City of Santa Cruz/Soquel Creek Water District
Alternative Water Supply Study

EXECUTIVE SUMMARY

March 2002

in Association with
Black and Veatch Engineers and
Hopkins Groundwater Consultants

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CITY OF SANTA CRUZ/SOQUEL CREEK WATER DISTRICT
ALTERNATIVE WATER SUPPLY STUDY

EXