoir, which is owned and operated by the Santa Clara Valley Water District. The reservoir has a total storage capacity of approximately 8,100 acre-feet. The reservoir is operated for water supply purposes, but does provide some incidental flood control benefit due to peak flow attenuation.

The upland areas of the Llagas Creek watershed have soils developed on sedimentary rock, basic igneous rocks and serpentine rocks. The main soils are of the Los Gatos, Gaviota, Vallecitos and Haymen associations. They range in depth from shallow to deep, and are located on steep to very steep slopes. The vegetative cover includes grasses, oak, pine, brush and hardwoods. The infiltration rates of water in the upland areas is generally slow. The upland soils are classified as having a high to very high erosion potential.

The upland portions of the Llagas Creek watershed have very little development at this time, and the County General Plan calls for only limited development in the future with mostly open space. On the valley floor, most of the Llagas Creek channel and its tributaries are leveed or perched channels with channel banks higher than adjacent areas on one side or both sides of the stream channel. Therefore, overflows from the channel tend to flow away from and parallel to the channel.

Based on information from the Federal Emergency Management Agency (FEMA) Flood Insurance Study for Santa Clara County, there are extensive areas of floodplain from Llagas Creek and its tributaries. The most serious of these are within the City of Morgan Hill from West Little Llagas Creek, and in the City of Gilroy from West Branch Llagas Creek.

The Santa Clara Valley Water District and the Soil Conservation Service have completed a flood control project for the Llagas Creek watershed. The downstream reach from Bloomfield Road to the Ronan Channel has been improved to 100-year design standards, and the reach from the Ronan Channel to Route 101 has been improved to 10-year design standards. In addition, 100-year design channels have been provided in the urban areas of Morgan Hill and Gilroy. Improvements in Gilroy included diversion of West Branch Llagas Creek to the Ronan Channel, and channel improvements upstream to Day Road. The project was designed to eliminate most flooding in Gilroy south of Day Road. This project has been completed, and FEMA is in the process of changing the Flood Insurance Rate Maps for this area.

Site Drainage and Flooding Conditions

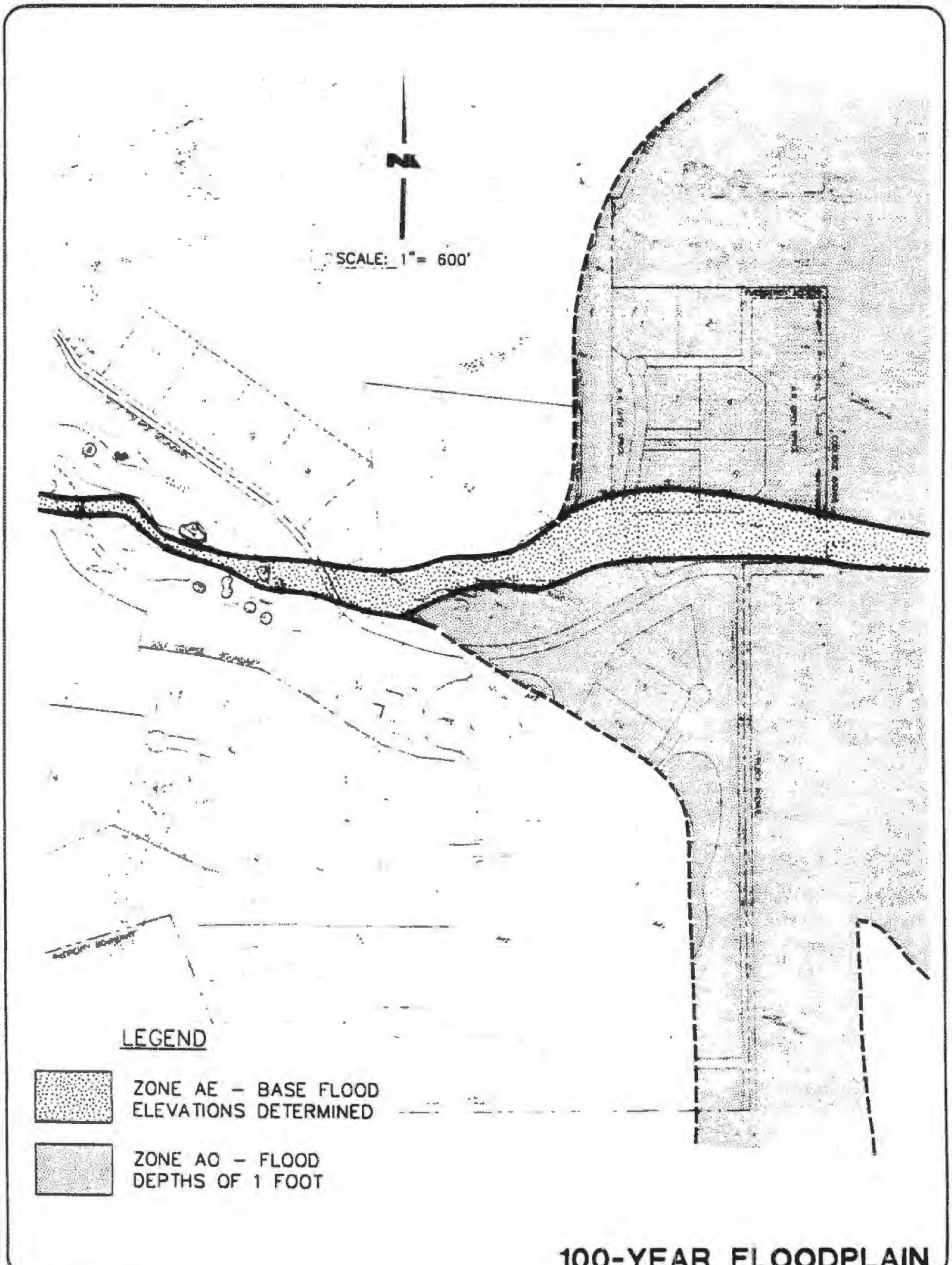
The project site drains to two separate drainages. The western portion of the site drains to the west to Hayes Creek near Watsonville Road while the majority of the site drains via the east to the West Branch Llagas Creek. A network of intermittent and ephemeral streams flow from the higher elevations on the perimeter of the central valley into the West Branch of Llagas Creek. The Creek has 8 primary tributaries, 4 of which drain the hills north of the valley and with the other 4 originating on the southern ridgeline. These tributary streams flow during winter and spring months for varying periods and are dry the remainder of the year. West Branch Llagas Creek discharges to the Ronan Channel which joins Llagas Creek near Highway 152 east of Gilroy. Hayes Creek drains to Llagas Creek near Watsonville Road, south of Morgan Hill. The are no detailed floodplain studies for Hayes Creek. The area is designated as Zone D on the Flood Insurance Rate Map. Zone D is defined as an area of undetermined flood hazard.

As flows in West Branch Llagas Creek reach the eastern project boundary at Coolidge Avenue, they pass under the road through a 3.5' x 6' concrete box culvert. Since the culvert is relatively small compared to the incoming 100-year flow, the creek backs up submerging the culvert and overtopping the northern bank of the channel and flooding the orchard located just north of the channel. As the flow ponds up in the orchard, it crosses Coolidge Avenue at a dip section located approximately 1,200 feet north of the creek. The dip section in the road has a 24-inch reinforced concrete pipe culvert to convey the smaller nuisance flows under the road.

At the southeast corner of the site, ground elevations are low resulting in natural drainage flows toward this corner of the site. As the flows pond up in the corner they enter a 16-inch corrugated metal pipe which conveys the flows from the project site to the adjacent property to the south. The flows then enter two 12-inch pipes that convey the flows under Turlock Avenue to the east. Since the 100-year flow in this area is 161 cfs, which is more than the capacity of the pipes, the road is overtopped at the nearby low point or dip section in the road.

The Flood Insurance Rate Maps for West Branch Llagas Creek do not include detailed floodplain studies upstream of Golden Gate Avenue, approximately 2 miles south of Highland Avenue. The stream channel on the project site is designated as Zone A, approximate 100-year floodplain. At Turlock Avenue, the floodplain is shown as approximately 300 feet wide along the channel north of Highland Avenue.

West Branch Llagas Creek has been restudied by FEMA to update the existing Flood Insurance Rate Maps. The draft work maps are currently in the review process and are not expected to be become effective until late 1996. The SCVWD is using the revised maps as the best available information in the interim. The proposed 100-year floodplain for West Branch Llagas Creek near Highland Avenue is significantly larger on the revised maps than on the current maps. The proposed floodplain includes shallow flooding from the channel commencing at the ranch complex on the project site and including the area south of Highland Avenue, west of Turlock Avenue, and the area north of Highland Avenue west of Coolidge Avenue (see Figure 13).



100-YEAR FLOODPLAIN

FIGURE 13

The hydrology for the detailed floodplain study shows an estimated 100-year peak flow rate of 850 cubic feet per second for West Branch Llagas upstream of Turlock Avenue. An estimated 400 cfs overflows Highland Avenue toward the south upstream of Turlock Avenue. An additional 355 cfs overflows from the channel toward the north upstream of Coolidge Avenue. The northern overflow crosses Coolidge Avenue north site and flows overland to the east and south to the West Branch Llagas Creek channel at Highland Avenue. The majority of the overflow to the south flows overland to the south and east and crosses Turlock Avenue to rejoin the West Branch Llagas Creek floodplain between Highland Avenue and Golden Gate Avenue. A portion of the overflow continues south along the west side of Turlock Avenue.

<u>A more detailed floodplain study was undertaken for the project by Pacific Advanced Civil Engineering in conjunction with preparation of the Master Drainage Plan for the project in January 1998. This floodplain study was based on detailed topographic mapping and ground surveying of the site and adjacent roadways. Therefore, the findings of this study are considered to be the best available information on flooding potential for the project site. This floodplain study estimates the 100-year peak flowrate leaving the eastern edge of the site to be 1,068 cubic feet per second. This includes 797 cfs that overflows the channel of West Branch Llagas Creek east of Coolidge Avenue and north of Highland Avenue, 110 cfs that flows through the culvert at Coolidge Avenue, and 161 cfs that overflows Turlock Avenue in the southeast corner of the site. According to this study there would be no extensive sheet flooding across the eastern portions of the site during the 100-year event, as shown in Figure 13, except for the ponding within 200 feet of Coolidge Avenue north of Highland Avenue and in the extreme southeast corner of the site.</u>

Ordinances and Regulations that Address Drainage and Flooding

<u>County Drainage Manual</u>: This manual contains guidelines for design and installation of drainage facilities for projects. Projects must demonstrate that drainage will be handled adequately in order to avoid drainage and flooding problems. These guidelines ensure that there are no on- or off-site drainage problems associated with a project.

<u>Grading Ordinance</u>: The ordinance requires that all drainage structures and devices be consistent with the adopted County Drainage Manual and its standards. It outlines disposal requirements for both on- and off-site drainage; provides for slope protection and erosion control; and the design of dikes, swales and ditches.

<u>Land Development Regulations</u>: The County Land Development Engineer reviews all projects to ensure no on- or off-site drainage impacts would occur as a result of the proposed project.

<u>Zoning Ordinance</u>: For projects requiring a use permit, Section 47-5(d) of the Zoning Ordinance ensures that adequate storm drainage exists or shall be provided as a part of the project; and that no on- or off-site drainage impacts would result from the project.

<u>Special Flood Hazard Area Ordinance</u>: This ordinance applies to all areas of special flood hazard (i.e., within the 100-year flood zone as established by FEMA) within the unincorporated area of Santa Clara County. No new development shall occur, or structure or improvement shall be constructed in a flood zone without compliance with this ordinance.

Significance Criteria

With respect for flooding and drainage impacts, Appendix G of the CEQA Guidelines states that a project will normally have a significant effect on the environmental if it will: "(g) Cause substantial flooding, erosion or siltation."

Impacts and Mitigation

<u>Impact 1.</u> **The project would potentially result in increased downstream flooding during the 100-year, 10-year, <u>and more frequent storm events</u>. (Potential Significant Impact)**

The proposed residential development on the project site would increase the amount of impervious area on the site and therefore increase the runoff from the site.

The cluster residential development area south of Highland Avenue would be served by storm drains which would discharge to the 20-acre lake proposed for the main subdivision area. The overflows from the lake would discharge via storm drains to West Branch Llagas Creek upstream of Coolidge Avenue. In addition, there are approximately 73 acres of hillside area upstream of this residential development area. Drainage from this area would also be collected by the storm drain system and discharge to the lake. The total area of this drainage area is approximately 240 acres.

The golf course would also be located entirely within the West Branch Llagas Creek watershed which drains to the east. There would be no development in the western portion of the site which drains to the west to Hayes Creek. The West Branch Llagas Creek watershed upstream of Turlock Avenue is approximately 1,060 acres or 1.66 square miles. The golf course development would include approximately 240 acres, the majority of which would be landscaping and turf. The upstream hillside areas would not be affected. The existing creek channel and pond would be largely maintained in their existing configurations. A new pond would be constructed west of the existing pond to serve as an irrigation water reservoir and to detain runoff from the undeveloped area upstream. The new pond would include approximately 9 acre-feet of detention storage.

To analyze potential drainage and flooding impacts, the project site was divided into the following 3 drainage areas: the cluster residential subdivision south of Highland Avenue; the area upstream of the existing pond; the area upstream of the proposed new irrigation reservoir; and the area downstream of the pond golf course reservoir. Discharge rates were estimated for the 10-year and 100-year storms for existing and project conditions.

The results of the flooding analysis show that the proposed golf course would reduce the flow from the site to West Branch Llagas Creek. The golf course would decrease the estimated peak runoff from the watershed because the proposed irrigated turf would maintain a dense layer of thatch which would act as a sponge and reduce runoff, whereas the existing unirrigated range grasses tend to be sparse, with exposed dirt between grass clumps, which does not retain as much runoff. The estimated 100-year peak flow from the golf course area would decrease from 780 cubic feet per second to 765 cubic feet per second, a decrease of 2 percent. The 10-year peak flow rate would decrease from 375 cubic feet per second to 360 cubic feet per second, a decrease of 4 percent.

The proposed golf course irrigation reservoir would also act as a detention facility to reduce the estimated peak flow rate from the western portion of the watershed. For purposes of analysis, the existing pond was assumed to be full at the start of the storm and to have minimal effect on the flood hydrograph. The proposed irrigation reservoir was assumed to be full to spillway elevation at the start of the storm, and to have a 12-foot wide spillway. The estimated storage capacity of the pond is 9-acre-feet with 3 feet of flow over the spillway. The detention storage in the irrigation reservoir would reduce the estimated 100-year peak flow at the pond from 59 cubic feet per second to 39 cubic feet per second, a reduction of 20 cubic feet per second. However when routed downstream and combined with the larger watershed downstream, the detention storage reduces the peak by approximately 10 cubic feet per second. This is due to the difference in timing between the peak flow in the upper watershed and the lower portion of the watershed. The peak flow from the upper watershed is delayed by the travel time along the creek channel and arrives after the peak from the lower watershed. Therefore the peaks do not add directly. The detention storage in the upper watershed acts to increase the timing difference of the upper watershed.

The proposed golf course grading would also include local detention areas to contain runoff from the turf areas for water quality purposes. These would also act to reduce runoff from the site, particularly for small storms. The effect of these detention areas on larger storms would depend on the design and placement of each area and whether the upstream hillside areas would drain to the detention areas or directly to the creek. Therefore, the effects of potential detention storage on the golf course other than the larger pond were not considered in the hydrograph analysis.

The flooding analysis indicated that the proposed cluster residential development would result in a potential increase in the peak runoff from the development site. The 100-year peak flow from the entire watershed would increase from 236 cubic feet per second to 301 cubic feet per second, an increase of 28 percent. The 10-year peak flow rate would increase from 120 cubic feet per second to 160 cubic feet per second, an increase of 33 percent. The increase in peak runoff is due to both the increased impervious area in the development, and the more efficient drainage system which collects runoff faster than the existing overland flow conditions.

~~However, the cluster residential subdivision would include a proposed lake, and runoff would be drained to the lake, then released to West Branch Llagas Creek. Only the proposed equestrian center in the southeastern corner of the site would be below the lake elevation and would drain toward Turlock Avenue. There is no storm drain system along Turlock Avenue, but runoff flows along the road under existing conditions~~.

~~The residential cluster subdivision is located in a drainage area of 240 acres, which would drain to the proposed lake. Without the lake, increased peak runoff from the cluster residential subdivision would potentially increase the peak flow in West Branch Llagas Creek downstream of the project~~.

Mitigation 1. ~~**The on-site lake proposed for the southern residential cluster subdivision would be designed to provide sufficient detention storage for increased peak runoff resulting from site development. In addition, a diversion structure would be constructed in the creek channel to divert a substantial portion of storm flows exceeding existing 10-year flow rates to the residential lake, which would be sized to accommodate about one-half of the flows from the 100-year event. With these facilities, the peak flow rates leaving the project site during significant storm events would be substantially lower than under existing conditions.**~~ **In order to control and detain flood flows generated at the site, a diversion channel, a detention basin and a lake are proposed. These structures would be designed to minimize the extent of flooding within the project boundaries, and would reduce peak flood flows leaving the site during the 100-year, 10-year, and 2-year events relative to existing conditions.**

~~The potential increased runoff from the residential area during the 100-year event would be 65 cubic feet per second, without the proposed lake. The proposed lake would have a normal water surface elevation less than the top of bank elevation of West Branch Llagas Creek at the outfall from the pond. The diversion structure in the creek would be designed such that a substantial portion of the flows in the creek less than the existing 10-year peak flow would pass under the structure and would not be able to enter the side channel to the lake. Flows exceeding the 10-year peak flow would be blocked by the structure and diverted to the lake for temporary storage (see Figure 13a). This would reduce the 100-year flow rate leaving the site from approximately 800 cfs under existing conditions to approximately 400 cfs. This substantial reduction in flood flows leaving the site would significantly reduce flooding problems along the West Branch of Llagas Creek downstream of the site. However, there still would be overland and downstream flooding during the 100-year event, but the extent and volume of flooding would be reduced as a result of the proposed diversion and storage. Once the storage capacity of the lake is reached, any additional flows would be prevented from entering the lake. Instead, these extreme flood flows would be allowed to overspill the creek, as would occur under existing conditions. The outflow from the lake would only occur when the water level in the creek is low. Therefore, the outflow from the pond would not contribute to the existing flood problems from the creek channel.~~

~~Since the residential lake would be sized to contain a substantial portion of the 100-year peak flow, the shallow flooding that occurs along the Turlock and Coolidge Avenue frontage areas of the site during the 100-year event would be significantly reduced (see discussion under 'Impact 2' below).~~

~~The equestrian center area in the southeast portion of the project site would not drain to the pond in the residential development area. Due to the site topography, there would be a berm between the equestrian center and the pond to contain the pond. The maximum height of the berm would be approximately 7 feet. The equestrian center would continue to drain to Turlock Avenue and ultimately to West Branch Llagas Creek. Because of the limited impervious area associated with the equestrian center, there should be no increase in runoff from the area after the project. In addition, the proposed equestrian center would include a detention pond for water quality purposes.~~

The main flood control features include a diversion channel to be constructed along the West Branch of Llagas Creek, a detention basin along Coolidge Avenue, and a lake/detention basin in the residential area south of Highland Avenue. These features are shown in Figure 13a and described below.

Diversion Channel along West Branch Llagas Creek

To eliminate flooding along the banks of the creek at the eastern end of the project, a diversion channel is planned to run parallel and south of the existing creek channel. The new channel would branch southward off the existing channel at a point just west of the roadway ('A Street') for the Rural Residential subdivision where it crosses the existing creek channel. The diversion structure would consist of a spillway 75 feet in length. Flows to the existing creek channel would be limited by two 24-inch reinforced concrete pipes that would serve as culverts under 'A Street'. This would serve to divert the major portion of the flood flows to the new parallel channel. The diversion channel would be trapezoidal with a 10-foot bottom width, and would be grass lined. The diversion channel would rejoin the main channel at a point just west of Coolidge Avenue. The existing creek channel would not be altered.

Detention Basin Along Coolidge Avenue

Since the existing culvert at Coolidge Avenue does not have sufficient capacity to convey larger storm events, flooding occurs to the north in the orchard along Coolidge Avenue with flood flows crossing eastward over the roadway at a low point approximately 1,200 feet north. To control this on-site flooding and to reduce flooding over Coolidge Avenue, a 5.5-acre detention basin is planned adjacent to the roadway. The detention basin would be approximately 200 feet wide and 1,000 feet long and have a storage capacity of 23 acre-feet. Flood flows would enter the detention basin at a spillway alongside the creek channel near Coolidge Avenue. Once the detention basin is filled, it would overflow at the northeast corner where flood flows would cross Coolidge Avenue at the low point or dip section. The detention basin is not intended to eliminate flooding altogether, but it would significantly reduce flood flows crossing the dip section of the roadway relative to existing conditions (see Table 4a). The flow reduction is greatest for the 2-year event, which would undergo a reduction of 63 percent as a result of these improvements.

Lake/Detention Basin South of Highland Avenue

The 20-acre lake planned for the residential area south of Highland Avenue would be designed to provide 50 acre-feet of flood storage during major storm events. Drainage from the adjacent residential area and the tributary area in the hills to the west would be conveyed to the residential lake. Once the lake has reached capacity, flows would enter a swale at the south end of the lake which would convey flows to the southeast corner of the site. Overflows from the swale would cross Turlock Avenue at a low point or dip section in the roadway as occurs under existing conditions. However, the flood flows crossing the dip section would be significantly reduced for all major storm events, relative to existing conditions (see Table 4a). The flow reductions are greatest for the 10-year and 2-year events, which would undergo reductions of 62 percent and 81 percent, respectively, as a result of these improvements.

TABLE 4A

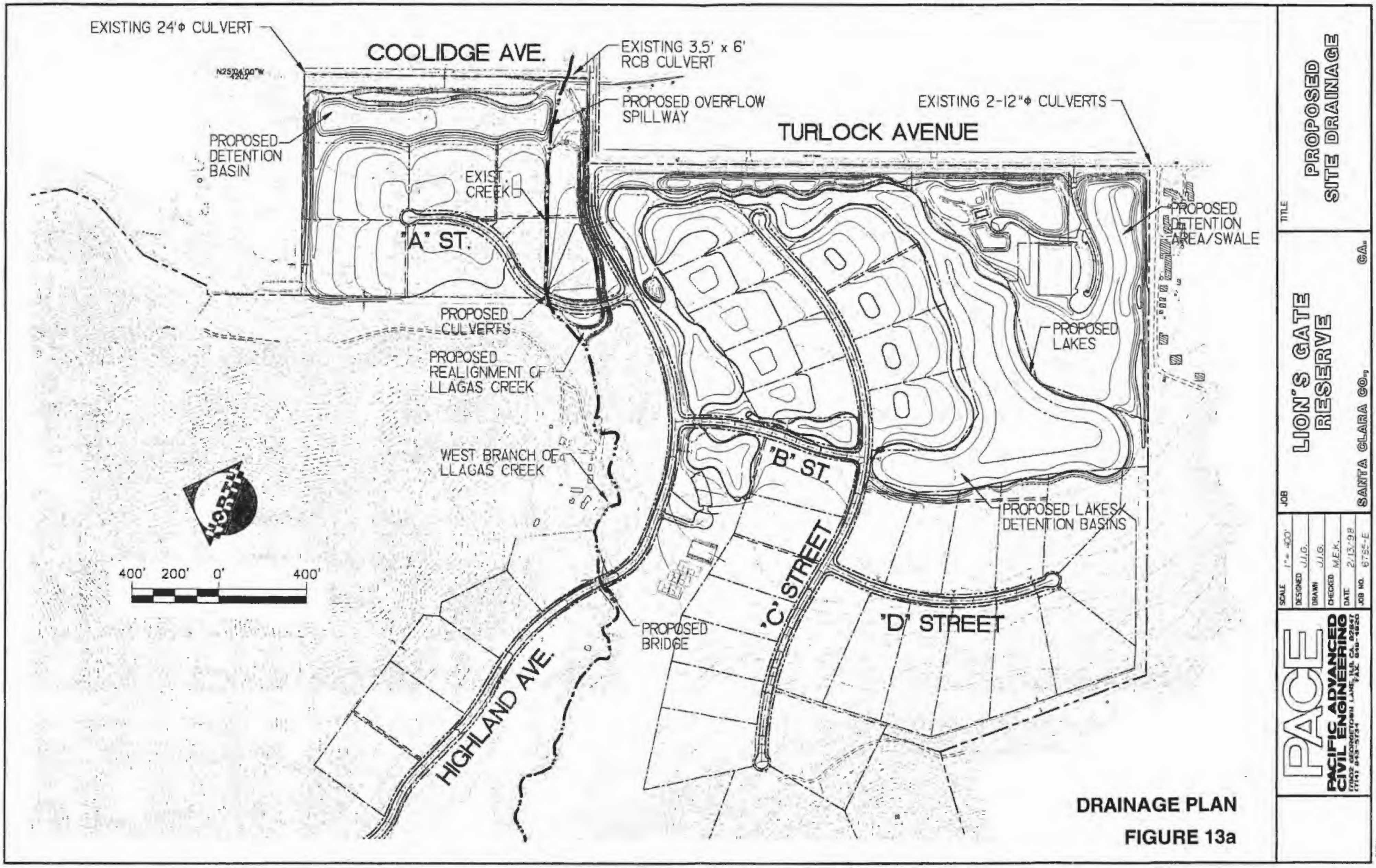
COMPARISION OF FLOOD FLOWS LEAVING THE SITE FOR THE EXISTING AND DEVELOPED CONDITIONS

	Existing Condition Flow (cfs)	**Developed Condition** Flow (cfs)	**Percentage Flow Reduction**
Coolidge Ave. Dip			
100-year	797	753	6%
10-year	332	294	11%
2-year	86	32	63%
Turlock Ave. Dip			
100-year	161	128	20%
10-year	73	28	62%
2-year	31	6	81%

Impact 2. **Portions of the residential cluster subdivisions ~~would~~ may be subject to shallow flooding (one foot average depth) during a 100-year event, and the proposed dwellings could also potentially obstruct this sheet flow through the site. However, the total area of the site that may be subject to shallow flooding would be reduced by flood control improvements included in the project. (Potential Significant Impact)**

Based on the revisions to the existing Flood Insurance Rate Map, shown in Figure 13, the West Branch Llagas Creek would overflow to the south upstream of Turlock Avenue (i.e., at the on-site ranch complex). For the 100-year flood, the FIRM shows that approximately 400 cubic feet per second would cross through the northeastern portion of the cluster residential development, in particular through Lots 12, 13 and 14 at the northeast corner of the subdivision. This mapped overflow crosses the site and Turlock Avenue to rejoin West Branch Llagas Creek 500 to 1,000 feet downstream of Highland Avenue. The overflow is indicated as shallow flooding with an average depth of one foot, indicating that the proposed lots would be prone to flooding. In addition, grading for the residential lots in the overflow area could adversely affect the sheetflow through the area if the flow is obstructed. Similarly, grading for the access road the project and landscaping along Turlock Avenue could affect the sheetflow across the site.

The revised flood maps also show an overflow to the north from West Branch Llagas Creek upstream of Coolidge Avenue. For the 100-year flood, approximately 355 cubic feet per second would cross through proposed the rural residential development north of Highland Avenue and west of Coolidge Avenue. The overflow would flow overland to rejoin West Branch Llagas Creek at the culvert under Highland Avenue. Part of the overflow is designated as shallow flooding with an average depth of one foot, and a small sliver along the north boundary is indicated for flood depths of 0.5 to 2.5 feet. All six of the 5-acre lots are within the mapped 100-year floodplain area and thus would be prone to flooding. Also, grading for the residential lots and cul-de-sac in the floodplain could have an adverse affect on the sheetflow if flow is obstructed.



DRAINAGE PLAN

FIGURE 13a

Both the area subject to potential sheet flooding and the volume of flood water spilled would be substantially reduced by the flood diversion and storage facilities described under 'Mitigation 1' above. The Coolidge detention basin, the residential lake would detain the increment of runoff generated by the project in addition to ~~approximately 400 cfs~~ a portion of the peak flow during the 100-year event (see Table 4A above).~~, which would represent approximately one half of the overland flows overspilling the creek west of Coolidge/Turlock Avenues on the project site during the 100-year event. The precise reduction in flood plain area would be calculated in conjunction with the preparation of the Final Master Drainage Plan for the project.~~

As noted under 'Environmental Setting', a more detailed floodplain study was undertaken for the project by Pacific Advanced Civil Engineering in conjunction with preparation of the Master Drainage Plan for the project in January 1998. This floodplain study was based on detailed topographic mapping and ground surveying of the site and adjacent roadways. Therefore, the findings of this study are considered to be the best available information on flooding potential for the project site. According to this study there would be no extensive sheet flooding across the eastern portions of the site during the 100-year event, as shown in Figure 13, except for the ponding within 200 feet of Coolidge Avenue north of Highland Avenue and in the extreme southeast corner of the site.

Mitigation 2. **Potential impacts to the residential subdivisions from shallow flooding would be mitigated by constructing building pads on fills raised above flood elevations. The potential obstruction of sheetflows by the proposed development would be mitigated by balancing fills with cuts within the flood-prone areas.**

The potential impact of placing a portion of the proposed residential development within the 100-year floodplain areas, as shown on the revised Flood Insurance Rate Maps, would be mitigated by balancing the grading within the 100-year floodplain. This would mean that fills required to elevate building pads above flood elevations would need to be balanced by cut areas to allow flood flows between the buildings. This procedure is generally most effective in shallow flooding areas with limited building coverage as in the proposed project. If the buildings cover a large percentage of the floodplain and are in deeper flood area, and effective balance between cut and fill would be problematic. For instance, if a building obstructs 50 percent of the floodplain in 3 feet of flood depth, the building pads would have to be elevated 3 feet, and the remainder of the floodplain would have to be excavated 3 feet to balance the cut and fill. This would lead to an elevation difference of 6 feet between the building pads and the adjacent ground. In the proposed project, the building densities would be very low with 2- to 3-acre residential lots. Thus, building elevations of 1 to 2 feet above existing grade would become 2 to 3 feet or less above the new ground elevations because of the larger area available to balance the fill.

In addition, the frontage berms proposed along Coolidge and Turlock Avenues would include sufficient breaks within the flood-prone sections such that the direction of sheet flow during major storm events would not be altered relative to existing conditions.

Although the most recent floodplain study undertaken by Pacific Advanced Civil Engineering in conjunction with the Master Drainage Plan indicates that the eastern portions of the would not be subject to extensive sheet flooding during the 100-year event, the building pads for the dwellings will be above the flood elevations shown on the revised Flood Insurance Rate Maps.

Conclusion. **With implementation of the above mitigations as proposed in the project, the potential flooding impacts of the project would be reduced to less-than-significant levels.**

F. WATER QUALITY

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Impacts and Mitigations

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Impact 3. **The project would generate urban nonpoint pollutants which may be carried in stormwater runoff from paved surfaces to downstream waterbodies. (Potential Significant Impact)**

The introduction of traffic and parking areas would increase accumulated hydrocarbon byproducts and heavy metals from automobiles, which would be flushed into drainages and streams. At the maintenance facility, washwater, lubricants and hazardous materials may be generated. Unless controlled, these urban pollutants would contribute to cumulative nonpoint contaminant loads in downstream drainages and waterbodies.

Mitigation 3. The project would include stormwater controls at the parking lots and maintenance facilities.

Sheet flows over the clubhouse ~~and practice range~~ parking lots would be collected and ~~piped to nearby stormwater retention basins. The collected runoff would not be discharged into the West Branch Llagas Creek, but would percolate into the soil or evaporate. The retention basins would be cleaned of accumulated sediments and debris as needed~~ conveyed to the underground drainage system for the golf course. Prior to discharge into the main creek channel, the parking lot runoff would pass through a biofilter consisting of cobbles and gravel to remove sediments and debris.

J. VISUAL AND AESTHETICS

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Impacts and Mitigation

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<u>Impact 1.</u> **The project would result in visual changes to some areas of the site open to public view. (Potential Significant Impact)**

As discussed under 'Environmental Setting' above, the most visually accessible areas of the site are located along Coolidge Avenue (Santa Teresa Boulevard) and Turlock Avenue at the eastern end of the site, and along Watsonville Road to the west. The interior valley area of the site is not visible from off-site vantage points except for the single home that overlooks the site from the northern ridge. The hillside areas nearest to the flanking roadways are also visible.

The residential subdivisions proposed for the eastern end of the site would be partially visible from adjacent land uses and roadways. In the Rural Residential subdivision proposed adjacent to Coolidge Avenue, north of Highland, the 6 proposed lots would be set back from the roadway at least 300 feet toward the adjacent hillside to the west. The setback area would remain as permanent open space, with a landscaped berm ~~and a planted vineyard~~ providing visual screening for these lots. <u>A stormwater detention basin would occupy the open space area between the roadside berm and the residential lots; however, the basin would be entirely screened from the roadway by the intervening landscaped berm.</u>

The residential cluster subdivision proposed for the field west of Turlock Avenue would also be partially visible to passing motorists. However, this subdivision would be set back 200 feet to 1,400 feet from the roadway, and would be screened by the landscaped berms planted with black walnut trees. Nevertheless, the roof lines of the nearest dwellings would be visible from Turlock Avenue and Santa Teresa Boulevard, at least until the black walnuts have matured enough to provide more complete screening (see Figure 16). Since two of the proposed lots (Lots 24 and 25) extend into the adjacent hillside area, it is possible that future custom homes to be built on these lots may be visible from Turlock Avenue and Santa Teresa Boulevard.

<u>The small horse stable planned for the northwest corner of the site would be sited in a small side valley along the toe of the eastern hillsides. The nearest existing land uses include a nursery business located approximately 500 feet east and two single-family dwellings located approximately 800 feet to the northeast and the southeast. The existing nursery with its dense boundary landscaping almost completely screens the stable from view of Coolidge Avenue and the residences in the vicinity.</u>

The package wastewater treatment plant and residential lake occupy the area between the roadside berm and the residential subdivision. However, these project components would be low in profile and almost completely shielded from view by the landscaped berm along Turlock Avenue.

The only other visual changes that would occur at the eastern end of the site would be the roadway improvements and entry features along the Highland Avenue entry way. However, any improvements would be subject to Architecture and Site Approval to ensure that signs, fences, lighting and other features would be compatible with their surroundings. Also, the existing mature landscaping trees around the ranch complex would be retained and incorporated into the project.

From Watsonville Road to the west, very little of the project, if anything, would be visible. All of the area with ¾ mile of the roadway is proposed to be maintained as permanent open space. The golf course would be located to the east of the low saddle that crosses the western portion of the valley, and thus would not be visible from Watsonville Road. It is possible that the maintenance facility proposed for the western end of the golf course may be partially visible from Watsonville Road, ¾ mile to the west. The only evidence of the project alongside Watsonville Road would be the new maintenance access road to be constructed from Watsonville Road to the golf course maintenance facility. There would be no structural entry features such as signage here since no public access to the golf course would be permitted from this direction.

In the interior area of the valley, the golf course, clubhouse and overnight units would not be visible from off-site vantage points~~, except for~~ even from the single dwelling that overlooks the valley from the adjacent ridge to the north. From the vantage point of this residence, the clubhouse/overnight complex would be completely blocked by the intervening low hills and ridges just north of the complex.

Mitigation 1. **The project would be designed and landscaped in a manner to help it blend in with the natural and rural surroundings, and to reduce its visibility from off-site locations.**

The site planning measures proposed as part of the project, including buffer zones from all adjacent roadways, as well as the proposed landscaping and berming, would minimize the potential visual effects of the project. The design of the residential areas reflects many of the guidelines of the San Martin Integrated Design Plan (see Section *II. Consistency with Plans, Policies and Regulations.)*

All structural elements such as signs, fences, lighting or other entry features would be subject to Architectural and Site Approval to ensure their compatibility with the surroundings. In addition, any structures proposed within 100 feet of adjacent scenic roads would be subject to the County's Design Guidelines.

L. NOISE

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Impacts and Mitigation

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Impact 3. **Noise generated by golf course mowers would have a potentially adverse effect on the nearest dwellings proposed on the project site. (Potential Significant Impact)**

The mowing machines used at the golf course would be the loudest noise sources. These pieces of equipment typically generate noise levels no higher than 70 dBA at a distance of 50 feet.

The closest existing residence to the proposed golf course would be the existing ranch house near the eastern limits of the project site, which would be 900 feet from the golf course at its nearest point. At this location, the highest noise levels from mowers would be approximately 45 dBA. The average noise levels would be less.

The second closest residence would be the home to the north of the central area of the site on the nearest off-site ridge, which would look down on the golf course, and which would be 1,600 feet from the golf course at its nearest point. For this home, the maximum noise levels generated by lawn mowers would be 41 dBA at the nearest point near the project boundary. This noise level would be barely audible with the windows open.

The closest new lots in the proposed cluster subdivision along the north side of the main access road would be ~~200~~ approximately 70 feet from the golf course at the nearest points, and the dwellings themselves would be ~~at least~~ ~~230~~ approximately 120 to 150 feet away (~~given the minimum required front setback distance of 30 feet~~ since these estate homes would be set back at 50 to 80 feet from the roadway). At the ~~nearest~~ front of the dwellings, the maximum noise from mowers would be approximately ~~58~~ 61 to 63 dBA (noise levels drop off by 6 dBA for each doubling of distance from the source). The maximum levels of mowing noise would exceed the County's 55 dBA threshold for the new proposed lots in the subdivision located north of Holes ~~8~~ 1 and ~~9~~ 2 at the eastern end of the golf course. According to the County's noise ordinance, however, ~~the maximum mowing~~ noise levels of ~~58~~ 60 to 65 dBA would not constitute a noise impact if the residences were subject to these noise levels for less than ~~15~~ 5 minutes in any hour, and noise levels of 55 to 60 dBA would not constitute a noise impact if the residences were subject to these noise levels for less than 15 minutes in any hour. Since maximum noise levels would drop off to 55 dBA at a distance of approximately ~~330~~ 350 feet from the source, ~~the noise threshold would be exceeded by the mowing of a band of turf 100 feet wide (or less, depending on the location of individual dwellings relative to the fairway)~~ turf mowing would be in compliance with the noise ordinance if it occurred for no more than 5 minutes in any hour along the northern-most 30 feet of fairway along the roadway, and for no more than 15 minutes in any hour with the band between 30 feet and 230 feet from the roadway. It is expected that the gang reel mowers would complete mowing of ~~that strip within 15 minutes~~ these areas within the alloted times with respect to any of the individual residences affected. It should be noted that the average noise level generated by mowers would be less than 5 dBA above the background level in the area of the proposed residences. In addition, fairway mowing would typically occur in the afternoon.

Mitigation 3. **The hours of mowing within ~~330~~ 350 feet of any existing or proposed residences, would be restricted to weekdays between the hours of 8:00 a.m. and 5:00 p.m., with total noise generating activities in those areas restricted in accordance with the limits set forth in the County's Noise Ordinance.**

Beyond the requirements of the County's Noise Ordinance, the CC&Rs for the project should establish clear guidelines for operational golf course noise to minimize potential annoyance and inconvenience for all concerned.

Impact 4. **Activities at the clubhouse would increase noise levels in the interior of Hayes Valley. (Less-than-Significant Impact)**

Events at the clubhouse, such as weddings or banquets, would generate noise from music played at such events. There are two existing residences in the vicinity which would be within audible range of the clubhouse. One residence is located approximately ~~3,600~~ 3,000 feet from the clubhouse on the northern ridge overlooking the valley. An on-site ridge located mid-way between the clubhouse and this residence would break the line of sight between these two structures and would provide noise shielding under normal atmospheric conditions. The second potentially affected residence is the existing on-site ranch house located approximately ~~2,400~~ 3,000 feet east of the clubhouse, along West Branch Llagas Creek. The line of sight between the clubhouse and the ranch house would be unbroken by intervening terrain.

To evaluate potential noise impacts to these existing residences, worst-case meteorological conditions were assumed. The conditions of maximum sound propagation would be a temperature inversion with a light wind blowing toward the receiver. Under these conditions the sound levels would bend down from the atmosphere toward the receptor, thus negating shielding by intervening hills, buildings and other barriers. It was calculated by Illingworth & Rodkin that the sound level of a loud rock band inside the clubhouse with the windows open would be about 35 to 40 dBA outside the on-site ranch to the east, and about 35 dBA outside the ridgetop house to the north. Under the vast majority of meteorological conditions, sound levels would be 10 to 20 dBA lower, and essentially inaudible. Under conditions of good sound propagation, the sound of a very loud event at the clubhouse could be audible outdoors at these residences. However, it is also most likely that under these conditions the windows in the clubhouse would be closed because it would have to be quite cold to create the type of inversion needed to result in the highest sound levels. Therefore, it is expected that sound from the clubhouse would be audible at the nearest residences, but only under rare circumstances.

The nearest residences proposed within the project itself would be located ~~1,200~~ 1,500 feet to the east of the clubhouse. Under the worst-case meteorological conditions described above, the noise level at the nearest residence would be about 40 to 45 dBA, outside the residence. This noise level would still be well under the County's noise criteria of 55 dBA for residential land uses.

Mitigation 4. **No mitigation required.**

<u>Impact 5.</u> **Noise levels would be temporarily elevated during grading and construction. (Potential Significant Impact)**

Most of the existing noise receptors in the area are far from the main grading and construction area of the golf course. The major exception is the existing ranch house at the east end of the site. During construction, maximum noise levels generated by grading, paving, and other activities would be 5 to 10 decibels lower. If average levels do not exceed 55 dBA, there would be no interference with outdoor activity or indoor activity, although the construction may be occasionally audible. Noise levels at the existing ranch could reach as high as 80 dBA with average levels of up to 75 dBA. During most of construction, however, noise levels would be significantly below 55 dBA.

The existing residence on the ridge to the north of the project site would be approximately 1,200 feet from the nearest grading activity for the golf course. At this distance, the sound of equipment would be noticeable but would not exceed 55 dBA.

At the eastern end of the project site, existing dwellings in the vicinity would be subject to short-term grading and construction noise impacts from construction of the perimeter berms, <u>the detention basin along Coolidge Avenue, the package wastewater treatment plant and lake/detention basin along Turlock Avenue,</u> and to a lesser extent the proposed residential subdivisions which would be set back from the site boundary.

At the western end of the site, the construction of the maintenance access road to Watsonville Road would generate noise from grading and paving. The nearest existing dwelling would be 700 feet from this maintenance road at its nearest point, and would not be subject to construction noise impacts, although the noise would be audible.

Mitigation 5. **Short-term construction noise impacts would be reduced through compliance with the County's Noise Ordinance with respect to hours of operation and maximum noise levels at adjacent property lines. <u>At the eastern edge of the project, the berms proposed along the project boundary would be constructed during the early phases of grading to provide a noise barrier for existing residences nearby.</u>**

~~For example,~~ <u>T</u>he Noise Ordinance stipulates that construction noise generated between 7 am and 7 pm on weekdays and Saturdays should reach noise levels no greater than 75 dBA at an adjoining property line of a single-family or two-family dwelling.

These hours would be enforced by the grading inspector, and also the County Department of Environmental Health in the event of a violation of the County Noise Ordinance.

To minimize noise generation, construction equipment should be maintained in good operating condition and properly muffled.

To further reduce construction noise impacts, the berms proposed for the eastern project boundaries would be constructed during the early phases of grading in order to provide shielding from construction and grading in the interior of the project. <u>This would be</u>

particularly effective in attenuating noise from grading and excavation for the detention basin along Coolidge Avenue, and the package wastewater treatment plant and lake/detention basin along Turlock Avenue.

Conclusion. **Implementation of the above mitigation measures would reduce noise impacts resulting from the project to less-than-significant levels.**

N. HAZARDOUS MATERIALS, PUBLIC HEALTH AND SAFETY

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Impacts and Mitigations

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Impact 4. **The ~~equestrian facility~~ horse stable could result in potential vector and odor impacts. (Potential Significant Impact)**

Vectors such as flies and rodents could become a problem if the stable~~s are~~ is not properly managed. Offensive odors could develop from a large accumulation of manure or other poor husbandry practices.

Mitigation 4. **The ~~equestrian facility~~ stable would employ vector control measures, and would be operated in accordance with a manure management plan in conformance with State law, which would also be reviewed and approved by the County Department of Environmental Health.**

A manure management plan would be required under Title 23, Chapter 15 of the California Code of Regulations. The stable would be operated as cleanly as possible to reduce vectors and the potential for odor. Specific vector controls would include baiting for flies, manure management and rodent trapping. Hay would be stored in a small barn and all feed grain would be stored in enclosed containers to reduce availability to rodents.

Manure management practices would consist of cleaning up manure daily and placing it in debris boxes which would be emptied ~~daily or every other day~~ on an as-needed basis and taken to a local landfill.

The ~~equestrian facility~~ stable would be subject to Article 47 of the County zoning ordinance which requires that stables not create a nuisance, and that they be set back from water courses and neighboring uses. The ordinance requires that erosion control plans be prepared for stables, and that they by subject to Architecture and Site Approval.

APPENDICES

APPENDIX C

Geologic and Geotechnical Site Review

Prepared by

Twining Laboratories

May 1998

GEOLOGIC AND GEOTECHNICAL SITE REVIEW
NEW CLUBHOUSE AND OVERNIGHT LODGES
CORDEVALLE GOLF CLUB AND HOTEL
SAN MARTIN, CALIFORNIA

Project Number: D34301.03

Prepared by The Twining Laboratories

for:

Lion's Gate Limited Partnership, LLC
395 Oyster Point Boulevard, Suite 309
South San Francisco, California 94080

May 29, 1998

D34301.03-02

May 29, 1998

Lion's Gate Limited Partnership, LLC
395 Oyster Point Boulevard, Suite 309
South San Francisco, California 94080

Attention: Mr. Sky Joyner

Subject: Geologic and Geotechnical Site Review:
New Clubhouse and Overnight Lodge Area
Cordevalle Golf Club and Hotel
San Martin, California

Dear Mr. Joyner:

The Twining Laboratories (Twining) is pleased to submit this report of Geologic and Geotechnical Site Review evaluating potential geologic and geotechnical hazards that could impact the new Clubhouse and Overnight Lodges site at the Cordevalle Golf Club and Hotel. The proposed site location is on a gently sloping hillside, north of the golf course. Geologic and geotechnical hazards were previously evaluated by Twining for a Clubhouse and Overnight Lodge facilities site about 1,000 feet south of the currently proposed site location. Twining has also performed several geologic and geotechnical investigations for other projects near the proposed Clubhouse and Overnight Lodge site (see section 2.0). In addition, Twining performed a geologic site reconnaissance for the subject site and is currently conducting a preliminary geotechnical investigation. Twining was requested and authorized to perform this site review by Mr. Ron Davis, with the Cordevalle Golf Club and Hotel, on May 20, 1998.

The potential hazards investigated included expansive soils, erosive soils, shallow groundwater, landslides and slope stability, seismic ground shaking, fault rupture, earthquake induced liquefaction and seismic settlement. Our assessment indicates that the proposed development is feasible with respect to the geologic and seismic hazards evaluated, provided the conclusions and proposed mitigative measures described in this report are implemented.

We appreciate the opportunity to be of service to Lion's Gate Limited Partnership, LLC. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience.

Sincerely,
THE TWINING LABORATORIES, INC.

Kenneth J. Clark, CEG
Engineering Geologist
Geotechnical Engineering Division

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1.0 INTRODUCTION

This investigation evaluates potential geologic and geotechnical hazards that could impact the new Clubhouse and Overnight Lodges site located at the Cordevalle Golf Club and Hotel.

The Geotechnical Engineering Division of Twining, headquartered in Fresno, California, performed the investigation. This report is provided specifically for the new Clubhouse and Overnight Lodges site located at the Cordevalle Golf Club and Hotel, referenced in the "Background Information" section of this report.

2.0 PURPOSE AND SCOPE OF INVESTIGATION

The purpose of the investigation was to evaluate geologic hazards and general geotechnical conditions relevant to the proposed development of the subject property in support of an environmental impact report for the project.

This investigation did not include a design level geotechnical engineering investigation, pavement design, floodplain investigation, compaction tests for construction, environmental investigation, or environmental audit.

The actions undertaken during the investigation are summarized as follows.

I. The following documents prepared by others were reviewed:

- o Prepurchase Site Assessment of Geologic Hazards, Ground Water Supply and Environmental/Toxic Contamination, Hayes Valley Property, Santa Clara, California, Project 4297, prepared for LAND USE, by TERRATECH, INC., January 20 1988.

- o Supplemental Geological Reconnaissance Investigation for Proposed Hayes Valley Dams, Santa Clara County, California, prepared by Kaldveer Associates Geoscience Consultants, August 4, 1989.

- o Geologic Input to Draft Environmental Impacted Report, Lions Gate Development, project HRC-101B, prepared by Wahler Associates for HR Development Partners, April 17, 1990.

- o Geologic Input to EIR, prepared by ENGEO Incorporated, April 13, 1993.

- o Geologic Feasibility Investigation, Golf Course Maintenance Building, The Lion's Gate Reserve, San Martin , California, Project 1385/6G, prepared for Hayes Valley Development Partners, by Pacific Geotechnical Engineering, December 1995.

- o Preliminary Geologic Feasibility Evaluation, Homesites on Parcels #24, #25, and #26, The Lion's Gate Reserve, San Martin, California, Project 1385/7G, prepared for Hayes Valley Development Partners, by Pacific Geotechnical Engineering, December 1995.

- o Geologic Feasibility Investigation, Clubhouse and Overnight Lodges, The Lion's Gate Reserve, San Martin, California, Project 1385/5G, prepared for Hayes Valley Development Partners, by Pacific Geotechnical Engineering, December 1995.

- o Administrative Draft Environmental Impact Report, Volume IIa Technical Appendices, Lion's Gate Reserve, December 1995.

- o Draft Environmental Impact Report, Volume II Technical Appendices B through E, Lion's Gate Reserve, March 1996.

II. The following geologic and geotechnical reports prepared by The Twining Laboratories were reviewed:

- o Report entitled Preliminary Geotechnical Engineering Investigation, Golf Coarse, dated March 18, 1997, and Addendums No. 1 and No. 2.

- o Letter report entitled "Review of Site Geologic Conditions and Grading Plans, Golf Course Phase", dated May 6, 1997.

- o Report entitled "Preliminary Geotechnical Engineering Investigation, Clubhouse and Overnight Lodges" (former proposed site), dated October 30, 1997.

- o Letter report entitled "Preliminary Evaluation of Geotechnical and Geological feasibility, Clubhouse and Overnight Lodge Area" (proposed new site), dated April 16, 1998.

III. Reviewed pertinent published geologic literature and maps for the project site area.

IV. A site reconnaissance and subsurface exploration were conducted on May 5 and 20, 1998.

V. Mr. Ron Davis and Mr. Sky Joyner with Cordevalle Golf Club and Hotel, and Mr. Bert Verrips with Nolte Engineering, and Mr. Loren Kroeger with Backen & Gillam Architects, were consulted during the investigation.

VI. The data obtained from the investigation were evaluated and this report was prepared to present our findings and recommendations.

3.0 BACKGROUND INFORMATION

The following background information is based on our review of the documents listed in section 2.0, consultation with the project planners, and geologic reconnaissance, and our preliminary geotechnical investigation of the site. The site description, anticipated construction, previous studies, and regional geologic conditions are summarized in the following subsections.

3.1 Site Description: The proposed Clubhouse and Overnight Lodges site occupies a portion of the 1,676 acre Cordevalle Golf Club and Hotel. The Cordevalle Golf Club and Hotel is located west of the intersection of Highland and Turlock Avenues, about two miles southwest of the City of San Martin in Santa Clara County, California. A site location map is provided as Drawing No. 1. The proposed Clubhouse and Overnight Lodges are to be located on a gently sloping colluvial swale and a sloping hillside, north of the proposed golf course. A Pro Shop is proposed east of the Clubhouse, and a Range House is proposed north of the Clubhouse. An ephemeral creek is located between the Clubhouse and the Pro Shop. Llagas Creek is located about 75 to 100 feet south of the proposed facilities. Native slope gradients range from about 2.5 horizontal (H) to 1 vertical (V) on the hillside, to nearly flat near the ephemeral creek. Drawing No. 2 provides a conceptual plan of the facilities.

3.2 Anticipated Construction: We understand that design of the proposed Clubhouse and Overnight Lodges is currently underway, and final details have not been finalized. Anticipated construction includes the construction of the Clubhouse, Overnight Lodges, and associated asphaltic paved roads, parking lots, driveways, and cart paths. The proposed construction will include a Clubhouse building which is largely one-story and 34,000 square feet in plan dimension. The Clubhouse will include an approximate 7,000 square foot second-story, and a 9,000 square foot partial basement for a wine cellar and cart storage. We anticipate the Clubhouse will have a slab-on-grade floor at the basement level, and concrete floor slabs on a steel framed metal pan deck for the ground floor level.

Forty-five Overnight Lodges are planned for the south facing hillside slope, west of the proposed Clubhouse. The Overnight Lodge units will be 550 to 600 square feet in plan dimension. Five meeting rooms with plan dimensions of about 500 square feet will be connected to individual Overnight Lodge units. The lodges and meeting rooms will be slab-on-grade, wood-frame structures.

Grading plans were not available at the time this report was prepared; however, earthwork cuts on the order of 10 feet or less and fills of 10 to 20 feet are anticipated for the Clubhouse and Overnight Lodges.

3.3 Previous Studies: We have reviewed the geologic reports listed under "Purpose and Scope". Most of the cited reports present descriptions of regional geologic and tectonic conditions, and general site geologic conditions. Our summary of these regional conditions are presented below. Geologic conditions applicable to the subject site, which are described in these reports, and conditions noted during our field reconnaissance and geotechnical investigation of the site, are summarized in the "Evaluation" section of this report.

3.4 Regional Geologic Conditions: The earth materials underlying the project site region are composed of rocks belonging to the Franciscan Complex of Jurassic to Cretaceous age. Bedrock types found within the Hayes Valley area include sandstone, shale, chert, limestone, greenstone, and low grade metamorphic rocks. Many areas of bedrock terrane include a mixture of different rock types in a sheared matrix. This formational mixture is termed a melange, and was formed as a result of intense shearing and faulting. Serpentine type rock is also found within this assemblage of rocks.

The regional trend of geologic structures in the Hayes Valley area is roughly east-west, acute to the overall geologic structure of north 40 degrees east for the Santa Cruz Mountains as a whole. Physiographic features, bedrock contacts, and faults are generally parallel to this structural trend.

The distribution of geologic units and structures (including faults) depicted on the ENGEO map is generally suitable for planning purposes for the proposed project. This map is included as Figure No. 2 of the report entitled "Geologic Input for EIR For Lion's Gate Property", dated April 13, 1993 (contained in the Draft Environmental Report [DEIR]).

The Sargent-Berrocal faults are located approximately 2.5 miles southwest of the site and the active San Andreas Fault is located approximately 5 miles southwest of the site. The active Calaveras and Hayward faults are both located approximately 8 miles northeast of the site. Regional geologic maps prepared by U.S. Geological Survey and the California Division of Mines and Geology show a bedrock fault and bedrock contacts within the melange terrane on the north side of Hayes Valley. The fault and contacts are shown on the Geologic Index Map (Figure 1) of the Geologic Feasibility Investigation for the Clubhouse and Overnight Lodges, prepared by Pacific Geotechnical Engineering, dated December 1995.

4.0 FIELD INVESTIGATION

4.1 Geologic Field Reconnaissance: A geologic field reconnaissance of the proposed Clubhouse and Overnight Lodges area was conducted in conjunction with our geotechnical investigation of the golf course and surrounding areas, performed on April 28, 1997. The reconnaissance, which included confirming previously mapped geologic features and noting potential geologic hazards, was performed by Mr. Kenneth J. Clark, a Twining Certified Engineering Geologist. Field reconnaissance of the subject site was also performed by Mr. Clark on May 5, 1998. The results of the reconnaissance are provided in the Section 6.0 "Evaluation".

4.2 Geotechnical Investigation Test Borings: On May 5, 1998, five test borings were drilled by Twining within the proposed Clubhouse and Overnight Lodges area to investigate soil, rock, and groundwater conditions. The borings were advanced to a depth of 20 feet below site grade, or until refusal was encountered on hard bedrock materials. Disturbed and undisturbed soil samples were collected for geotechnical laboratory analyses in conjunction with the geotechnical engineering investigation. In addition, two bulk samples were collected for R-value testing (for pavement design). The test borings were drilled, and R-value samples were collected at the locations shown in Drawing No. 2.

4.3 Exploratory Trenching: On May 7, 1998, two exploratory trenches (Trenches A and B) were excavated across the proposed Clubhouse and Overnight Lodges sites to assess potential faults and general subsurface soil and rock conditions. Three additional exploratory trenches (Trenches C, D, and E) were excavated on May 20, 1998 in the area of the Overnight Lodges. Soil and rock exposed in the trench walls were observed by Twining's engineering geologist on May 20, 1998. The locations of the trenches are shown on Drawing No. 2.

5.0 FINDINGS

5.1 Site Soil and Rock Conditions: The project site spans two general geologic units. The proposed locations of the eastern half of the Clubhouse, Range Building, and the entire Pro Shop are located predominantly on Quaternary alluvial soils located in the lower drainage areas. Soils below the proposed Clubhouse and Pro Shop locations comprised gravelly and sandy lean clays. The clays were generally soft to medium stiff from the ground surface to a depth of about 3 feet BSG. The underlying clays were stiff to very stiff as indicated by Standard Penetration Resistance blow counts documented during collection of soil samples. Weathered greenstone was encountered between depths of 7 to 19 feet below site grade (BSG) in the borings drilled at the proposed clubhouse and Pro Shop sites. If treated as a soil, the weathered greenstone was dense to very dense as indicated by blow counts.

The Overnight Lodges are to be located on the hillside portions of the site. The hillside areas are comprised primarily of relatively shallow soils overlying greenstone rocks of the Franciscan Complex. Exploratory trenches revealed bedrock at a depth of about 2 to 4 feet BSG on the hillside areas investigated.

5.2 Groundwater Conditions: Groundwater occurs in the alluvial soils on the eastern portion of the site, in the area of the proposed Clubhouse, Range House, and Pro Shop. Groundwater was encountered at depths of 5.5 and 16 feet BSG in two borings drilled in the eastern portion of the Clubhouse area. One test boring drilled in the western portion of the Clubhouse did not encounter groundwater above the depth of refusal on bedrock at 7 feet BSG.

Groundwater was not encountered in exploratory trenches excavated on the hillside locations of the Overnight Lodges. However, the presence of near surface (standing) water and phreatophyte vegetation suggest that groundwater seepage may occur from native slopes in the project area. In addition, springs were reported (ENGEO, Incorporated, 1993) occurring along a fault lineation within the proposed site area. Seepage would likely be exacerbated on cut slopes constructed for the project.

Erosion may be accelerated and slope stability compromised where groundwater daylights (seeps) onto slopes. Conditions in the site area favoring seeps include relatively shallow bedrock (or other impermeable layer) with an overlying permeable soil, and inactive fault zones which can act to concentrate subsurface water.

5.3 Faults: Two subparallel fault traces have been mapped in the immediate site area (Wahler Associates, 1990, and Kaldveer Associates, 1989). The locations of these mapped faults are shown on Drawing No. 2 with respect to the proposed facilities. The northern trace is located near the axis of the ravine, north of the clubhouse. The southern fault is located through the area of the Overnight Lodges.

Two brecciated zones indicative of faulting were noted in Exploratory Trench A. Rocks in the brecciated zone were a light grey color and appeared to be sheared and chemically altered greenstone. The location of the brecciated zone is approximately coincident with the south fault trace, the springs noted by Wahler (1990), and our field reconnaissance. A dark brown lean clay soil horizon was developed on both the weathered greenstone, as well as rocks in the brecciated zone. The lean clay soil did not appear to be offset or disrupted above the brecciated zone which would suggest recent movement.

6.0 EVALUATION

This section presents information regarding potential geologic, geotechnical, and seismic hazards at the Clubhouse and Overnight Lodges area.

6.1 Geologic Hazards: Geologic and geotechnical hazards including expansive soils, erosion, landslides, seiches, tsunamis, and volcanic activity are evaluated in the following subsections.

6.1.1 Expansive Soils: The predominant soil type anticipated at the site area is lean clay. In general, the clayey soils (revealed during previous investigations near the site area) exhibited, moderate compressibility, and the potential for low to moderate swell. The primary geotechnical concerns at the site are the medium expansion potential of the lean clays. Over time the near surface clays will experience cyclic drying and wetting as the dry and wet seasons pass. The clay soils encountered at the site are anticipated to experience volumetric changes (shrink/swell) as the moisture content of the clay soils fluctuate. These shrink/swell cycles can impact foundations and lightly loaded slabs-on-grade even though the expansion potential is classified as medium. Expansive soils cause more damage to structures, particularly light buildings and pavements, than any other natural hazard, including earthquakes and floods (Jones and Holtz, 1973). Expansion potential may not manifest itself until months or years after construction. At most sites there exists a depth to which the moisture content of the subgrade remains essentially constant throughout the year; thus, the clays would not undergo a significant volume change below this depth. Therefore, the depth, referred to as the "critical depth", to which significant moisture fluctuation occurs influences the selection of suitable foundation and floor slab alternatives for this site. Climatic conditions, groundwater conditions, landscape irrigation, and the soil conditions effect the critical depth. Our review of moisture data and observations of near surface clay soils did not clearly demonstrate a critical zone depth. Based on experience, it is expected that the critical zone would be approximately, 36 inches BSG in the site region, and that seasonal moisture fluctuation would effect soils to a depth of 3 feet BSG. The above estimate of the critical depth should be reevaluated based on soil sample test data to be generated for the proposed geotechnical and geological investigation.

6.1.2 Erosion Hazard: Erosional features indicative of the unusually rapid erosion of the earth materials at the site were not noted during our field reconnaissance. Based on our geologic and geotechnical investigation of the site, the soil and rock conditions are not prone to excessive erosion. Accordingly, the potential erosion hazard at the site is low.

6.1.3 Landslides and Slope Stability: Landslides on the proposed development site were mapped by others (Kaldveer Associates, 1989, and Wahler Associates, 1990). The locations of these landslides are shown on Figure No. 2 of the report entitled "Geologic Input for the Lion's Gate Property" (DEIR Volume II) which is a compilation of site data generated prior to April 1993. Two previously mapped landslide features near the site were observed during our geologic field reconnaissance and appeared to comprise relatively shallow rotational block slides and slumps. The two mapped landslide masses are located north of the proposed Clubhouse and Overnight Lodges site. These slides are separated from the subject site by a ravine which would preclude impact to the site if the masses were remobilized. Accordingly, the documented slide masses do not present a hazard to the project site.

Other existing slide features, which could potentially affect the proposed project, were not noted during our geologic field reconnaissance. Based on our field observations native slopes in the vicinity of the project site appear to be relatively stable.

6.1.4 **Inactive Faults as Foundation Discontinuities**: Subsection 6.2.2 indicates that faults noted in the subject area are inactive and the potential is low for ground rupture due to earthquake faulting, or rupture due to seismic ground motion induced movement across an inactive fault. However, structures built across faults may be supported on soil or rock materials with highly variable foundation properties and excessive differential settlement can result. Variable foundation properties may result from dissimilar earth materials juxtaposed across the fault, or by structurally weak zones of sheared rock coincident with the faults. Potential differential settlement due to weak shear zones may be mitigated by soil foundation modification, using deep foundations, or modifying the location of a structure away from the shear zone. Mitigation measures are described in subsection 7.7.

6.1.5 **Serpentinite**: Twining's field investigation did not encounter serpentinite type rock materials in the project area. In addition, the "Aerial Geologic Map" prepared by Kaldveer Associates (1990) does not indicate serpentinite in the area of the proposed Clubhouse and Overnight Lodges. Accordingly, the potential for encountering naturally occurring asbestos materials during grading for the project is low.

6.1.6 **Seiches and Tsunamis**: A seiche is a wave generated by the periodic oscillation of a body of water whose period is a function of the resonant characteristics of the containing basin as controlled by its physical dimensions. These periods generally range from a few minutes to an hour or more. The site is not near any large bodies of water, so seiches are not considered a significant hazard at the site.

Tsunamis are waves generated in oceans from seismic activity. Due to the inland location of the site, there is no potential hazard from tsunamis.

6.1.7 **Volcanic Activity**: The closest known post Quaternary volcanic areas are near the Mammoth Mountain area in the Sierra Nevada Mountains, approximately 130 miles east of the site. Based on the distance of potential volcanic sources from the site, the prospects for lava flows or significant ash falls are low.

6.2 **Seismic Hazards**: The potential seismic hazards of ground shaking, ground rupture, liquefaction, and seismic settlement are evaluated in the following subsections.

6.2.1 **Ground Shaking**: For any given earthquake, the rock in the immediate vicinity will respond with a certain maximum acceleration and with a predominant period that depends on the nature of the rock and on the source mechanism. Away from the focus of the earthquake, the shock waves begin to attenuate. The way in which the earthquake wave is altered depends to a great degree on source characteristics and to a lesser degree on the travel path.

A detailed seismic analysis was conducted using two different methods, historic and probabilistic. Discussion of the analyses and the results are presented in the following subsections.

6.2.1.1 **Historic Seismic Activity**: The general area of the site has experienced recurring seismic activity. Based on historical earthquake catalogs published by the California Division of Mines and Geology, and supplemental data from Townley and Allen (1939) and the U.S. Geological Survey's earthquake database system, approximately 684 historical earthquakes with magnitude 4.0 or greater were recorded from 1800 through 1996 within a 100 mile radius of the site. A map showing the location of the project site with relation to the approximate historical earthquake epicenter locations is presented on Drawing No. 3. The source data presented include: latitude, longitude, date, time, depth, Magnitude, computed site acceleration, computed site Modified Mercalli intensity, and the approximate earthquake-to-site distance in miles and kilometers. This analysis was performed by a computer program titled EQSEARCH (1989).

An attenuation relationship, developed by Boore et al. (1993), was used to estimate the peak horizontal ground acceleration that may have occurred at the site from each of the historical earthquakes within the 100 mile search radius.

The nearest event (Mag.= 5.0, Acc.= 0.234g) found during the search occurred in 1938 approximately 1 mile southeast of the site. The largest magnitude earthquake identified in the search was the magnitude 8.25, 1906 San Francisco earthquake event occurring approximately 62 miles northwest of the site.

6.2.1.2 **Probabilistic Seismic Hazards Analysis**: The level of ground motion typically used for design of non-essential commercial developments is the ground motion with a 10% probability of being exceeded in 50 years, which is termed the "maximum probable earthquake". Determination of the Maximum Probable Earthquake requires probabilistic methods.

The computation of attenuated ground motion is based on the closest distance between the site and various measures of potential fault-plane ruptures along selected faults. The twenty (20) faults selected for this analysis are listed on Table No. 1. These selected faults comprise the local potentially active faults and regional faults with higher activity and magnitudes. The computations were conducted using FRISK (McGuire, 1978). FRISKSP version 3.00 programs (Blake, 1995) was used to set up the input data files and generate the output.

TABLE NO. 1
Summary of Fault Source-Model Parameters

FAULT NAME	Site to Fault Distance, Miles	SLIP RATE (millimeters per year)	Upper Bounds Magnitude
Sargent-Berrocal	3	1.0	7.0
San Andreas (Northern)	5	19	8.0
Hayward	8	9	7.0
Calaveras	8.5	7	7.5
San Andreas (Creeping)	10	34	7.0
Greenville	20	0.5	7.3
Monterey Bay Zone	24.5	2	6.0
Ord Terrace	26	0.16	5.5
Rinconada	27	1	7.0
Palo Colorado-San Gregorio	30	10	7.7
Chupines	31	2	6.0
Seaside	31	0.01	5.5
Navy-Turlarcitos	31	0.13	5.5
Ortigalita	31	0.04	7.0
Cypress Point	35	0.01	5.0
Coast Range-Sierran Block	38	3.0	7.0
Las Positas	38	0.2	6.3
Miller Creek-Palomares	41	1.2	6.3
Vernalis	45	0.4	6.5
Concord	59	4	6.7
Antioch	59	1	6.7
Green Valley	73	4	7.0

Fault parameters (such as fault length, magnitude, and rupture area) of faults capable of impacting the site were determined from published geologic papers (see bibliography), and the maximum magnitudes (100 year) were estimated using a characteristic fault model relationship (Youngs and Coppersmith, 1985). Due to the relative age of the faults and the absence of historic event data, subjective probabilities reflecting the relative slip rates reported were applied to account for the questionable activity of potentially active faults. The primary parameters used in the analysis are included in Table No. 1. The location of faults used in this analysis are provided on Drawing No. 4.

The ground motion attenuation relationship used in the analysis to estimate site response values was developed by Boore et al. (1993) for a Class A site (soil). The relationship for the larger component plus one standard deviation (as opposed to mean) was used. Boore et al. (1993) defines a class for each site based on the shear wave velocities of the upper 30 meters of material (about 200 feet). A Class A site has a shear wave velocity of 750 meters per second (m/s) or greater; a Class B site has a shear wave velocity of between 360 m/s and 750 m/s; a Class C site has a shear wave velocity of between 180 m/s and 360 m/s; and a Class D site has a shear wave velocity of less than 180 m/s. Our understanding of the shallow bedrock conditions in the site area suggest the subject site should be classified as a Class A site.

The horizontal site acceleration that has a 10 percent probability of being exceeded in 50 years (maximum credible event) was determined to be about 0.38g. The Probability of Exceedance vs. Acceleration for exposure periods of 25, 50, 75, and 100 years for the site are shown on Drawing No. 5. In addition, the Average Return Period versus Ground Acceleration is shown on Drawing No. 6.

6.2.2 **Ground Rupture**: Earthquakes are caused by the sudden displacement of earth along faults with a consequent release of stored strain energy. The fault slippage can often extend to the ground surface where it is manifested by sudden and abrupt relative ground displacement. Damage resulting from fault rupture occurs only where structures are located astride the fault traces that move.

The project site is located in a seismically active region with numerous active and potentially active faults. Two subparallel bedrock faults associated with melange terrane have been mapped near the proposed Clubhouse and Overnight Lodges (Wahler Associates, 1990, and Kaldveer Associates, 1989). The locations of these mapped faults are shown on Drawing No. 2 with respect to the proposed facilities. The northern trace is located near the axis of the ravine, north of the clubhouse. The southern fault is located through the area of the Overnight Lodges. According to Wahler (1990) the bedrock faults and sheared zones are apparently an extension of the Ben Trovato fault zone mapped northwest of the site. The Ben Travato Fault is designated as preQuaternary (Jennings, 1994), and is therefore considered inactive. During our geologic field investigation we noted evidence of several northwest-southeast trending faults and/or shear zones delineated based on linear distribution of springs, linear zones of contrasting vegetation, and topographic expressions. Terratech (1988) reported photolineaments in alluvium along inferred fault traces. However, trenching by Wahler (1973) across projections of the lineaments in bedrock areas did not identify evidence of geologically recent fault activity. Wahler (1973) judged both the Hayes Valley Fault and the fault on the north side of the valley (near the subject site) to be inactive.

Data presented in the cited reports of previous investigations do not indicate that the bedrock faults in the site area are active. The nearest mapped active or potentially active fault is the Sargent, located about 3 miles east of the site. The project site is not located in a Fault-Rupture Hazard Zone or former Alquist-Priolo Special Studies Zones. Accordingly, the potential for surface fault rupture at the site is low.

6.2.3 Liquefaction: Liquefaction in this instance describes a phenomenon in which a saturated, cohesionless soil loses strength during an earthquake as a result of induced shearing strains. Lateral and vertical movement of the soil mass, combined with loss of bearing usually results. Research has shown that liquefaction potential of soil deposits induced by earthquake activity depends on soil types, void ratio, groundwater conditions, duration of shaking, and confining pressure over the potentially liquefiable soil mass. Fine, well sorted, loose sand, shallow groundwater conditions, higher intensity earthquakes, and particularly long duration of ground shaking are the requisite conditions for liquefaction.

Studies of liquefaction potential during earthquakes address the liquefaction "susceptibility" and "opportunity" of a given site. Liquefaction susceptibility is a function of the mechanical properties of the underlying soils, particularly grain size distribution and relative density determined from standard penetration blow counts. Liquefaction opportunity expresses the probability of exceeding a critical level of shaking and is described in terms of a function which accounts for peak ground acceleration, or acceleration and duration. Accelerations of at least 0.10g and ground shaking durations of at least 30 seconds are generally required to initiate liquefaction.

The potential for the occurrence of an earthquake with the intensity and duration characteristics capable of promoting liquefaction "opportunity" is considered likely for the project life of the proposed Clubhouse and Overnight Lodges. Considering that granular soils were not identified, and that liquefaction will not occur in areas of very shallow bedrock, the "susceptibility" for liquefaction is considered very low.

6.2.4 Seismic Settlement: Seismic shaking may induce settlement of loose, unconsolidated sediments. This can occur in unsaturated and saturated granular soils. Considering that loose or granular soils were not identified at the site during or field exploration, in conjunction with the shallow depth to bedrock, the potential for seismic induced settlement is considered very low.

7.0 CONCLUSIONS

Based on the data collected during our investigation and our understanding of the anticipated construction, we present the following general conclusions and mitigation measures. Considering the conclusions and mitigation measures, the proposed project is feasible with respect to geotechnical, geologic, and seismic hazards.

7.1 The site appears geologically and geotechnically suitable for the proposed Clubhouse and Overnight Lodges facility considering the conclusions and mitigation measures presented in this report. The geotechnical and geologic issues requiring mitigation are discussed below.

7.2 Soil and rock conditions at the site vary from alluvial soils on the eastern portion of the site to soils developed on colluvium, and residual soils (lean clays) developed on the shallow bedrock on the western (hillside) portions of the site. Soils below the Clubhouse and Pro Shop comprised gravelly and sandy lean clays. Weathered greenstone bedrock was encountered between depths of 7 to 19 feet BSG at the proposed Clubhouse and pro shop sites. Rock was encountered at depths of 2 to 4 feet BSG on the hillside portion of the site.

7.3 Testing of lean clay soils collected from sites near the proposed Clubhouse and Overnight Lodges have been reported to have a low to moderate shrink-swell potential. Lean clay soils at the site may exhibit low to moderate expansion characteristics. To mitigate the potential for structural damage resulting from expansive soils, non-expansive materials can be placed below slabs, and foundations can be extended below the depth where moisture changes in soil cause volumetric changes. This depth is preliminarily estimated to be approximately 36 inches below site grade. To minimize the potential for fluctuations in soil moisture near buildings, grading should be conducted to direct drainage away from the buildings and prevent ponding near the building. Landscaping setbacks can also be instituted to minimize the potential for ponding of water near the foundation.

7.4 As evidenced by springs and seeps, shallow groundwater may be encountered during grading of the hillside slopes.

7.5 Potential erosion and slope stability hazards which may be caused by shallow groundwater at the site can be mitigated by the following methods:

- Road subgrades: Trenched cut-off walls and subdrains
- Native slopes: Upslope trench cut-off wall or horizontal wick drains
- Cut slopes: Retaining wall with filter drain and weep holes
- Fill slopes: Cut-off drains placed in keyways and other locations where subflow impinges on fill slopes.

Subsequent to rough grading, areas with evidence for subsurface groundwater flow should be identified by Twining's civil engineer or engineering geologist. Soil textures exhibiting a selective removal of fine particles from currently dry soils may indicate subsurface groundwater flow during wetter periods. Mitigative measures can be selected by Twining's civil engineer or engineering geologist for specific areas, when adverse shallow groundwater conditions are identified.

7.6 The soils are estimated to have a low erosion hazard. Based on our understanding of the anticipated construction, soil erosion is not expected to significantly affect the project.

7.7 Trenching exploration of the subject site did not reveal evidence of active faults (see section 6.2.2), however, brecciated and sheared zones were noted indicating older (inactive) faults within the greenstone bedrock. These shear zones are typical for Franciscan Complex (melange terrane) materials. Differential settlement across and within an inactive fault zone may occur, and damage may occur to buildings constructed across those zones. Potential differential settlement due to weak shear zones may be mitigated by overexcavation and recompaction of foundation soils over the fault discontinuity, or deep foundations such as drilled shafts or driven piles. In addition, mitigation may include modifying the location of a structure away from the shear zone. Specific foundation recommendations can be provided in the design level geotechnical engineering report.

7.8 Native slopes in the vicinity of the project site appear to be relatively stable and suitable for the proposed construction based on maximum cut and fill slopes of 2 horizontal (H) to 1 vertical (V). Existing landslide features were not noted which could affect the project. Further evaluation of slope stability should incorporate the proposed site grading plan. In addition, Twining's engineering geologist should be contacted to observe soil, rock and associated groundwater conditions revealed after mass grading. If unstable native slopes are encountered, they can be mitigated by removal of the unstable material, buttressing the material, or providing subflow cut-off drains and limiting infiltration of surface water. Cut and fill slopes of not greater than 2 horizontal (H) to 1 vertical (V) can be constructed in accordance with the Uniform Building Code to provide stable foundations for construction. Steeper cut or fill slopes, if required, may be feasible contingent on evaluation on a case-by-case basis.

7.9 The potential to encounter serpentine and asbestos at the project site is low. However, if asbestos containing materials are encountered during grading, the locations should be documented and the asbestos content in the serpentine should be assessed by Twining's engineering geologist. Serpentine rock is typically a green or yellow, highly sheared and altered rock, with a fibrous appearance. Where final graded areas expose asbestos-containing serpentine, or where asbestos-containing fill material is used, the potential for human exposure to asbestos can be mitigated by placing a layer of non-asbestos containing material over the asbestos containing material.

7.10 There is little or no potential for hazards due to volcanic activity, seiches, and tsunamis at the site.

7.11 A maximum probable peak horizontal ground acceleration of 0.38g is estimated for the proposed development site. Building design and construction in accordance with the Uniform Building Code can mitigate the potential effects of the maximum probable peak horizontal ground acceleration estimated for the site.

7.12 Mitigation for potential surface rupture of an active fault typically requires establishing building setbacks. However, trenching exploration of the subject site did not reveal evidence of active faults. The site is not located in a Proposed Seismic Hazard Zone or an Alquist-Priolo Special Studies Zone. Therefore, the potential for ground rupture associated with a known active fault is very low, and building setbacks would not be warranted.

7.11 Based on the soil and rock conditions at the site, the potential for liquefaction and seismic settlement are considered low. Accordingly, it is not anticipated that mitigation of potential liquefaction and seismic settlement would be required. In the event soil conditions susceptible to liquefaction or seismic settlement are revealed during design level geotechnical studies, the potential for liquefaction and seismically induced settlement can be mitigated. Mitigation can be achieved through site preparation, including densifying site soils by either overexcavation and compaction, ground modification techniques, using deep foundation (piles) founded below liquefiable zones, or using reinforced structures.

8.0 NOTIFICATION AND LIMITATIONS

The conclusions presented in this report are based on the information provided regarding the proposed construction, the results of the research of background information, and our evaluation of site conditions revealed during our reconnaissance and subsurface geotechnical engineering investigation. This report does not present design level geologic or geotechnical data.

The focus of our investigation was the proposed Clubhouse and Overnight Lodges area and pertains only to geologic and geotechnical concerns of this site. Potential geotechnical and geologic hazards to structures on or outside of the subject site were not evaluated in this report.

If variations or undesirable conditions are encountered during construction, Twining should be notified promptly so that these conditions can be reviewed and our recommendations reconsidered where necessary. It should he noted that unexpected conditions frequently require additional expenditures for proper construction of the project.

If the proposed construction is relocated or redesigned, or if there is a substantial lapse of time between the submission of our report and the start of work (over 12 months) at the site, or if conditions have changed due to natural cause or construction operations at or adjacent to the site, the conclusions and preliminary recommendations contained in this report should be considered invalid unless the changes are reviewed and our conclusions and recommendations modified or approved in writing.

Changed site conditions, or relocation of proposed structures, may require additional investigations to determine if our conclusions are applicable considering the changed conditions or time lapse.

The conclusions contained in this report are valid only for the project discussed in the "Anticipated Construction" section of this report. The entity or entities that use or cause to use this report or any portion thereof for a structure or site other than those indicated in the "Background" section of this report shall hold Twining, its officers and employees harmless from any and all claims and provide Twining's defense in the event of a claim.

This report is issued with the understanding that it is the responsibility of the client to transmit the information and preliminary recommendations of this report to developers, owners, buyers, architects, engineers, designers, contractors, subcontractors, and other parties having interest in the project so that the steps necessary to carry out these preliminary recommendations in the design, construction and maintenance of the project are taken by the appropriate party.

Reliance on this report by a third party (i.e., that is not a party to our written agreement) is at the party's sole risk. If the project and/or site is purchased by another party, the purchaser must obtain written authorization and sign an agreement with Twining in order to rely upon the information provided in this report for design or construction of the project.

Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally-accepted engineering principles and practices in Santa Clara County, California at the time of the investigation. This warranty is in lieu of all other warranties either expressed or implied.

9.0 CLOSING

We appreciate the opportunity to be of service to Lion's Gate Limited Partnership, LLC. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience.

Sincerely,
THE TWINING LABORATORIES, INC.

REGISTERED GEOLOGIST
KENNETH JAMES CLARK
No. EG 1834
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA
Exp 5-31-99

Kenneth J. Clark, CEG
Project Geologist

cc: Mr. Ron Davis, Lion's Gate Limited Partnership, LLC
cc: Mr. Bert Verrips with Nolte Engineering

10.0 REFERENCES

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Blake, T.F., 1993, EQSEARCH: a computer program for the estimation of peak acceleration from digitized California historical earthquake catalogs.

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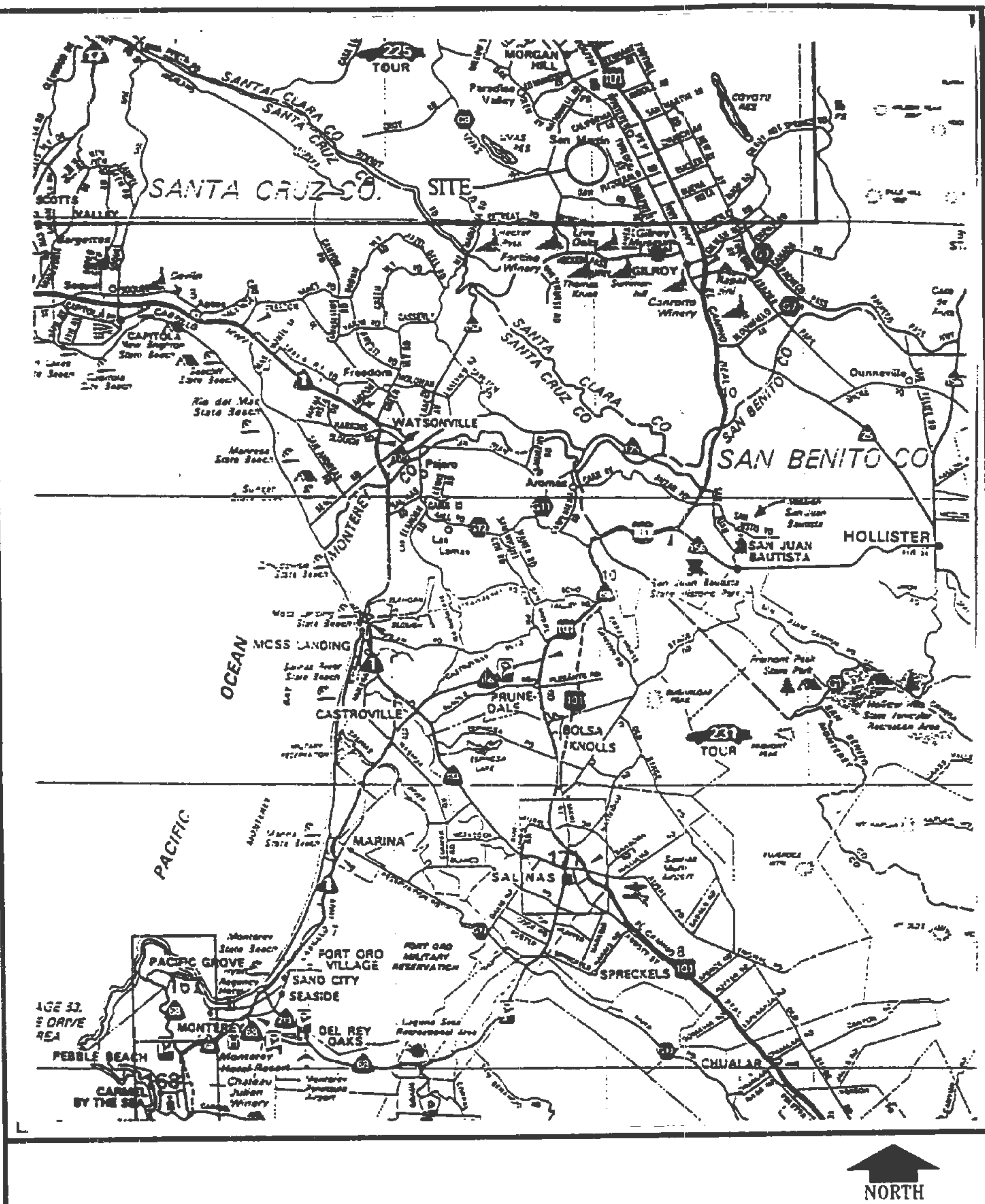

SITE LOCATION MAP NEW CLUBHOUSE AND OVERNIGHT LODGES CORDEVALLE GOLF CLUB AND HOTEL SAN MARTIN, CALIFORNIA	FILE NO.: N/A	DATE: 5-29-98	THE TWINING LABORATORIES, INC. EST. 1896 FRESNO/MODESTO/VISALIA/BAKERSFIELD/SALINAS
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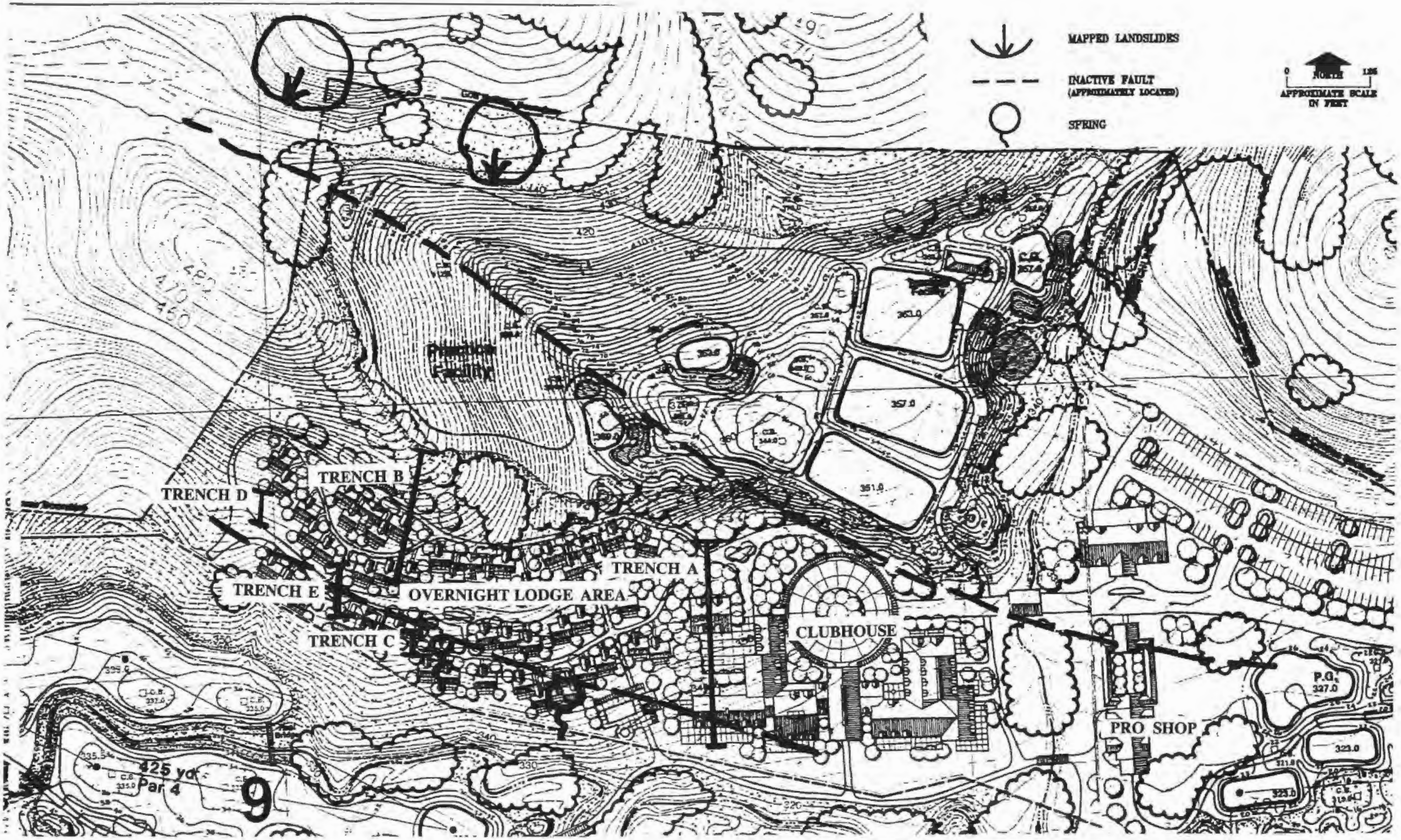

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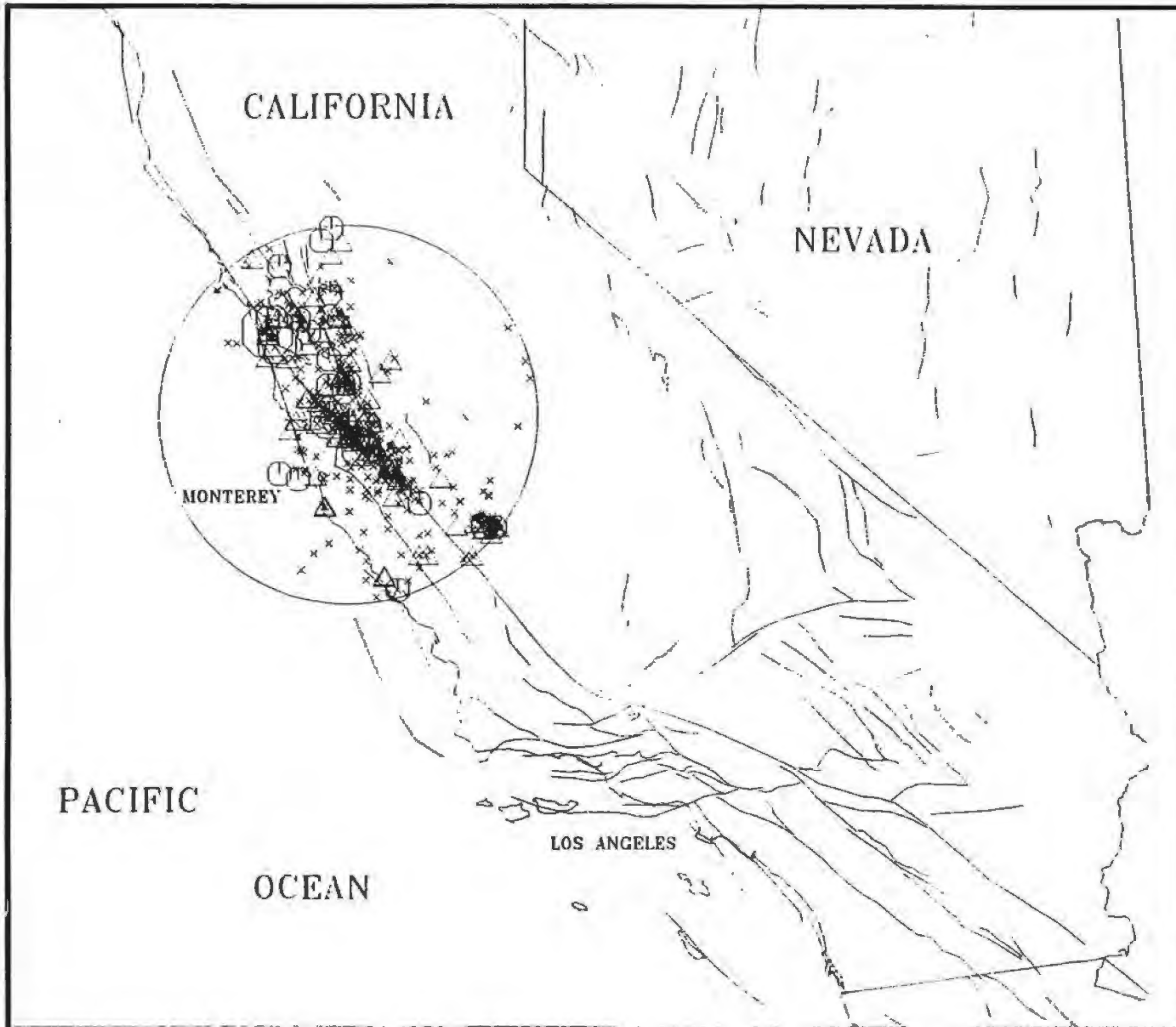
THE TWINING LABORATORIES, INC.



N

0 50 100

SCALE
(MILES)

EXPLANATION

M = 8.0 +

M = 7.0–7.9

M= 6.0–6.9

M = 5.0–5.9

M = 4.0–4.9

SITE LOCATION (+):

LATITUDE – 37.0678 N
LONGITUDE – 121.7000 W

HISTORICAL EARTHQUAKES 1800 TO 1996

JOB NO.: D34301.03

HISTORICAL EARTHQUAKE EPICENTER LOCATION NEW CLUBHOUSE AND OVERNIGHT LODGES CORDEVALLE GOLF CLUB AND HOTEL SAN MARTIN, CALIFORNIA	FILE NO.: 34301-04	DATE: 5-29-98	THE TWINING LABORATORIES, INC. EST. 1898 FRESNO/MODESTO/VISALIA/BAKERSFIELD/SALINAS
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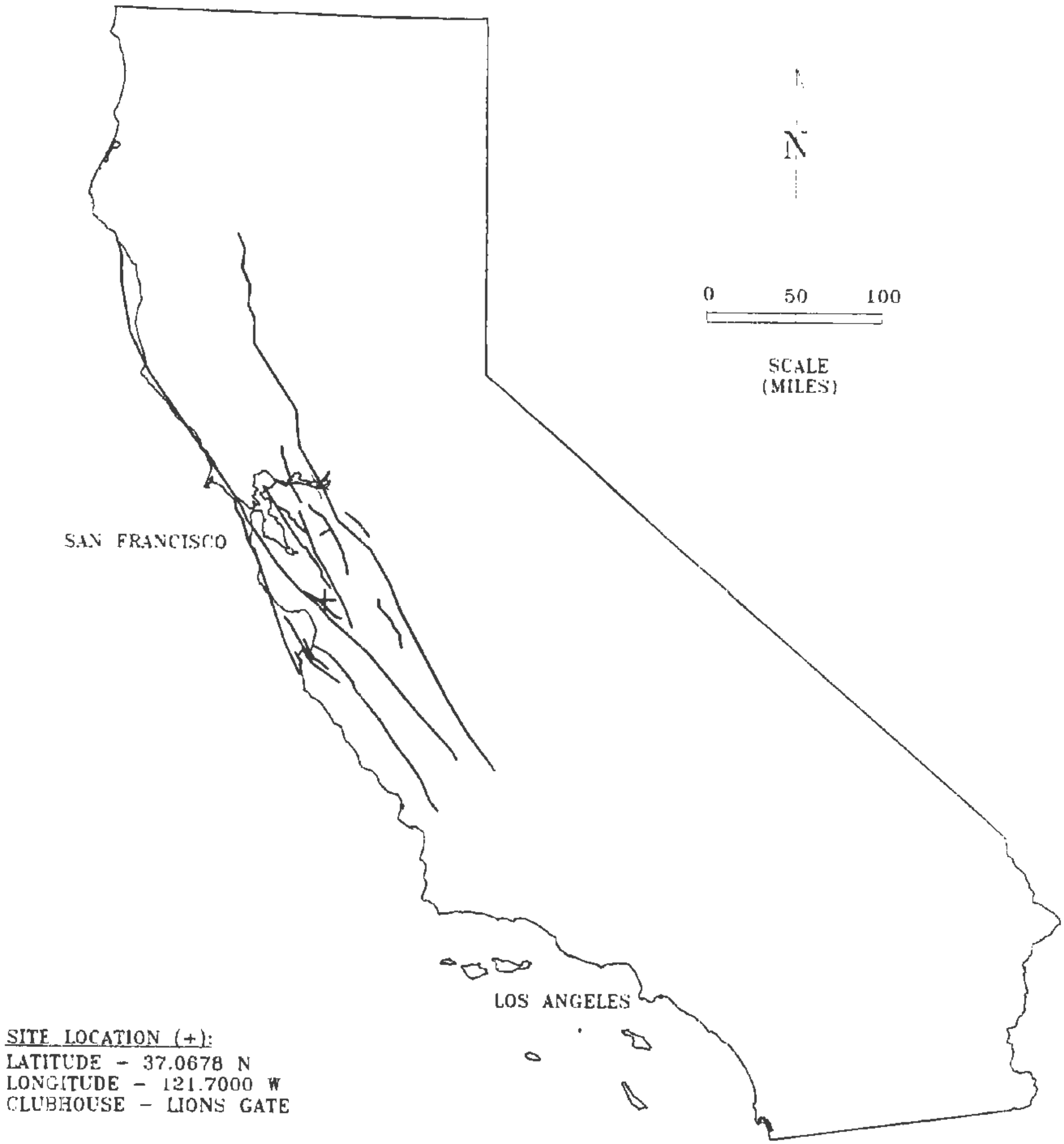

FRISKSP FAULT MAP

JOB No: D34301.03

FAULT LOCATION MAP NEW CLUBHOUSE AND OVERNIGHT LODGES CORDEVALLE GOLF CULB AND HOTEL SAN MARTIN, CALIFORNIA	FILE NO.: 34301-03	DATE: 5-29-98	THE TWINING LABORATORIES, INC. EST. 1898 FRESNO/MODESTO/VISALIA/BAKERSFIELD/SALINAS
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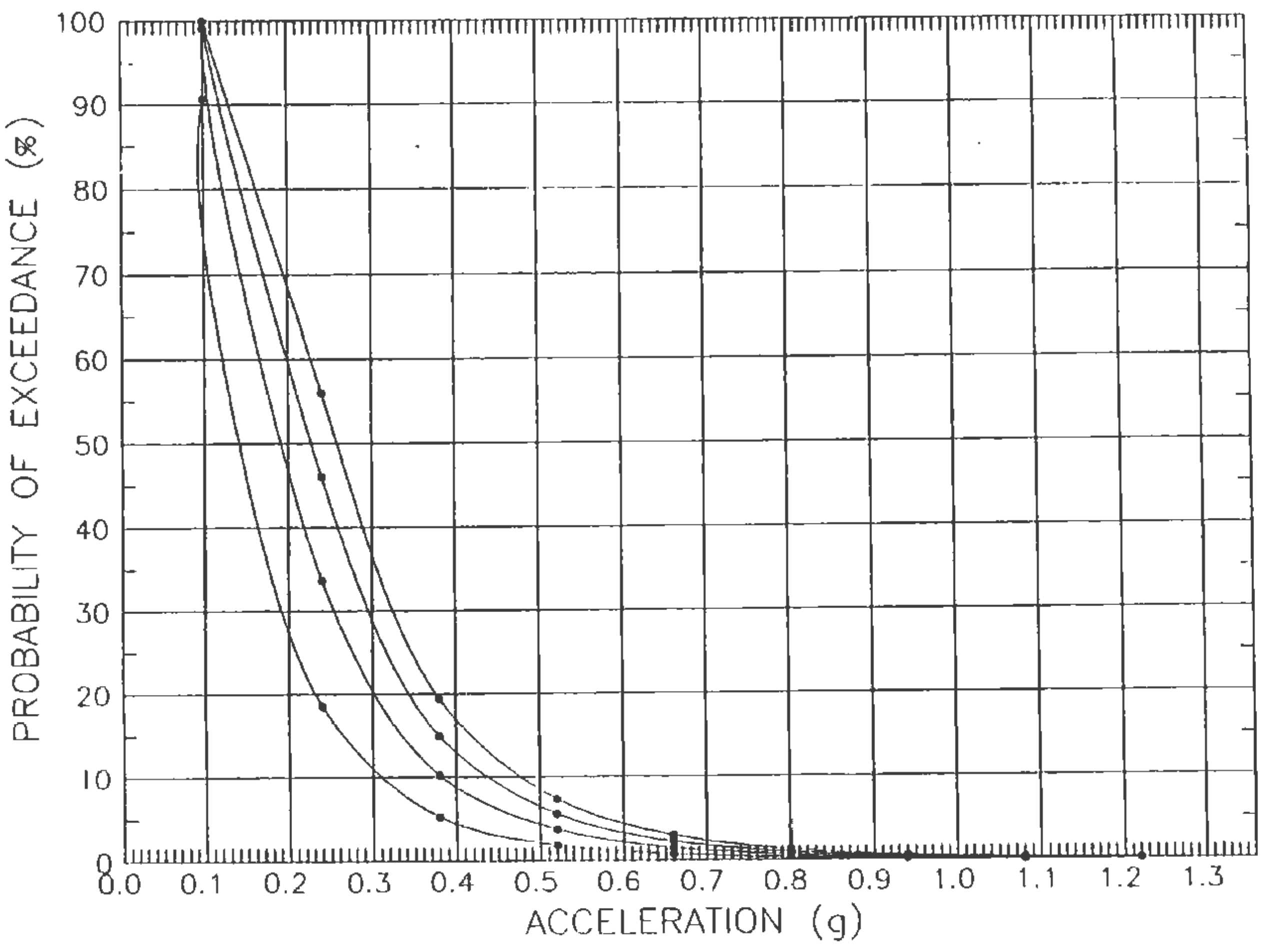


BOORE ET AL. (1993) LRG. S - A

EXPOSURE PERIODS:

25 years	75 years
50 years	100 years

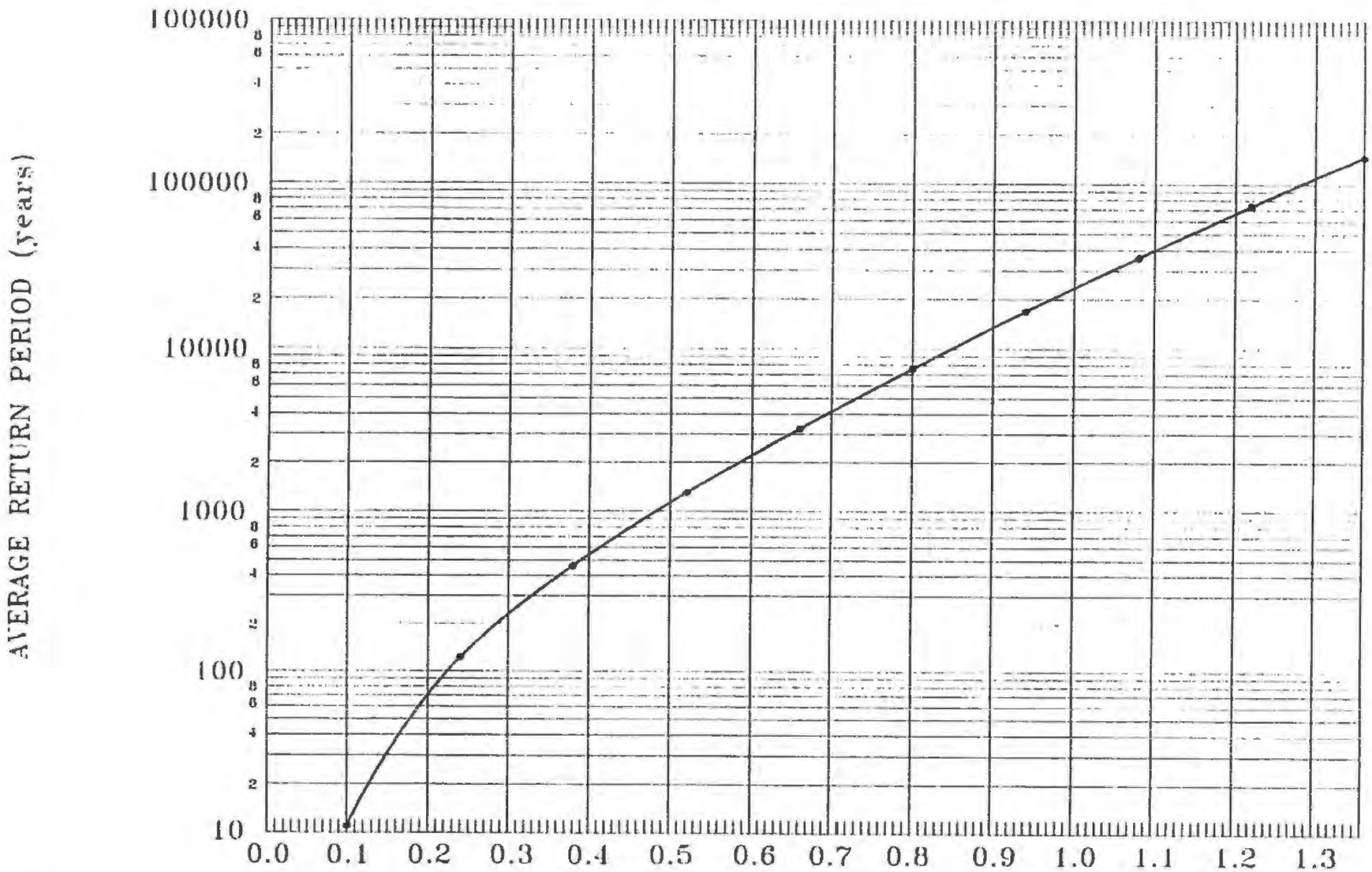
JOB No.: D34301.03

PROBABILITY OF EXCEEDANCE vs. ACCELERATION
NEW CLUBHOUSE AND OVERNIGHT LODGES
CORDEVALLE GOLF CLUB AND HOTEL
SAN MARTIN, CALIFORNIA

FILE NO.: 34301-06	DATE: 5-29-98
DRAWN BY: PNG	APPROVED BY: KC
PROJECT NO. D34301.03	DRAWING NO. 5

THE TWINING LABORATORIES, INC.
EST 1898
FRESNO / MODESTO / VISALIA / BAKERSFIELD / SALINAS

AVERAGE RETURN PERIOD vs. ACCELERATION



CLUBHOUSE - LIONS GATE

ACCELERATION (g)

BOORE ET AL. (1993) LRG. S - A

JOB No.: D34301.03

AVERAGE RETURN PERIOD vs. ACCELERATION
NEW CLUBHOUSE AND OVERNIGHT LODGES
CORDEVALLE, GOLF CLUB AND HOTEL
SAN MARTIN, CALIFORNIA

FILE NO.: 34301-05	DATE: 5-29-98
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PROJECT NO. D34301.03	DRAWING NO. 6

THE TWINING LABORATORIES, INC.
EST. 1898
FRESNO, MODESTO, VISALIA, BAKERSFIELD, SALINAS

APPENDIX D

Master Drainage Plan

Prepared by

Pacific Advanced Civil Engineering

January 1998 (with Addendum dated April 1998)

LION'S GATE RESERVE

MASTER DRAINAGE PLAN

Prepared by

PACE

PACIFIC ADVANCED CIVIL ENGINEERING

17902 Georgetown Lane

Huntington Beach, CA 92648

714/843-5734

REGISTERED PROFESSIONAL ENGINEER
SASHA SAVICH
C056094
Exp. 12-31-00
CIVIL
STATE OF CALIFORNIA

Date:

January 1998

Table of Contents

INTRODUCTION

Lion's Gate Reserve, formerly know as Hayes Valley Ranch, is located at the base of Lion's Peak, 11 miles south of San Jose, adjacent to the City of Morgan Hill and approximately 2 miles west of Highway 101. Of the 1,676 acres, only approximately 420 are being developed and are located at the valley floor. The development plan for Lion's Gate Reserve includes a golf course, clubhouse, overnight lodge and 41 executive homes. The West Branch of Llagas Creek, an ephemeral stream, runs east-west through the project.

Pacific Advanced Civil Engineering, Inc. (PACE) has been retained by Hix-Rubenstein Companies to complete a Master Drainage Plan for Lion's Gate Reserve. The scope of this drainage report is to define the drainage area tributary to the project, estimate the flows and design drainage structures necessary to safely convey the flows through the project. Analysis of the golf course has been completed previously and is included in *the Lion's Gate Reserve Golf Course Drainage Report, PACE, May 1997.*

HYDROLOGY

Given the size of the watershed tributary to the project (2.37 square miles), the Corps of Engineers (COE) HEC-1 computer program was used. In general, HEC-1 is better suited for analysis of watersheds over 200 acres than other methods such as the Rational Method. The hydrology portion of this report discusses the drainage sub basin delineation and description, precipitation, soil parameters and routing used in the HEC-1 hydrologic model as well as the resulting flows that were calculated. Two separate models were created: Existing Condition and Developed Condition. The Existing Condition models the watershed under the present undeveloped conditions. The Developed Condition models the watershed assuming full residential and golf course improvements are in place.

Drainage sub-basin delineation

The drainage sub basin boundaries were developed by utilizing a 1"=400' topographic map of the project site as well as a 1"=2000' USGS map to determine any offsite flows that drain through the project site. Exhibit 1, USGS Map, located in the Appendix, shows the offsite drainage sub-basins. Exhibits 2 and 3 show the drainage sub basins for the entire watershed for the existing and developed conditions. The Tables 1 and 2 below list all the drainage basins along with area, time of concentration and Clark Storage coefficient "R" calculations (necessary for the Clark Unit Graph modeling of the drainage sub basins in the HEC-1 model). The equations used in the calculation of time of concentration and Clark storage coefficient were obtained from the Santa Clara County Water District and are shown below:

$$Tc = 0.01377\, L^{0.47}\, N^{0.47}\, S^{-0.235}$$

$$R/(R + Tc) = X$$

Where Tc = time of concentration (in hours)
L = length of drainage sub basin (in feet)
N = overall watershed roughness (resistance to overland flow)
S = drainage sub basin slope (in feet/foot)
R = Clark Storage coefficient (in hours)
X = 0.6 for rural areas

Table 1
Existing Condition Drainage Sub-Basin Description

Sub-basin	Area (mi^2)	Basin Length (ft)	Basin Slope (ft/ft)	Overland Flow Roughness N	Tc (hours)	R (hours)
1	0.1078	2100	0.1500	0.4000	0.5093	0.7639
2	0.0905	2175	0.0667	0.4000	0.6264	0.9396
3	0.0498	1800	0.1139	0.4000	0.5054	0.7580
4	0.2879	3605	0.1148	0.4000	0.6991	1.0487
5	0.0691	3085	0.1378	0.4000	0.6225	0.9337
6	0.1210	3853	0.1376	0.4000	0.6913	1.0369
7	0.1312	3500	0.1871	0.4000	0.6147	0.9221
8	0.0924	2477	0.2806	0.4000	0.4751	0.7126
9	0.0399	1170	0.1154	0.4000	0.4115	0.6172
10	0.1404	3326	0.2315	0.4000	0.5709	0.8563
11	0.0898	3640	0.0810	0.4000	0.7623	1.1434
12	0.1382	2655	0.2203	0.4000	0.5195	0.7793
13	0.0868	2750	0.2691	0.4000	0.5039	0.7559
14	0.0807	2825	0.2088	0.4000	0.5417	0.8125
15	0.0787	2200	0.0318	0.4000	0.7495	1.1242
16	0.0776	2940	0.1810	0.4000	0.5708	0.8562
17	0.1281	4050	0.0770	0.4000	0.8111	1.2167
18	0.1542	2530	0.1420	0.4000	0.5631	0.8446
19	0.0423	3000	0.1090	0.4000	0.6429	0.9737
20	0.3678	3870	0.1320	0.4000	0.6995	1.0493

Table 2
Developed Condition Drainage Sub-Basin Description

Sub-basin	Area (mi^2)	Basin Length (ft)	Basin Slope (ft/ft)	Overland Flow Roughness N	Tc (hours)	R (hours)
1	0.1078	2100	0.1500	0.3500	0.4783	0.7175
2	0.0905	2175	0.0667	0.4000	0.6264	0.9396
3	0.0498	1800	0.1139	0.3500	0.4746	0.7119
4	0.2879	3605	0.1148	0.3750	0.6782	1.0174
5	0.0691	3085	0.1378	0.3500	0.5846	0.8769
6	0.1210	3853	0.1376	0.3750	0.6706	1.0059
7	0.1312	3500	0.1871	0.3750	0.5964	0.8945
8	0.0924	2477	0.2806	0.3750	0.4609	0.6913
9	0.0399	1170	0.1154	0.3500	0.3864	0.5797
10	0.1404	3326	0.2315	0.4000	0.5709	0.8563
11	0.0898	3640	0.0810	0.3250	0.6914	1.0371
12	0.1382	2655	0.2203	0.4000	0.5195	0.7793
13	0.0868	2750	0.2691	0.3750	0.4889	0.7333
14	0.0807	2825	0.2088	0.3750	0.5255	0.7882
15	0.0787	2200	0.0318	0.3250	0.6798	1.0197
16	0.0776	2940	0.1810	0.3750	0.5537	0.8306
17	0.1281	4050	0.0770	0.4000	0.8111	1.2167
18N	0.0779	2530	0.1420	0.4000	0.5631	0.8446
18S	0.0763	2530	0.1420	0.4000	0.5631	0.8446
19	0.0423	3000	0.1090	0.2500	0.5205	0.7807
20	0.3678	3870	0.1320	0.2500	0.5609	0.8413

Precipitation

Per the *Santa Clara County Drainage Manual*, for watersheds between 200 and 2560 acres, the minimum return period for a design storm is 10 years. *Technical Paper No. 40, Rainfall Atlas of the United States, US Weather Bureau, US Department of Commerce* lists the following precipitation depths for the area.

Table 3
Rainfall Depths for Storms of Various Return Periods

Storm	Total Rainfall (in)
2 year 24-hour storm	3
5 year 24-hour storm	4
10 year 24-hour storm	5
25 year 24-hour storm	6
50 year 24-hour storm	7
100 year 24-hour storm	8

The rainfall distribution used in the HEC-1 modeling is the based on the C.O.E. standard storm.

Soils

Soil Conservation Service Soils Map for Santa Clara County indicates that the soils in the area consist of predominantly: Gilroy, Garretson, Keefers and Los Robles. *Technical Release 55, Urban Hydrology for Small Watersheds by Soil Conservation Service, US Department of Agriculture* lists these soils as belonging to hydrologic soil groups C and D. Group D soils have high runoff potential. They have very low infiltration rates when thoroughly wetted and consist chiefly of clay soils with high swelling potential, soils with permanently high water table, soils with claypan or clay layer near the surface, and shallow soils over nearly impervious material. These soils have a very low rate of water transmission (0.0-0.5 in/hr). Group C soils have a slightly lower runoff and higher infiltration rates than group D soils. Each drainage sub basin was analyzed for the soil group. The highest runoff soil group present in the sub-basin was conservatively selected as representative for the entire sub basin.

HEC-1 modeling for the watershed requires the use of SCS Curve numbers for description of the individual drainage basins within the watershed. Per *Table 5-2(a) Runoff Curve numbers for Urban Areas, Engineering Hydrology by Victor Miguel Ponce*, golf courses on group C soils with grass cover greater than 75% are considered to have an SCS curve number of 74. *Table 5-2(d) Runoff Curve Numbers for Arid and Semi Arid Rangelands* for herbaceous, mixture of grass, weeds and low growing brush with more than 70% ground cover on group C soils also have a SCS Curve number of 74. Group D soils have an SCS curve number of 85. Table 3 below lists the SCS curve numbers that were assigned to the various drainage sub basins. All areas were assumed to be 5% impervious for the existing condition. Drainage sub basins which will contain residential development and club house are assumed to be 15% impervious. For all storms events except the 100 year 24 hour storm, antecedent moisture condition AMC II (average soil moisture level) was used. For the 100 year 24 hour storm event, AMC III (wet condition) was used. The AMC III increased the SCS curve numbers from 74 and 85 to 88 and 94 respectively. Higher SCS curve numbers generate higher runoff.

Table 4
Existing Condition Soil Description

Drainage Sub Basin	Hydrologic Soils Group	Existing SCS Curve Number	AMC III Existing SCS CN	Existing Percent Impervious
1	D	85	94	5
2	D	85	94	5
3	C	74	88	5
4	D	85	94	5
5	D	85	94	5
6	D	85	94	5
7	C	74	88	5
8	C	74	88	5
9	C	74	88	5
10	C	74	88	5
11	C	74	88	5
12	C	74	88	5
13	C	74	88	5
14	C	74	88	5
15	C	74	88	5
16	C	74	88	5
17	C	74	88	5
18	D	85	94	5
19	D	85	94	5
20	C	74	88	5

Table 5
Developed Condition Soil Description

Drainage Sub Basin	Hydrologic Soils Group	Developed SCS Curve Number	AMC III Existing SCS CN	Existing Percent Impervious
1	D	85	94	15
2	D	85	94	5
3	C	74	88	5
4	D	85	94	5
5	D	85	94	5
6	D	85	94	5
7	C	74	88	5
8	C	74	88	5
9	C	74	88	5
10	C	74	88	5
11	C	74	88	5
12	C	74	88	5
13	C	74	88	5
14	C	74	88	5
15	C	74	88	15
16	C	74	88	15
17	C	74	88	5
18N	D	85	94	15
18S	D	85	94	15
19	D	85	94	15
20	C	74	88	25

Channel Routing

Runoff flows from the drainage basins were routed using the Storage Routing procedure in the HEC-1 models. Table below shows the routing parameters used in the HEC-1 model for various reaches.

Table 6
Channel Routing Parameters

Reach	Length (ft)	Slope (ft/ft)	Manning's n	Bottom width (ft)	Side Slope (H:V)
RO5	600	0.0333	0.030	20	5:1
RO7	1050	0.0140	0.030	10	5:1
RO11	1440	0.0086	0.030	10	5:1
RO10-1	2620	0.0267	0.035	20	5:1
RO10-2	3600	0.0333	0.035	15	5:1
RO9	800	0.0125	0.030	25	5:1
RO3	1000	0.0400	0.035	20	5:1
RO15	1450	0.0138	0.030	15	5:1
RO13	2500	0.0280	0.035	20	5:1
RO14	2150	0.0279	0.035	20	5:1
ROCP16	1770	0.0056	0.030	20	5:1

Flows

HEC-1 models for both the existing condition and developed condition were completed for storm events ranging from the 2 year 24 hour to the 100 year 24. Differences between the existing and developed condition models include:

1. Percent impervious
2. Time of concentration Tc and Roughness R
3. SCS curve numbers
4. Inclusion of detention areas

Runoff from each of the drainage sub basins is summarized in the table below for both existing and developed conditions for the 100 year 24 hour design storm.

Table 7
Runoff From Individual Drainage Sub-Basins for the 100 year 24 hour storm event for the Existing and Developed Conditions

Drainage Sub Basin	Existing Condition Peak Runoff (cfs)	Developed Condition Peak Runoff (cfs)
SUB1	54	54
SUB2	42	42
SUB3	24	24
SUB4	128	129
SUB5	32	33
SUB6	54	54
SUB7	59	60
SUB8	45	46
SUB9	20	20
SUB10	65	65
SUB11	38	39
SUB12	66	66
SUB13	42	42
SUB14	38	38
SUB15	34	34
SUB16	36	37
SUB17	54	54
SUB18	74	n/a
SUB18N	n/a	37
SUB18S	n/a	37
SUB19	19	21
SUB20	159	173

Peak flows in the various reaches are summarized in the table below.

Table 8
Flows in various Channel Reaches for the 100 year 24 hour storm

Reach	Existing Condition Peak Flow (cfs)	Developed Condition Peak Flow (cfs)
RO5	81	82
RO7	268	271
RO11	311	314
RO10-1	64	64
RO10-2	64	64
RO9	479	469
RO3	41	41
RO15	563	551
RO13	40	40
RO14	37	37
ROCP16	678	649
RO19	764	730

HYDRAULICS

Rainfall runoff from the project site exits the property at three locations: 1) South-east corner of the site across Turlock Avenue, 2)Llagas Creek and 3) across Coolidge Avenue north of Llagas Creek. The hydraulics section of this report analyzes the flows in the West Branch of Llagas Creek and the flows leaving Lion's Gate under existing and developed conditions.

Existing Condition

Under the existing condition, rainfall runoff confluences in two major locations: 1)West branch of Llagas Creek and 2) south-east corner of the project site. Exhibit 4 located in the Appendix shows the 100 year water surface at the project site under existing conditions.

West Branch of Llagas Creek

Flows in the West Branch of Llagas Creek traverse the middle of the project in a west to east direction. As flows reach the eastern project boundary at Coolidge Avenue, they pass under the road through a 3.5' x 6' reinforced concrete box culvert. Since the culvert is relatively small compared to the incoming 100 year flow, the creek backs up submerging the culvert and overtopping the northern bank of the channel and flooding the orchard located just north of the channel. As the flow ponds up in the orchard, it crosses Coolidge Avenue at a dip section located approximately 1.200' north of the creek. The dip section in the road has a 24" reinforced concrete pipe culvert to convey the smaller nuisance flows under the road.

To correctly assess the extent of the ponding and flooding under a 100 year storm event several different calculation and modeling procedures were completed. A HEC-RAS model was completed for the creek. Output from model (Lion8.prj) including cross-sections, profile and summary table are included in the Appendix. Since the culvert at the end of the channel has insufficient capacity to convey all of the 100 year flow (783 cfs), and from recent storm events it is known that that the creek does not overtop the road at the box culvert it, was necessary to determine the maximum flow through the culvert. The HEC-RAS model was used to calculate a rating table of water surface elevation versus flow for the culvert. The rating table is included in the Appendix. A flow of 110 cfs was assumed to pass through the culvert with the remaining 673 cfs overtopping the bank and entering the orchard. The flows that enter the orchard pond up and overtop Coolidge Avenue at the dip section some 1,200 feet to the north. It was then necessary to determine the extent of the flooding and ponding in this area. To determine the flow depth and width across Coolidge Avenue a rating table was developed. The road centerline profile was input into Flowmaster (Manning's Equation) and a critical depth and top width were calculated for various flow rates. The correct flow rate was then looked up in the HEC-1 model which includes the diversion from the creek, flows tributary to that area as well as storage effects from ponding. A flow of 797 cfs crosses Coolidge Avenue at the dip section. This flow was then looked up in the rating table which shows that the flow would be over 1,050' wide and over 6" deep.

Southeast corner of Project Site

The south east corner of the project site is a low point and the natural drainage path for the rainfall runoff of drainage sub-basin 20. As the flows pond up in the corner they enter a 16" corrugated metal pipe which coveys the flows from the project site. The flows then enter 2 12" pipes that convey the flows under Turlock Avenue. Since the 100 year flow expected in this area is 161 cfs, which is more than the capacity of the pipes, the road is overtopped. To determine the flow depth and width across Turlock Road, a rating table was developed. The road centerline profile was input into Flowmaster (Manning's Equation) and a critical depth and top width were calculated for various flow rates. The correct flow rate was then looked up in the HEC-1 model which includes the storage effects from ponding. A flow of 161 cfs crosses Turlock Road at the dip section. This flow was then looked up in the rating table which shows that the flow would be over 250' wide and 5" deep.

Developed Condition

In order to mitigate the problem of flooding and ponding at the project site, it was decided that the flows needed to be controlled and detained. A diversion channel, a detention basin and lake are proposed. These structures are intended to minimize the extent of flooding within the project boundaries as well as reduce the extent of flooding across Coolidge Avenue and Turlock Road, that exists under the present conditions. The natural flow path of the creek remains, as well as the natural crossings leaving the project site. Exhibit 5 located in the Appendix shows the proposed drainage structures as well as the 100 year water surface under developed conditions.

West Branch of Llagas Creek

As under the existing condition, the West Branch of Llagas Creek is the main runoff conveyance system for the project. To mitigate the problem of flooding at the orchard which presently exists, a diversion channel is proposed to parallel the creek. Since a road is proposed to cross the creek at approximately station 21+13, only 2 24" pipes are proposed to be placed there. The remainder of the flow is expected to cross under the road further south through a larger culvert, and parallel the creek. The proposed culvert is a concrete arch bridge with a 24' span and 8' rise. The creek is to remain as is, and will continue to covey flows during all storm events. The difference is that the flows during larger storm events will be lower. The proposed diversion channel is to be trapezoidal with 3:1 side slopes, a bottom width of 10', and be grass lined. The diversion channel and the creek confluence at the culvert at Coolidge Avenue. Since the Coolidge Avenue culvert is not capable of conveying the runoff from larger storm events, a side spillway is proposed to route the flows north, into the proposed Coolidge Detention Basin. The spillway is set at elevation 272.7 and is 200' in length. The calculated depth of flow over the spillway is 1.1'. Calculations are included in the Appendix. The maximum water surface in the detention basin is set at elevation 273' allowing for a 25% submergence of the weir. The detention basin outflow is through a 18" low flow outlet pipe and a 83' spillway set at elevation 271'. The detention basin outlets at the dip section at Coolidge Avenue where it leaves the project site. The basin is designed to intercept the diverted flows from the creek as well as flows from drainage sub-basin 18S. Flows from drainage sub-basin 18N go around the northern edge of the detention basin, to the dip section at Coolidge Avenue, where they confluence with the outflow from the basin. These flow paths are included in the developed condition HEC-1 model dev100.hc1 located in the Appendix. The detention basin design is summarized in the table below.

Table 9
Coolidge Avenue Detention Basin Summary Table

Storm Event	Peak Inflow (cfs)	Peak Outflow (cfs)	Peak Storage (AF)	Peak Stage (feet)
100-year storm	674	663	23	273.00
10-year storm	261	254	18	272.03
2-year storm	63	15	9	269.70

The table shows that only a minor reduction in flow across the road is attained for the 10 and 100 year storms. The significant reduction in flows leaving the property over Coolidge avenue will be achieved for storm events more frequent than the 10 year storm.

To obtain a water surface profile for the West Branch of Llagas Creek a HEC-RAS model was completed. The model includes the proposed diversion channel. Output from the HEC-RAS model lion14.prj including profile, cross-sections and summary table are included in the Appendix.

Since the peak flow over Coolidge Avenue is known, the rating table for the Coolidge avenue crossing was consulted and the flow depth and width over the road was obtained. The flow was found to be 753 cfs with a flow top width of over 1,050' and depth of over 6".

Southeast Corner of Project Site

The natural flow path of drainage sub-basin 20 continues to be the south east corner of the project site under developed condition. The developed condition includes a 16 acre lake which serves as a detention basin. The normal water surface for the lake is set at elevation 275' with the 100 year water surface set at elevation 277.99'. The lake has a peak storage of 50 acre feet. Flows leave the lake over a 54' spillway set at elevation 277' and enters a swale which conveys the flow to the southeast corner of the project. The spillway has a 2' notch set at elevation 275.5' to allow the lake to empty to within 6" of its normal water surface following a storm. The swale itself has a minor flow attenuation effect as the flows pond up in the southeast corner prior to leaving the project site. The swale has a 15' spillway set at elevation 276'and a 16" RCP for a low flow outlet set at elevation 272'.Both the lake and swale are included in the developed condition HEC-1 model dev100.hc1 which is included in the Appendix. Tables 10 and 11 summarize the flow through the lake and swale.

Table 10
Lake Summary Table

Storm Event	Peak Inflow (cfs)	Peak Outflow (cfs)	Peak Storage (AF)	Peak Stage (feet)
100-year storm	173	144	50	277.99
10-year storm	85	41	40	277.43
2-year storm	40	6	25	276.53

Table 11
Swale Summary Table

Storm Event	Peak Inflow (cfs)	Peak Outflow (cfs)	Peak Storage (AF)	Peak Stage (feet)
100-year storm	144	128	22	277.96
10-year storm	41	28	11	276.49
2-year storm	6	6	0	272.69

Since the storage volume available in the lake and swale is small when compared to the volume of the incoming 100 year storm event, only a small reduction in peak flow is attained. However, the 10 and 2 year storm event peak flows are reduced by 67% and 85% respectively. Once the flow out of the swale was calculated, the rating table for the Turlock Road dip section was consulted for the flow top width and depth. The 100 year flow top width calculated was 250' with a maximum flow depth of about 4".

SUMMARY

The purpose of this drainage report is to quantify and characterize the storm runoff flows through the project site under present conditions, and design the drainage structures necessary to minimize onsite flooding and eliminate any increase in runoff leaving the project site as a result of development. The drainage infrastructure proposed includes a diversion channel to divert larger flows from the West Branch of Llagas Creek and route them through the proposed Coolidge Avenue Detention Basin eliminating flooding in the orchard and reducing the flow over Coolidge Avenue. Also included is a lake which intercepts and detains flows from drainage sub-basin 20 and reduces the flows crossing Turlock Road. The natural drainage paths for storm runoff that leave the project site remain in place. Given the large volume of the 10 and 100 year storm events in comparison to the available storage volume in the proposed lake and Coolidge Detention Basin, only a minor flow attenuation is obtained for these storm events. For more frequent storm events the reduction in flow is much more significant. The table below compares the flows leaving the property between the existing and developed conditions for the 2, 10 and 100 year storm events.

Table 12
Comparison of Existing and Developed Discharges Leaving the Property

	Existing Condition			Developed Condition			
	Flow (cfs)	Top Width of flow over road (ft)	Depth of flow over road (ft)	Flow (cfs)	Top Width of flow over road (ft)	Depth of flow over road (ft)	Percentage Flow Reduction
Coolidge Avenue Dip Section							
100 year	797	1050+	0.6	753	1050+	0.6	6%
10 year	332	980	0.4	294	800	0.4	11%
2 year	86	290	0.2	32	150	0.1	63%
Coolidge Avenue Box Culvert							
100 year	110	n/a	n/a	110	n/a	n/a	0%
10 year	110	n/a	n/a	110	n/a	n/a	0%
2 year	110	n/a	n/a	110	n/a	n/a	0%
Turlock Avenue Dip Section							
100 year	161	260	0.4	128	245	0.3	20%
10 year	73	210	0.3	28	110	0.1	62%
2 year	31	120	0.2	6	n/a	n/a	81%

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NOTE

The technical appendices and full scale exhibits of the Master Drainage Plan are not included in this EIR Addendum. These are contained in the full Master Drainage Plan document which is available for review at the County of Santa Clara Advance Planning Office.

ADDENDUM TO LION'S GATE RESERVE MASTER DRAINAGE PLAN

Prepared by

PACIFIC ADVANCED CIVIL ENGINEERING
17902 Georgetown Lane
Huntington Beach, CA 92648
714/843-5734

Date

April 1998

REGISTERED PROFESSIONAL ENGINEER
JOSEPH I. GUTIERREZ
No. 55604
Exp. 12-31-00
CIVIL
STATE OF CALIFORNIA

Table Of Contents

I. Introduction

This addendum to the Lion's Gate Master Drainage Plan (MDP), December 1997, covers the design of the A Street and Highland Avenue Bridges, as well as the design of the diversion structure. The analysis and design of these structures was not included in the original MDP. The hydraulic models included in this addendum supersede all hydraulic models in the MDP, and are to be considered final. Also included in this addendum are calculations for golf course drainage piping.

II. Diversion Structure

The proposed diversion structure, to be located approximately between stations 2449 and 2180, serves to divert a portion of the flow from the existing creek into the proposed diversion channel. The diversion channel parallels the existing creek through the orchard. The proposed diversion channel was necessary to mitigate the flooding problems resulting from the existing creek's insufficient conveyance capacity, in the area of the orchard. Major storm event flows regularly overtop the creek banks and flood the orchard. The design of the diversion channel is included in the main body of the Lion's Gate Master Drainage Plan. The design of the diversion structure is included in this addendum.

For environmental reasons, the existing creek will still convey flows during all storm events. The diversion structure serves to divert major flows from the existing creek into the proposed diversion channel, thereby eliminating flooding in the orchard area. This mitigation was accomplished by proposing two separate structures. First, at the A Street crossing, two 24" reinforced concrete pipes serve as a culvert and convey the flows under A Street to the existing creek during all storm events. The culvert, due to its relatively small conveyance capacity, also serves to back up the water in the West Branch of Llagas Creek, upstream of A Street. This backwater effect forces the water to spill over the side weir spillway diversion structure, into the proposed diversion channel. This proposed diversion structure is a side spillway, to be constructed as a rip rap reinforced berm, with the top of spillway set at elevation 279'. The expected water surface in this area is 283'. Plan and cross-section views of the proposed diversion structure are shown on Exhibit 4. The spillway is to be 75' in length. To verify that the spillway has a sufficient capacity to convey the flows, a weir calculation is included in the Appendix.

III. Bridges

The design of the A Street and Highland Avenue Bridges is included in this section of the addendum. The design was completed utilizing the HEC-RAS computer program. The hydraulic model used in the Lion's Gate Master Drainage plan was modified to include the two proposed bridges. Output from the HEC-RAS computer model (Lion15.prj) including a summary table, profile and cross-sections is included in the Appendix. To accurately size the bridges, scour calculations were also completed. The bridge design is summarized in the table below. The proposed bridges are cast-in place concrete arch bridges by Con Arch, Inc. A minimum of 2 feet of freeboard is provided between the water surface and bridge soffit.

	Highland Avenue Bridge	A Street Bridge
Station	3728	2161
Channel Invert Elevation	287.91	276.73
Calculated Bridge Scour (feet)	4.5	6.3
Bridge Footing Elevation	283.5	270.50
Flow Depth (feet)	3.26	6.3
Calculated Water Surface Elevation	291.17	282.01
Proposed Bridge Arch Height & Span	14.5 x 42	13.5 x 24
Bridge Soffit Elevation	298	284
Roadway Elevation	301	287
Available Freeboard (feet)	6.83	2.00

IV. GOLF COURSE DRAINAGE PIPING

The overall grading and pipe placement for the golf course was designed by the architect, Robert Trent Jones II, to convey nuisance flows through the course to the West Branch of Llagas Creek. Sizing of the pipes was completed by PACE and is summarized by the following:

For flow analysis of the golf course drainage pipes, both HEC-1 and the rational method was utilized. Tributary areas to each pipe inlet were determined. If the tributary area to a pipe corresponded to one of the drainage sub-basins delineated for the HEC-1 model, then the flows from the model were utilized to size the pipe. Otherwise, per the Santa Clara County Drainage Manual the following equation was used:

$Q = RCIA$

Where $R = 1$ (Table 6)
Assuming a 100-year design storm
$I = 1.75$ (per Figure 10)
$C = 0.2 + 0.15 + 0.05 + 0.1 = 0.5$ (per Table 4)
A = drainage area in acres

The equation simplifies to $Q = 0.875$ A.

Using Mannings Equation, pipe sizes were determined based on the assigned flows and the slopes. All pipes were designed to be partially full.

Piping Calculation Summary

Pipe Num.	Trib. Area (acres)	100-Yr. Flow (cfs)	Pipe Dia. (in)	Num. of Inlets	Inlet Size (in)	Slope (ft/ft)	Depth of Flow (ft)	Velocity (ft/s)
1	36.00	31.50	24	-	-	0.056	0.94	21.60
2	0.61	32.03	24	1	10	0.025	1.22	15.96
3	9.03	7.90	15	3	18	0.012	0.90	8.37
4	2.73	10.29	18	2	12	0.010	0.97	8.49
5	2.01	12.05	18	1	12	0.010	1.09	8.72
6	5.99	5.25	12	2	18	0.088	0.43	16.32
7	1.86	6.87	15	1	12	0.030	0.60	11.68
8	1.03	7.77	15	1	12	0.008	1.07	6.96
9	2.86	2.50	12	1	18	0.061	0.32	11.66
10	1.32	3.66	12	1	12	0.050	0.41	12.05
11	10.03	193.18	2-36	-	-	0.025	1.86	21.01
12	2.02	182.19	2-36	-	-	0.025	1.79	20.75
13	97.34	174.42	2-36	-	-	0.025	1.67	20.27
14	3.15	89.25	36	-	-	0.020	1.90	18.92
15	98.85	86.49	36	-	-	0.017	1.97	17.62

16	1.47	1.29	8	1	12	0.070	0.26	10.50
17	0.92	2.09	8	1	12	0.029	0.45	8.43
18	1.18	3.12	10	1	12	0.020	0.55	8.16
19	8.76	10.79	18	-	-	0.025	0.75	12.22
20	2.53	2.21	10	-	-	0.040	0.36	9.78
21	3.04	2.66	10	1	12	0.017	0.52	7.35
22	0.20	2.84	10	1	8	0.018	0.54	7.67
23	0.53	3.30	10	1	8	0.022	0.55	8.57
24	0.63	3.85	12	1	8	0.035	0.47	10.71
25	1.12	4.83	12	1	10	0.013	0.76	7.57
26	0.69	0.60	6	1	10	0.010	0.35	4.15
27	0.28	0.85	8	1	8	0.025	0.27	6.45
28	4.11	3.60	12	-	-	0.018	0.55	8.20
29	1.74	1.52	8	1	12	0.011	0.51	5.31
30	2.83	4.00	12	2	12	0.010	0.72	6.64
31	0.64	4.56	12	1	10	0.010	0.81	6.72
32	2.39	2.09	12	1	12	0.018	0.40	7.13
33	3.50	13.85	18	-	-	0.025	0.87	12.97
34	1.44	15.11	24	-	-	0.030	0.75	14.13
35	8.65	7.57	15	2	18	0.042	0.58	13.54
36	0.71	8.19	15	1	10	0.032	0.66	12.46
37	7.38	49.47	30	-	-	0.057	1.08	24.32
38	2.13	51.34	30	1	12	0.030	1.33	19.31
39	2.42	2.12	10	1	12	0.024	0.41	8.01
40	1.42	1.24	8	1	12	0.020	0.35	6.59
41	1.39	4.58	12	1	12	0.015	0.68	8.04
42	38.93	34.83	24	-	-	0.067	0.95	23.69
43	0.87	0.76	6	1	10	0.190	0.17	13.29
44	0.92	0.81	8	1	10	0.045	0.22	7.87
45	16.96	14.84	18	-	-	0.142	0.55	25.18
46	14.91	29.62	24	-	-	0.090	0.80	25.31
47	1.98	16.58	18	1	12	0.075	0.70	20.93
48	0.53	0.46	8	1	8	0.009	0.26	3.70
49	0.42	0.37	8	1	8	0.043	0.15	6.19
50	0.36	1.15	10	1	8	0.032	0.27	7.55
51	0.38	0.32	6	1	8	0.051	0.15	6.50
51A	-	20.00	21	-	-	0.058	1.29	10.53
52	0.67	20.90	21	1	8	0.023	1.05	13.91
53	1.44	23.31	24	1	12	0.016	1.37	10.15
54	0.49	0.43	6	1	8	0.036	0.19	6.22
55	3.57	26.86	30	2	12	0.010	1.26	10.88
56	1.29	1.12	8	1	10	0.053	0.25	9.14
57	0.65	27.42	30	1	10	0.010	1.27	10.94
58	0.90	29.35	30	1	10	0.010	1.32	11.12
59	0.62	0.55	6	1	8	0.020	0.26	5.34
60	0.84	0.74	6	1	10	0.010	0.42	4.23
61	0.41	0.35	6	1	8	0.013	0.23	3.99
62	0.84	0.73	6	1	10	0.012	0.38	4.58

63	0.73	0.63	6	1	10	0.038	0.23	6.99
64	0.65	0.57	6	1	8	0.010	0.33	4.11
65	2.8	2.45	10	2	12	0.010	0.60	5.87
66	6.6	8.25	15	2	18	0.010	1.01	7.80
67	0.56	0.49	6	1	8	0.010	0.30	3.98
68	0.65	9.31	18	1	10	0.010	0.91	8.31
69	1.27	1.11	8	1	12	0.010	0.41	4.88
70	21.12	18.48	24	2	24	0.010	1.15	9.89
71	0.54	20.07	24	1	10	0.010	1.21	10.07
72	7.03	6.15	15	2	18	0.010	0.79	7.47
73	-	59	24	-	-	0.048	1.51	23.21
74	1.3	1.14	8	-	12	0.026	0.31	7.10
75	0.62	1.68	10	-	10	0.050	0.29	9.85
76	13.04	11.41	15	3	18	0.035	0.79	14.01
77	0.74	0.64	6	-	10	0.041	0.23	7.28
78	1.66	13.5	18	-	12	0.040	0.74	15.41
79	2.11	1.85	8	-	12	0.147	0.25	15.18
80	1.27	2.96	8	-	12	0.186	0.31	18.74
81	1.29	4.08	10	-	12	0.179	0.33	19.94
82	1.84	5.69	10	-	12	0.083	0.51	16.27
83	1.36	6.88	12	-	12	0.053	0.58	14.46
84	0.54	0.47	6	-	10	0.067	0.17	7.97
85	0.68	1.06	6	-	10	0.055	0.29	9.14
86	1.51	9.26	15	-	12	0.019	0.84	10.57
87	10.85	9.5	12	2	24	0.110	0.57	20.59
88	0.51	9.95	15	-	10	0.038	0.71	13.91
89	4.07	13.51	18	2	12	0.010	1.22	8.81

Depending on the pipe location, it was either sized with grated drain inlets or headwalls. Drain piping outlets will discharge into Llagas Creek through outlet structures. A rip rap outlet structure detail is shown on Exhibit 5.

APPENDIX F

Biological Report

Prepared by

H.T. Harvey & Associates

May 1998

29 May 1998

Mr. Bert Verrips
Nolte and Associates, Inc.
1 N. First Street, Suite 450
San Jose, CA 95113
voice: 510.652.1666
facsimile: 510.547.6677

SUBJECT: Hayes Valley (Lions Gate): reconnaissance-level biotic constraints survey

Dear Mr. Verrips:

We have finished our reconnaissance-level field survey of the project modification areas. Three specific areas were surveyed, including: (1) the newly-proposed location of the clubhouse, (2) creek by-pass channels, and, (3) new location of the stable/corral complex. The purpose of our survey was to determine if these proposed changes to the original project resulted in significant impacts to biotic resources on site. Survey personnel included Dr. Patrick Boursier, plant ecologist. A detailed project description and field review of each location was supplied by Mr. Ron Davis. All of these three sites occur within the project boundaries intensively surveyed by H. T. Harvey & Associates staff in 1994-95 in preparation of our report entitled *Hayes Valley, Biological Resources Report* (30 Nov 95; PN 385-11). Each of the project modification sites are discussed below.

1. Clubhouse Site: The proposed location is within habitat previously identified in our report as non-native annual grassland situated near the confluence of two riparian corridors. It is our understanding that no trees will be removed within this area, the previously-approved riparian setback distance of 75 feet will be maintained, the creek crossing will occur at the same location as that initially proposed for the golf cart path crossing, however, the crossing will be widened somewhat to accommodate two-lane traffic. One two-lane bridge crossing is to be removed. This proposed modification will not result in any additional direct or indirect impacts to biotic resources.
2. Creek By-pass Channel: The by-pass channel occurs within the portion of the project site originally identified as agricultural, situated along Highland Avenue near its intersection with Coolidge Avenue. It is understanding that water from the native channel will be diverted above the 2.3-year flood event, all existing riparian vegetation will remain, water will be placed into a series of on-site retention basins. This proposed modification will not result in any additional direct or indirect impacts to biotic resources.

3. Stable/Corral Complex: The access road and stable/corral complex occurs within a habitat identified in our 1995 report as non-native annual grassland. The access road will utilize a currently-existing, unimproved dirt road. The access road will cross two seasonal drainage channels with existing culvert and/or bridge crossings. These crossings will be upgraded to handle increased traffic and may result in relatively minor impacts to seasonal wetland habitats within one of the drainages (on the order of 10-25 square feet). This proposed modification will not result in any additional direct or indirect impacts to biotic resources.

In summary, the proposed modifications discussed above will not result in significant impacts to existing biological resources, beyond those already identified and addressed in the approved Environmental Impact Report.

If you our your staff have any questions please feel free to contact me or Rick Hopkins.

Sincerely,

Patrick J. Boursier, Ph.D.
Division Head, Botany and Wetlands

APPENDIX G

Archaeological Report

Prepared by

Basin Research Associates

May 1998

BASIN
RESEARCH
ASSOCIATES

1933 DAVIS STREET
SUITE 210
SAN LEANDRO, CA 94577
VOICE (510) 430-8441
FAX (510) 430-8443

29 May, 1998

Mr. Bert Verrips
Nolte and Associates
1 North First Street
Suite 450
San Jose, CA 95113

RE: Review of Previous Cultural Resources Studies
Proposed Location of Club House, Horse Stables and Creek Bypass Channel
Lions Gate/Cordevalle Project, Santa Clara County

Dear Mr. Verrips,

Please let this letter serve as our review of the proposed location changes for the Club House and Horse Stables as well as the addition of a Creek Bypass Channel for the above project.

As you are aware, the project is situated in an area which has undergone a number of archival reviews and archaeological inventories as a result of cultural resource compliance requirements. Four archaeological sites, CA-SCl-76, SCl-77, SCl-305/H and SCl-568, have been recorded within the boundaries of the proposed project although only one prehistoric site, CA-SCl-76, was relocated during the various field programs. This site was also the subject of a presence/absence testing program to determine its horizontal and vertical extent [Fig. 1]. The three other reported sites for the project area, CA-SCl-77, SCl-305/H and SCl-568, did not have any visible surface indicators of a prehistoric occupation at their recorded location nor did auger testing expose the presence of subsurface cultural materials at their reported locations.

A review of the archival material on file at our office for the project indicates that none of the planned changes for the location of the Club House and Horse Stables will affect any known cultural resources. The Creek Bypass Channel is in the immediate and near vicinity of CA-SCl-76.

It is Basin Research Associates' considered opinion that the construction planned for the project can proceed as planned. No further archaeological research appears necessary and monitoring during subsurface construction at the Club House and Horse Stables does not appear warranted. However, archaeological monitoring of the first three to five feet of subsurface trenching for the Creek Bypass Channel is recommended by a professional archaeologist. The frequency and duration of the monitoring should be at the discretion of the archaeologist and dependent on his/her subsurface observations during trenching.

It is also recommended that if any unanticipated prehistoric or significant historic era cultural materials are exposed during construction, operations should stop within 20 feet of the find and a qualified professional archaeologist contacted for evaluation and further recommendations. Potential recommendations could include evaluation, collection, recordation, analysis, etc. of any significant cultural materials followed by a professional report.[1]

If I can provide any additional information or be of further service please don't hesitate to contact me.

Sincerely yours,
BASIN RESEARCH ASSOCIATES, INC.

Colin I. Busby
Principal

CIB/dg

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1. Significant prehistoric cultural resources are defined as human burials, features or other clusterings of finds made, modified or used by Native American peoples in the past. The prehistoric and protohistoric indicators of prior cultural occupation by Native Americans include artifacts and human bone, as well as soil discoloration, shell, animal bone, sandstone cobbles, ashy areas, and baked or vitrified clays. Prehistoric materials may include:

   a. Human bone - either isolated or intact burials.
   b Habitation (occupation or ceremonial structures as interpreted from rock rings/features, distinct ground depressions, differences in compaction (e.g., house floors).
   c. Artifacts including chipped stone objects such as projectile points and bifaces; groundstone artifacts such as manos, metates, mortars, pestles, grinding stones, pitted hammerstones; and, shell and bone artifacts including ornaments and beads.
   d. Various features and samples including hearths (fire-cracked rock; baked and vitrified clay), artifact caches, faunal and shellfish remains (which permit dietary reconstruction), distinctive changes in soil stratigraphy indicative of prehistoric activities.
   e. Isolated artifacts

   Historic cultural materials may include finds from the late 19th through early 20th centuries. Objects and features associated with the Historic Period can include.

   a. Structural remains or portions of foundations (bricks, cobbles/boulders, stacked field stone, postholes, etc.).
   b. Trash pits, privies, wells and associated artifacts.
   c. Isolated artifacts or isolated clusters of manufactured artifacts (e.g., glass bottles, metal cans, manufactured wood items, etc.).
   d. Human remains.

   In addition, cultural materials including both artifacts and structures that can be attributed to Hispanic, Asian and other ethnic or racial groups are potentially significant. Such features or clusters of artifacts and samples include remains of structures, trash pits, and privies.

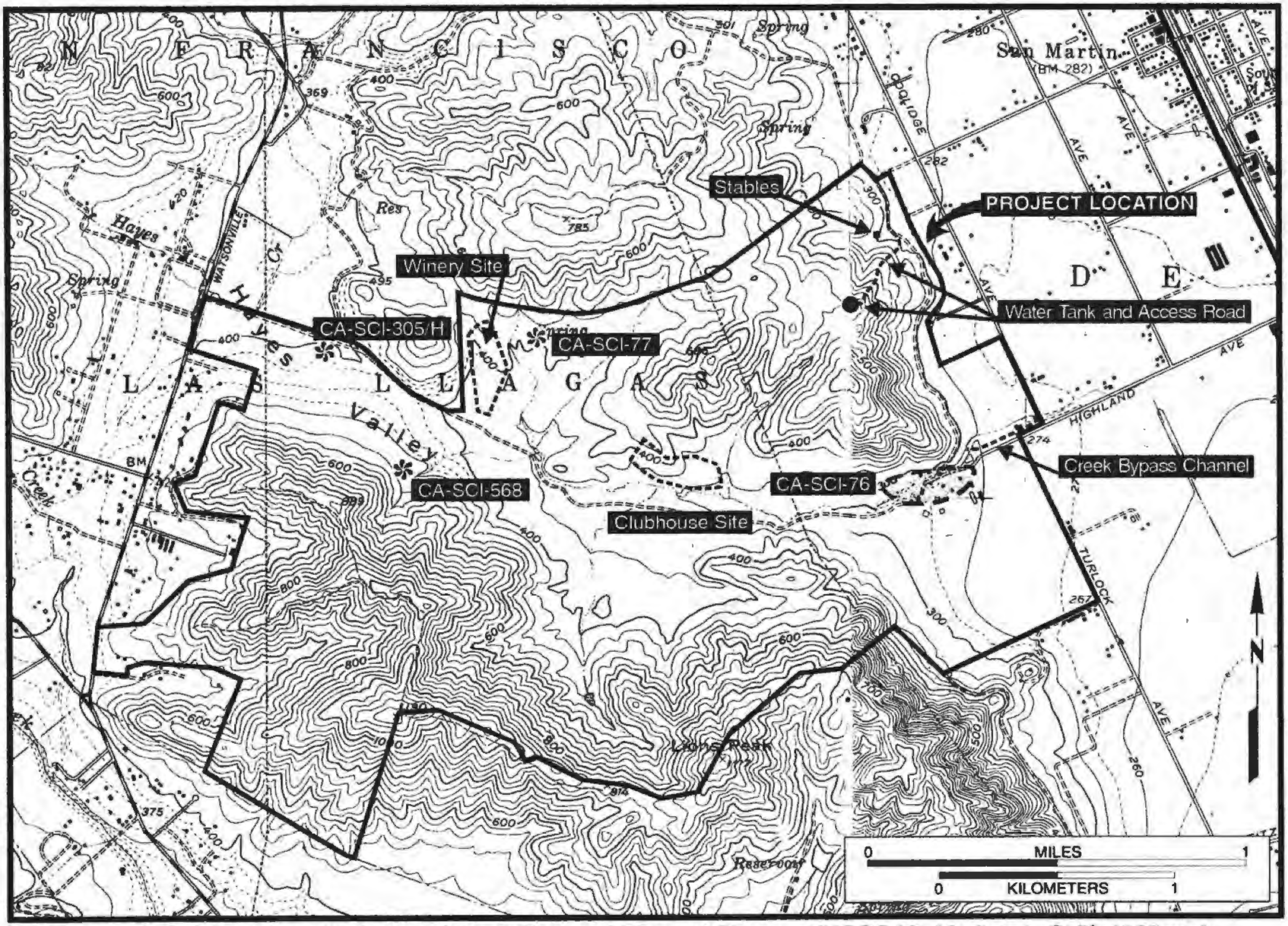


Figure 1: Project Location with Archaeological Sites and Planned Changes (USGS Mt. Madonna, Calif. 1980 and Gilroy, Calif. 1981)

APPENDIX H

Traffic Report

Prepared by

TJKM Transportation Consultants

May 1998

TJKM Transportation Consultants

May 27, 1998

Mr. Bert Verrips
Nolte Associates
1 North First Street, Ste 450
San Jose, CA 95113

Subject: Traffic Impact due to Incremental Square Footage in Restaurant Space at the Proposed Hayes Valley Country Club in the County of Santa Clara

Dear Mr. Verrips:

TJKM Transportation Consultants is pleased to present this traffic evaluation based on changes to the development proposal since our February 1996 traffic study report on the proposed Hayes Valley development. The new proposal calls for the restaurant space in the golf club house facility to be roughly 5,800 square feet as opposed to 4,000 square feet as was previously proposed. This letter report presents our evaluation of the impact of that incremental development. In summary, the impact of the additional space is negligible. No change in intersection delay or level of service occurs.

Note that this analysis uses the same trip generation and capacity analysis methodologies as the previous study. This is done to maintain consistency with this study despite minor recent changes in the ITE trip generation rates and the adoption by the county of new capacity analysis software.

Previous Impacts

In our earlier study, the proposed project was not found to have significant impacts at any of five study intersections:

1) Santa Teresa Boulevard/Sunnyside Avenue/Watsonville Road
2) Coolidge Avenue/San Martin Avenue
3) Monterey Road/San Martin Avenue
4) Santa Teresa Boulevard/Highland Avenue
5) Monterey Road/San Martin Avenue

In fact, even in the ultimate scenario which evaluated Existing plus Approved plus Proposed Project Traffic plus Expected Growth, only the p.m. peak conditions at the intersection of Monterey Road/San Martin Avenue fell below LOS B (at LOS C-).

Impact of Incremental Development

In order to determine whether the additional restaurant space, roughly 2,000 square feet, would produce an impact it is only necessary to add the incremental traffic generation and re-evaluate the project impact. Because the most project traffic is routed through the intersection of Monterey Road/San Martin Avenue, and this is the most congested intersection, a determination that there would be no p.m. peak impact at that intersection is a necessary and sufficient condition of determining that there would be no impact at any location.

4234 Hacienda Drive, Suite 101, Pleasanton, California 94588-2721, (510) 463-0611, Fax (510) 463-3690
Pleasanton . Santa Rosa

Using the trip generation assumptions of our previous analysis, the incremental trip generation due to the additional restaurant space would consist of 2 additional trips in the a.m. peak (1 in, 1 out) and 15 additional trips in the p.m. peak (10 in, 5 out). The 15 p.m. trips are of importance here -- 12 p.m. peak trips would be assigned to Monterey Road/San Martin Avenue. Assigning this additional traffic to the intersection and replicating the capacity analysis from the previous study reveals that all measures of delay and level of service are unchanged from the previous study (24 seconds of delay, LOS C-). Detailed calculation sheets from the latest analysis and the previous study are presented in Attachment A.

Conclusion

As has been shown, the impacts of the previous study are not changed given the additional restaurant space, the conclusion of no impact and therefore no mitigation measures also holds.

I hope that this analysis has been helpful. If there are any questions or comments, please feel free to give me a call.

Sincerely,

Michael Carroll
Transportation Engineer

rhm
Attachments
146-026l.1mc

Attachment A

Detailed Calculation Sheets

11-16-95

Previous Proposal

C A P S S I
COMPREHENSIVE ANALYSIS PROGRAM
FOR A SINGLE SIGNALIZED INTERSECTION *

Santa Clara County
EX + AP + PR + EXPECTED GROWTH
SOLUTION USING REQUIRED CYCLE TIME

FLN:3eg_p

3. Monterey/San Martin — P.M Peak Hour — Scenario 1

Movement	EBT	EBL	EBR	SBT	SBL	SBR	WBT	WBL	WBR	NBT	NBL	NBR
Phase 1 - 43 secs	X	X	X	.	.	.	X	X	X	.	.	.
Phase 2 - 5 secs	.	.	.	.	X	.	.	.	.	.	X	.
Phase 3 - 13 secs	.	.	.	X	X	X	.	.	.	.	.	.
Phase 4 - 26 secs	.	.	.	X	.	X	.	.	.	X	.	X
Phase 5 - 0 secs	.	.	.	.	.	.	.	.	.	.	.	.
Phase 6 - 0 secs	.	.	.	.	.	.	.	.	.	.	.	.
Critical Mvmt-**					****		****			****		
Peak 15 Vol -vph	86	17	16	519	222	41	82	175	253	426	12	188
Saturation -vph	1000	Shrd	1800	3600	1700	1800	1300	Shrd	Shrd	3600	1700	Shrd
Lost time -sec	4.00	-	2.00	6.00	4.00	2.00	4.00	-	-	6.00	4.00	-
Relative Sat 'X'	0.23	-	0.02	0.38	0.81	0.05	0.88	-	-	0.74	0.61	-
Effective Gr-sec	39	-	41	33	14	37	39	-	-	20	1	-
Move Time -sec	43	-	43	39	18	39	43	-	-	26	5	-
Min/Ped Time-sec	26	-	26	26	4	26	26	-	-	26	4	-
Prog Factor PAF	1.00	-	1.00	1.00	1.00	1.00	1.00	-	-	1.00	1.00	-
AvDelay/veh -sec	11	-	9	15	38	11	26	-	-	26	60	-
Level of Service	B-	-	B+	B-	D-	B-	D+	-	-	D+	F	-
Av.'Q'/ lane veh	1	-	0	4	5	1	7	-	-	6	0	-
Veh Stopping %	62	-	53	73	97	59	91	-	-	93	100	-
Do Veh Clear ?	YES	-	YES	YES	YES	YES	YES	-	-	YES	YES	-

Whole Intersection - Weighted Av Delay (sec) = 24 Level of Service = C-
Critical Movements - Weighted Av Delay (sec) = 28 Level of Service = D+
'' '' - Intersection Capacity Utilization (ICU) = 0.83

Required Cycle Length is 87 seconds (All Minimum times are satisfied)

* CAPSSI (Release 11) - Based on Delay Methodology Per 1985 Highway Capacity Manual

C A P S S I
COMPREHENSIVE ANALYSIS PROGRAM
FOR A SINGLE SIGNALIZED INTERSECTION *

05-26-98 Revised Proposal

SOLUTION USING PREDETERMINED CYCLE TIMES

FLN:rico

monterey/san martin A.M Peak Hour Scenario 1

Movement	EBT	EBL	EBR	SBT	SBL	SBR	WBT	WBL	WBR	NBT	NBL	NBR
Phase 1 - 45 secs	X	X	X	.	.	.	X	X	X	.	.	.
Phase 2 - 5 secs	.	.	.	.	X	.	.	.	.	.	X	.
Phase 3 - 13 secs	.	.	.	X	X	X	.	.	.	.		.
Phase 4 - 24 secs	.	.	.	X	.	X	.	.	.	X	.	X
Phase 5 - 0 secs	.	.	.	.	.	.	.	.	.	.	.	.
Phase 6 - 0 secs	.	.	.	.	.	.	.	.	.	.	.	.

	EBT	EBL	EBR	SBT	SBL	SBR	WBT	WBL	WBR	NBT	NBL	NBR
Critical Mvmt-**					****		****			****		
Peak 15 Vol -vph	87	17	16	520	222	41	83	180	253	427	12	191
Saturation -vph	1000	Shrd	1800	3600	1700	1800	1300	Shrd	Shrd	3600	1700	Shrd
Lost time -sec	4.00	-	2.00	6.00	4.00	3.00	4.00	-	-	6.00	4.00	-
Relative Sat 'X'	0.22	-	0.02	0.41	0.81	0.06	0.84	-	-	0.83	0.6[illegible]	-
Effective Gr-sec	41	-	43	31	14	34	41	-	-	18	[illegible]	-
Move Time -sec	45	-	45	37	18	37	45	-	-	24	5	-
Min/Ped Time-sec	20	-	20	20	0	20	20	-	-	20	0	-
Prog Factor PAF	1.00	-	1.00	1.00	1.00	1.00	1.00	-	-	1.00	1.00	-
AvDelay/veh -sec	10	-	9	16	38	13	23	-	-	31	60	-
Level of Service	B-	-	B+	C+	D-	B-	C-	-	-	D	F	-
Av.'Q'/ lane veh	1	-	0	4	5	1	7	-	-	6	0	-
Veh Stopping %	59	-	51	75	97	62	88	-	-	96	100	-
Do Veh Clear ?	YES	-	YES	YES	YES	YES	YES	-	-	YES	YES	-

Whole Intersection - Weighted Av Delay (sec) = 24 Level of Service = C-
Critical Movements - Weighted Av Delay (sec) = 29 Level of Service = D+
" " - Intersection Capacity Utilization (ICU) = 0.83

Predetermined Cycle Length is 87 seconds (Min. times may not be satisfied)

* CAPSSI (Release 11) - Based on Delay Methodology Per 1985 Highway Capacity Manual

APPENDIX I

<u>Noise Report</u>

Prepared by

Illingworth & Rodkin

May 1998

ILLINGWORTH & RODKIN, INC.
Acoustics • Air Quality

May 29, 1998

Bert Verrips
Nolte & Associates
1 North First Street, Suite 450
San Jose CA 95113

Subject: Hayes Valley Ranch EIR

Dear Bert:

This letter is in response to the proposed change in the clubhouse location at Hayes Valley Ranch. The clubhouse under the current plan would be moved approximately 600 feet closer to the home located on the ridge to the east of the Hayes Valley. Noise generated at the clubhouse area would be perceived at a level about 2 decibels louder than the location farther from the home. The resulting level would not be noticeably different than generated at the previous location and the resulting noise levels would be within the range predicted at our previous study as noted in our letter dated February 5, 1996.

Sincerely,

Richard R. Illingworth, PE

RRI:lk
(95-012)

85 Bolinas Road, #11 • Fairfax, California 94930 • (415) 459-5507 • FAX (415) 459-6448

THIRD ADDENDUM TO

ENVIRONMENTAL IMPACT REPORT

LION'S GATE RESERVE

(CordeValle)

LEAD AGENCY: COUNTY OF SANTA CLARA

File #4039-67-28-93
SCH #94043016

October 1998

TABLE OF CONTENTS*

* The contents only include sections of the EIR that have been revised in this Addendum.

INTRODUCTION

Description of Project Modifications

This Third EIR Addendum has been prepared to address the changes to the Lion's Gate Reserve (CordeValle) project that have been proposed since the time that the EIR on the project was certified by the County Board of Supervisors in August 1996, and since the first EIR Addendum was prepared in January 1997 and the second EIR Addendum was prepared in June 1998.

The main changes to the project addressed in this EIR Addendum include the addition of two new project elements as follows: 1) construction of a winery/grape processing center on approximately 18 acres in the northwest portion of the site; 2) installation of a 400,000-gallon water storage tank (with maintenance access road and water pipeline) in the northeast portion of the site. These new project elements are described in detail below, followed by a summary evaluation of potential impacts resulting from these new facilities. The changes to the EIR resulting from these project elements are addressed in the body of this addendum.

<u>New Winery Facility</u>

The Lion's Gate project was approved by the County of Santa Clara subject to a condition that approximately 82.5 acres of the project's designated permanent open space area be planted in vineyards. In order to process the grapes from this on-site vineyard, the applicant proposes to construct a winery/grape processing center on approximately 18 acres in the northwest portion of the site, north of the golf course maintenance facility. Having the winery/processing center on-site would eliminate the need to truck grapes off-site for processing.

The land to be occupied by the winery has been removed from the permanent open space area of the Lion's Gate/CordeValle project and incorporated into the parcel containing the golf course and related facilities. (This aspect of the winery project was previously addressed in the Second Addendum to the EIR of June 1998).

The winery site is a located on gently sloping terrain covered in annual grasses and a few scattered oaks. The winery facilities would include a 25,000 square-foot production facility, which would be equipped for all phases of the wine-making process and would include administrative offices, meeting rooms, and a reception area. The winery's architectural image is planned to be of high quality and would complement the style of the larger project. Building materials would primarily consist of stucco walls and tile roofs, with some external elements clad in stone veneer. The facility would include a grape receiving area at the north end of the winery building and a truck dock at the south end for receiving barrels and shipping finished product. Twenty parking spaces would be provided for employees and visitors. A landscaped berm would be installed east of the winery building to screen the parking area from view of the nearby golf course.

The facility would include a 5,000 square-foot stand-alone structure for the storage and maintenance of mobile vineyard equipment. The equipment storage building would be located just north of the golf course maintenance facility and would not include fuel storage tanks. Fuel for the winery equipment would be obtained from the golf course maintenance facility.

Access to the winery would be exclusively from the controlled access maintenance road to Watsonville Road, and would include a 20-foot wide crushed gravel driveway extending north from the golf course maintenance facility.

The winery would be equipped for all phases of the wine-making process including crushing, fermentation, barrel aging, and bottling. The production capacity of the facility is estimated to be 45,000 cases per year, which is sufficient to process the grapes from approximately 100 acres of vineyards, and would be adequate to handle the annual grape harvest from the site. The winery would have approximately 8 full-time staff, with an additional 6 temporary workers employed each fall for the harvest and crush.

The winery would include a hospitality area that would be open to trade representatives and the public by invitation only. A small tasting room for the winery would also be included in the main golf course clubhouse complex and would be open to golf course guests only.

The traffic generated by the winery would include trips by employees and visitors, as well as about 40 truck trips to transport finished product (cases of wine) which would occur periodically throughout the year. In comparison, if all the grapes grown on-site had to be trucked to off-site processing centers, this would involve approximately 200 truck loads using 18-wheeled trucks.

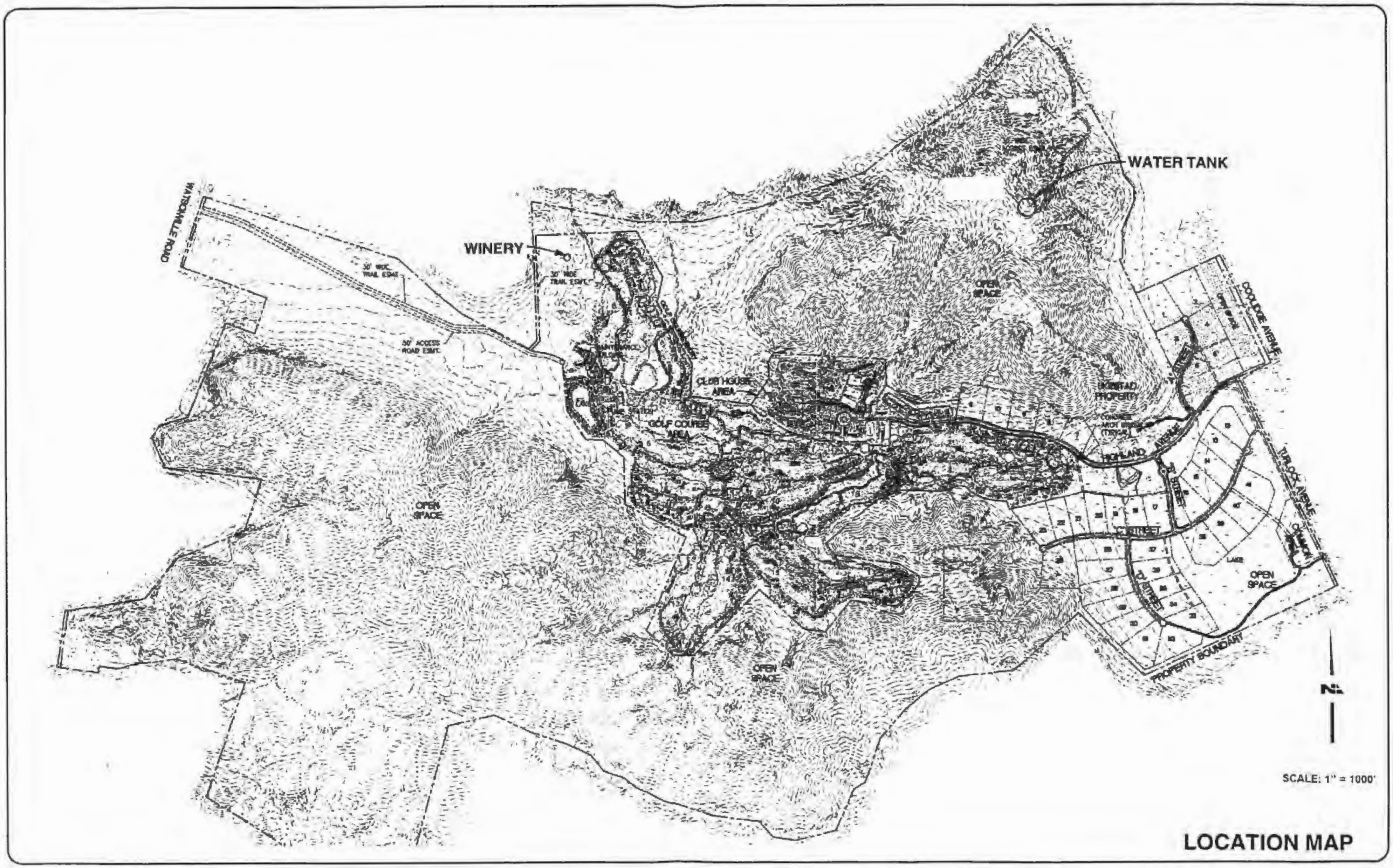
The winery would utilize approximately 700,000 gallons of non-potable water per year (which represents approximately 0.5 percent of total project water use). Most of this water would be used for irrigating the vineyard, although a small portion would be used for washing down the vats and equipment at the winery. This non-potable water would be obtained from the golf course irrigation reservoir located south of the golf course maintenance facility. Use of domestic water at the winery would relatively minor and the water would be obtained from the water line serving the golf course maintenance facility.

Current plans are to treat domestic wastewater generated by the winery at a new septic tank and leachfield system south of the winery building. Alternatively, wastewater from the winery would be piped to the septic system at the maintenance facility. The siting and design of the septic tank and leachfield system would be subject to the criteria and standards of the Santa Clara County Department of Environmental Health.

Washdown water from cleaning the vats and equipment would be screened for solids and then piped to two small treatment ponds occupying a 0.5-acre area south of the winery building. This washdown water would include some residue from cleaning detergents and minor amounts of chemicals used in the wine making process. The treatment ponds would include aerators to prevent stagnation and odor generation, which would also prevent mosquito breeding. Some of the treated washwater would evaporate at the ponds and the remainder would be used for irrigation or frost protection in the vineyards. The organic material screened from the washwater would be applied on the vineyards or would be used in making mulch for soil amendment. The treatment ponds would include landscaped berms to the east and west to screen them from view of the golf course and winery access road, respectively. The design and installation of the wastewater treatment ponds would be subject to the approval of the Central Coast Regional Water Quality Control Board.

Various chemicals are used in the wine making process and detergents are used for cleaning the vats and equipment. None of these substances would be used in significant quantities and therefore would not require a hazardous materials permit. The used chemicals and cleaning detergents would be piped with the washwater to the on-site treatment ponds.

Grading for the winery facilities and access road would include cuts and fills of up to about 8 feet, and would involve approximately 11,300 cubic yards of cut and 14,800 cubic yards of fill. The 3,500 cubic yards of fill to be brought to the site would be obtained from surplus earthwork from other areas of the CordeValle project. Retaining walls would be required at several locations primarily to prevent tree removal along the winery access road and around the winery building. The height of the retaining walls would vary with the



Source: PACE Engineering

terrain and would be no higher than about 8 feet. No trees would be removed as a result of constructing the winery facility.

The access road and drainage facilities for the winery would be designed by a qualified civil engineer in accordance with County requirements and standards. Electric power and telephone service would be extended underground to the winery from the golf course maintenance facility. Fire hydrants would be provided in conformance with the requirements of the County Fire Marshal's Office.

Water Storage Tank

A new 400,000-gallon domestic water storage tank for the West San Martin Water Works is planned for the upper elevations in the northeast portion of the site. This tank is needed to provide adequate water pressure and fire flows to the Lion's Gate project, and would also improve fire flows for existing residences east of the project site.

The maintenance access road to the tank would commence from the western extension of San Martin Avenue and would follow an existing dirt track up the hillside to the tank site. The new water main from the tank would be installed in the tank access road to the toe of the eastern hillside where it would split into two mains heading north and south. The southern main would follow the base of the hill to the residential portion of the CordeValle project located north of Highland Avenue. The northern main would follow the maintenance access road to San Martin Avenue where it would tie into an existing water line.

The water tank site is located on an broad eastward sloping swale just below the ridgeline. The tank would be 70 feet in diameter and 34 feet high, with approximately one-half of the overall tank height located below adjacent native ground level. The tank walls would be reinforced concrete supported on a spread foundation, and the tank would have an aluminum domed roof which would be rigidly connected to the tank walls.

Cuts of up to 23 feet would be required to achieve a level pad for the tank. The tank foundation would bear entirely on cut. A french drain would be installed outside the perimeter of the tank to control subsurface drainage.

The tank site takes advantage of existing trees to provide visual screening from the valley floor to the east. Additional trees would be planted as needed to increase visual screening. No trees would be removed for the tank, access road, or water mains.

Summary Evaluation of Potential Impacts Resulting from Winery and Water Tank

The proposed winery and water tank would not result in any new significant environmental impacts compared with the project evaluated in the EIR. The environmental effects of the new project elements are briefly evaluated below.

Land Use: The winery and water tank represent a very minor addition in square footage of the project, and would not significantly increase the land use intensity of what is already a very low density development. The winery and water tank sites are not adjacent to existing off-site development, and as discussed under 'Aesthetics' below, would be visible only in the distance from a few existing residences. Since the winery and water tank would not result in significant land use impacts, no changes are required to EIR Section *III. A. Land Use.*

Agriculture: The winery would provide a facility for processing grapes from the vineyards that were stipulated as a mitigation for loss of prime farmland in the EIR. The EIR Section *III. B. Agriculture* has been amended to include mention of the winery's role in processing the grapes produced on-site.

Parks, Recreation and Open Space: As discussed in the second EIR Addendum of June 1998, the removal of the winery site from the project's permanent open space area would result in a very small reduction of the open space area. However, the total open space allocation of the project still exceeds the 1,226 acres required to fulfill the 90 percent open space requirement for the Hillside cluster subdivision. The winery site is located in close proximity to the on-site segment of the San Martin Cross-Valley Trail which will run along the northern project boundary. The winery site has been designed to leave a strip of permanent open space between the winery site and northern and western site boundary that is of ample width to accommodate the 30-foot wide cross-site trail easement. The water tank would have no impact on the cross-site trail or any other open space amenity. The water tank and related facilities are also located well away from the cross-site trail easement and would not have a significant impact on recreation and open space. No changes are required to EIR Section *III. C. Parks, Recreation and Open Space*.

Geology and Soils: The sites of the winery and water tank were evaluated for geologic constraints by Twining Laboratories in October 1998. The study found that there are no earthquake faults or bedrock fault contacts in the vicinity of either the winery or the water tank sites. Likewise, there are no landslides in the vicinity of the winery or water tank, and the native slopes in the vicinity of both facilities appear relatively stable. Neither site is susceptible to liquefaction or seismic settlement, and both sites are located well away from the mapped area of serpentine bedrock located elsewhere on the Lion's Gate site. The near-surface soils at both the winery and water tank sites have medium potential for soils expansion. This would not pose a problem at the water tank site since the tank site will be subexcavated well below the surface soil. At the winery site, mitigation for expansive soils would consist of overexcavation for footings and floor slabs. Shallow groundwater is present at the water tank site, which would be mitigated by the installation of proper surface and subsurface drainage facilities. The winery site does not appear to be subject to high groundwater. The EIR Section *III. D. Geology and Soils* has been amended to incorporate the pertinent findings of the Twining report, insofar as these issues have not already been covered in the EIR. The Twining report is included in Appendix C of this EIR Addendum.

Hydrology and Drainage: No part of either the winery or water tank sites are located within or across existing drainage courses. The winery site is located west of an intermittent drainage courses in the northwestern portion of the project. Proper drainage facilities for the winery site will be designed by a civil engineer in accordance with County requirements. The water tank is located at the head of a swale just below a broad ridgeline. The tank site has a tributary drainage area of only 3.0 acres, so minimal storm flow will pass through the tank vicinity. The tank site will be designed to convey surface and subsurface drainage around the tank to the swale below. Neither the winery nor water tank would result in significant increases in site runoff or alteration of site drainage patterns. No changes are required to the EIR Section *III. E. Hydrology and Drainage*.

Water Quality: The water tank and winery facilities would result in relatively small areas of additional paved surfaces where non-point pollutants could accumulate and wash off to the adjacent watershed. These effects are adequately covered in the existing EIR Section *III. E. Water Quality*. (See 'Wastewater Treatment and Disposal' below for discussion of treatment and disposal of domestic wastewater and washdown water.)

Biological Resources: The proposed winery and water tank elements (including the tank access road and pipeline alignments) have been evaluated by H.T. Harvey and Associates. The biologists surveyed the sites and found no sensitive species or habitats that would be affected by these new project elements. No trees would be removed as a result of either of these new project elements. Therefore, the winery and water tank would result in no new potential impacts to biological resources. No changes are required to EIR Section *III. F. Biological Resources*. The letter report prepared by Harvey and Associates which addresses these new project elements is contained in Appendix F of this EIR Addendum.

Archaeology: The winery and water tank facilities (including the tank access road and pipeline alignment) are not within areas of archaeological sensitivity and there are no known archaeological resources in the vicinity of these sites. Therefore, the winery and water tank would result in no new potential impacts to archaeological resources. No changes are required to the EIR Section *III. E. Archaeology*. A letter report on these project elements prepared by Basin Research Associates is contained in Appendix G of this EIR Addendum.

Aesthetics: The winery site is located in the northwest corner of the project site where it is all but invisible from public vantage points. The winery would only be visible from a single residence on the off-site ridge to the north, at a distance of at least 2,000 feet. The winery would be designed to conform to the architectural style of the CordeValle clubhouse complex, and no trees would be removed for the winery. The water tank would be installed at a relative high elevation; however, the visibility of the tank would be minimized by its location in a broad swale just below the ridgeline. Approximately one-half of the tank would be buried so only the upper portion of the tank would extend above ground elevation. The tank site takes advantage of existing trees to provide visual screening from the valley floor. Additional trees would be planted as needed to increase visual screening. The tank may be partially visible in the distance from the valley floor to the east and also from some residences in the Hayes Valley Ranch to the north and west, which would be at least 2,000 feet away. Thus neither the winery nor the water tank would result in significant visual impacts. The EIR Section *III. J. Visual and Aesthetics* has been modified to include discussions of the winery and water tank.

Traffic: The traffic generated by the winery would include trips by employees and visitors, as well as about 40 truck trips to transport finished product (cases of wine), which would occur periodically throughout the year. There would also be occasional trips by delivery vehicles. This level of trip generation would not have a significant effect on traffic operations along Watsonville Road. The EIR Section *III. K. Traffic and Circulation* has been amended to include a discussion of traffic generated by the winery.

Noise: Neither the winery nor the water tank would result in significant new operational noise sources. The winery operation would be conducted entirely indoors, including the crushing of grapes during the harvest season. There would be occasional noise generated by trucks traveling to the winery, but this noise would not be audible from off-site locations. The operation of the water tank likewise would not generate noise audible from off-site locations, and truck traffic from maintenance vehicles visiting the tank would be infrequent. Therefore, no changes would be made to the EIR Section *III. L. Noise* with respect to operational noise.

The construction noise generated during installation of the winery and the water tank would be noticeable but not significant at the nearest residences which are located at least 1,000 feet away in both cases. Construction of the portion of the tank access road along the base of the hillside may temporarily elevate noise levels at the nearest residences to the east along the western extension of San Martin Avenue. These residences would also be subject to temporary noise from truck traffic generated during the construction of the water tank. This may result in a short-term noise impact at these residences, although the impact would be mitigated by measures contained in the EIR. The EIR Section *III. L. Noise* has been amended to include a discussion of this potential construction noise impact.

Air Quality: The slight increase in traffic resulting from the addition of the winery facility would cause a very small increase in the generation of vehicle emissions. However, according to air quality consultant M'OC Physics Applied, this increase would not be significant in terms of either local carbon monoxide concentrations or in term of pollutants of regional concern. No changes are required to EIR Section *III. M. Air Quality*. The winery operation would not result in the creation of noxious odors. The grape crushing would occur entirely within the winery building, and the fermentation process would occur in fully enclosed vats. At close range the winery would exude the pleasant smell of oak and fruit. However, at the nearest residence located at least 1,000 feet north no winery odors would be detectable.

Hazards: Various chemicals would be used in the wine making process and detergents would be used for cleaning of the vats and equipment. In addition, small amounts of oils and lubricants would be used by the vineyard tractors and equipment (fuel would be obtained from the nearby golf course maintenance facility). These chemicals or hydrocarbons would not be used in significant quantities and therefore would not require a hazardous materials permit. No changes to the EIR Section *III. N. Hazardous Materials, Public Health and Safety* are required.

Water Supply: The winery would use approximately 210 gallons of domestic water daily for the maximum of 14 staff who would be on-site during the harvest and crush. In addition, a daily average of approximately 2,000 gallons of non-potable water would be used for washing down the vats and equipment. This additional water consumption represents less than 0.5 percent of the total water consumption estimate for the CordeValle project and would be readily accommodated by the surplus water supply available to the project as calculated in the EIR. The EIR Section *III. P. Water Supply* has been amended to include the additional water demand for the winery.

Wastewater Treatment and Disposal: Current plans are to treat domestic wastewater generated at the winery at a new septic tank and leachfield system to be located south of the winery building. However, wet weather percolation tests have not yet been conducted to determine whether on-site soils are suitable for leachfields. Alternatively, wastewater from the winery would be piped to the septic system at the nearby golf course maintenance facility. The siting and design of the septic tank and leachfield system would be subject to the criteria and standards of the Santa Clara County Department of Environmental Health.

Washdown water from cleaning the vats and equipment would be screened for solids and then piped to two small treatment ponds occupying a 0.5-acre area south of the winery building. The treatment ponds would include aerators to prevent stagnation and odor generation, which would also prevent mosquito breeding. Some of the treated washwater would evaporate at the ponds, and the remainder would be used for irrigation or frost protection in the vineyards. The organic material screened from the washwater would be applied to the vineyards or used in making mulch for soil amendment. The design and installation of the wastewater treatment ponds would be subject to the approval of the Central Coast Regional Water Quality Control Board. The EIR Section *III. Q. Wastewater Treatment and Disposal* has been amended to include a discussion of wastewater treatment and disposal for the winery.

Rationale for Preparation of an EIR Addendum

This document has been prepared in accordance with the requirements of the California Environmental Quality Act (CEQA) which sets forth specific requirements for the documentation of potential environmental impacts which may result from modifications made to a proposed project after an EIR on the project has been certified. Under these circumstances, Sections 15162 through 15164 of the CEQA Guidelines provide for the preparation of one of three types of documents depending on the situation. The criteria to be met for each type of document are as follows: 1) a 'Subsequent EIR' shall be prepared if the changes to the project are substantial, and will result in major revisions to the EIR due to the involvement of new significant environmental effects or a substantial increase in the severity of previously identified significant effects; 2) a 'Supplement to an EIR' shall be prepared if the conditions described in #1 above apply but only minor changes or revisions to the EIR are necessary; and 3) an 'Addendum to an EIR' shall be prepared if some minor changes and additions are necessary, but the conditions which would necessitate the preparation of a Supplement to an EIR are not present. In the present case, the proposed modifications may or may not be considered substantial, but in no instance would new significant environmental effects be involved or the severity of a significant effect be increased substantially, as discussed above and in the body of this document. In addition, the changes to the EIR required to address the proposed project modifications are minor in nature. Thus two of the required criteria for preparing a Subsequent EIR and one of the required criteria for preparing a Supplement to an EIR would not apply. Therefore, according to CEQA criteria noted above, the type of environmental document that should be prepared in this instance is an 'Addendum to an EIR'.

Organization of This Document

Since this is the Third Addendum to the EIR, this document identifies revisions to the certified EIR, as modified by the First and Second Addendums, which reflect the changes in project description and environmental analysis resulting from the proposed modifications to the project. In order to facilitate the reader's comprehension without having to refer back to the certified EIR and the previous Addendums, this document contains the affected portion of the EIR to provide a context for the text changes. Revisions to the text are indicated by ~~strikethrough~~ for deletions and <u>underline</u> for additions.

I. PROJECT DESCRIPTION

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B. DESCRIPTION OF THE PROPOSED PROJECT

Overview

1. ~~A~~ Use Permit applications for a public access championship golf course, including a clubhouse with restaurant, 45 units of overnight accommodations, a practice range, a maintenance facility, ~~and~~ a swim and tennis center, a winery/grape processing center, and a water storage tank.

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Winery/Grape Processing Center

In order to process the grapes from the on-site vineyards, the applicant proposes to construct a winery/grape processing center in the northwest portion of the site, north of the golf course maintenance facility. Having the winery/processing center on-site would eliminate the need to truck grapes off-site for processing.

The winery site comprises approximately 18 acres of gently sloping terrain covered in annual grasses and a few scattered oaks (see Figures 9a, 10e and 10f). The winery facilities would include a 25,000 square-foot production facility, which would be equipped for all phases of wine making and would include administrative offices, meeting rooms, and a reception area. The winery's architectural image is planned to be of high quality and would complement the style of the larger project. Building materials would primarily consist of stucco walls and tile roofs, with some external elements clad in stone veneer. The facility would include a grape receiving area at the north end of the winery building and a truck dock at the south end for receiving barrels and shipping finished product. Twenty parking spaces would be provided for employees and visitors. A landscaped berm would be installed east of the winery building to screen the parking area from view of the nearby golf course.

The facility would include a 5,000 square-foot stand-alone structure for the storage and maintenance of mobile vineyard equipment. The equipment storage building would be located just north of the golf course maintenance facility and would not include fuel storage tanks. Fuel for the equipment would be obtained from the golf course maintenance facility.

Access to the winery would be exclusively from the controlled access maintenance road to Watsonville Road, and would include a 20-foot wide crushed gravel driveway extending north from the golf course maintenance facility.

The winery would be equipped for all phases of the wine-making process including crushing, fermentation, barrel aging, and bottling. The production capacity of the facility is estimated to be 45,000 cases per year, which is sufficient to process the grapes from approximately 100 acres of vineyards, and would be adequate to handle the annual grape harvest from the site. The winery would have approximately 8 full-time staff, with an additional 6 temporary workers employed each fall for the harvest and crush.

The winery would include a hospitality area that would be open to trade representatives and the public by invitation only. A small tasting room for the winery would also be included in the main golf course clubhouse complex and would be open to golf course guests only.

The traffic generated by the winery would include trips by employees and visitors, as well as about 40 truck trips to transport finished product (cases of wine) which would occur periodically throughout the year. In comparison, if all the grapes grown on-site had to be trucked to off-site processing centers, this would involve approximately 200 truck loads using 18-wheeled trucks.

The winery would utilize approximately 700,000 gallons of non-potable water per year (which represents approximately 0.5 percent of total project water use). Most of this water would be used for irrigating the vineyard, although a small portion would be used for washing down the vats and equipment at the winery. This non-potable water would be obtained from the golf course irrigation reservoir located south of the golf course maintenance facility. Use of domestic water at the winery would relatively minor and the water would be obtained from the water line serving the golf course maintenance facility.

Current plans are to treat domestic wastewater generated by the winery at a new septic tank and leachfield system south of the winery building. Alternatively, wastewater from the winery would be piped to the septic system at the maintenance facility. The siting and design of the septic tank and leachfield system would be subject to the criteria and standards of the Santa Clara County Department of Environmental Health.

Washdown water from cleaning the vats and equipment would be screened for solids and then piped to two small treatment ponds occupying a 0.5-acre area south of the winery building. This washdown water would include some residue from cleaning detergents and minor amounts of chemicals used in the wine making process. The treatment ponds would include aerators to prevent stagnation and odor generation, which would also prevent mosquito breeding. Some of the treated washwater would evaporate at the ponds and the remainder would be used for irrigation or frost protection in the vineyards. The organic material screened from the washwater would be applied on the vineyards or would be used in making mulch for soil amendment. The treatment ponds would include landscaped berms to the east and west to screen them from view of the golf course and winery access road, respectively. The design and installation of the wastewater treatment ponds would be subject to the approval of the Central Coast Regional Water Quality Control Board.

Various chemicals are used in the wine making process and detergents are used for cleaning the vats and equipment. None of these substances would be used in significant quantities and therefore would not require a hazardous materials permit. The used chemicals and cleaning detergents would be piped with the washwater to the on-site treatment ponds.

Grading for the winery facilities and access road would include cuts and fills of up to about 8 feet, and would involve approximately 11,300 cubic yards of cut and 14,800 cubic yards of fill. The 3,500 cubic yards of fill to be brought to the site would be obtained from surplus earthwork from other areas of the CordeValle project. Retaining walls would be required at several locations primarily to prevent tree removal along the winery access road and around the winery building. The height of the retaining walls would vary with the terrain and would be no higher than about 8 feet. No trees would be removed as a result of constructing the winery facility.

The access road and drainage facilities for the winery would be designed by a qualified civil engineer in accordance with County requirements and standards. Electric power and telephone service would be extended underground to the winery from the golf course maintenance facility. Fire hydrants would be provided in conformance with the requirements of the County Fire Marshal's Office.

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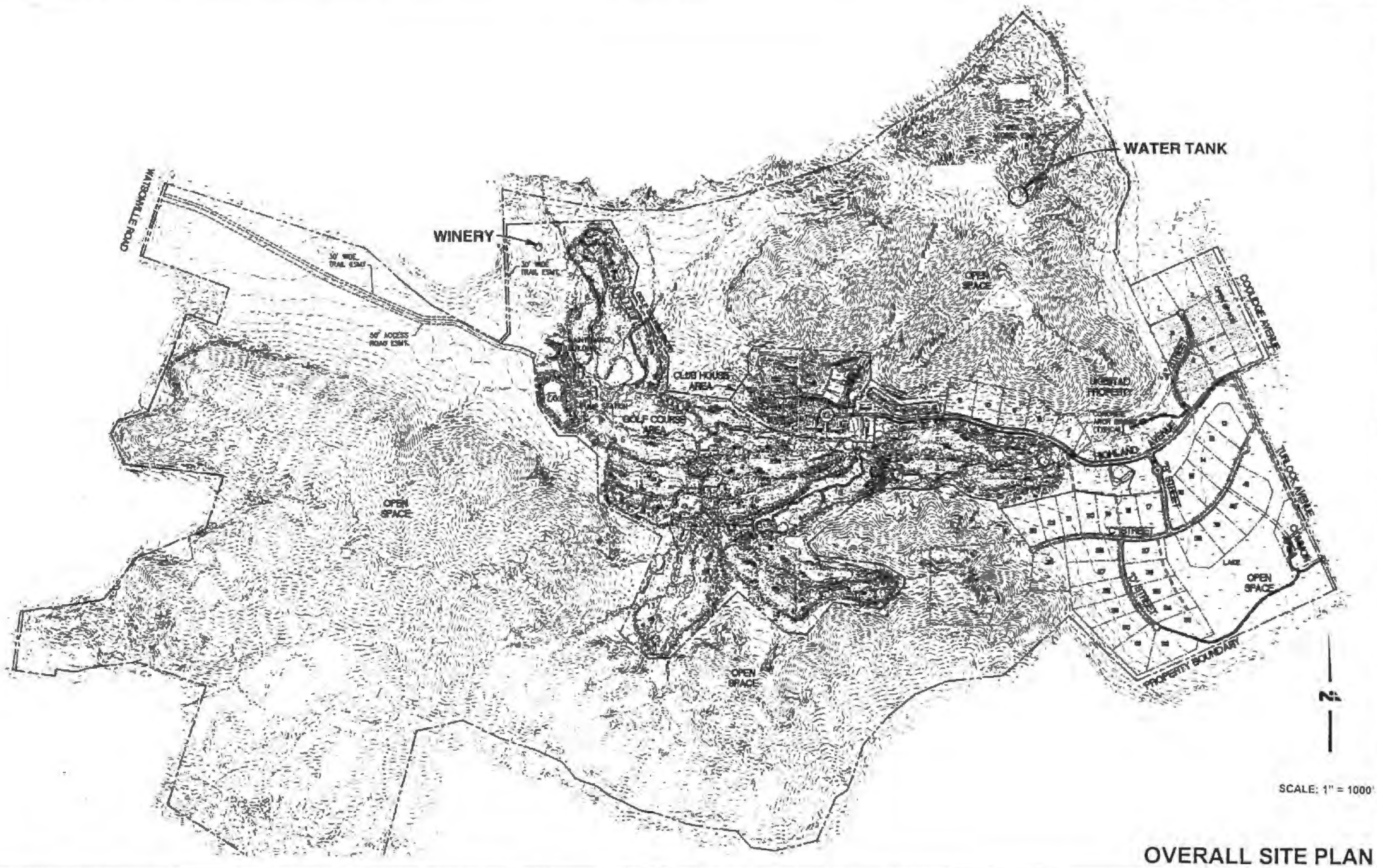
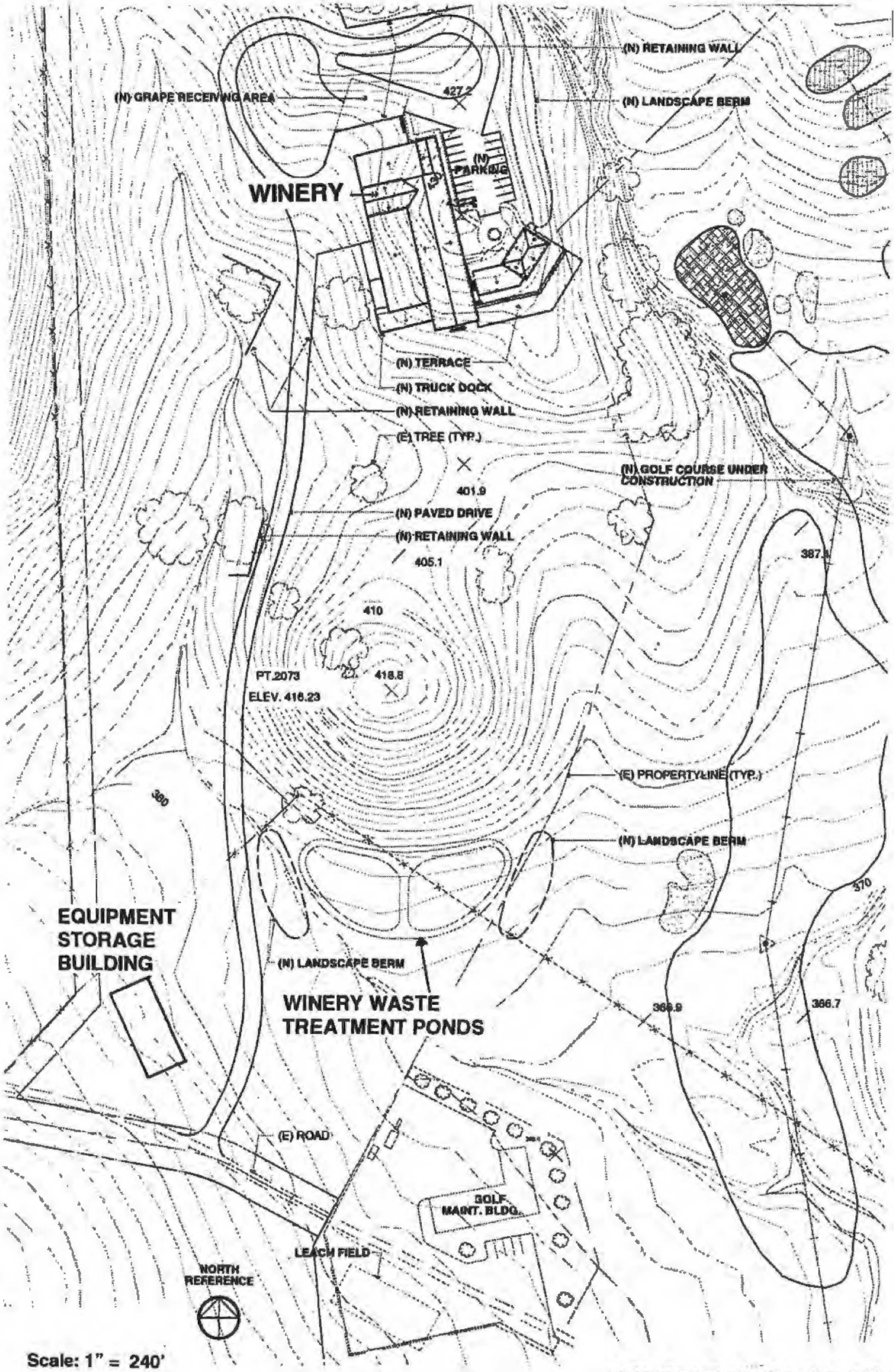


OVERALL SITE PLAN

Source: PACE Engineering

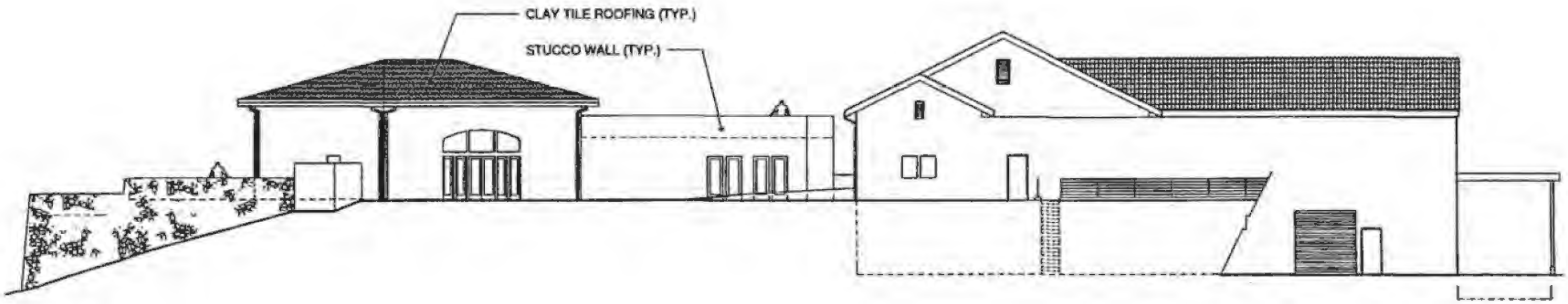
FIGURE 9a



WINERY SITE PLAN

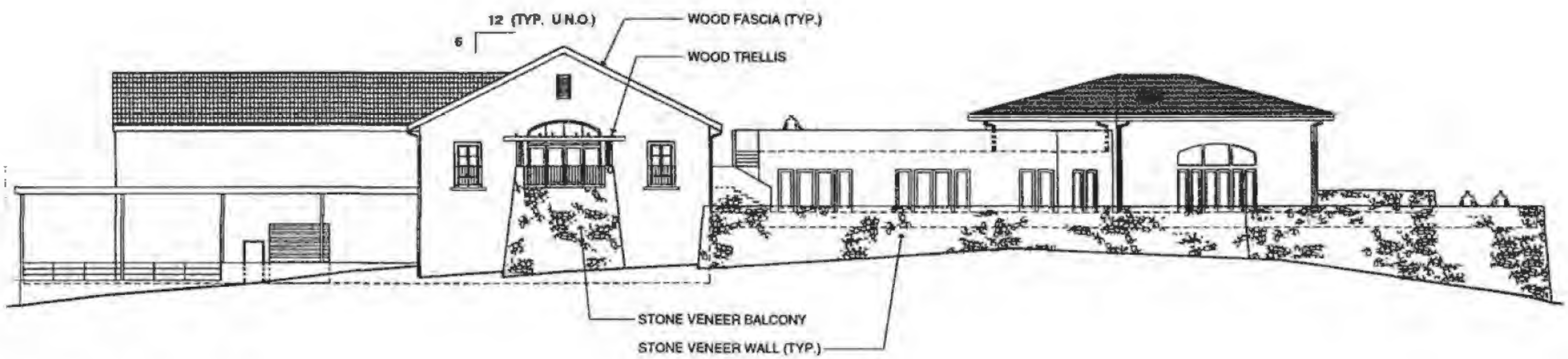
Source: Hall & Bartley Architects

FIGURE 10e



NORTH ELEVATION

1" = 30'-0"



SOUTH ELEVATION

1" = 30'-0"

WINERY ELEVATIONS

Source: Hall & Bartley Architects

FIGURE 10f

Associated Improvements and Programs

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Water Storage Tank

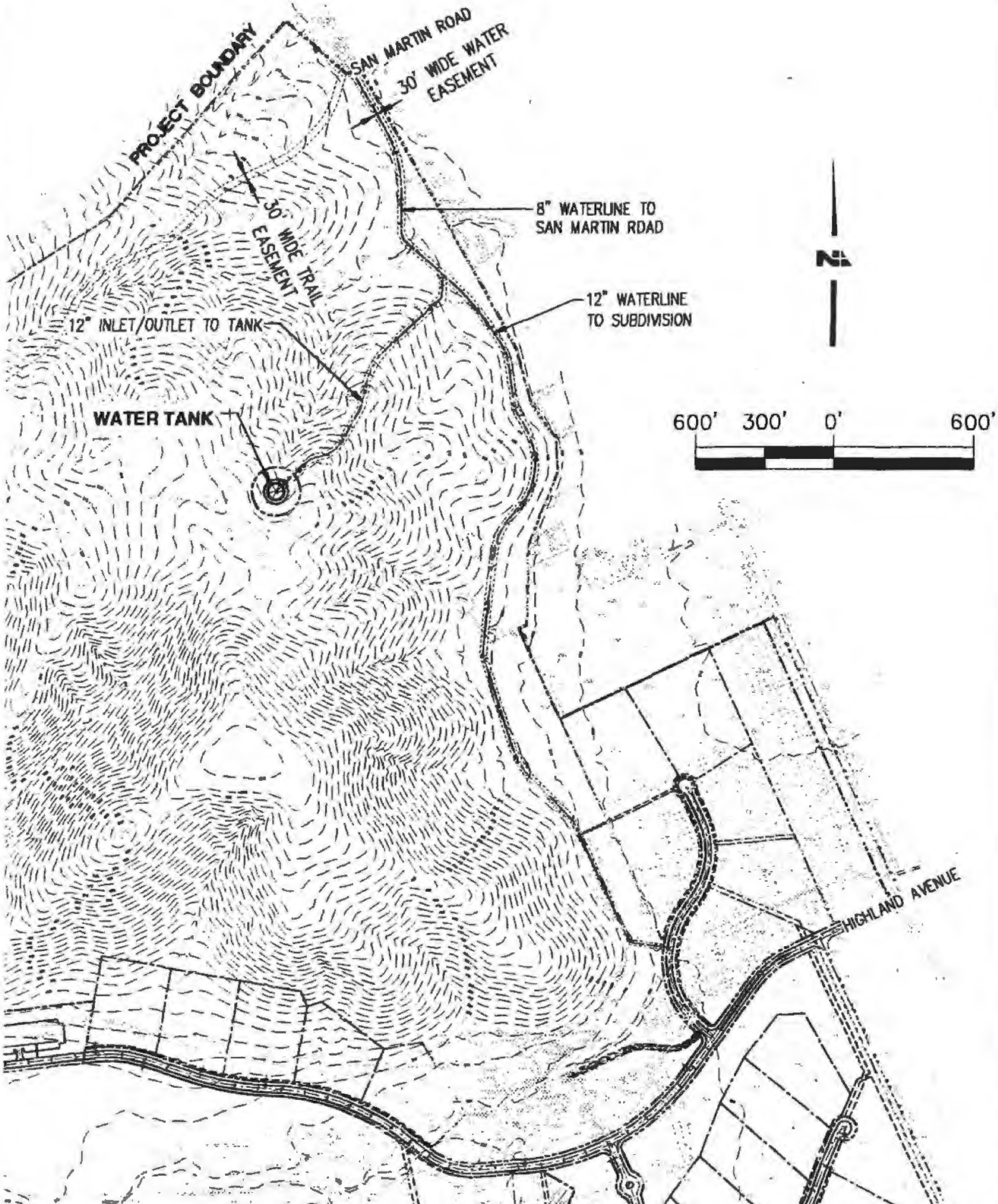
A new 400,000-gallon domestic water tank for the West San Martin Water Works is planned for the upper elevations in the northeast portion of the site (see Figures 9a, 10g and 10h). This tank is needed to provide adequate water pressure and fire flows to the Lion's Gate project, and would also improve fire flows for existing residences east of the project site.

The maintenance access road to the tank would commence from the western extension of San Martin Avenue and would follow an existing dirt track up the hillside to the tank site. The new water main from the tank would be installed in the tank access road to the toe of the eastern hillside where it would split into two mains heading north and south. The southern main would follow the base of the hill to the residential portion of the CordeValle project located north of Highland Avenue. The northern main would follow the maintenance access road to San Martin Avenue where it would tie into an existing water line.

The water tank site is located on an broad eastward sloping swale just below the ridgeline. The tank would be 70 feet in diameter and 34 feet high, with approximately one-half of the overall tank height located below adjacent native ground level. The tank walls would be reinforced concrete supported on a spread foundation, and the tank would have an aluminum domed roof which would be rigidly connected to the tank walls.

Cuts of up to 23 feet would be required to achieve a level pad for the tank. The tank foundation would bear entirely on cut. A french drain would be installed outside the perimeter of the tank to control subsurface drainage.

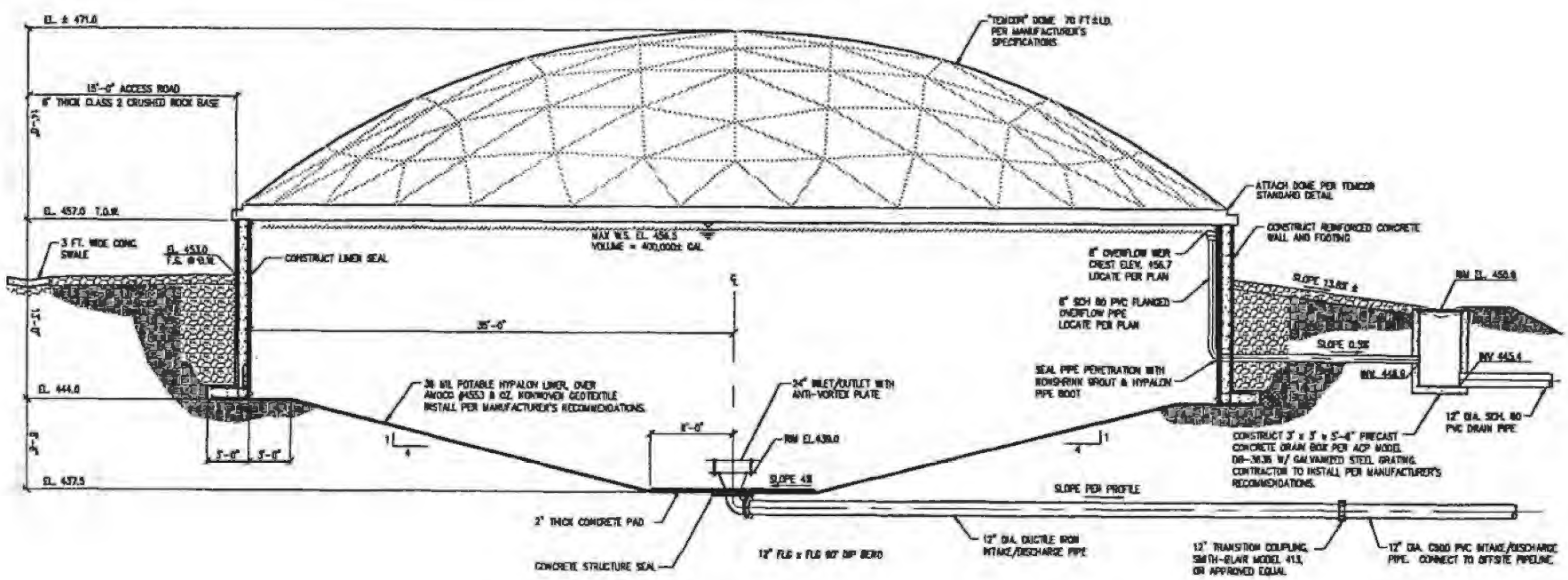
The tank site takes advantage of existing trees to provide visual screening from the valley floor to the east. Additional trees would be planted as needed to increase visual screening. No trees would be removed for the tank, access road, or water main.



WATER TANK AND PIPELINE

FIGURE 10g

Source: PACE Engineering



WATER TANK CROSS SECTION

Source: PACE Engineering

FIGURE 10h

III. ENVIRONMENTAL SETTING, IMPACTS AND MITIGATION MEASURES

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B. AGRICULTURE

Impacts and Mitigation

Mitigation 1. **The loss of approximately 110 acres of prime farmland would be offset by the planting of vineyards in areas not proposed for development.**

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<u>The grapes produced in the on-site vineyards would be processed at the winery planned for the northwest portion of the site. The capability to process the grapes on-site would eliminate the estimated 200 truckloads (by 18-wheel trucks) that would otherwise need to be transported to off-site processing facilities.</u>

D. GEOLOGY AND SOILS

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Impacts and Mitigation

<u>Impact 7</u>. **Expansive soils present on the site may cause movement or heaving, potentially resulting in damage to foundations, concrete pads and pavements. (Potential Significant Impact)**

The majority of the near-surface soil on the site consists of silty or sandy clay, which is moderately to highly expansive. The higher clay content gives the soil the capacity to absorb and release large amounts of moisture with associated volume changes. During the rainy season these soils swell as water is absorbed, and during the dry season they shrink as water is removed by evapotranspiration. Highly expansive soils are evident during the dry season by the formation of open shrinkage cracks on the ground surface.

The expansion (or swell) of soils could exert pressures against foundation elements, and on slopes that could result in creep of the soils. The shrinking of soils could result in consolidation beneath the foundation elements. Structures built on foundations that are not designed for such soil movements can be deformed and damaged.

The north-central area of the site contains colluvial materials which are potentially highly expansive. Any development proposed for this area, such as the maintenance facility, <u>the water storage tank, and the winery/grape processing center</u>, would require special attention during design and construction of building foundations and pavements, but would probably not require site plan modifications.

Mitigation 7. **The potential damage to foundations and pavements would be avoided by following the requirements of the Uniform Building Code, and may necessitate removal of the expansive soils from areas where buildings, slabs-on-grade or pavements are planned to be constructed.**

Site-specific geotechnical studies would be conducted prior to permit approvals to determine if expansive soils are present within the proposed development areas. To mitigate potential foundation problems associated with expansivity of soils, the project geotechnical engineer may recommend that all foundations bear on low expansivity subsoils or bedrock, necessitating the removal of any expansive soils from those areas.. This would result in reduced foundation requirements and lower foundation costs. If removal of expansive soils is not possible, the foundations should be designed to accommodate movements caused by the expansive soils.

At the water tank site, the tank pad would be cut to a depth of about 23 feet, which would remove the majority of the expansive surface soils. Any remaining expansive soils would be removed and replaced with engineered fill as appropriate.

Any locations where ~~the~~ internal access roads traverses expansive soils would require stripping of the expansive soil in the foundation subgrade.

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Impact 10. **Shallow groundwater conditions in areas of the site may adversely affect below-ground structures and utilities. (Potential Significant Impact)**

The relatively shallow groundwater conditions are expected to affect below-ground structures including basements and utilities located at depths of greater than 10 feet below original ground surface in spring areas and in the valley floor. Excavation for stormwater retention basins or ponds, requiring cuts greater than a depth of 10 feet, may encounter groundwater.

Since the water storage tank site is near the top of a broad swale, it is expected that some shallow groundwater may occur near the tank pad elevation. However, the amount of groundwater is anticipated to be relatively small and the potential pore pressure would not be great.

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Mitigation 10. **Groundwater problems would be minimized by avoiding subsurface construction during or just after the rainy season, and through implementation of grading and drainage measures to improve surface and subsurface drainage.**

The grading and drainage plan would include provisions for improving surface and subsurface drainage to alleviate the seasonal groundwater problem.

At the water storage tank site, shallow groundwater conditions would be adequately addressed by installing a french drain on the outside of the tank wall foundation.

J. VISUAL AND AESTHETICS

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Impacts and Mitigation

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<u>Impact 1</u>. **The project would result in visual changes to some areas of the site open to public view. (Potential Significant Impact)**

As discussed under 'Environmental Setting' above, the most visually accessible areas of the site are located along Coolidge Avenue (Santa Teresa Boulevard) and Turlock Avenue at the eastern end of the site, and along Watsonville Road to the west. The interior valley area of the site is not visible from off-site vantage points except for the single home that overlooks the site from the northern ridge. The hillside areas nearest to the flanking roadways are also visible.

The residential subdivisions proposed for the eastern end of the site would be partially visible from adjacent land uses and roadways. In the Rural Residential subdivision proposed adjacent to Coolidge Avenue, north of Highland, the 6 proposed lots would be set back from the roadway at least 300 feet toward the adjacent hillside to the west. The setback area would remain as permanent open space, with a landscaped berm providing visual screening for these lots. A stormwater detention basin would occupy the open space area between the roadside berm and the residential lots; however, the basin would be entirely screened from the roadway by the intervening landscaped berm.

The residential cluster subdivision proposed for the field west of Turlock Avenue would also be partially visible to passing motorists. However, this subdivision would be set back 200 feet to 1,400 feet from the roadway, and would be screened by the landscaped berms planted with black walnut trees. Nevertheless, the roof lines of the nearest dwellings would be visible from Turlock Avenue and Santa Teresa Boulevard, at least until the black walnuts have matured enough to provide more complete screening (see Figure 16). Since two of the proposed lots (Lots 24 and 25) extend into the adjacent hillside area, it is possible that future custom homes to be built on these lots may be visible from Turlock Avenue and Santa Teresa Boulevard.

<u>The water storage tank planned for the northeastern hillside area of the site may be partially visible in the distance from the valley floor to the east and from two or three residences in the Hayes Valley Ranch project to the north and west. The visibility of the tank would be minimized by its location in a broad swale just below the ridgeline. Approximately one-half of the tank would be buried so only the upper portion would extend above ground elevation. The tank site also takes advantage of existing trees downslope to the east for visual screening, and additional trees would be planted as needed to increase visual screening.</u>

The small horse stable planned for the northwest corner of the site would be sited in a small side valley along the toe of the eastern hillsides. The nearest existing land uses include a nursery business located approximately 500 feet east and two single-family dwellings located approximately 800 feet to the northeast and the southeast. The existing nursery with its dense boundary landscaping almost completely screens the stable from view of Coolidge Avenue and the residences in the vicinity.

The package wastewater treatment plant and residential lake occupy the area between the roadside berm and the residential subdivision. However, these project components would be low in profile and almost completely shielded from view by the landscaped berm along Turlock Avenue.

The only other visual changes that would occur at the eastern end of the site would be the roadway improvements and entry features along the Highland Avenue entry way. However, any improvements would be subject to Architecture and Site Approval to ensure that signs, fences, lighting and other features would be compatible with their surroundings. Also, the existing mature landscaping trees around the ranch complex would be retained and incorporated into the project.

From Watsonville Road to the west, very little of the project, if anything, would be visible. All of the area with ¾ mile of the roadway is proposed to be maintained as permanent open space. The golf course would be located to the east of the low saddle that crosses the western portion of the valley, and thus would not be visible from Watsonville Road. It is possible that the maintenance facility proposed for the western end of the golf course may be partially visible from Watsonville Road, ¾ mile to the west. The only evidence of the project alongside Watsonville Road would be the new maintenance access road to be constructed from Watsonville Road to the golf course maintenance facility. There would be no structural entry features such as signage here since no public access to the golf course would be permitted from this direction.

In the interior area of the valley, the golf course, clubhouse and overnight units would not be visible from off-site vantage points, even from the single dwelling that overlooks the valley from the adjacent ridge to the north. From the vantage point of this residence, the clubhouse/overnight complex would be completely blocked by the intervening low hills and ridges just north of the complex. <u>However, the winery complex would be visible from the residence, although it would be at least 2,000 feet away.</u>

Mitigation 1. **The project would be designed and landscaped in a manner to help it blend in with the natural and rural surroundings, and to reduce its visibility from off-site locations.**

The site planning measures proposed as part of the project, including buffer zones from all adjacent roadways, as well as the proposed landscaping and berming, would minimize the potential visual effects of the project. The design of the residential areas reflects many of the guidelines of the San Martin Integrated Design Plan (see Section *II. Consistency with Plans, Policies and Regulations.)*

All structural elements such as signs, fences, lighting or other entry features would be subject to Architectural and Site Approval to ensure their compatibility with the surroundings. In addition, any structures proposed within 100 feet of adjacent scenic roads would be subject to the County's Design Guidelines.

K. TRAFFIC AND CIRCULATION

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Impacts and Mitigation

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<u>Impact 1</u>. **The project would result in increased traffic generation at the project site. (Potential Significant Impact)**

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<u>The winery would generate a small volume of traffic which would primarily consist of daily trips by 8 permanent employees and 6 additional temporary employees during the harvest and crush season. There would also be a small number of trips (an average of 5 per week) generated by guests, who would visit the winery by appointment only. Truck trips generated would include approximately 40 truck loads of finished product, which would occur periodically throughout the year, and occasional trips by delivery and service vehicles. Since all vehicles would access the winery site from Watsonville Road, they would not contribute to traffic on roadways east of the CordeValle site. The small increment of traffic from the winery would not significantly affect traffic operations on Watsonville Road.</u>

L. NOISE

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Impacts and Mitigation

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<u>Impact 5</u>. **Noise levels would be temporarily elevated during grading and construction. (Potential Significant Impact)**

Most of the existing noise receptors in the area are far from the main grading and construction area of the golf course. The major exception is the existing ranch house at the east end of the site. During construction, maximum noise levels generated by grading, paving, and other activities would be 5 to 10 decibels lower. If average levels do not exceed 55 dBA, there would be no interference with outdoor activity or indoor activity, although the construction may be occasionally audible. Noise levels at the existing ranch could reach as high as 80 dBA with average levels of up to 75 dBA. During most of construction, however, noise levels would be significantly below 55 dBA.

The existing residence on the ridge to the north of the project site would be approximately 1,200 feet from the nearest grading activity for the golf course. At this distance, the sound of equipment would be noticeable but would not exceed 55 dBA.

At the northeastern corner of the site, existing dwellings along and near the western extension of San Martin Avenue (west of Coolidge Avenue) would be subject to short-term noise from the grading and construction of the water tank access road commencing southwestward from the end of San Martin Avenue. These residences would also be subject to temporary noise from truck traffic generated during the construction of the water tank.

At the eastern end of the project site, existing dwellings in the vicinity would be subject to short-term grading and construction noise impacts from construction of the perimeter berms, the detention basin along Coolidge Avenue, the package wastewater treatment plant and lake/detention basin along Turlock Avenue, and to a lesser extent the proposed residential subdivisions which would be set back from the site boundary.

At the western end of the site, the construction of the maintenance access road to Watsonville Road would generate noise from grading and paving. The nearest existing dwelling would be 700 feet from this maintenance road at its nearest point, and would not be subject to construction noise impacts, although the noise would be audible.

Mitigation 5. **Short-term construction noise impacts would be reduced through compliance with the County's Noise Ordinance with respect to hours of operation and maximum noise levels at adjacent property lines. At the eastern edge of the project, the berms proposed along the project boundary would be constructed during the early phases of grading to provide a noise barrier for existing residences nearby.**

The Noise Ordinance stipulates that construction noise generated between 7 am and 7 pm on weekdays and Saturdays should reach noise levels no greater than 75 dBA at an adjoining property line of a single-family or two-family dwelling.

These hours would be enforced by the grading inspector, and also the County Department of Environmental Health in the event of a violation of the County Noise Ordinance.

To minimize noise generation, construction equipment should be maintained in good operating condition and properly muffled.

To further reduce construction noise impacts, the berms proposed for the eastern project boundaries would be constructed during the early phases of grading in order to provide shielding from construction and grading in the interior of the project. This would be particularly effective in attenuating noise from grading and excavation for the detention basin along Coolidge Avenue, and the package wastewater treatment plant and lake/detention basin along Turlock Avenue.

P. WATER SUPPLY

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Impacts and Mitigations

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Impact 1. **The proposed project would increase the demand for water at the site. (Potential Significant Impact)**

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Maintenance Facility: It is estimated that the maintenance facility would use 225 gpd for domestic use, based on 15 employees at 15 gpd per employee. The washdown estimates are provided below.

Winery/Grape Processing Center: Maximum domestic water used at the winery would be based on the maximum number of employees (14) at 15 gpd per employee, for a daily consumption of 210 gpd. In addition, an average of approximately 2,000 gpd of non-potable water would be used for washing down the vats and equipment.

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Mitigation 1a. Increased water supplies to meet project demand for domestic water would be provided by the West San Martin Water Works, without adversely affecting existing or future users.

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The project includes a 400,000-gallon water tank to be constructed in the northeast portion of the project approximately 4,000 feet northwest of the Coolidge Avenue/Highland Avenue intersection. ~~In the near future, the water company plans to construct a new 300,000 gallon water tank at an existing tank site on Hayes Lane, approximately ¾ mile north of the proposed clubhouse.~~ This tank is being constructed to improve existing low pressure problems in the system, to enhance fire protection capability, and to provide for projected future growth in the San Martin area. With the completion of this tank, the water company would have sufficient capacity to meet the estimated water demands and fire flow requirements for the Lion's Gate project.

Q. WASTEWATER TREATMENT AND DISPOSAL

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Impacts and Mitigations

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Mitigation 1. **Increased wastewater from the project would be treated and disposed of with new facilities to be constructed in conjunction with the project.**

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Maintenance Facility

The maintenance facility would not be connected to the centralized wastewater system, but would have its own septic tank and leachfield system. Based on a generation rate of 15 gpd for 15 employees, maximum flows would be 225 gpd. Preliminary soils and groundwater studies indicate that there is adequate depth to groundwater, and that the soils in the vicinity have acceptable percolation rates for the planned leachfield.

Winery/Grape Processing Center

The winery is also planned to have its own septic tank and leachfield system. Based on a generation rate of 15 gpd for a maximum of 14 employees during the harvest and crush, maximum flows would be 210 gpd. Wet weather percolation tests have not yet been conducted to determine the suitability of the soils at the winery site for leachfields (these tests are planned to be conducted in the winter of 1999). In the event the soils are found unsuitable, the alternative plan is to pipe the domestic effluent to the nearby golf course maintenance facility septic system for treatment and disposal. The siting and design of the septic tank and leachfield system for the winery would be subject to the criteria and standards of the Santa Clara County Department of Environmental Health.

Washdown water from cleaning the vats and equipment would be piped to two small treatment ponds occupying a 0.5-acre area south of the winery building. This washdown water would be screened for organic material before being piped to the ponds. The washwater would include some residue from cleaning detergents and minor amounts of chemicals used in the wine making process. The treatment ponds would include aerators to prevent stagnation and odor generation, which would also prevent mosquito breeding. Some of the treated washwater would evaporate from the pond, and the remainder would be used for irrigation or frost protection in the vineyards. The organic material screened from the washwater would be applied to the vineyards or used in making mulch for soil amendment. The design and installation of the wastewater treatment ponds would be subject to the approval of the Central Coast Regional Water Quality Control Board.

APPENDICES

APPENDIX C

Geotechnical Reports

Prepared by

Twining Laboratories

August and October 1998

THE TWINING LABORATORIES, INC.
EST. 1898
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October 7, 1998

Lion's Gate Limited Partnership
395 Oyster Point Boulevard, Suite 309
South San Francisco, California 94080

Attention: Mr. Ron Davis

Subject: Preliminary Evaluations for
Geotechnical and Geological Feasibility:
Proposed Potable Water Storage Tank
and Proposed Winery Buildings
Cordevalle Estates
San Martin, California

Dear Mr. Ron Davis:

This letter report addresses the geotechnical feasibility of the proposed water storage tank and the winery buildings to be located at the Cordevalle Estates. The proposed water tank is to be located on an eastward sloping swale, about 4,000 feet northwest of the intersection of Highland and Turlock Avenues, and about one-half mile north of the golf course. The winery is to be located northwest of the northwest portion of the golf course on gently rolling terrain.

The Twining Laboratories (Twining) prepared a Geotechnical Engineering Investigation report for the proposed water storage tank, which included test trenching, soil sampling, and laboratory testing of soils. Two test borings have been completed at the site of the proposed winery buildings, however, a complete geotechnical engineering investigation has not been performed. We understand that additional test borings, soil sampling and associated laboratory testing are proposed for the winery to support a design level geotechnical engineering report for that site.

PURPOSE AND SCOPE

This letter report is provided to facilitate evaluation by Santa Clara County with respect to the geotechnical and geological feasibility of the two sites. The report provides our preliminary evaluation of the geotechnical and geological feasibility of the sites.

CORPORATE OFFICE 2527 Fresno Street Fresno, CA 93721 (209) 268-7021 • Fax 268-7126
MODESTO 4230 Kiernan Ave., #105 Modesto, CA 95256 (209) 545-1050 • Fax 545-1147
VISALIA 130 North Kelsey St., #H6 Visalia, CA 93291 (209) 651-8280 • Fax 651-8288
BAKERSFIELD 3701 Pegasus Drive, #124 Bakersfield, CA 93308 (805) 390-5088 • Fax 393-4643
SALINAS 520 #A Crazy Horse Canyon Rd. Salinas, CA 93907 (408) 449-5284 • Fax 449-5092

The following tasks were performed in support of our evaluation:

I. The following previous geologic investigation reports, prepared by others, were reviewed:

Supplemental Geological Reconnaissance Investigation for Proposed Hayes Valley Dams, Santa Clara County, California, prepared by Kaldveer Associates Geoscience Consultants, August 4, 1989.

Geologic Input to Draft Environmental Impacted Report, Lions Gate Development, project HRC-101B, prepared by Wahler Associates for HR Development Partners. April 17, 1990.

Geologic Input to EIR, prepared by ENGEO Incorporated, April 13, 1993.

Geologic Feasibility Investigation, Golf Course Maintenance Building, The Lion's Gate Reserve, San Martin , California, Project 1385/6G, prepared for Hayes Valley Development Partners, by Pacific Geotechnical Engineering, December 1995.

Geologic Feasibility Investigation, Clubhouse and Overnight Lodges, The Lion's Gate Reserve, San Martin, California, Project 1385/5G, prepared for Hayes Valley Development Partners, by Pacific Geotechnical Engineering, December 1995.

Administrative Draft Environmental Impact Report, Volume IIa Technical Appendices, Lion's Gate Reserve, December 1995.

Draft Environmental Impact Report, Volume II Technical Appendices B through E, Lion's Gate Reserve, March 1996.

II. The following geologic and geotechnical reports prepared by The Twining Laboratories were reviewed:

Report entitled Preliminary Geotechnical Engineering Investigation, Golf Coarse, dated March 18, 1997, and Addendums No. 1 and No. 2.

Letter report entitled "Review of Site Geologic Conditions and Grading Plans, Golf Course Phase", dated May 6, 1997

Report entitled Preliminary Geotechnical Engineering Investigation, Clubhouse and Overnight Lodges, dated October 30, 1997.

Preliminary Evaluation of Geotechnical and Geological Feasibility: Clubhouse and Overnight Lodges Area, dated April 16, 1998.

Geotechnical Engineering Investigation, Maintenance Building, Cordevalle Golf Club and Hotel, San Martin, California, dated July 8, 1998.

Geotechnical Engineering Investigation, Proposed Potable Water Tank, dated August 11, 1998.

Geotechnical Engineering Investigation, Proposed Cart Path, Pedestrian, and Utility Bridges, Cordevalle Estates, San Martin, California, dated September 25, 1998.

This report is provided specifically for the water storage tank and winery buildings at the proposed Cordevalle Estates, referenced in the Proposed Construction section of this report.

This investigation did not include design level geotechnical engineering investigation, floodplain investigation, agricultural compatibility assessment, compaction tests, environmental investigation, or environmental audit. This investigation was intended only to evaluate the static physical characteristics of the soils and rock at the project sites.

BACKGROUND

Site Descriptions

Water Storage Tank Site: The potable water tank site is located on an eastward sloping swale, about 4,000 feet northwest of the intersection of Highland and Turlock Avenues, and about one-half mile north of the golf course. The swale slopes at about 4 horizontal (H) to 1 vertical (V). The west edge of the proposed tank is approximately 125 feet downslope from the top of a broad ridgeline. Oak trees are present on the hillside near the near the proposed tank site. Dry brown grasses of up to 3 feet high covered the surface soils at the time of our field investigation.

According to a geologic map of the site region prepared by Kaldveer Associates (scale: 1 inch = 500 feet, 1989) for the proposed Hayes Valley Dam, the tank is located on Franciscan Complex greenstone. A serpentinite belt is located approximately 500 feet west of the proposed tank site. The nearest mapped active or potentially active fault is the Sargent-Berrocal Fault, located about 2.5 miles east of the site.

Winery Buildings Site: The winery site is located west of the northwestern most portion of the golf course, approximately 300 feet west of the number 6 green. The winery buildings are to be located near the axis of a gently sloping, north-south trending, ridge line. Slope gradients at this location range from nearly flat at the top of the ridge to a maximum slope of about 5 horizontal to 1 vertical. Dry native grasses and scattered oak trees were observed during our site reconnaissance. A pre-engineered building to be used for vineyard agricultural purposes will be located about 400 feet southwest of the winery buildings.

Anticipated Construction

Water Storage Tank: We understand the proposed potable water tank will include a 70-foot diameter reinforced concrete walled tank with an approximate capacity of 420,000 gallons. Approximately two-thirds of the tank height will be below the adjacent native grade level. The tank is proposed to have an aluminum "TEMCOR" domed roof and a 36 mil Hypalon liner covering the sides and bottom of the tank. An 8 ounce nonwoven geotextile is proposed to be placed below the bottom portion of the tank liner. The bottom surface of the tank will be sloped toward the center at a 4H to 1V gradient. The reinforced concrete walls will be supported on a 3-foot wide perimeter spread foundation. A french drain will be installed outside the entire perimeter of the reinforced concrete tank wall. A perimeter access road with a Class II aggregate base surface will be constructed around the tank.

Cuts of up to about 23 feet are proposed to achieve a level pad for the tank. The tank foundation is proposed to bear entirely on cut. Fills of about 2 to 5 feet are proposed along the downslope perimeter on the pad, beneath the perimeter access road.

Winery Buildings: We understand that the winery will comprise an approximate 20,000 square foot, wood-frame, main winery building, and a pre-engineered building to be used for vineyard agricultural purposes. Anticipated grading would include cuts and fills of up to about 5 to 8 feet.

General Geologic Conditions

The earth materials underlying the proposed water storage tank and winery sites are composed of rocks belonging to the Franciscan Complex of Jurassic to Cretaceous age. Bedrock types found within the Hayes Valley area include sandstone, shale, chert, limestone, greenstone, and low grade metamorphic rocks. Many areas of bedrock terrain include a mixture of different rock types in a sheared matrix. This formational mixture is termed a melange and was formed as a result of intense shearing and faulting. Serpentine is also found within this assemblage of rocks.

The regional trend of geologic structures in the Hayes Valley area is roughly east-west, acute to the overall geologic structure of North 40 degrees East for the Santa Cruz Mountains as a whole. Physiographic features, bedrock contacts, and faults are generally parallel to this structural trend.

The proposed water storage tank and winery buildings are located approximately 7 to 8 miles northeast of the San Andreas Fault and 6 miles southwest of the Calaveras Fault. Other active faults in the site region include the Hayward and Sargent-Berrocal faults. Regional geologic maps prepared by U.S. Geological Survey and the California Division of Mines and Geology show a bedrock fault and bedrock contacts within the melange terrain on the north side of Hayes Valley. The faults and contacts are also shown on the Geologic Index Map (Figure 1), of the Geologic Feasibility Investigation for the Clubhouse and Overnight Lodges, prepared by Pacific Geotechnical Engineering, dated December 1995.

Soil Conditions in the Site Areas

Soil conditions in the areas of the proposed water tank and winery building were revealed in test borings conducted by Twining during July and August, 1998, respectively.

Water Storage Tank Site: Near surface soils comprise silty sands at the water storage tank site. The sands extend from the ground surface to depths of approximately 1 to 2 feet below site grade (BSG). The root systems of grasses and weeds extended to depths of about 18 to 24 inches. Sandy and gravelly lean clays were present beneath the silty sands. Weathered greenstone bedrock was encountered in test pits and a test boring at depths of 5 to 7 feet BSG, extending to the maximum depths explored (41.5 feet BSG).

Winery Buildings Site: Silty sands with gravel are present at the proposed winery buildings site to depths of about 0.5 to 2.5 feet BSG. Highly weathered greenstone bedrock was encountered below the silty sand in both soil borings drilled, to the maximum depths of exploration of 6.5 and 10.5 feet BSG.

Review of Previous Geologic Investigation Reports

We have reviewed the geologic reports listed under "Purpose and Scope". Most of the cited reports present descriptions of regional geologic and tectonic conditions, and general site geologic conditions. Our summary of these regional conditions are presented above under the "Background" section of this report. Geologic conditions applicable to the subject sites, which are described in these reports, and conditions noted during our geologic field reconnaissance of the site areas are summarized in the "Evaluation" section of this report.

Geologic Field Reconnaissance

A geologic field reconnaissance of the proposed water storage tank and winery buildings areas was performed in conjunction with our review of the proposed golf course and surrounding areas performed on April 28, 1997. The reconnaissance, which included confirming previously mapped geologic features and noting potential geologic hazards, was performed by Kenneth J. Clark, Certified Engineering Geologist. The results of the geologic field reconnaissance suggest that the geologic map prepared by ENGEO is generally suitable for planning purposes for the proposed water storage tank and winery buildings project. This map is included as Figure No. 2 of the report entitled "Geologic Input for EIR For Lion's Gate Property", dated April 13, 1993 (contained in the Draft Environmental Report [DEIR]). However, we do not warrant the accuracy of the aforementioned map. Additional reconnaissance of the water storage tank and winery buildings sites was performed by Mr. Kenneth Clark on July 7, 1998, and May 5, 1998, respectively.

EVALUATION

This section presents our evaluation of potential geotechnical and geologic concerns pertinent to the water storage tank and winery buildings area, and a discussion of potential measures to mitigate the adverse conditions.

Soil and Rock Conditions

Water Storage Tank Site: The predominant soil types at the water storage tank site are silty sands and sandy and gravelly lean clays. The soils overlie weathered greenstone bedrock at depths of 5 to 7 feet BSG. The sandy soils, to depths of 1 to 2 feet BSG were generally loose. The clayey soils are anticipated to have a medium expansion potential, moderate compressibility, and the potential for moderate to high swell. However, we anticipate that the pad will be cut to a maximum depth of about 23 feet to achieve the designed tank bottom surface. This excavation would remove the majority of the loose near-surface silty sands and clayey soils. Along the perimeter of the pad (where fill is to be placed) care should be taken to remove the loose silty sand soils to a minimum depth of 1 foot BSG prior to placement of the fill. Field and laboratory data suggest that weathered greenstone rock will provide an adequate foundation material to support the water storage tank.

Based on our observations of the weathered rock in test pits, temporary cut slopes into the rock material will likely be stable up to gradients of about 3/4H to 1V. Temporary cut slopes in lean clay or silty sand soils will likely be stable to about 1H to 1V. If sloughing of the cut slope occurs, the temporary excavations should be shored or slopes flattened.

Winery Buildings Site: Soil borings indicate the winery buildings site includes silty sand with gravel underlain by highly weathered greenstone bedrock at a depth of about 0.5 to 2.5 feet BSG. The silty sands are generally loose. The loose silty sand soils will not adequately support fills, foundations, or floor slabs. These soils should be removed prior to placement of engineered fill, floor slabs, or shallow footings. Slabs and foundations should bear either entirely on engineered fill or entirely on firm native weathered bedrock.

Although not noted in the soil borings, lean clay soils (prevalent at the Cordevalle Estates project site) may be encountered during further investigation and/or grading for the winery buildings. Over time near surface clays will experience cyclic drying and wetting as the dry and wet seasons pass. Clays soils are anticipated to experience volumetric changes (shrink/swell) as the moisture content of the clay soils fluctuate. These shrink/swell cycles can impact foundations and lightly loaded slabs-on-grade even though the expansion potential is classified as medium. Expansive soils cause more damage to structures, particularly light buildings and pavements, than any other natural hazard, including earthquakes and floods (Jones and Holtz, 1973). Expansion potential may not manifest itself until months or years after construction. At most sites there exists a depth to which the moisture content of the subgrade remains essentially constant throughout the year; thus, the clays would not undergo a significant volume change below this depth. Therefore, the depth, referred to as the "critical depth", to which significant moisture fluctuation occurs influences the selection of suitable foundation and floor slab alternatives for this site. Climatic conditions, groundwater conditions, landscape irrigation, and the soil conditions effect the critical depth. Our review of moisture data and observations of near surface clay soils did not clearly demonstrate a critical zone depth. Based on experience, it is expected that the critical zone would be approximately 24 inches BSG in the site region, and that seasonal moisture fluctuation would effect soils to a depth of 2 feet BSG. The above estimate of the critical depth should be reevaluated based on soil sample test data to be generated for the proposed geotechnical and geological investigation.

Potentially expansive clayey soils may be present near the proposed locations of floor slabs or lightly loaded foundations at the winery buildings site. If clay soils are present, footings should be extended to bear at the bottom of the critical zone, at least 24 inches BSG. Over-excavation and backfilling with non-expansive engineered fill soils my be required below floor slabs. Based on soils data generated for other sites within the Cordevalle project, we anticipate that 12 to 24 inches of nonexpansive granular soil would be required between floor slabs and clayey soils. Recommendations for footings and over-excavation and placement of non-expansive engineered fill should be provided with the report of Geotechnical Engineering Investigation of the winery buildings site

Faulting

The water storage tank and winery buildings sites are located in a seismically active region with numerous active and potentially active faults. The nearest mapped active or potentially active fault is the Sargent-Berrocal Fault, located 2 to 3 miles east of the site. Several bedrock faults associated with melange terrace have been mapped by others on the Cordevalle development site. Our field reconnaissance and review of the aforementioned geologic reports, prepared by others, do not indicate the presence of faults in the immediate areas of the proposed water storage tank and winery buildings. Additionally, our review of data presented in geologic reports previously generated for the development project indicates that the bedrock faults in the site area are inactive.

The subject sites are not located in an area containing any of the State of California Earthquake Fault Zones (formerly Alquist-Priolo Special Studies Zones), established to delineate earthquake fault zones.

Considering the presence of mapped bedrock faults (inactive) in the vicinity of subject sites, it is possible that site grading for the projects may reveal shear zones or faults in the bedrock. It is generally not recommended to build a structure across a fault, active or inactive. Although the mapped faults are judged to be inactive, geologic evaluation should be conducted during grading operations. Exposed bedrock should be observed by an engineering geologist to assess the presence or absence of faults. Structures built across faults may be supported on soil or rock materials with highly variable foundation properties, and excessive differential settlement can result. Potential differential settlement may be reduced by over-excavation and placement of engineered fill over the fault, or modifying the location of the structure away from the fault.

Seismic Ground Motion

Seismic ground motion may occur at the site as a result of earthquakes on nearby active faults. The intensity of ground shaking depends on factors such as earthquake magnitude, distance to causative fault, depth to bedrock, physical characteristics of underlying soil and bedrock, and local topography. Terratech (1988) indicated that ground motions were likely to exceed 0.5 g.

Our deterministic evaluation of the potential magnitude of seismic ground motion indicates that the upper bounds earthquake event would likely produce a peak horizontal ground acceleration at the site in the range of 0.4g to 0.5g.

Native Slope Stability

Landslides on the proposed development site were mapped by others (Kaldveer Associates, 1989, and Wahler Associates, 1990). The locations of these landslides are shown on Figure No. 2 of the report entitled "Geologic Input for the Lion's Gate Property" (DEIR Volume II) which is a compilation of site data generated prior to April 1993. Previously mapped landslides were observed during our geologic field reconnaissance near the two subject sites. These slides appeared to comprise relatively shallow rotational block slides and slumps.

The aforementioned mapping studies do not indicate landslides have occurred in the immediate vicinity of the water storage tank or winery buildings sites. Field reconnaissance performed by Twining did not indicate the occurrence of notable landslides near the subject sites. Native slopes in the areas proposed for the water storage tank and winery buildings appear relatively stable.

Shallow Groundwater

Considering the proposed water storage tank location near the top of a broad swale, we anticipate that some shallow groundwater may occur near the tank pad elevation. However, the amount of groundwater is anticipated to be relatively small and it appears that potential pore pressure and nuisance conditions could be adequately addressed by installing a french drain proposed on the outside of the tank wall foundation. A french drain outside the tank wall foundation is included on the civil engineering plans for the project.

Groundwater was not encountered in exploratory borings drilled (August, 1998) at the winery buildings site. Considering the elevated topographic location of the proposed building sites, we anticipate that shallow groundwater will not have an adverse impact on the winery buildings project.

Subsequent to rough grading of the water tank and winery buildings sites, slope, soil, and rock conditions should be reviewed by Twining's civil engineer or engineering geologist for evidence of subsurface groundwater flow. Conditions favoring seeps include relatively shallow bedrock (or other impermeable layer) with an overlying permeable soil. Soil textures exhibiting a selective removal of fine particles from currently dry soils may indicate subsurface groundwater flow during wetter periods. Erosion may be accelerated and slope stability compromised where groundwater daylights (seeps) on cut slopes.

The following mitigation methods may be employed where shallow groundwater impinges on:

- Road subgrades: Trenched cut-off walls and subdrains
- Native slopes: Upslope trench cut-off wall or horizontal wick drains
- Cut slopes: Retaining wall with filter drain and weep holes

Mitigative measures should be designed by Twining's civil engineer or engineering geologist for specific areas, when adverse shallow groundwater conditions are identified.

Liquefaction and Seismic Settlement

Liquefaction describes a phenomenon in which a saturated, cohesionless soil loses strength during an earthquake as a result of induced shearing strains. Lateral and vertical movement of the soil mass, combined with loss of bearing usually results. Liquefaction can cause damage to structures during earthquake events. Foundations can literally loose support due to bearing capacity failure. The resulting displacements can induce excessive differential settlements in floor slabs and foundations. Research has shown that liquefaction potential of soil deposits induced by earthquake activity depends on soil types, void ratio, groundwater conditions, duration of shaking, and confining pressure over the potentially liquefiable soil mass. Fine, well sorted, loose sand, high groundwater conditions, higher intensity earthquakes, and particularly long duration of groundshaking are the requisite conditions for liquefaction.

Based on the anticipated shallow bedrock and paucity of well sorted loose sandy soils, as suggested by Twining's previous investigations, the water storage tank and winery buildings sites do not appear to be susceptible to liquefaction.

Seismic settlement occurs when loose, poorly graded, granular soils consolidate as a result of cyclic ground shaking associated with an earthquake. Based on the anticipated shallow bedrock and lack of well sorted loose sandy soils, the water storage tank and winery buildings sites do not appear to be susceptible to seismic settlement.

Serpentine Rock

The proposed water storage tank and winery buildings sites are not located near mapped exposures of serpentinite. Accordingly, we do not anticipate that grading operations would reveal asbestos bearing serpentinite materials. However, naturally occurring asbestos materials may be associated with serpentine rock which has been documented by previous investigators at other locations on the development property. Serpentine rock is typically a green or yellow, highly sheared and altered rock, with a fibrous appearance. In the event site grading exposes asbestos bearing materials, the location of the locations of these materials should be documented and the asbestos content should assessed by Twining.

If asbestos bearing materials are exposed during grading of the sites, or where asbestos-containing fill material is used, the potential for human exposure to asbestos should be mitigated. Exposed cuts with asbestos-containing serpentine should be gunited or covered with 12 inches of asbestos free fill. Asbestos materials used as fill should be covered with 12 inches of serpentine free fill.

CONCLUSIONS

Based on our reconnaissance and geotechnical evaluation of the water storage tank area, our reconnaissance and limited field exploration at the winery buildings site, and our understanding of the anticipated construction at the two sites, we present the following general conclusions.

- The water storage tank and winery buildings sites appear suitable for the proposed construction provided the recommendations contained in this report and design level geotechnical engineering reports are followed. It should be noted that the recommended design consultation and construction monitoring by Twining are integral to this conclusion.

- The predominant soil types at the water storage tank site are silty sands, and sandy and gravelly lean clays overlying weathered greenstone bedrock at depths of 5 to 7 feet BSG.

- Silty sands with gravel are present at the proposed winery building locations to depths of about 0.5 to 2.5 feet BSG. The silty sands are underlain by highly weathered greenstone bedrock to the maximum depths of exploration of 6.5 and 10.5 feet BSG.

- Some shallow groundwater may impact the proposed water storage tank site. Based on our estimate of the quantity and location of this shallow groundwater, the french drain proposed for behind the tank wall foundation would provide adequate subsurface drainage for the tank structure. Shallow groundwater is not anticipated to impact winery buildings. However, subsequent to rough grading of the water tank and winery buildings sites, slope, soil, and rock conditions should be reviewed by Twining's civil engineer or engineering geologist for evidence of subsurface groundwater flow. Adverse shallow groundwater can be controlled using the methods listed in the "Evaluation" section.

- Potentially expansive clayey soils may be present near the proposed locations of floor slabs or lightly loaded foundations at the winery buildings site. If clays are encountered at these locations foundations will need to be extended to the base of the critical zone (approximately 24 inches). Over-excavation and backfilling with non-expansive engineered fill soils my be required. Based on soils data generated for other sites within the Cordevalle project, we anticipate that 12 to 24 inches of nonexpansive granular soil would be required between floor slabs and clayey soils. A recommendation for over-excavation and placement of non-expansive engineered fill should be provided with the report of Geotechnical Engineering Investigation of the winery buildings site.

- Data presented in the cited reports of previous investigations do not indicate the presence of bedrock faults in the vicinity of the subject sites. The reports indicate that evidence suggests these bedrock faults are not active.

- The subject sites do not lie within published special study zones for ground surface rupture. Our literature investigation suggests that the potential for ground rupture at the subject sites associated with a known fault is low.

- A preliminary deterministic seismic evaluation indicates that the "upper bounds" earthquake event would produce peak horizontal ground acceleration at the subject sites in the range of 0.4g to 0.5g.

- Soil and rock conditions revealed at the site are not conducive to liquefaction or seismic settlement, and suggest a low potential for liquefaction and significant seismic settlement.

- We do not anticipate that grading operations would reveal asbestos bearing serpentinite materials. However, if asbestos bearing material is revealed during grading, the potential for human exposure to asbestos can be mitigated. In areas where final grading exposes asbestos-containing serpentine, or where asbestos-containing fill material is used, the potential for human exposure can be mitigated by covering with 12 inches of asbestos free engineered fill.

RECOMMENDATIONS

Based on our investigation of the water storage tank and winery buildings sites, the following recommendations are presented for use in project design. Recommendations for the proposed winery buildings are subject to change based on the results of the proposed geotechnical engineering investigation.

When applying the preliminary recommendations for design, the background information, procedures used, findings, evaluation, and conclusions should be considered. The recommended design consultation and construction monitoring by Twining are integral to the proper application of recommendations made for the subject site.

Additional Investigations

A geotechnical field investigation, laboratory investigation of soils, and evaluations should be conducted, and recommendations for site preparation, foundations, slabs-on-grade, and pavements should be prepared prior to construction of the proposed winery buildings. A geotechnical engineering investigation report has been prepared for the water storage tank site (Twining, August 11, 1998).

Even after submittal of the geotechnical and geological engineering investigation report, conditions may be encountered during grading, or the scope of the project may change such that additional or altered recommendations may be warranted as an addendum to the geotechnical engineering investigation report. Twining should observe the project sites after rough grading to assess the potential presence of faults, asbestos containing soils, shallow groundwater, loose soils, or expansive clayey soils.

Potential mitigative measures for adverse conditions are described in the "Evaluation" section of this report. Mitigative measures should be designed by Twining's civil engineer or engineering geologist for specific areas, if necessary.

When grading plans have been generated, Twining should be provided the opportunity to review the plans. Conclusions and recommendations presented in this report, as well as the geotechnical engineering investigation reports, should be incorporated into the final design of the water storage tank and winery buildings.

Twining should be contacted to provide an inspection of final grading.

CONSTRUCTION MONITORING

It is recommended that Twining be retained to observe the excavation and earthwork phases of the subject project to determine that the subsurface conditions are compatible with those referenced in this report and identified in the proposed geotechnical and geological engineering investigation report. These services should include site review by an engineering geologist at least monthly.

Twining should conduct the necessary observation, field testing services and provide results so that action necessary to remedy potential deficiencies can be taken in accordance with the plans and specifications. Upon completion of the work, a written summary of observations should be prepared including field testing and conclusions regarding the conformance of the completed work to the intent of the plans and geologic and geotechnical specifications.

Upon the completion of work, a final engineering geology report should be prepared by Twining. This report is essential to ensure that recommendations are incorporated into the project construction, and to note any deviations from the project plans and specifications. The client should notify Twining upon the completion of work to provide this report.

DESIGN CONSULTATION

Twining should be provided the opportunity to review those portions of the contract drawings and specifications that pertain to earthwork and foundations prior to finalization to determine whether they are consistent with our recommendations.

If Twining is not afforded the opportunity for review, we assume no liability for the misinterpretation of our conclusions and recommendations. This review is documented by a formal plan/specification review report provided by Twining.

NOTIFICATION AND LIMITATIONS

The conclusions and recommendations presented in this report are based on the information provided regarding the proposed construction, and the results of the research of background information, combined with interpolation of the subsurface conditions from investigations conducted at nearby sites. A design level geotechnical investigation is necessary for prior to construction of the winery buildings.

The focus of our investigation was the proposed water storage tank and winery buildings sites and pertains only to geologic and geotechnical concerns of this site. Potential geotechnical and geologic hazards to structures on or outside of the subject site were not evaluated in this report.

If variations or undesirable conditions are encountered during construction, Twining should be notified promptly so that these conditions can be reviewed and our recommendations reconsidered where necessary. It should be noted that unexpected conditions frequently require additional expenditures for proper construction of the project.

If the proposed construction is relocated or redesigned, or if there is a substantial lapse of time between the submission of our report and the start of work (over 12 months) at the site, or if conditions have changed due to natural cause or construction operations at or adjacent to the site, the conclusions and preliminary recommendations contained in this report should be considered invalid unless the changes are reviewed and our conclusions and recommendations modified or approved in writing.

Changed site conditions, or relocation of proposed structures, may require additional investigations to determine if our conclusions and recommendations are applicable considering the changed conditions or time lapse.

The conclusions and recommendations contained in this report are valid only for the project discussed in the "Anticipated Construction" section of this report. The entity or entities that use or cause to use this report or any portion thereof for a structure or site other than those indicated in the "Background" section of this report shall hold Twining, its officers and employees harmless from any and all claims and provide Twining's defense in the event of a claim.

This report is issued with the understanding that it is the responsibility of the client to transmit the information and preliminary recommendations of this report to developers, owners, buyers, architects, engineers, designers, contractors, subcontractors, and other parties having interest in the project so that the steps necessary to carry out these preliminary recommendations in the design, construction and maintenance of the project are taken by the appropriate party.

This report presents the results of a preliminary investigation of geotechnical and geological feasibility, and should not be construed as a geotechnical report, or an environmental audit or study.

Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally-accepted engineering principles and practices in Santa Clara County, California at the time of the investigation. This warranty is in lieu of all other warranties either expressed or implied.

Reliance on this report by a third party (i.e., that is not a party to our written agreement) is at the party's sole risk. If the project and/or site is purchased by another party, the purchaser must obtain written authorization and sign an agreement with Twining in order to rely upon the information provided in this report for design or construction of the project.

CLOSING

We appreciate the opportunity to be of service to Lion's Gate Estate Partners. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience.

Sincerely,
THE TWINING LABORATORIES, INC.

REGISTERED GEOLOGIST
KENNETH JAMES CLARK
No. EG 1664
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA
Exp 5-31-99

Kenneth J. Clark, CEG
Project Geologist

cc: Mr. Burt Verrips

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REFERENCES

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GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED POTABLE WATER TANK
CORDEVALLE GOLF CLUB AND HOTEL
SAN MARTIN, CALIFORNIA

Project Number: D34301.02

Prepared for:

Lions Gate Estate Partners, LLC
405 El Camino Real, Suite 127
Menlo Park, California 94025

August 11, 1998

CORPORATE OFFICE
2527 Fresno Street
Fresno, CA 93721
(209) 268-7021 • Fax 268-7126

MODESTO
4230 Kiernan Ave., #105
Modesto, CA 95256
(209) 545-1050 • Fax 545-1147

VISALIA
130 North Kelsey St., #H6
Visalia, CA 93291
(209) 651-8280 • Fax 651-8288

BAKERSFIELD
3701 Pegasus Drive, #124
Bakersfield, CA 93308
(805) 393-5088 • Fax 393-4643

SALINAS
520 #A Crazy Horse Canyon Rd.
Salinas CA 93907
(408) 449-5284 • Fax 449-5092

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August 11, 1998 D34301.02-06

Lion's Gate Estate Partners, LLC
405 El Camino Real, Suite 127
Menlo Park, California 94025

Attention: Mr. Tom Hix

Subject: Geotechnical Engineering Investigation
Proposed Potable Water Tank
Cordevalle Golf Club and Hotel
San Martin, California

Dear Mr. Hix:

We are pleased to submit this geotechnical engineering investigation report prepared for the proposed potable water tank at the Cordevalle Golf Club and Hotel located west of the City of San Martin, in Santa Clara County, California. The contents of this report include the purpose of the investigation, scope of services, background information, investigative procedures, our findings, evaluation, conclusions, and recommendations.

We recommend that those portions of the plans and specifications that pertain to earthwork, and foundations be reviewed by The Twining Laboratories, Inc. (Twining) to determine if they are consistent with our recommendations. This service is part of this current contractual agreement and the client should provide these documents for our review prior to their issuance for construction bidding purposes.

In addition, it is recommended that Twining be retained to provide inspection and testing services for the excavation, earthwork, and foundation phases of construction. These services are necessary to determine if the subsurface conditions are consistent with those used in the analysis and formulation of recommendations for this investigation, and if the construction complies with our recommendations. This service is not, however, part of this current contractual agreement.

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Fresno, CA 93721
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4230 Kiernan Ave., #105
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We would appreciate the opportunity to provide a proposal for this additional service after construction documents are completed. Mr. Harry Moore with our firm (800-268-7021) will contact you in the near future regarding these services.

We appreciate the opportunity to be of service to Lion's Gate Estate Partners, LLC. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience.

Sincerely,
THE TWINING LABORATORIES, INC.

Kenneth J. Clark, CEG
Engineering Geologist

EXECUTIVE SUMMARY

The proposed Cordevalle Golf Club and Hotel will include a 70-foot diameter reinforced concrete potable water tank with an approximate capacity of 420,000 gallons. The tank will have an aluminum "TEMCOR" domed roof and a 36 mil Hypalon liner covering the sides and bottom of the tank. The tank is to be located in a gentle swale with approximately two-thirds of the tank height below the adjacent native grade level. The reinforced concrete walls will be supported on a 3-foot wide perimeter spread foundation. A french drain will be installed outside the entire perimeter of the reinforced concrete tank wall. The tank will have a perimeter access road surfaced with Type II aggregate base.

The Twining Laboratories, Inc. (Twining) was authorized on December 12, 1997, by Mr. Thomas Hix with Lion's Gate Estate Partners, LLC, to conduct this geotechnical engineering investigation.

The purpose of this investigation was to provide geotechnical engineering parameters for earthwork, site preparation, and preliminary information for preparation of related construction documents. The investigation included a field exploration and laboratory testing program, evaluation of the data collected during the field and laboratory portions of the investigation, and preparation of this report.

Reconnaissance of the site consisted of walking the site and noting visible surface and slope features. The reconnaissance was conducted by Mr. Kenneth Clark on July 7, 1998. Maximum native slopes within the swale at the proposed tank location range from about 4 horizontal (H) to 1 vertical (V) to 5H to 1V. Our reconnaissance did not reveal evidence of existing slope failure near the proposed tank location. Native slopes in the area of the proposed tank appear relatively stable.

On July 7, 1998 one (1) test boring was drilled near the proposed tank location to a depth of 41.5 feet below site grade (BSG). In addition, two (2) test pits were excavated below the proposed tank location to depths of 11 and 12 feet BSG. Soil samples were collected from the boring and pits for testing.

Soil conditions encountered during the field investigation were relatively consistent across the project site. The near surface soils were silty sands to depths of about 1 to 2 feet BSG. Gravelly and sandy lean clays were encountered below the silty sands to depths of 5 to 7 feet BSG. The lean clay soils exhibit low to moderate shear strength and moderate compressibility characteristics.

Weathered greenstone bedrock was encountered below the lean clays to the maximum depths of exploration in the boring and test pits.

Based on the Potable Tank Section diagram provided by PACE it appears the grading for the tank pad will extend to a maximum of about 24 feet below the existing site grade. Soil and rock conditions revealed in the test boring and test pits suggest variable degrees of weathering and generally rippable conditions for the greenstone bedrock to the anticipated elevations required for pad preparation. However, during test pit exploration, the backhoe was unable to excavate the pits to an even depth across the bottom of the test pits and encountered refusal at 8 to 12 feet BSG on relatively fresh greenstone rock in some portions of pits.

Field data suggests that some shallow groundwater may occur near the tank pad elevation. However, the amount of groundwater is anticipated to be relatively small and it appears that potential pore pressure and nuisance conditions would be adequately addressed by the french drain proposed outside of the wall foundation.

From a geotechnical standpoint, the site is suitable for the proposed tank provided the recommendations contained in this report are followed.

We anticipate that the pad will be cut to a maximum depth of about 23 feet BSG to achieve the designed tank bottom surface (at the center of the tank). The tank wall footings will be at depths ranging from about 10 to 23 feet below existing site grades. Field data indicates the base of the footings will be on variably weathered greenstone bedrock. Conditions at the proposed footing depths are anticipated to be predominantly competent greenstone. However, due to the irregular weathering profile, some lean clay soils are anticipated. Where lean clay soils are exposed at the bottom of foundation excavations, these soils should be excavated down to firm rock material and the excavations should be backfilled with a lean (2-sack) cement slurry to establish a level foundation bottom.

To address potential differential settlement of the tank bottom liner the tank pad should be prepared by ripping and moisture conditioning to a depth of 8 inches below pad grade and compacting soils as engineered fill. The intent of pad preparation is also to provide a uniform base free of sharp rocks which could puncture the bottom liner.

After excavation of the tank pad, and prior to placement of footings and the bottom liner, the subgrade should be reviewed by our firm to confirm the removal of soft or pliant areas.

A 25-foot high cut slope with a gradient of 2 horizontal (H) to 1 vertical (V) is proposed upslope of the tank. The majority of this cut is anticipated to be into weathered greenstone rock. Our observations of the greenstone rock materials exposed in test pits suggest that the proposed cut slope would be stable.

For stability, permanent fill and cut slopes should be constructed at 2H to 1V, horizontal to vertical, or flatter. Where fill is placed on native slopes steeper than 5H:1V a minimum 6 foot wide keyway should be constructed at the toe of fill slopes.

Analytical results of a near surface soil sample indicate the soils are "mildly corrosive". Buried metal objects should be protected in accordance with the manufacturer's recommendations based on a "mildly corrosive" corrosion potential of the soil. The evaluation was limited to the effects of soils to metal objects; corrosion due to other potential sources, such as stray currents and groundwater, was not evaluated.

Corrosion of concrete due to sulfate attack is not anticipated based on concentration of sulfates indicating a "negligible" exposure, as determined for the near-surface soils. Type I or II cement may be used as specified in Table No. 19-A-3 of the 1994 Uniform Building Code.

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GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED POTABLE WATER TANK

CORDEVALLE GOLF CLUB AND HOTEL

SAN MARTIN, CALIFORNIA

Project Number: D34301.02

1.0 INTRODUCTION

This report presents the results of a geotechnical engineering investigation for the proposed proposed potable water tank to be located at the Cordevalle Golf Club and Hotel, Subdivision and Country Club, San Martin, California.

The Twining Laboratories, Inc. (Twining) was authorized by written agreement on December 12, 1997 by Mr. Thomas Hix, with Lion's Gate Estate Partners, LLC, to conduct this geotechnical engineering investigation.

The contents of this report include the purpose of the investigation and the scope of services provided. The site history, previous studies, existing site features, and anticipated construction are discussed. In addition, a description of the investigative procedures used and the subsequent findings obtained are presented. Finally, the report provides an evaluation of the findings, general conclusions, and related recommendations. The three report appendices contain the drawings (Appendix A), the logs of test pits and borings (Appendix B), and the results of laboratory tests (Appendix C).

The Geotechnical Engineering Division of Twining, headquartered in Fresno, California, performed the investigation.

2.0 PURPOSE AND SCOPE OF INVESTIGATION

The purpose of the investigation was to conduct a field exploration and laboratory testing program, evaluate the data collected during the field and laboratory portions of the investigation, and provide the following:

a) General subsurface soil and groundwater conditions;

b) Recommendations for site preparation including preparation of subgrade soils as well as placement, moisture conditioning, and compaction of engineered fill soils;

c) Recommendations for cut and fill slopes;

d) Recommendations for temporary excavations and trench backfill;

e) Geotechnical parameters for use in design of foundations; and

f) Evaluation of soil corrosivity.

This report is provided specifically for the proposed potable water tank of the proposed Cordevalle Golf Club and Hotel, referenced in the Proposed Construction section of this report.

This investigation did not include a geologic/seismic hazards evaluation, floodplain investigation, compaction tests, environmental investigation, environmental audit, or investigation of soil conditions for a tank access road. The Cordevalle Golf Club and Hotel will consist of the development of a 41-lot residential development, 18 hole golf course and associated maintenance facilities, club house, tennis courts, overnight lodges, equestrian center, and winery.

Our proposal, dated December 12, 1997, outlined the scope of our services. The actions undertaken during the investigation are summarized as follows.

1. The following documents prepared by others were reviewed:

   o Prepurchase Site Assessment of Geologic Hazards, Ground Water Supply and Environmental/Toxic Contamination, Hayes Valley Property, Santa Clara, California, Project 4297, prepared for LAND USE, by TERRATECH, INC., dated January 20, 1988;

   o Supplemental Geological Reconnaissance Investigation for Proposed Hayes Valley Dams, Santa Clara County, California, prepared by Kaldveer Associates Geoscience Consultants, dated August 4, 1989;

   o Geologic Input to Draft Environmental Impacted Report, Lion's Gate Development, project HRC-101B, prepared by Wahler Associates for HR Development Partners, dated April 17, 1990;

   o Geologic Input to EIR, prepared by ENGEO Incorporated, date April 13, 1993;

   o Geologic Feasibility Investigation, Golf Course Maintenance Building, The Lion's Gate Reserve, San Martin , California, Project 1385/6G, prepared for Hayes Valley Development Partners, by Pacific Geotechnical Engineering, dated December 1995;

- o Preliminary Geologic Feasibility Evaluation, Homesites on Parcels #24, #25, and #26, The Lion's Gate Reserve, San Martin, California, Project 1385/7G, prepared for Hayes Valley Development Partners, by Pacific Geotechnical Engineering, dated December 1995;

- o Geologic Feasibility Investigation, Clubhouse and Overnight Lodges, The Lion's Gate Reserve, San Martin, California, Project 1385/5G, prepared for Hayes Valley Development Partners, by Pacific Geotechnical Engineering, dated December 1995;

- o Administrative Draft Environmental Impact Report, Volume IIa Technical Appendices, Lion's Gate Reserve, dated December 1995;

- o Draft Environmental Impact Report, Volume II Technical Appendices B through E, Lion's Gate Reserve, dated March 1996; and

- o Final Grading Plan, prepared by Pacific Advanced Civil Engineering, dated May 5, 1998.

II. The following geologic and geotechnical reports prepared by Twining were reviewed:

- o Report entitled Preliminary Geotechnical Engineering Investigation, Golf Course, dated March 18, 1997, and Addendums No. 1 and No. 2;

- o Letter report entitled "Review of Site Geologic Conditions and Grading Plans, Golf Course Phase", dated May 6, 1997;

- o Report entitled "Preliminary Geotechnical Engineering Investigation, Clubhouse and Overnight Lodges" (former proposed site), dated October 30, 1997;

- o Letter report entitled "Preliminary Evaluation of Geotechnical and Geological Feasibility, Clubhouse and Overnight Lodge Area" (proposed new site), dated April 16, 1998; and,

- o Geologic and Geotechnical Site Review: New Clubhouse and Overnight Lodge Area, Cordevalle Golf Club and Hotel, dated May 29, 1998;

III. A site reconnaissance and subsurface exploration were conducted.

IV. Laboratory tests were conducted to determine selected physical and engineering properties of the subsurface soils.

V. Mr. Ron Davis (Lion's Gate Estate Partners, LLC), and Mr. Joseph Gutierrez (PACE) were consulted during the investigation.

VI. The data obtained from the investigation were evaluated to develop an understanding of the subsurface conditions and engineering properties of the subsurface soils.

VII. This report was prepared to present the purpose and scope, background information, field exploration procedures, findings, evaluation, conclusions, and recommendations.

3.0 BACKGROUND INFORMATION

The site history, previous studies, existing site features, and the anticipated construction are summarized in the following subsections.

3.1 Site History: The site appears to have been used for cattle grazing. No evidence of other site uses were noted during our field investigation.

3.2 Previous Studies: It is our understanding that no other studies have been conducted specifically for the proposed potable water tank site. Numerous engineering, geological, and environmental studies have been conducted for other portions of the Cordevalle development.

3.3 Site Description: The Cordevalle Golf Club and Hotel includes a 1,676 acre site of which 400 acres are to be developed, and the remainder is to remain undeveloped. The project site is located approximately west of the intersection of Highland and Turlock Avenues west of the City of San Martin in Santa Clara County, California. A site location map is presented on Drawing No. 1 in Appendix A.

The potable water tank site is located on an eastward sloping swale, about 4,000 feet northwest of the intersection of Highland and Turlock Avenues, and about one-half mile north of the golf course. The swale slopes at about 4H to 1V. The west edge of the proposed tank is approximately 125 feet downslope from the top of a broad ridgeline.

Oak trees are present on the hillside near the near the proposed tank site. Dry brown grasses of up to 3 feet high covered the surface soils at the time of our field investigation.

According to a geologic map of the site region prepared by Kaldveer Associates (scale: 1 inch = 500 feet, 1989) for the proposed Hayes Valley Dam, the tank is located on Franciscan Complex greenstone. A serpentinite belt is located approximately 500 feet west of the proposed tank site. The nearest mapped active or potentially active fault is the Sergant Fault, located 2.5 miles east of the site.

3.4 <u>Proposed Construction</u>: We understand the proposed potable water tank will include a 70-foot diameter reinforced concrete walled tank with an approximate capacity of 420,000 gallons. Approximately two-thirds of the tank height is below the adjacent native grade level. The tank will have an aluminum "TEMCOR" domed roof and a 36 mil Hypalon liner covering the sides and bottom of the tank. An 8 ounce nonwoven geotextile will be placed below the bottom portion of the tank liner. The bottom surface of the tank will be sloped toward the center at a 4H to 1V gradient. The reinforced concrete walls will be supported on a 3-foot wide perimeter spread foundation. A french drain will be installed outside the entire perimeter of the reinforced concrete tank wall.

A perimeter access road with a Class II aggregate base surface will be constructed around the tank.

Drawing 3 in Appendix A presents a cross section of the tank.

3.5 <u>Proposed Construction Grading</u>: Cuts of up to about 23 feet are proposed to achieve a level pad for the tank. The tank foundation is proposed to bear entirely on cut. Fills of about 2 to 5 feet are proposed along the downslope perimeter on the pad, beneath the perimeter access road.

4.0 INVESTIGATIVE PROCEDURES

The field exploration and laboratory testing program conducted for this investigation are summarized in the following subsections.

4.1 <u>Field Exploration</u>: The field exploration consisted of a site reconnaissance, drilling of a test boring, excavation of test pits, and soil sampling. The test boring and test pit locations are shown on Drawing No. 2 in Appendix A. Due to the relatively steep gradient of ground surface at the proposed tank site, the drill rig could not access test boring locations within the tank footprint. However, one test boring was drilled approximately 120 west of the proposed west edge of the tank, near a level natural ridge.

Test boring and pit locations were determined by pacing with reference to survey stakes placed at the center and on the perimeter of the proposed tank. The locations, as described, should be considered accurate to within 15 feet.

4.1.1 <u>Site Reconnaissance</u>: The site reconnaissance consisted of walking the site and noting visible surface features. The reconnaissance was conducted by Mr. Kenneth Clark on July 7 1998. The features noted are described in the background information (Section 3.0).

4.1.2 Drilling Test Borings: On July 7, 1998 one (1) test boring was drilled west of the proposed tank site, approximately 120 feet from the edge of the proposed tank. The boring was advanced to a depth of 41.5 feet BSG.

Under the direction of Twining's engineering geologist, the test boring was drilled using a CME-75 drill rig equipped with 6 5/8 inch hollow-stem augers. . The soils encountered in the test boring were logged by Twining's field engineer. The field soil classification was in accordance with the Unified Soil Classification System and consisted of particle size, color, and other distinguishing features of the soil.

The presence and elevation of free water, if any, in the borings were noted and recorded during drilling and immediately following completion of borings.

Elevations of the test borings were not measured as a part of the investigation. The test borings were loosely backfilled with material excavated during the drilling operations; thus, some settlement should be anticipated.

4.1.3 Excavation of Test Pits: On July 7, 1998, two (2) test pits were excavated below the plan area of the proposed tank. These pits were excavated to depths of 7 and 10 feet BSG. Under the direction of a Twining engineering geologist, the test pits were excavated using a backhoe equipped with a 24 inch wide bucket.

The test pits were loosely backfilled with excavation material; thus, some settlement should be anticipated. Portions of the pits located outside the cut areas should be re-excavated and replaced as engineered fill during earthwork operations.

4.1.4 Soil Sampling: Standard penetration tests were conducted, and both disturbed and undisturbed soil samples were obtained.

The standard penetration resistance, N-value, is defined as the number of blows required to drive a standard split barrel sampler into the soil. The standard split barrel sampler has a 2 inch O.D. and a 1-3/8 inch inside diameter (I.D.). The sampler is driven by a 140 pound weight free falling 30 inches. The sampler is lowered to the bottom of the bore hole and set by driving it an initial 6 inches. It is then driven an additional 12 inches and the number of blows required to advance the sampler the additional 12 inches is recorded as the N-value.

Relatively undisturbed soil samples for laboratory tests were obtained by pushing or driving a California modified split barrel ring sampler into the soil. The soil was retained in brass rings, 2.5 inches O.D. and 1 inch in height. The lower 6 inch portion of the samples were placed in close-fitting, plastic, air-tight containers which, in turn, were placed in cushioned boxes for transport to the laboratory.

Soil samples obtained were taken to Twining's laboratory for classification and testing.

4.2 Laboratory Testing: The laboratory testing was programmed to determine selected physical and engineering properties of the soils underlying the site. The tests were conducted on disturbed and undisturbed samples representative of the subsurface material.

The results of laboratory tests are summarized on Figure Nos. 1 through 4 in Appendix C. These data, along with the field observations, were used to prepare the final test boring and test pit logs in Appendix B.

5.0 FINDINGS

The findings of the field exploration and laboratory testing are summarized in the following subsections.

5.1 Soil Profile: Silty sands were encountered from the ground surface to depths of approximately 1 to 2 feet BSG. The root systems of grasses and weeds along with desiccation cracking extended to depths of about 18 to 24 inches. Beneath the silty sands, the soils encountered were sandy and gravelly lean clays. Weathered greenstone bedrock was encountered at depths of 5 to 7 feet BSG, extending to the maximum depths explored in the test boring (41.5 feet BSG) and test pits (7 and 10 feet BSG).

The foregoing is a general summary of the soil conditions encountered in the test pits drilled for this investigation. Detailed descriptions of the soils encountered at the test boring and test pits are presented on the logs of test borings in Appendix B. The stratification lines shown on the logs represent the approximate boundary between soil types; the actual in-situ transition may be gradual.

5.2 Soil Engineering Properties: The natural moisture content measured in a sample of the silty sand was 5 percent.

The natural moisture content measured in samples of the lean clay ranged from 2 to 13 percent. A maximum density/optimum moisture determination performed on one near-surface soil sample indicated a maximum dry density of 114.8 pounds per cubic foot at an optimum moisture content of 18.8 percent.

The natural moisture content measured in samples of the weathered greenstone rock ranged from 2 to 5 percent and one in-place density test revealed a dry density of 125 pounds per cubic foot. A direct shear test performed on a lean clay sample indicated and angle of internal friction of 28 degrees, with a cohesion value of 369 pounds per square foot. The weathered greenstone soils exhibited moderate compressibility characteristics with the addition of moisture as indicated by one consolidation test (about 8.2 percent consolidation under a load of 4 kips per square foot). Upon inundation, the soils exhibited low collapse potential (about 1.6 percent collapse under a load of 0.5 kips per square foot).

5.3 **Groundwater Conditions**: Groundwater was not encountered in the test pits excavated below the proposed tank location to a maximum depth of 10 feet BSG. Wet soil and rock material was encountered at a depth of 35 feet (estimated to be 15 to 20 feet below existing site grade at the proposed center of the tank. However, free groundwater was not encountered in the test boring to the maximum depth of exploration of 41.5 feet BSG.

It should be recognized that water table elevations and potentiometric conditions fluctuate with time, since they are dependent upon seasonal precipitation, irrigation, land use, and climatic conditions as well as other factors. Therefore, water level observations at the time of the field investigation may vary from those encountered both during the constructio . phase and the design life of the project. The evaluation of such factors was beyond the scope of this investigation and report.

6.0 EVALUATION

The data and methodology used to develop conclusions and recommendations for project design and preparation of construction specifications are summarized in the following subsections. The evaluation was based upon the subsurface conditions determined from the investigation and our understanding of the proposed construction.

6.1 **Soil, Rock, and Groundwater Conditions**: The soil conditions encountered in the test boring and test pits were relatively consistent across the project site as indicated on the test boring and test pit logs (Appendix B). The near surface soils were silty sands to depths of about 1 to 2 feet BSG. Hard gravelly and sandy lean clays were encountered below the silty sands to depths of 5 to 7 feet BSG. Weathered greenstone bedrock was encountered below the lean clays to the maximum depths of exploration in the boring and test pits. The lean clay soils exhibit low to moderate shear strength and moderate compressibility characteristics.

Bedrock was encountered during the field investigation at depths of 5 to 7 feet BSG. Based on the Potable Tank Section diagram provided by PACE it appears the grading for the tank pad will extend to a maximum of about 24 feet below the existing site grade. Soil and rock conditions revealed in the test boring and test pits suggest variable degrees of weathering and generally rippable conditions for the greenstone bedrock to the anticipated elevations required for pad preparation. However, during test pit exploration, the backhoe was unable to excavate the pits to a consistent depth, suggesting variable weathering conditions across the test pits.

Based on field data (subsection 5.3), and considering the proposed tank location near the top of a broad ridgeline, we anticipate that some shallow groundwater may occur near the tank pad elevation. However, the amount of groundwater is anticipated to be relatively small and it appears that potential pore pressure and nuisance conditions would be adequately addressed by the french drain proposed on the outside of the wall foundation.

6.2 **Stability of Native and Proposed Cut Slopes:** Maximum native slopes within the swale at the proposed tank location range from about 4H to 1V to 5H to 1V. Our investigation did not reveal evidence of existing slope failure near the proposed tank location. Native slopes in the area of the proposed tank appear relatively stable.

A 25-foot cut slope with a gradient of 2H to 1V is proposed upslope of the tank. The majority of this cut is anticipated to be into weathered greenstone rock. Our observations of the greenstone rock materials exposed in test pits suggest that the proposed cut slope would be stable.

6.3 **Faults**: The project site is located in a seismically active region with numerous active and potentially active faults. The nearest mapped active or potentially active fault is the Sergant Fault, located 2.5 miles east of the site. Several bedrock faults associated with melange terrace have been mapped by others on the Cordevalle development site. Our review of data presented in geologic reports previously generated for the development project indicates that the bedrock faults in the site area are inactive.

6.4 **Liquefaction and Seismic Settlement:** Seismic shaking may induce settlement of loose, unconsolidated sediments. This can occur in unsaturated and saturated granular soils. Considering the shallow bedrock below the tank site (absence of loose or granular soils), the potential for significant seismic induced settlement or liquefaction is considered very low.

6.5 **Site Preparation:** Proposed grading indicated on the tank section plan (prepared by PACE, dated July 13, 1998) indicates that cuts of up to 23 feet will be required to construct the tank pad. Fills of up to about 5 feet are anticipated along the perimeter of the downslope portion of the access road. All fills should be placed as compacted engineered fill. Areas to receive fill soils should be prepared to receive these fills by stripping surface organics and loose soils, scarifying to a minimum depth of 8 inches and compacting as engineered fill. Due to organic material noted in the near surface soils, stripped soils are not considered suitable for use as fills in structural areas.

Stripped topsoil may be stockpiled and reused in landscape areas or as erosion resistant materials at the discretion of the owner or residential development architect. It should be anticipated that topsoil will settle about 1 inch per foot of thickness of stripped soils as a result of decay of organic material. Therefore, it is also preferred that stripped soils not be placed in areas which will experience frequent foot traffic. These stripped soils should be placed in out-of-way areas where the anticipated long term settlement will not pose a safety concern or require constant regrading.

Often when fill soils are placed on sloping terrain, the fill soils will migrate downhill due to gravity, with slippage occurring on the plane between the fill soils and the native subgrade. To reduce the potential for this movement, the fill soils placed on slopes steeper than 5H:1V should be prepared to receive engineered by benching and keying into the stiff and competent native soils or bedrock to reduce the potential for a failure plane between the fill soils and the native materials. Based on the native slope grades at the site and the thickness of fill soils proposed, construction of one keyway will be required prior to placement of engineered fill. Keyway or benches should be a minimum of 6 feet wide.

6.6 **Cut and Fill Slopes:** For slope stability, permanent slopes should be constructed such that both cut and fill slopes are 2H to 1V or flatter. If slopes are to be graded steeper than 2H to 1V, these slopes should be evaluated by the geotechnical engineer on a case by case basis. It is anticipated that relatively steep temporary cut slopes of about 10 feet in height will be required for construction of the concrete wall footing and french drains. Observations of test pits and soil and rock conditions suggest that near vertical cuts may be stable on a temporary bases. However, considering the weathered nature of the greenstone bedrock, temporary excavations in weathered rock should not be graded steeper than 3/4H to 1V unless evaluated by a geotechnical engineer. Temporary excavations in soils should not be graded steeper than 1H to 1V unless evaluated by a geotechnical engineer.

Run-on of surface water onto the proposed 2H to 1V cut slope could cause erosion, and increased moisture content and soil unit weight. These factors would tend to decrease the long term stability of the proposed cut slope. Accordingly, a brow ditch, should be provided to direct surface water away from the cut slope. In addition, the cut slope should be maintained and protected with proper cover, such as shallow rooted vegetation, to reduce erosion and aid in stability. If the slope is landscaped, irrigation should be drip type or one with equivalent lack of runoff.

6.7 **Tank Foundation and Bottom:** Tank wall footings are proposed at depths ranging from about 10 to 23 feet below existing site grades. Test boring and test pit data indicate that the base of the footings will be on variably weathered greenstone bedrock. Conditions at the proposed footing depths are anticipated to be predominantly competent greenstone. However, due to the irregular weathering profile, some lean clay soils are anticipated. Where lean clay soils are exposed at the bottom of foundation excavations, these soils should be excavated down to firm rock material. The excavations should be backfilled with a low-compressible engineered fill material or lean (2-sack) cement slurry to establish a level foundation bottom.

The tank walls and bottom are to be lined with a 36 mil potable Hypalon material. Considering that variable weathered soil and rock conditions are anticipated at the bottom of the tank, some differential settlement of the bottom liner may occur. Although it is anticipated that the proposed bottom liner can accommodate some differential settlement, soil materials exposed on the tank pad should be prepared to so that a relatively smooth (regular) tank bottom surface is maintained during filling and operation of the tank. Preparation should include ripping and moisture conditioning to a depth of 8 inches below pad grade and compacting soils as engineered fill. This would also provide a provide a uniform base relatively free of sharp rocks which could puncture the bottom liner. The liner manufacturer should be consulted to assess whether the site preparation recommendations are consistent with the tear resistance of the liner material.

After excavation of the tank pad, and prior to placement of footings or liner, the subgrade should be reviewed by our firm to confirm the removal of soft or pliant areas.

6.8 **Corrosion Protection:** The risk of corrosion of construction materials relates to the potential for soil-induced chemical reaction. The rate of deterioration depends on soil resistivity, texture, acidity, and chemical concentration. The evaluation of potential corrosion for the tank was based on the results of an analyses of a composite sample collected from the location of proposed lot 8 of the Cordevalle residential development, about 2,000 feet south of tank site. Review of soil chemical test data for similar soils in the Cordevalle project area suggest that these results represent soil chemical conditions at the subject site.

Results of the analysis indicate a resistivity value of 21,600 ohms/cm and a pH value of 6.0. These values indicate the soils are "mildly corrosive". In addition, the results of the two soil sample analyses indicated a "none-detected" concentration of sulfate (less than the detection limits of 0.01 weight percent), and a chloride concentration of 0.0013 weight percent. We recommend that these soil corrosion data be provided to the manufacturer's or supplier's of materials that will be in contact with soils (pipes or ferrous metal objects, etc.) to provide assistance in selecting the protection and materials for the proposed products or materials. If the manufacturer's or supplier's cannot determine if materials are compatible with the soil corrosion conditions, a professional consultant, i.e. a corrosion engineer, with experience in corrosion protection should be consulted to provide design parameters.

7.0 CONCLUSIONS

Based on the data collected during the field and laboratory investigations, our geotechnical experience in the vicinity of the project site, and our understanding of the anticipated construction, we present the following general conclusions.

7.1 The site is suitable for the proposed construction with regard to support of the proposed tank, foundations, and concrete slabs-on-grade, provided the recommendations contained in this report are followed. It should be noted that the recommended design consultation and construction monitoring by Twining are integral to this conclusion.

7.2 The soil/rock conditions encountered comprised silty sands to depths of about 1 to 2 feet BSG, underlain by lean clay to a depth of 5 to 7 feet BSG. Weathered greenstone bedrock was encountered below the lean clay to the maximum depth of exploration of 41.5 feet BSG.

7.3 Field data suggests that some shallow groundwater may occur near the tank pad elevation. The proposed french drain appears adequate to provide subsurface drainage away the wall foundation (retaining wall).

7.4 After excavation of the tank pad, and prior to placement of footings or liner, the subgrade should be reviewed by our firm to confirm the removal of soft or pliant areas.

7.5 The bottom of the tank may be supported on hard greenstone, on low compressive engineered fill, or on a 2-sack sand cement slurry extending to hard greenstone rock (slurry required to fill areas of overexcavated highly weathered greenstone).

7.6 The tank pad should be prepared by moisture conditioning and compacting exposed native soils as engineered fill to a depth of 8 inches.

7.7 Total and differential settlements for the proposed tank are estimated to be 1 inch or less.

7.8 The potential for liquefaction and seismic settlement are very low based on the absence of granular soils at the site.

7.9 Proposed permanent slopes of 2H to 1V or flatter are anticipated to remain stable during the design life of the structure. If permanent slopes are to be graded steeper than 2H to 1V, these should be evaluated by the geotechnical engineer on a case by case basis. Temporary excavations in lean clay or silty sand soils should not be graded steeper than 1H to 1V. Temporary excavations in weathered rock should not be graded steeper than 3/4H to 1V, unless evaluated by a geotechnical engineer.

7.10 The analytical result of a soil sample analysis indicates that the near-surface soils exhibit a "mildly corrosive" corrosion potential to buried metal objects.

7.11 The analytical result of a soil sample analyses indicate sulfate concentrations of "none detected" and a chloride concentration of 0.0013 percent by dry weight. Therefore, a low potential for sulfate attack on reinforced concrete placed in the near-surface soils is anticipated.

8.0 RECOMMENDATIONS

Based on the evaluation of the field and laboratory data and our geotechnical experience in the vicinity of the project, we present the following recommendations for use in the project design and construction. However, this report should be considered in its entirety. When applying the recommendations for design, the background information, procedures used, findings, evaluation, and conclusions should be considered. The recommended design consultation and construction monitoring by Twining are integral to the proper application of the recommendations.

8.1 Site Grading and Drainage

8.1.1 Develop and maintain site grades which will drain surface runoff away from the tank walls - both during and after construction. Adjacent exterior finished grades should be sloped a minimum of two percent for a distance of at least five feet away from the tank to preclude ponding of water adjacent to the tank.

8.1.2 Landscaping after construction should direct rainfall and irrigation runoff away from the structure and not promote ponding of water adjacent to the structures. Care should be taken to maintain a leak-free sprinkler system.

8.2 Site Preparation

8.2.1 All topsoil, vegetation, organics, and debris should be removed from the proposed tank and roadway areas. The general depth of stripping should be sufficiently deep to remove the root systems and organic topsoils. For estimate purposes, a minimum stripping depth of 6 inches should be used. The actual depth of stripping should be reviewed by our firm at the time of construction. Deeper stripping may be required in localized areas. Stripping should extend laterally a minimum of 5 feet outside the tank and roadway perimeters. These materials will not be suitable for use as engineered fill; however, stripped topsoil may be stockpiled and reused in landscape areas at the discretion of the owner. It should be anticipated that topsoil will settle about 1 inch per foot of thickness as a result of decay of organic material.

8.2.2 We anticipate that the pad will be cut to a maximum depth of about 23 feet to achieve the designed tank bottom surface. This excavation would remove the loose near surface silty sand soils. Along the perimeter of the pad (where fill is to be placed) care should be taken to remove the loose silty sand soils to a minimum depth of 1 foot BSG prior to placement of the fill.

8.2.3 If fill soils are to be placed on slopes steeper than 5H:1V the slopes should be prepared to receive engineered keying into the stiff and competent native soils or bedrock at the to of the fill slope.

8.2.4 After stripping, excavation of the tank pad, and prior to placement of engineered fill, the subgrade should be reviewed by Twining to confirm the removal of topsoil, organics, and soft or pliant areas.

8.2.5 The bottoms of keyways and footings should be reviewed by Twining prior to placement of overlying materials.

8.2.6 The exposed ground surface in areas to receive engineered fill material should be scarified to a depth of 8 inches, moisture conditioned in accordance with subsection 8.3.1, and compacted as engineered fill. The zone of scarification and compaction should extend laterally a minimum of 5 feet outside the perimeters of the fill area. The scarification and compaction should be conducted following stripping operations, removal of subsurface structures, over-excavation, and removal of all soft or pliant areas.

8.2.7 All fill required to bring the site to final grade should be placed as engineered fill. In addition, all native soils over-excavated should be compacted on-site as engineered fill.

8.3 Engineered Fill

8.3.1 All fills should be placed as compacted engineered fill. The on-site soils and rock encountered are predominantly silty sands, lean clays, and weathered greenstone. The silty sand and lean clay soils will be suitable for use as fill material to support the structural loads, provided they are free of organics and debris and the moisture content of the soil is two to five percent over optimum moisture content at the time of placement for the lean clays, or within 2 percent of optimum for the sandy soils. If soils other than those considered in this report are encountered, Twining should be notified to provide alternate recommendations. If the near surface silty sand soils are used, these soils should be moisture conditioned to within two percent of optimum moisture and compacted as engineered fill.

8.3.2 The compactability of the native soils is dependent upon the moisture contents, subgrade conditions, degree of mixing, type of equipment, as well as other factors. The evaluation of such factors was beyond the scope of this report; therefore, we recommend that they be evaluated by the contractor during preparation of bids and construction of the project.

8.3.3 Engineered fill soil should be placed in loose lifts approximately 8 inches thick, moisture-conditioned to 2 to 5 percent above optimum, and compacted to a dry density of at least 90 percent of the maximum dry density as determined by ASTM Test Method D1557-78. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable.

8.3.4 Backfill material behind the tank wall should be non-expansive sandy soils or crushed rock material. These non-expansive materials will have good draining characteristics. If an open graded material is used in the french drain, a filter fabric such as Mirifi 140NS should separate the drain material from the finer grained fill material to minimize mixing and volume losses.

8.4 Tank Foundation and Bottom

8.4.1 Structural loads from the tank may be supported on either a ring foundation, strip footings, or on gravel or sand over the native subgrade. Ring or strip foundations should be supported on a minimum of 12 inches of engineered fill. Spread and continuous footings, a minimum of 1 foot deep and 1 foot wide, may be designed for a maximum gross allowable soil bearing pressure of 3,000 pounds per square foot for dead-plus-live loads. Gross allowable soil bearing pressure is the maximum contact pressure at the base of the foundations. These values may be increased by one-third for short duration wind or seismic loads.

8.4.2 A structural engineer experienced in perimeter foundation design for tanks should recommend the reinforcement, thickness, design details and concrete specifications for the tank foundation.

8.4.3 A total settlement of 1 inch should be anticipated for design. A differential settlement of 1 inch from the center to edge of the tank should also be anticipated for design.

8.4.4 The tank bottom subgrade should be prepared by moisture conditioning and compacting soils as engineered fill to a depth of 8 inches.

8.4.5 The tank connections to exterior structures and pipelines should be designed with flexible connections such that a minimum of 2 inches of settlement can occur without causing damage (more than the predicted settlements to allow for variances in the actual settlement).

8.5 Frictional Coefficient and Earth Pressures

8.5.1 The bottom surface area of concrete footings or concrete slabs in direct contact with engineered fill can be used to resist lateral loads (areas of slabs underlain by a synthetic moisture barrier cannot be considered). An ultimate coefficient of friction of 0.36, reduced by an appropriate factor of safety, can be used for design.

8.5.2 The ultimate passive resistance of the native soils and engineered fill may be assumed to be equal to the pressure developed by a fluid with a density of 300 pounds per cubic foot. An appropriate factor of safety should be applied.

8.5.3 The passive pressure was calculated based on a minimum soil unit weight of 100 pounds per cubic foot. The soils within the passive zone at the foot of retaining walls (one footing width in front of the wall to a depth equal to the footing depth) should be tested to verify that the soils have the minimum unit weight of 100 pounds per cubic foot (with moisture). If the soils have a unit weight of less than 100 pounds per cubic foot, the soils within this zone should be over-excavated and replaced as engineered fill. These soils should be tested prior to backfilling behind the wall.

8.5.4 A minimum factor of safety of 1.5 should be used for the lateral resistance, or as required by the governing building codes. The frictional and passive resistance of the soil may be combined in determining the total lateral resistance. The upper 12 inches of subgrade should be neglected in determining the total passive resistance.

8.5.5 The active and at-rest pressures of the native soils and engineered fill may be assumed to be equal to the pressures developed by a fluid with a density of 43 and 65 pounds per cubic foot, respectively. These pressures assume level ground surface and do not include the surcharge effects of construction equipment, loads imposed by nearby foundations and roadways and hydrostatic water pressure.

8.5.6 The active and at-rest pressures were calculated based on a maximum soil unit weight of 130 pounds per cubic foot. The compacted soils behind the retaining walls should not have a compacted unit weight above 130 pounds per cubic foot (with moisture). If the soils have a unit weight of greater than 130 pounds per cubic foot, the soils should be over-excavated and replaced at a lower degree of compaction. If the backfill soils must be placed at a unit weight of over 130 pounds per cubic foot to achieve minimum compaction requirements the material should not be used as backfill behind retaining walls.

8.5.7 The at-rest pressure should be used in determining lateral earth pressures against walls which are not free to deflect. For walls which are free to deflect at least one percent of the wall height at the top, the active earth pressure may be used.

8.6 Temporary Excavations

8.6.1 It is the responsibility of the contractor to provide safe working conditions with respect to excavation slope stability.

8.6.2 Temporary excavations should be constructed in accordance with CALOSHA requirements. Temporary cut slopes in weathered rock should not be steeper than 3/4H to 1V, and flatter if possible. Temporary cut slopes in lean clay or silty sand soils should not be steeper than 1H to 1V. If excavations can not meet this criteria, the temporary excavations should be shored.

8.6.3 Shoring systems, if used, should be designed by an engineer with experience in designing shoring systems and registered in the State of California.

8.7 Utility Trenches

8.7.1 The type of pipe bedding, the initial backfill and compaction requirements of bedding and initial backfill material should be specified by the project Civil Engineer based on either the manufacturers requirements, or ASTM D-2321 for flexible polyvinylchloride (PVC) pipe, whichever is more stringent.

8.7.2 Utility trench backfill placed in or adjacent to building areas, exterior slabs or pavements should be moisture conditioned to within two percent of the optimum moisture content and compacted to at least 92 percent of the maximum dry density as determined by ASTM Test Method D1557-78. The contractor should use appropriate equipment and methods to avoid damage to utilities and/or structures during placement and compaction of the backfill materials.

8.7.3 When utility trench backfills are determined by Twining to be nonstructural backfills, they should be compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM Test Method D1557-78.

8.7.4 Trench backfill should be placed in 8 inch lifts, moisture conditioned to within 2 percent of optimum and compacted to achieve the minimum relative compaction. Lift thickness can be increased if contractor can demonstrate the minimum compaction requirements can be achieved.

8.7.5 On-site soils and approved imported engineered fill may be used as final backfill in trenches.

8.7.6 Jetting of trench backfill is not recommended to compact the backfill soils.

8.8 Cut and Fill Slopes

8.8.1 For stability, permanent fill and cut slopes should be constructed at 2H to 1V, or flatter.

8.8.2 Where fill is placed on native slopes steeper than 5H to 1V, a minimum 6 foot wide keyway should be constructed at the toe of fill slopes.

8.8.3 Based on the nature of the slopes in the vicinity of the tank pad a minimum setback of 20 feet should be sufficient. This setback could be adjusted based on our review of the final grading plans. The setback should be measured between the bottom of the tank foundation, horizontally to the slope face.

8.8.4 Develop and maintain site grades which will drain surface and roof runoff away from the slopes both during and after construction.

8.8.5 The slopes should be graded to promote sheet type flow. Brow ditches should be constructed at the top of the cut slope to intercept potential runon water and channel it away from the slope faces.

8.8.6 A shallow rooted ground cover type of vegetation should be planted on the slopes to prevent erosion and aid in stability. Areas particularly susceptible to erosion and not amenable to successful vegetation should be protected with other techniques such as the use of jute netting or geotextile erosion control mats. Irrigation should be of a drip type or micro sprinkler system which does not generate surface runoff.

8.8.7 During earthwork operations, keyways should be observed by our firm to determine if the subsurface conditions are compatible with those used in our evaluation and design.

8.9 Corrosion Protection

8.9.1 Based on the ASTM Special Technical Publication 741 and the analytical results of a near surface soil sample, the soils range are "mildly corrosive". Buried metal objects should be protected in accordance with the manufacturer's recommendations based on a "corrosive" corrosion potential of the soil. The evaluation was limited to the effects of soils to metal objects; corrosion due to other potential sources, such as stray currents and groundwater, was not evaluated.

8.9.2 Corrosion of concrete due to sulfate attack is not anticipated based on concentration of sulfates indicating negligible exposure, as determined for the near-surface soils. Type I or II cement may be used as specified in Table No. 19-A-3 of the 1994 Uniform Building Code.

8.9.3 We recommend that these soil corrosion data be provided to the manufacturer's or supplier's of materials that will be in contact with soils (pipes or ferrous metal objects, etc.) to provide assistance in selecting the protection and materials for the proposed products or materials. If the manufacturer's or supplier's cannot determine if materials are compatible with the soil corrosion conditions, a professional consultant, i.e. a corrosion engineer, with experience in corrosion protection should be consulted to design parameters.

9.0 DESIGN CONSULTATION

9.1 Twining should be provided the opportunity to review those portions of the contract drawings and specifications that pertain to earthwork and foundations prior to finalization to determine whether they are consistent with our recommendations. This service is part of this current contractual agreement.

9.2 It is the client's responsibility to provide plans and specification documents for our review prior to their issuance for construction bidding purposes.

9.3 If Twining is not afforded the opportunity for review, we assume no liability for the misinterpretation of our conclusions and recommendations. This review is documented by a formal plan/specification review report provided by Twining.

10.0 CONSTRUCTION MONITORING

10.1 It is recommended that Twining be retained to observe the excavation, earthwork, and foundation phases of work to determine that the subsurface conditions are compatible with those used in the analysis and design.

10.2 Twining can conduct the necessary observation, field testing services and provide results so that action necessary to remedy indicated deficiencies can be taken in accordance with the plans and specifications. Upon completion of the work, we will provide a written summary of our observations, field testing and conclusions regarding the conformance of the completed work to the intent of the plans and specifications. This service is not, however, part of this current contractual agreement.

10.3 The construction monitoring is an integral part of this investigation. This phase of the work provides Twining the opportunity to verify the subsurface conditions interpolated from the soil test pits and make alternative recommendations if the conditions differ from those anticipated.

10.4 If Twining is not afforded the opportunity to provide engineering observation and field testing services during construction activities related to earthwork, foundations, pavements and trenches; then, Twining will not be responsible for compliance of any aspect of the construction with our recommendations or performance of the structures or improvements if the recommendations of this report are not followed. We recommend that if a firm other than Twining is selected to conduct these services that they provide evidence of professional liability insurance of at least $1,000,000 and review this report. After their review, the firm should, in writing, state that they understand and agree with the conclusions and recommendations of this report and agree to conduct sufficient observations and testing to ensure the construction complies with this report's recommendations. Twining should be notified, in writing, if another firm is selected to conduct observations and field testing services prior to construction.

10.5 Upon the completion of work, a final report should be prepared by Twining per the requirements of the Uniform Building Code, Chapter 33A, "Excavation and Grading," Section 3318.1, "Final Reports". This report is essential to ensure that the recommendations presented are incorporated into the project construction, and to note any deviations from the project plans and specifications. The client should notify Twining upon the completion of work to provide this report. This service is not, however, part of this current contractual agreement.

11.0 NOTIFICATION AND LIMITATIONS

11.1 The conclusions and recommendations presented in this report are based on the information provided regarding the proposed construction, and the results of the field and laboratory investigation, combined with interpolation of the subsurface conditions between test pit locations.

11.2 The nature and extent of subsurface variations between test pits may not become evident until construction.

11.3 If variations or undesirable conditions are encountered during construction, Twining should be notified promptly so that these conditions can be reviewed and our recommendations reconsidered where necessary. It should be noted that unexpected conditions frequently require additional expenditures for proper construction of the project.

11.4 If the proposed construction is relocated or redesigned, or if there is a substantial lapse of time between the submission of our report and the start of work (over 12 months) at the site, or if conditions have changed due to natural cause or construction operations at or adjacent to the site, the conclusions and recommendations contained in this report should be considered invalid unless the changes are reviewed and our conclusions and recommendations modified or approved in writing.

11.5 Changed site conditions, or relocation of proposed structures, may require additional field and laboratory investigations to determine if our conclusions and recommendations are applicable considering the changed conditions or time lapse.

11.6 The conclusions and recommendations contained in this report are valid only for the project discussed in Section 3.4, Proposed Construction. The use of the information and recommendations contained in this report for structures on this site not discussed herein or for structures on other sites not discussed in Section 3.3, Site Description is not recommended. The entity or entities that use or cause to use this report or any portion thereof for another structure or site not covered by this report shall hold Twining, its officers and employees harmless from any and all claims and provide Twining's defense in the event of a claim.

11.7 This report is issued with the understanding that it is the responsibility of the client to transmit the information and recommendations of this report to developers, owners, buyers, architects, engineers, designers, contractors, subcontractors, and other parties having interest in the project so that the steps necessary to carry out these recommendations in the design, construction and maintenance of the project are taken by the appropriate party.

11.8 This report presents the results of a geotechnical engineering investigation only and should not be construed as an environmental audit or study.

11.9 Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally-accepted engineering principles and practices in Santa Clara County as of June 1998. This warranty is in lieu of all other warranties either expressed or implied.

11.10 Reliance on this report by a third party (i.e., that is not a party to our written agreement) is at the party's sole risk. If the project and/or site is purchased by another party, the purchaser must obtain written authorization and sign an agreement with Twining in order to rely upon the information provided in this report for design or construction of the project.

We appreciate the opportunity to be of service to Lion's Gate Estate Partners, LLC. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience.

Sincerely,
THE TWINING LABORATORIES, INC.

REGISTERED GEOLOGIST
KENNETH JAMES CLARK
No. EG 1864
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA
Ex. 5-31-99

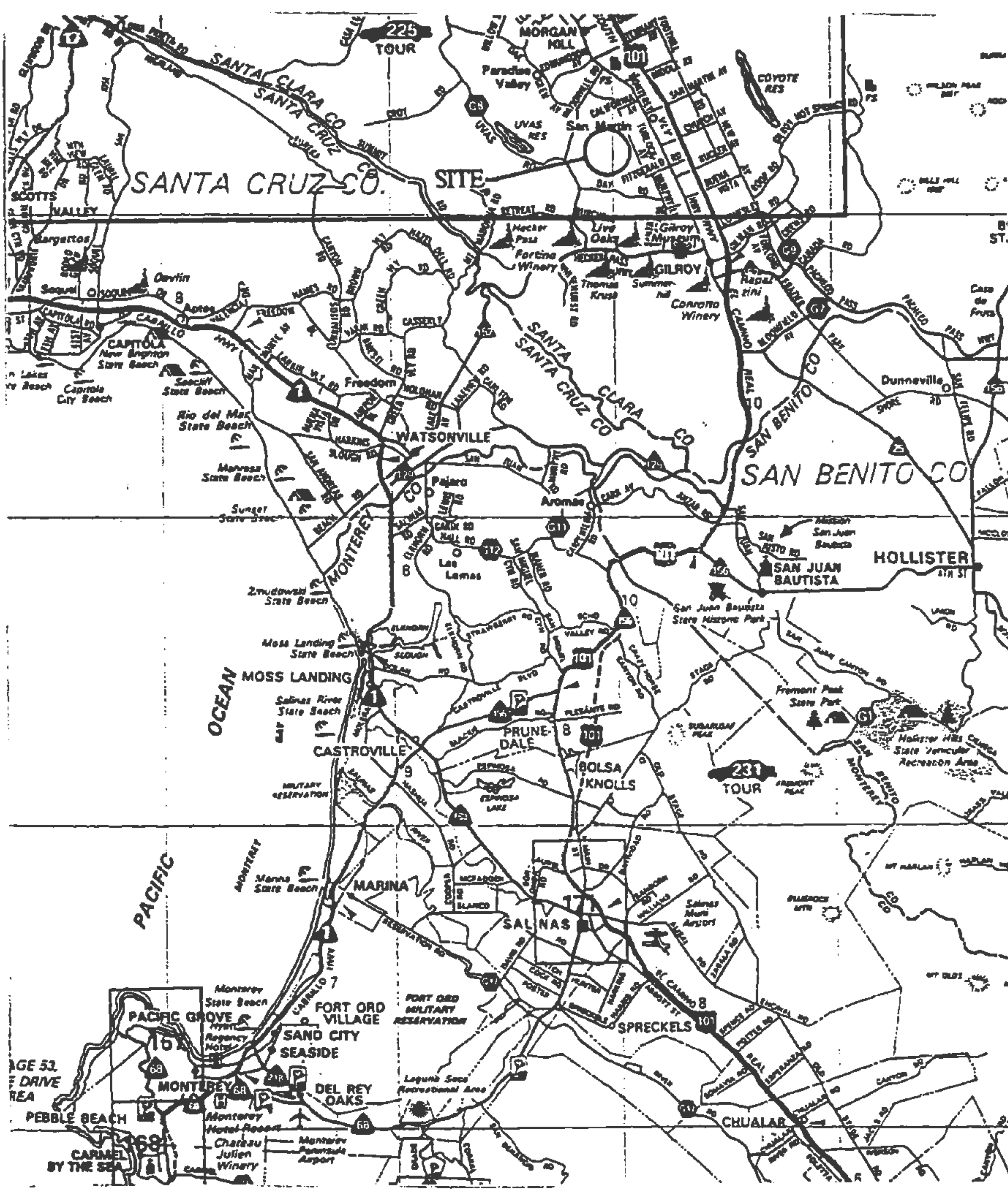
Kenneth J. Clark, CEG
Engineering Geologist
Geotechnical Engineering Division

REGISTERED PROFESSIONAL ENGINEER
SCOTT KRAUTER
No. GE2358
Exp. 12/31/02
GEOTECHNICAL
STATE OF CALIFORNIA

Scott W. Krauter, RCE, RGE
Manager
Geotechnical Engineering Division

APPENDIX A

DRAWINGS



SITE LOCATION MAP CORDEVALLE GOLF CLUB & HOTEL SAN MARTIN, CALIFORNIA	FILE NO.: 34301-02*	DATE: 8-11-98	THE TWINING LABORATORIES, INC. EST. 1896 FRESNO/MODESTO/VISALIA/BAKERSFIELD/SALINAS
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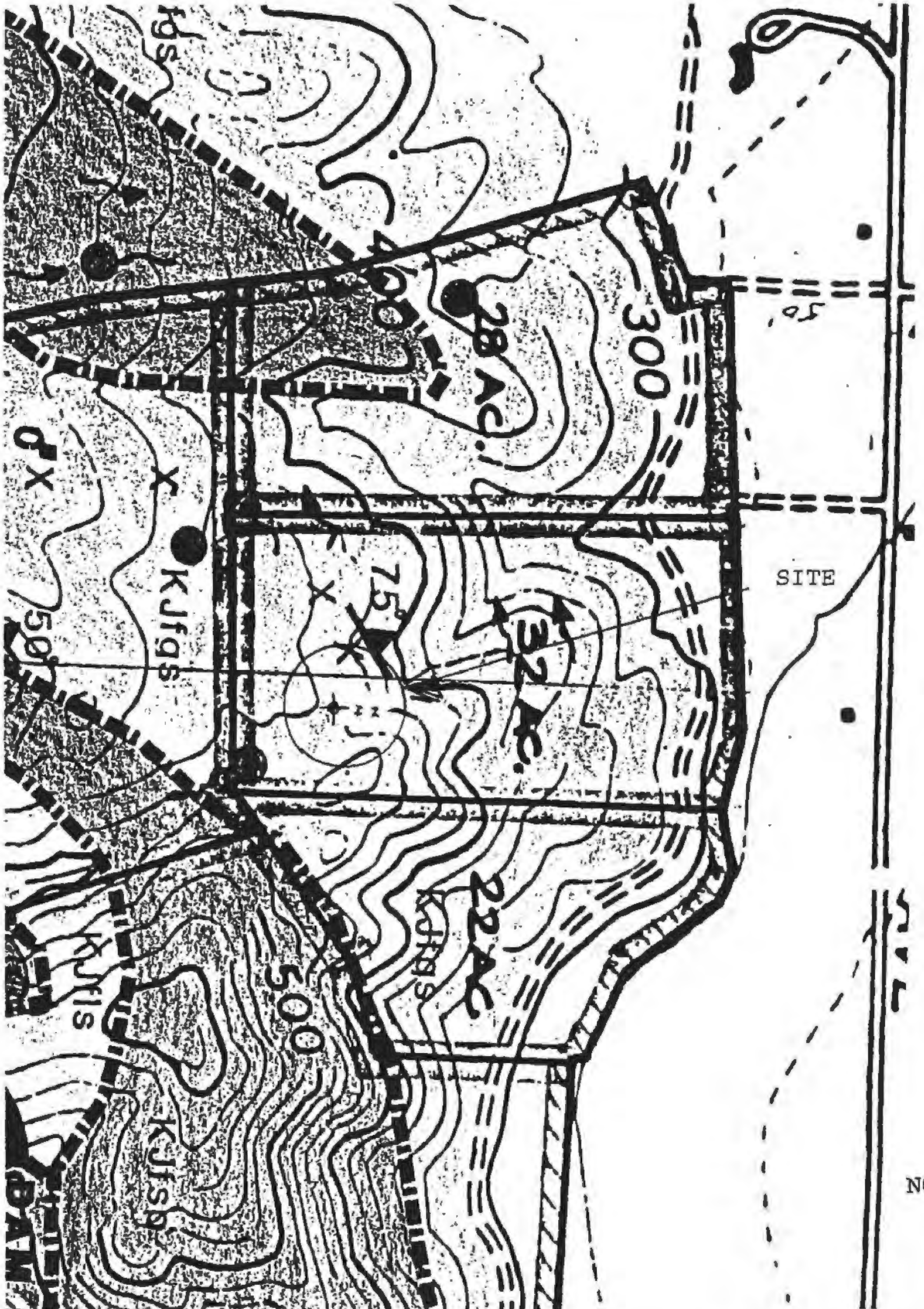
	PROJECT NO. D34301.02	DRAWING NO. 1	



TEST BORING   TEST PIT   SCALE: 1" = 500'

SITE PLAN WITH TEST BORING & TEST PIT LOCATIONS CORDEVALLE GOLF CLUB & HOTEL SAN MARTIN, CALIFORNIA	FILE NO.: 34301-02•	DATE: 8-11-98	THE TWINING LABORATORIES, INC. EST. 1898 FRESNO/MODESTO/VISALIA/BAKERSFIELD/SALINAS
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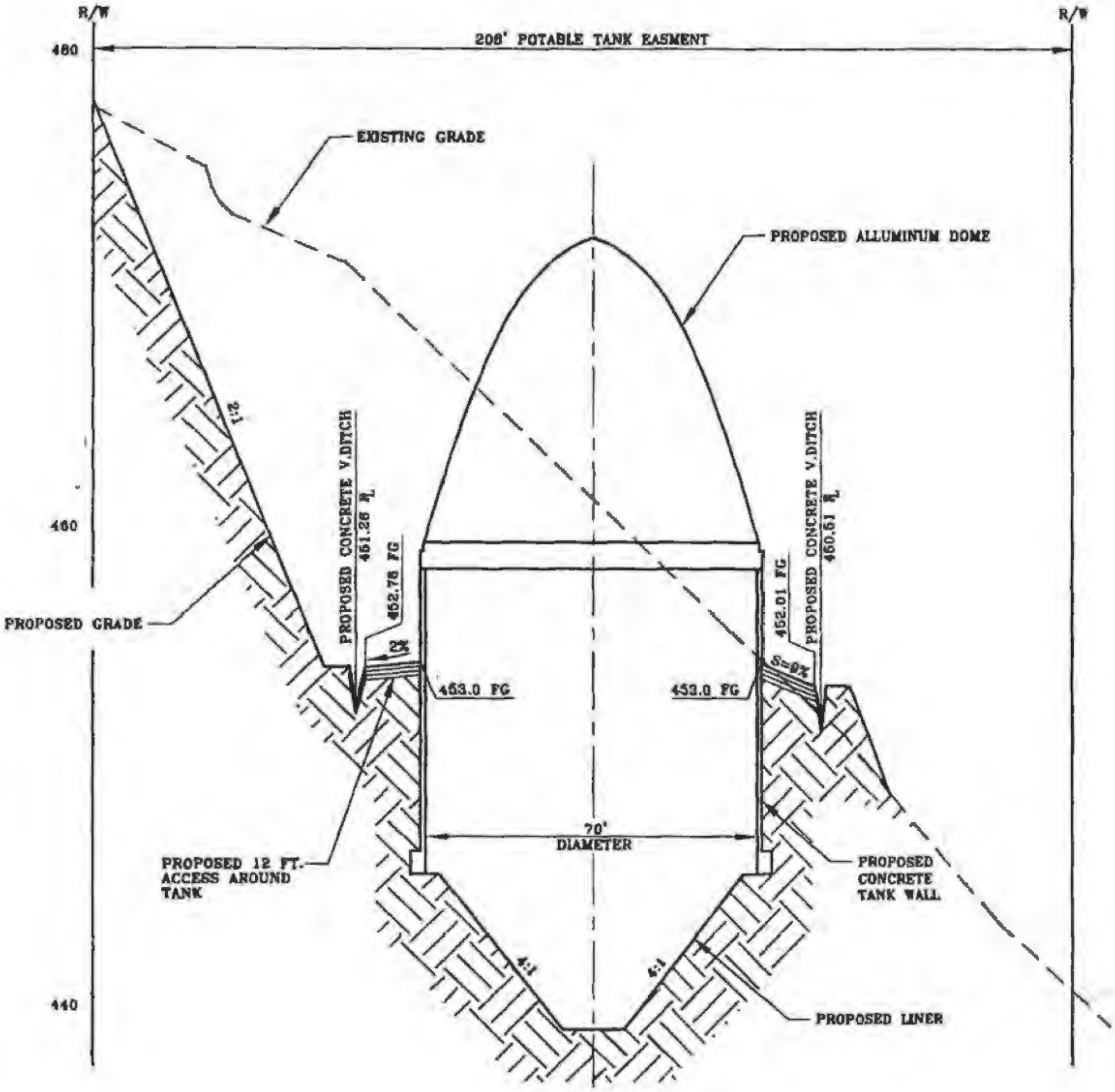
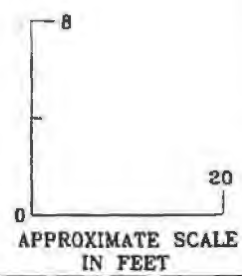
	PROJECT NO. D34301.02	DRAWING NO. 2	



TANK CROSS SECTION



TANK CROSS SECTION CORDEVALLE GOLF CLUB & HOTEL SAN MARTIN, CALIFORNIA	FILE NO.: 34301-02*	DATE: 8-10-98	THE TWINING LABORATORIES, INC. EST.1898 FRESNO/MODESTO/VISALIA/BAKERSFIELD/SALINAS
	DRAWN BY: PNG	APPROVED BY: RC	
	PROJECT NO. D34301.02	DRAWING NO. 3	

APPENDIX B

LOG OF TEST BORINGS AND PITS

This appendix contains the final logs of borings. These logs represent our interpretation of the contents of the field logs and the results of the field and laboratory tests.

The boring logs and related information depict subsurface conditions only at these locations and at the particular time designated on the logs. Soil conditions at other locations may differ from conditions occurring at these test boring locations. Also, the passage of time may result in changes in the soil conditions at these test boring locations.

In addition, an explanation of the abbreviations used in the preparation of the logs and a description of the Unified Soil Classification System are provided at the end of Appendix B.

THE TWINING LABORATORIES, INC. EST 1898

SOIL TEST BORING SYMBOLIC LOG

BORING B-1

Project: Cordevalle Potable Water Tank
Project Number: TL D34301.02A

Location: San Martin, CA
Date: 07/07/98

Logged By: M. Sekhon
Elevation: n/a

Drilled By: T. Conley
Depth to Groundwater: NE

Drill Type: CME 75
Cased to Depth: n/a

Auger Type: 6-5/8" OD HSA
Hammer Type: CME Trip

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
0		SM	SAND, Silty with gravels; very dense, slightly moist, fine-grained, light brown			
	17/6 25/6 32/6	CL	LEAN CLAY with gravels; hard, slightly moist, low plasticity, brown		57	2
5			gravel fraction increase			
	50/5			DD = 125.7 pcf ϕ = 28° c = 369 psf	>100	2
			weathered greenstone, very dense, damp, pale olive			
10	5/6 50/5				>100	4
15	28/6 40/2.5				>100	3
			stiffness decrease, moisture increase, gravel fraction decrease			
20	10/6 33/6 26/6				69	6
25	11/6 29/6 20/1				>100	2
30	7/6 12/6 22/6				34	

Notes: *Approximately 120 feet west of edge of tank.*

Figure Number B-1

THE TWINING LABORATORIES, INC.

EST 1898

SOIL TEST BORING SYMBOLIC LOG

BORING B-1

Project: Cordevalle Potable Water Tank
Project Number: TL D34301.02A
Location: San Martin, CA
Date: 07/07/98
Logged By: M. Sekhon
Elevation: n/a
Drilled By: T. Conley
Depth to Groundwater: NE
Drill Type: CME 75
Cased to Depth: n/a
Auger Type: 6-5/8" OD HSA
Hammer Type: CME Trip

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
35	50/5		water content increase, wet soil		>100	
40	24/6 30/6 46/6				76	
			Bottom of Boring			
45						
50						
55						
60						
65						

Notes: *Approximately 120 feet west of edge of tank.*

Figure Number B-1

THE TWINING LABORATORIES, INC.

SOIL TEST BORING SYMBOLIC LOG

BORING TP-1

Project: Cordevalle Potable Water Tank	**Project Number:** TL D34301.02A
Location: San Martin, CA	**Date:** 07/07/98
Logged By: K. Clark	**Elevation:** n/a
Drilled By: n/a	**Depth to Groundwater:** NE
Drill Type: Backhoe	**Cased to Depth:** n/a
Auger Type: n/a	**Hammer Type:** n/a

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
0		SM	SAND, Silty; damp, brown to yellow brown, rootlets and desication cracks			5
5		CL	LEAN CLAY, Sandy; moist, coarse sand with scattered gravel, moderate plasticity, reddish brown			13
10			Weathered rock, greenstone, highly weathered zones are silty clay, low plasticity, pale olive, very hard digging for backhoe at 9.5 feet below site grade			4
15			Bottom of Test Pit			
20						
25						
30						

Notes: *Center of proposed tank.*

SOIL TEST BORING SYMBOLIC LOG

THE TWINING LABORATORIES, INC. EST. 1898

BORING TP-2

Project: Cordevalle Potable Water Tank
Project Number: TL D34301.02A

Location: San Martin, CA
Date: 07/07/98

Logged By: K. Clark
Elevation: n/a

Drilled By: n/a
Depth to Groundwater: NE

Drill Type: Backhoe
Cased to Depth: n/a

Auger Type: n/a
Hammer Type: n/a

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
0		SM	SAND, Silty; damp, yellow brown, fine sand with some gravel, roots and desication cracks			
		CL	LEAN CLAY, gravelly; moist, reddish brown			
5			Weathered rock, greenstone, highly weathered zones are silty clay, low plasticity, pale olive, hard digging with backhoe at 6' below site grade			
10			Bottom of Test Pit			
15						
20						
25						
30						

Notes: *75 feet east of center of proposed tank location.*

KEY TO SYMBOLS

Symbol Description

Strata symbols

SAND, Silty (SM)

LEAN CLAY (CL)

Basalt (or generic rock)

Weathered Rock

Misc. Symbols

Boring continues

Soil Samplers

Standard penetration test

Symbol Description

California Modified split barrel ring sampler

Notes:

1. Test borings were drilled on 07/07/98 using a Backhoe equipped with n/a.
2. Groundwater was not encountered during drilling operations.
3. Boring locations were located by measuring wheel with reference to .
4. These logs are subject to the limitations, conclusions, and recommendations in this report.
5. Results of tests conducted on samples recovered are reported on the logs. Abbreviations used are:

DD = Natural dry density	LL = Liquid limit (%)
UC = Unconfined compression (psf)	PI = Plasticity index (%)
-4 = Percent passing #4 sieve (%)	pH = Soil pH
-200 = Percent passing #200 sieve (%)	SS = Soluble sulfates (%)
SR = Soil resistivity (ohm-cm)	Cl = Soluble chlorides (%)
c = Cohesion (psf)	ϕ = Angle of internal friction (degrees)
TS = Field Torvane Shear Strength test (tsf)	NE = None Encountered
ND = None Detected	

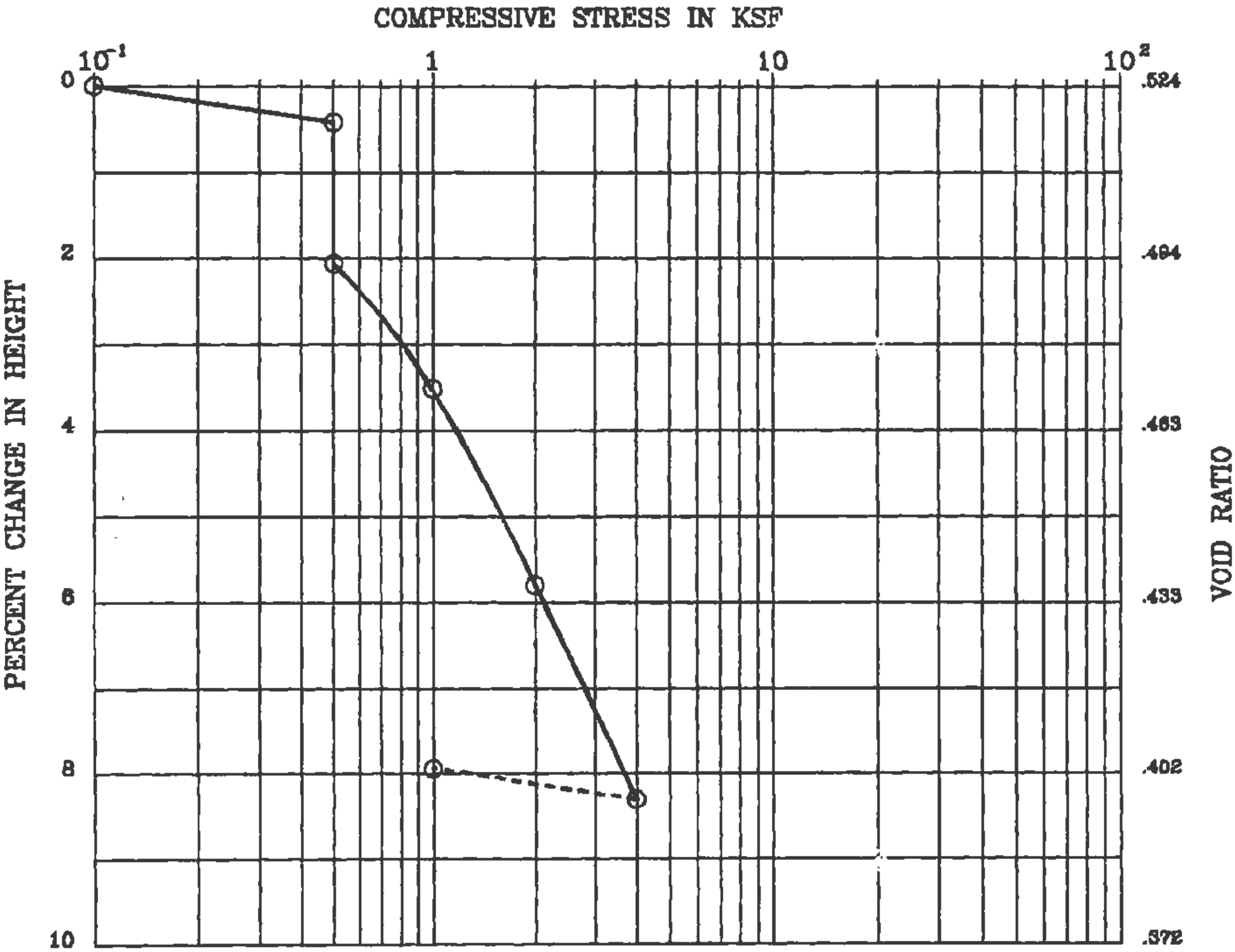
APPENDIX C

RESULTS OF LABORATORY TESTS

This appendix contains the individual results of the following tests. The results of the moisture content and dry density tests are included on the test boring logs in Appendix B. These data, along with the field observations, were used to prepare the final test boring logs in Appendix B.

These Included:	Number of Tests:	To Determine:
Natural Moisture (ASTM D2216)	9	Moisture contents representative of field conditions at the time the sample was taken.
Natural Density (ASTM D2216)	1	Dry unit weight of sample representative of in-situ or in-place undisturbed condition.
Direct Shear (ASTM D3080)	1	Soil shearing strength under varying loads and/or moisture conditions.
Consolidation (ASTM D2435)	1	The amount and rate at which a soil sample compresses when loaded, and the influence of saturation on its behavior.
Moisture-Density Relationship (ASTM D1557)	1	The optimum (best) moisture content for compacting soil and the maximum dry unit weight (density) for a given compactive effort.

These Included:	Number of Tests:	To Determine:
Sulfate Content (ASTM D4327)	1	Percentage of water-soluble sulfate as (SO_4) in soil samples. Used as an indication of the relative degree of sulfate attack on concrete and for selecting the cement type.
Chloride Content (ASTM D4327)	1	Percentage of soluble chloride in soil. Used to evaluate the potential attack on encased reinforcing steel.
Resistivity (ASTM D1125)	1	The potential of the soil to corrode metal.
pH (ASTM D4972)	1	The acidity or alkalinity of subgrade material.



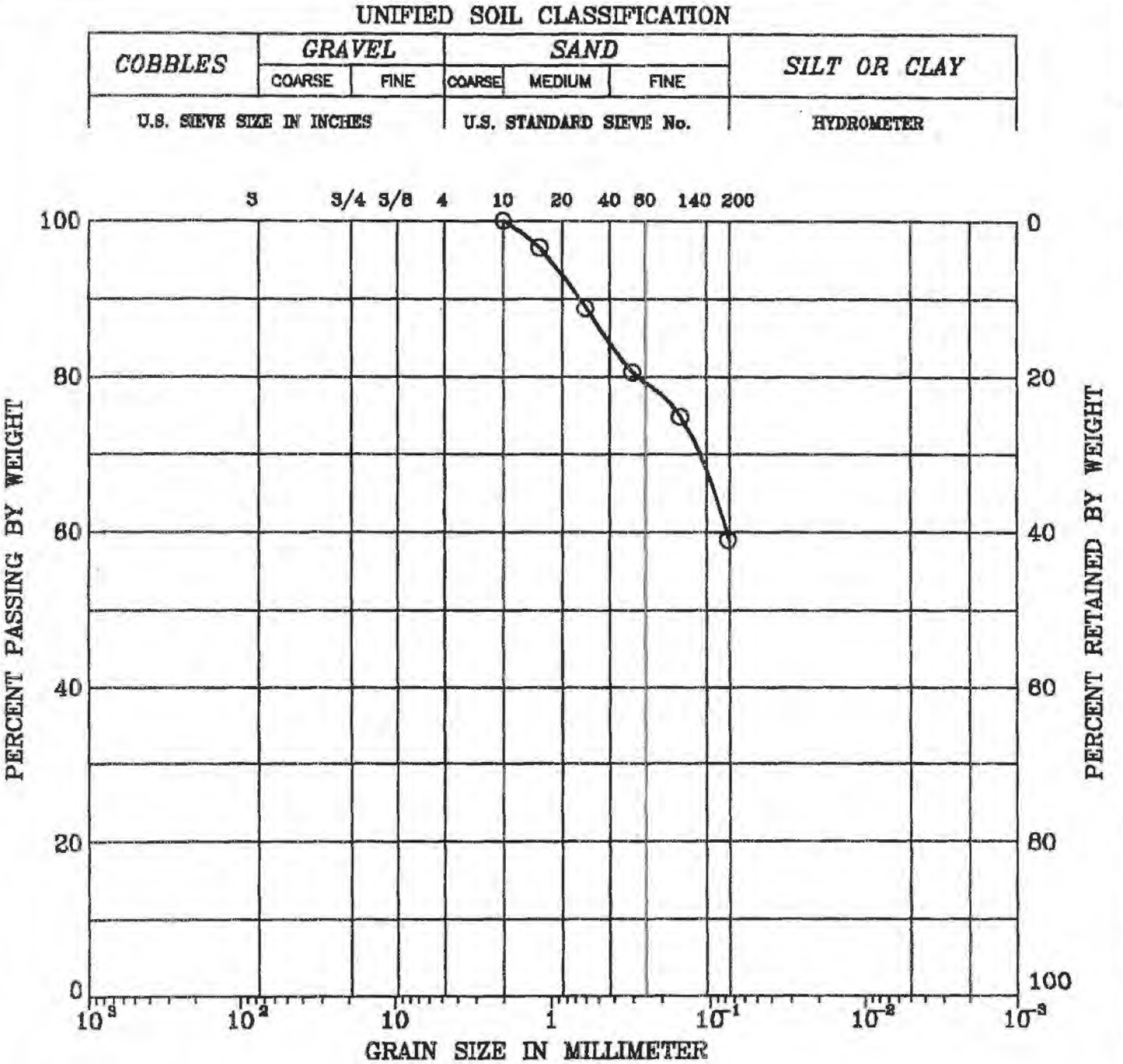
BORING : B-1
DEPTH (ft) : 5.0-5.5
SPEC. GRAVITY : 2.73

DESCRIPTION :
LIQUID LIMIT :
PLASTIC LIMIT :

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	12.7	110.7	64	.524
FINAL	15.3	120.2	100	.404

Remark : TEST METHOD: ASTM D2435 MOISTURE INCREASE AT 0.5 KSF

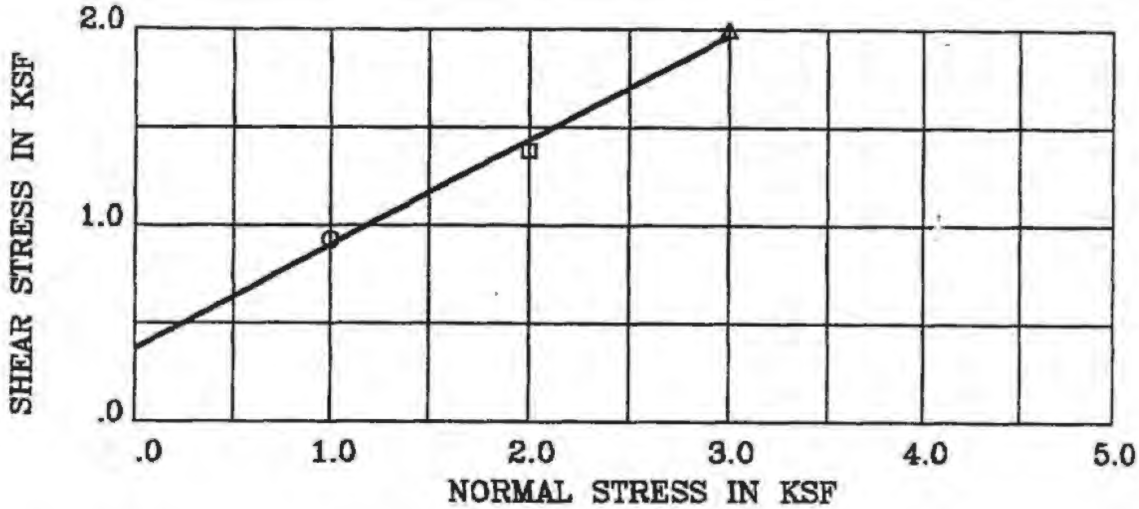
D34301.02	PROPOSED WATERTANK
The Twining Labs Inc. Fresno, CA	CONSOLIDATION TEST Figure No. 1

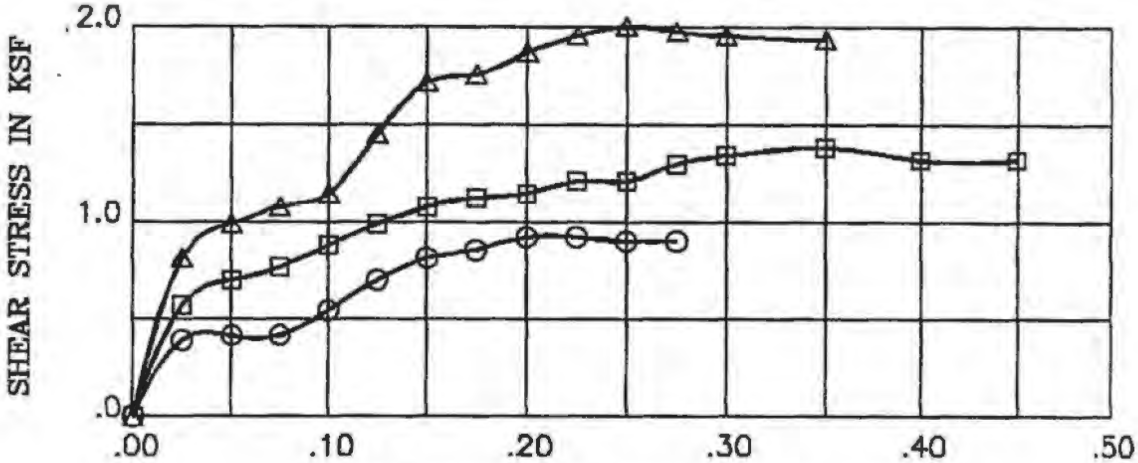


SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
O	TP-1	4.0-5.3			SILT, Sandy (ML)

Remark : TEST METHOD: ASTM D422

D34301.02	PROPOSED WATERTANK
The Twining Labs Inc. Fresno, CA	GRAIN SIZE DISTRIBUTION Figure No. 2





BORING/SAMPLE : B-1 DEPTH (ft) : 5-5.5

DESCRIPTION :

STRENGTH INTERCEPT (C) : .369 KSF

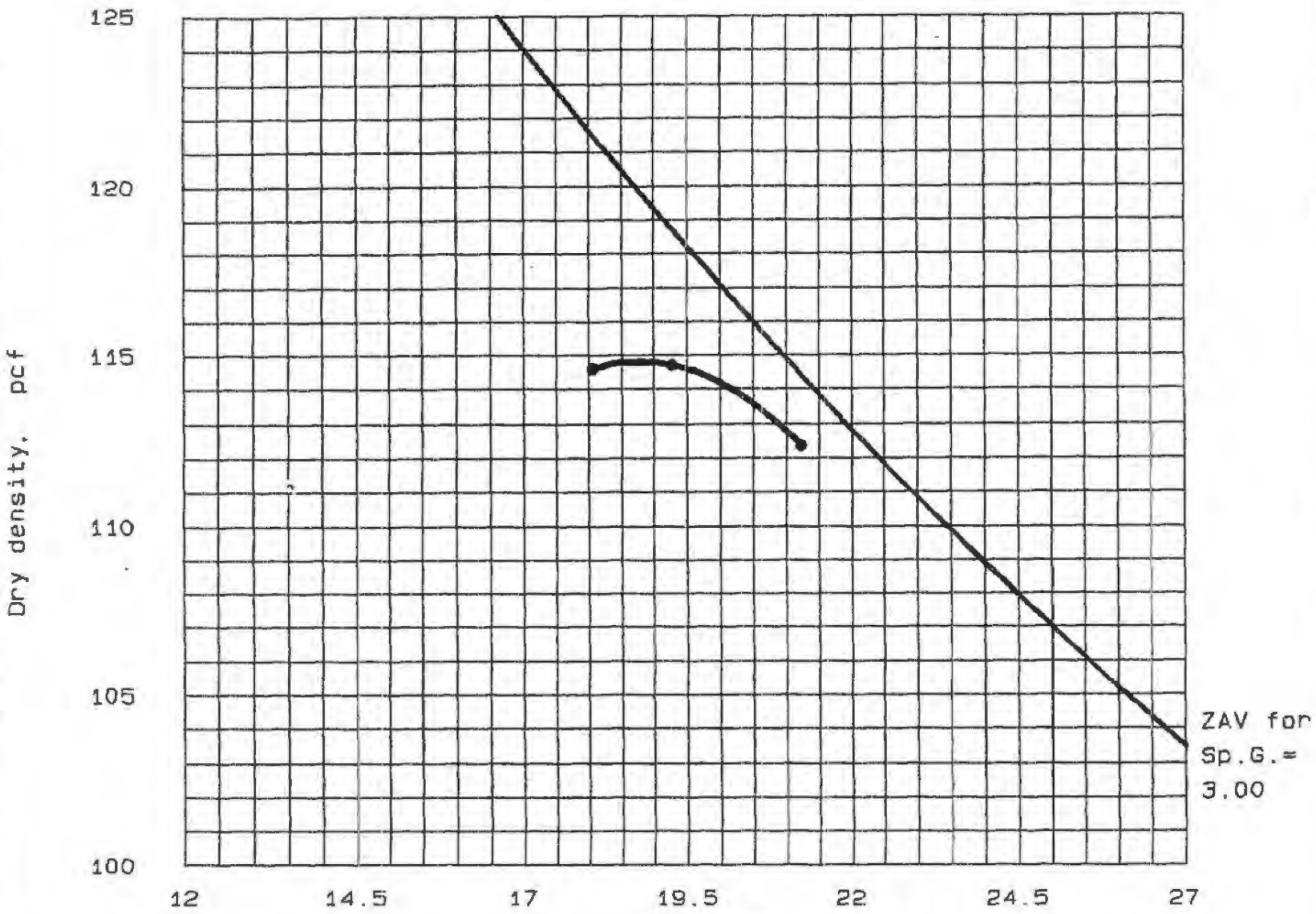
FRICTION ANGLE (PHI) : 28.1 DEG (PEAK STRENGTH)

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	VOID RATIO	NORMAL STRESS (ksf)	PEAK SHEAR (ksf)	RESIDUAL SHEAR (ksf)
○	28.7	103.6	.626	1.00	.93	.90
□	30.8	104.6	.611	2.00	1.38	1.31
△	31.2	102.3	.647	3.00	2.00	1.93

Remark : TEST METHOD: ASTM D3080

D34301.02	PROPOSED WATERTANK
The Twining Labs Inc. Fresno, CA	DIRECT SHEAR TEST Figure No. 3

PROCTOR TEST REPORT



"Modified" Proctor, ASTM D 1557, Method A

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
2-6.0				2.70				

TEST RESULTS	MATERIAL DESCRIPTION
Optimum moisture = 18.8 % Maximum dry density = 114.8 pcf	CLAY

Project No.: D34301.02

Project: PROPOSED WATERTANK

Location: TP-1

SAN MARTIN, CALIFORNIA

Date: AUGUST 6, 1998

Remarks:

PROCTOR TEST REPORT

THE TWINING LABORATORIES, INC.

Figure No. 4

APPENDIX F

Biological Report

Prepared by

H.T. Harvey & Associates

May 1998

H.T. HARVEY & ASSOCIATES
ECOLOGICAL CONSULTANTS

29 May 1998

Mr. Bert Verrips
Nolte and Associates, Inc.
1 N. First Street, Suite 450
San Jose, CA 95113
voice: 510.652.1666
facsimile: 510.547.6677

SUBJECT: Hayes Valley (Lions Gate): reconnaissance-level biotic constraints survey

Dear Mr. Verrips:

We have finished our reconnaissance-level field survey of the project modification areas. Two specific areas were surveyed, including: (1) the newly-proposed location of the water tank/access road, and; (2) winery site. The purpose of our survey was to determine if these proposed changes to the original project resulted in significant impacts to biotic resources on site. Survey personnel included Dr. Patrick Boursier, plant ecologist. A detailed project description and field review of each location was supplied by Mr. Ron Davis. Both of these two sites occur within the project boundaries intensively surveyed by H. T. Harvey & Associates staff in 1994-95 in preparation of our report entitled *Hayes Valley, Biological Resources Report* (30 Nov 95; PN 385-11). Each of the project modification sites are discussed below.

1. Water Tank/Access Road: The water tank/access road complex occurs within a habitat identified in our 1995 report as non-native annual grassland, and valley oak woodland. The access road will utilize a currently-existing, unimproved dirt road. The access road will cross a single seasonal drainage channel with an existing culverted crossing. The road and crossing will be upgraded to handle increased traffic and may result in relatively minor impacts to seasonal wetland habitats within the drainage (on the order of 10-25 square feet). It is our understanding that the steep portion of the existing road will remain as dirt or gravel during and after construction, some minor tree trimming of lower branches may be necessary to create a greater clearance for construction vehicles, no trees will be removed, and the only impact will include relatively minor loss of non-native annual grassland associated with the footprint of the proposed water tank. This proposed modification will not result in any additional direct or indirect impacts to biotic resources.
2. Winery Site: The winery site which includes a wine processing facility and minor planting of vineyards for aesthetic purposes occurs within the non-native annual grassland habitat. Our understanding is that no trees will be removed and

☐ Alviso Office
906 Elizabeth Street • P.O. Box 1180
Alviso, CA 95002 • 408-263-1814 • Fax: 408-263-3823

☐ Fresno Office
423 West Fallbrook, Suite 206
Fresno, CA 93711 • 209-449-1423 • Fax: 209-449-8248

construction will not result in any additional impacts to wetland habitats. This proposed modification will not result in any additional direct or indirect impacts to biotic resources.

In summary, the proposed modifications discussed above will not result in significant impacts to existing biological resources, beyond those already identified and addressed in the approved Environmental Impact Report.

If you our your staff have any questions please feel free to contact me or Rick Hopkins.

Sincerely,

Patrick J. Boursier, Ph.D.
Division Head, Botany and Wetlands

APPENDIX G

Archaeological Report

Prepared by

Basin Research Associates

May 1998

BASIN
RESEARCH
ASSOCIATES

1933 DAVIS STREET
SUITE 210
SAN LEANDRO, CA 94577
VOICE (510) 430-8441
FAX (510) 430-8443

29 May, 1998

Mr. Bert Verrips
Nolte and Associates
1 North First Street
Suite 450
San Jose, CA 95113

RE: Review of Previous Cultural Resources Studies
Proposed Location of Water Tank and Winery
Lions Gate/Cordevalle Project, Santa Clara County

Dear Mr. Verrips,

Please let this letter serve as our review of the proposed location changes for the Water Tank and Winery for the above project.

As you are aware, the project is situated in an area which has undergone a number of archival reviews and archaeological inventories as a result of cultural resource compliance requirements. Four archaeological sites, CA-SCl-76, SCl-77, SCl-305/H and SCl-568 have been recorded within the boundaries of the proposed project although only one prehistoric site, CA-SCl-76, was relocated during the various field programs. This site was also the subject of a presence/absence testing program to determine its horizontal and vertical extent [Fig. 1]. The three other reported sites for the project area, CA-SCl-77, SCl-305/H and SCl-568, did not have any visible surface indicators of a prehistoric occupation at their recorded location nor did auger testing expose the presence of subsurface cultural materials at their reported locations.

A review of the archival material on file at our office for the project indicates that none of the planned changes for the location of the Water Tank and Winery will affect any known cultural resources. Both locations are within areas that were previously subject to an archaeological inventory with negative results.

It is Basin Research Associates' considered opinion that the construction planned for the project can proceed as planned. No further archaeological research appears necessary and monitoring during subsurface construction at either the Water Tank or Winery does not appear warranted. It is recommended that if any unanticipated prehistoric or significant historic era cultural materials are exposed during construction, operations should stop within 20 feet of the find and a qualified professional archaeologist contacted for evaluation and further recommendations. Potential recommendations could include evaluation, collection, recordation, analysis, etc. of any significant cultural materials followed by a professional report.[1]

---

1. Significant prehistoric cultural resources are defined as human burials, features or other clusterings of finds made, modified or used by Native American peoples in the past. The prehistoric and protohistoric indicators of prior cultural occupation by Native Americans include artifacts and human bone, as well as soil discoloration, shell, animal bone, sandstone cobbles, ashy areas, and baked or vitrified clays. Prehistoric materials may

If I can provide any additional information or be of further service please don't hesitate to contact me.

Sincerely yours,
BASIN RESEARCH ASSOCIATES, INC.

Colin I. Busby
Principal

CIB/dg

---

include:

a. Human bone - either isolated or intact burials.
b. Habitation (occupation or ceremonial structures as interpreted from rock rings/features, distinct ground depressions, differences in compaction (e.g., house floors).
c. Artifacts including chipped stone objects such as projectile points and bifaces; groundstone artifacts such as manos, metates, mortars, pestles, grinding stones, pitted hammerstones; and, shell and bone artifacts including ornaments and beads.
d. Various features and samples including hearths (fire-cracked rock; baked and vitrified clay), artifact caches, faunal and shellfish remains (which permit dietary reconstruction), distinctive changes in soil stratigrapby indicative of prehistoric activities.
e. Isolated artifacts

Historic cultural materials may include finds from the late 19th through early 20th centuries. Objects and features associated with the Historic Period can include.

a. Structural remains or portions of foundations (bricks, cobbles/boulders, stacked field stone, postholes, etc.).
b. Trash pits, privies, wells and associated artifacts.
c. Isolated artifacts or isolated clusters of manufactured artifacts (e.g., glass bottles, metal cans, manufactured wood items, etc.).
d. Human remains.

In addition, cultural materials including both artifacts and structures that can be attributed to Hispanic, Asian and other ethnic or racial groups are potentially significant. Such features or clusters of artifacts and samples include remains of structures, trash pits, and privies.

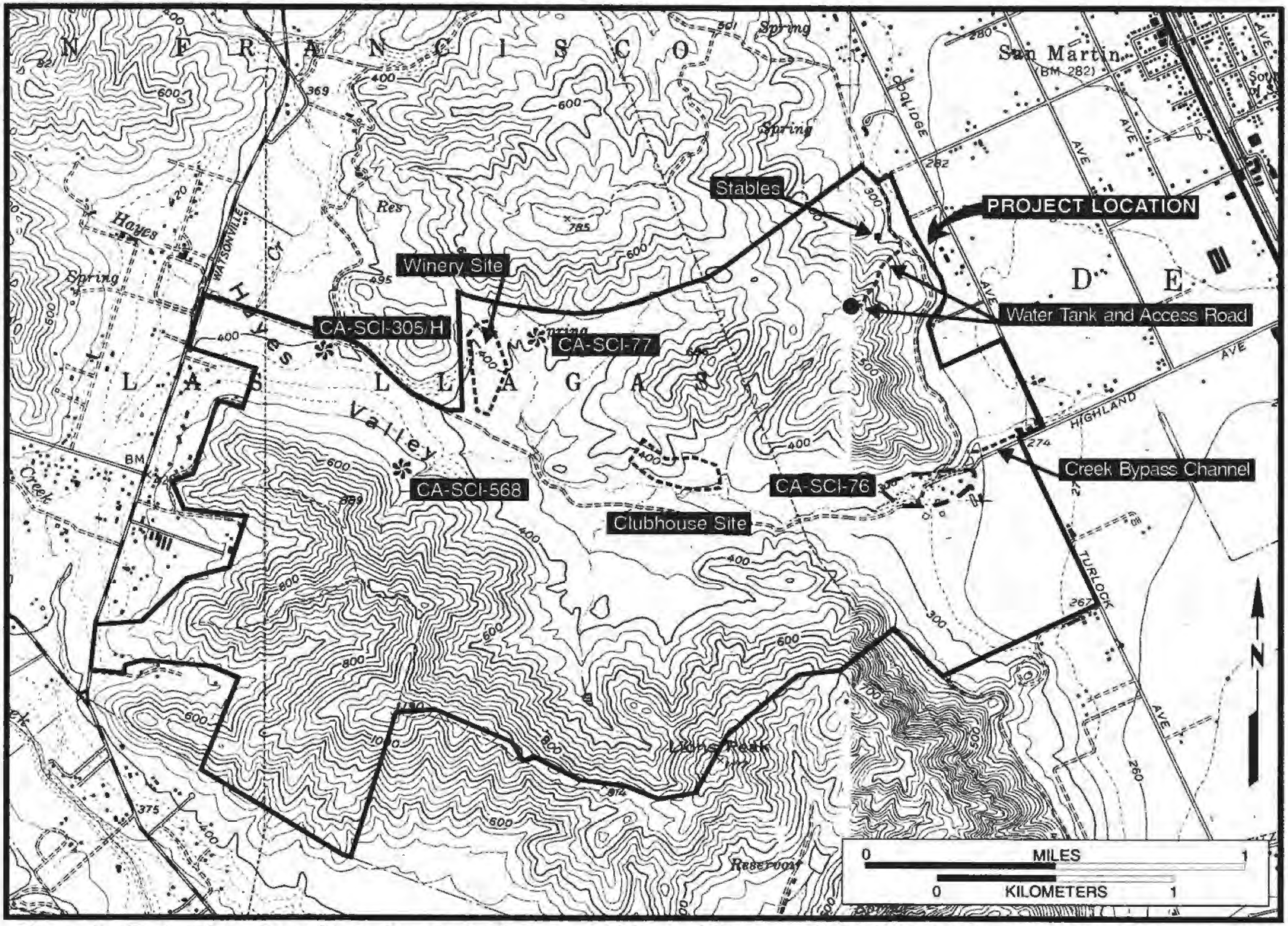


Figure 1: Project Location with Archaeological Sites and Planned Changes (USGS Mt. Madonna, Calif. 1980 and Gilroy, Calif. 1981)

TREE MITIGATION TABLE

THE FOLLOWING SPECIES/QUANTITIES OF TREES HAVE BEEN INSTALLED WITHIN THE GOLF COURSE AREA:

SPECIES	SIZE	QUANTITY
QUERCUS AGRIFOLIA	72" 84" 96" 108" 36"	2 6 3 4 6
QUERCUS RUBRUM		10
PLATANUS SPP.		65
SALIX SPECIES		50
QUERCUS AGRIFOLIA	15GAL	250
QUERCUS LOBATA	15GAL	100

WETLAND MITIGATION TABLE

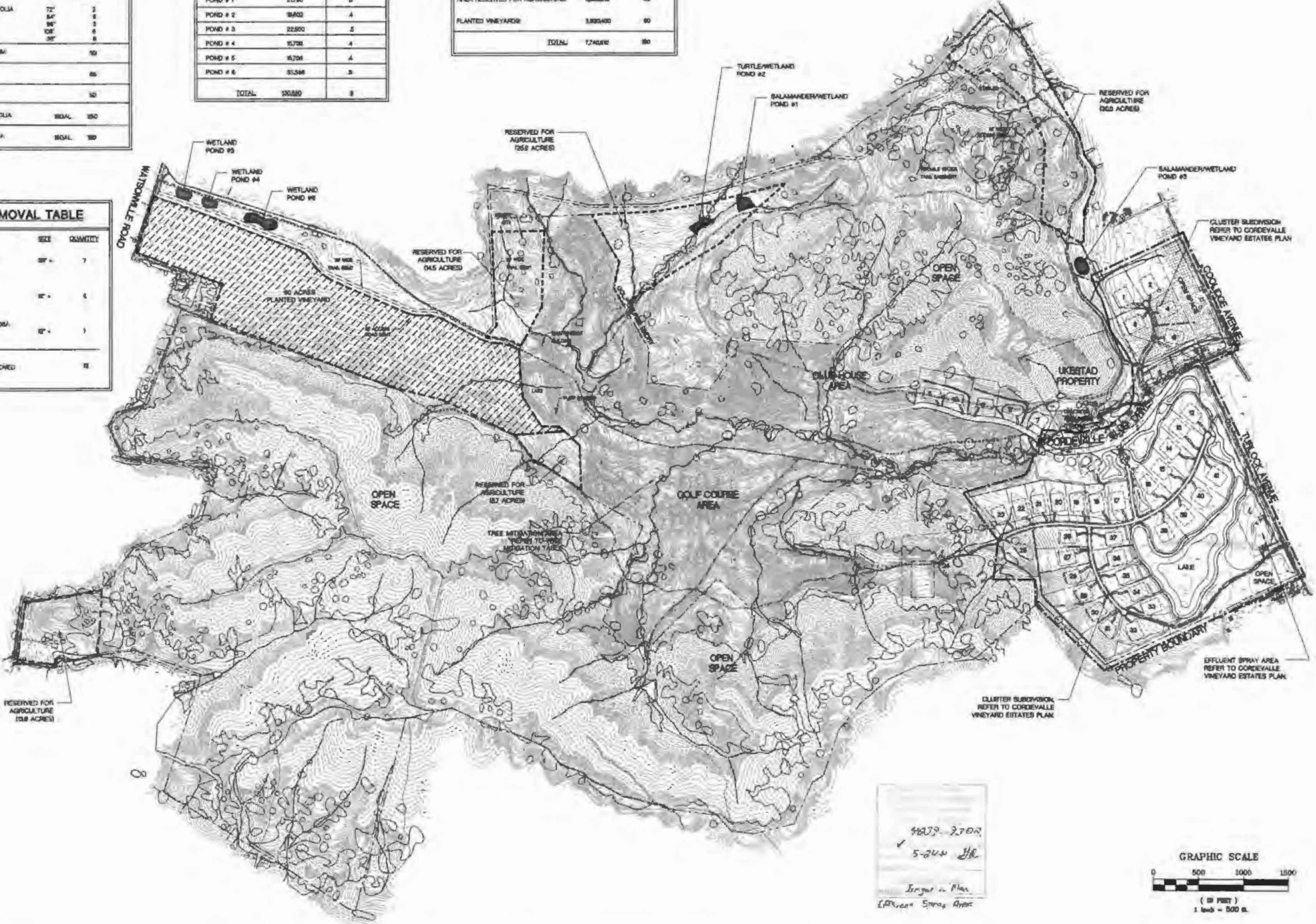
POND	AREA (SQ. SF.)	AREA (ACRE)
POND # 1	21,780	.5
POND # 2	18,402	.4
POND # 3	22,500	.5
POND # 4	15,738	.4
POND # 5	15,708	.4
POND # 6	33,348	.8
TOTAL	130,680	3

AGRICULTURE MITIGATION TABLE

DESCRIPTION	AREA (SQ. SF.)	AREA (ACRE)
AREA RESERVED FOR AGRICULTURE:	3,603,372	90
PLANTED VINEYARDS:	3,920,400	90
TOTAL:	7,740,800	180

TREE REMOVAL TABLE

SPECIES	SIZE	QUANTITY
QUERCUS LOBATA (VALLEY OAK)	30" +	7
SALIX SPP. (WILLOW)	12" +	4
PLATANUS RACEMOSA (SYCAMORE)	12" +	1
TOTAL TREES REMOVED		12



4039-9702 ✓
5-24-00 JR
Irrigat... Plan
Effluent Spray Area

GRAPHIC SCALE
0 500 1000 1500
(IN FEET)
1 inch = 500 ft.

RIPLEY DESIGN GROUP

Landscape Architecture
Land Planning

1646 Mt. Diablo Boulevard
Walnut Creek
California 94596
Tel 925.938.7377
Fax 925.938.7436

DEVELOPER:

CORDEVALLE VINEYARD ESTATES L.L.C

2001 WINWARD WAY,
SUITE 200
SAN MATEO, CA
94404

TEL. 650.377.5100
FAX 650.349.2442

PROJECT:

CORDEVALLE

SAN MARTIN, CALIFORNIA

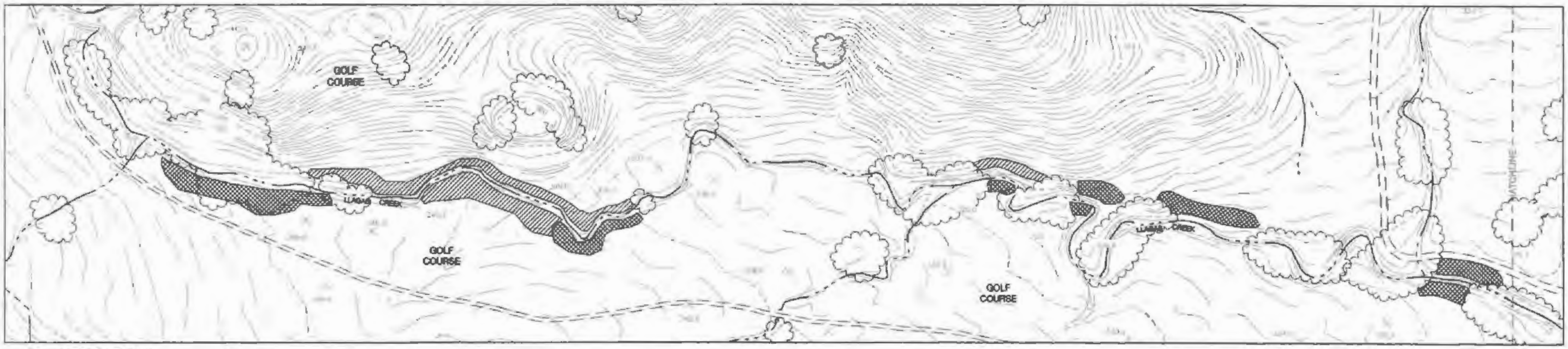
MITIGATION PLAN

LICENSED LANDSCAPE ARCHITECT
JAMES A. RIPLEY
State of California

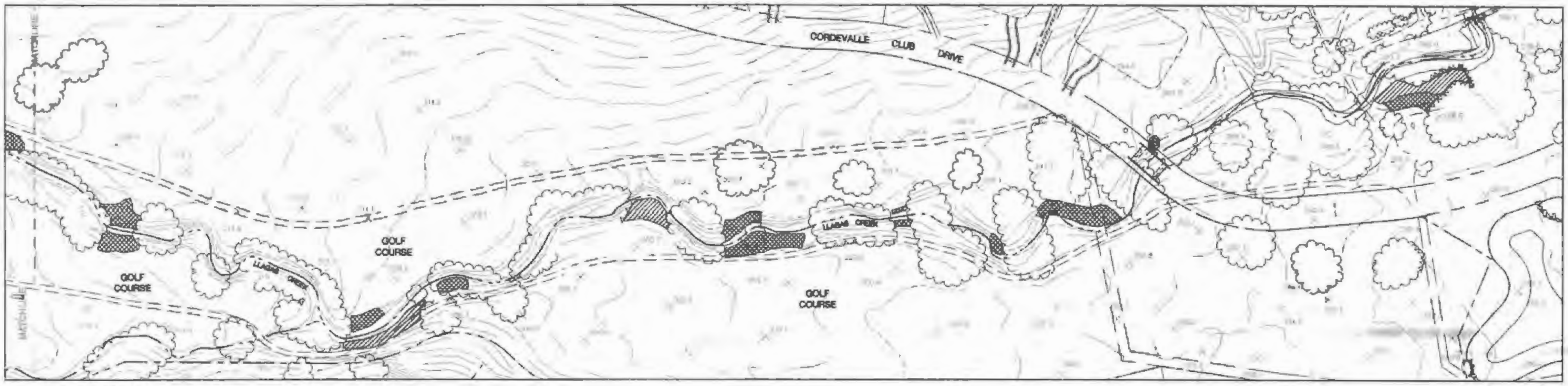
PROJECT #: 200699
DATE: April 24, 2000
SCALE: 1" = 500'-0"
DRAWN BY: GD
CHECKED BY: JR

REVISIONS:

SHEET
1
OF 1 SHEETS



WESTERN HALF



EASTERN HALF

LOCATION OF TREE AND SHRUB PLANTING AREAS

BOTANICAL NAME	COMMON NAME	AVERAGE SPACING	PLANT SIZE
TREES			
PLATANUS RACEMOSA	SYACAMORE	25 FEET	CITRUS POT OR 15 GAL. CONTAINER
QUERCUS LOBATA	VALLEY OAK	25 FEET	CITRUS POT OR 15 GAL. CONTAINER
SHRUBS			
BACCHARIS SALICIFOLIA	MULEFAT	5 FEET	DEEP POT OR 1 GAL. CONTAINER
RHAMNUS CALIFORNICA	CALIFORNIA COFFEE-BERRY	5 FEET	DEEP POT OR 1 GAL. CONTAINER
ROSA CALIFORNICA	CALIFORNIA ROSE	5 FEET	DEEP POT OR 1 GAL. CONTAINER
RUBUS URSINUS	CALIFORNIA BLACKBERRY	5 FEET	DEEP POT OR 1 GAL. CONTAINER
SYMPHORICARPOS ALBUS SSP.LAEVIGATUS	SNOWBERRY	5 FEET	DEEP POT OR 1 GAL. CONTAINER

EXISTING TREES

NOTE: ALL INFORMATION DERIVED FROM CONCEPTUAL CREEK REVEGETATION/ENHANCEMENT PLAN - LION'S GATE DEVELOPMENT PROJECT, SANTA CLARA COUNTY
BY LSA ASSOCIATES, INC.
MARCH 4, 1998

GRAPHIC SCALE
0 100 200 250
(IN FEET)
1 inch = 100 ft.

N

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1614 Mt. Diablo Boulevard
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Tel 925.938.7377
Fax 925.938.7436

DEVELOPER:

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2001 WINWARD WAY,
SUITE 200
SAN MATEO, CA
94404

TEL. 650.377.0300
FAX 650.349.2442

PROJECT:

CORDEVALLE

SAN MARTIN, CALIFORNIA

LLAGAS CREEK RIPARIAN HABITAT RESTORATION & ENHANCEMENT PLAN

LICENSED LANDSCAPE ARCHITECT
JAMES A. RIPLEY
State of California

PROJECT #: 200699
DATE: April 24, 2000
SCALE: 1" = 100'-0"
DRAWN BY: GD
CHECKED BY: JR

REVISIONS:

SHEET
1
OF 1 SHEETS

File 4039-97EIR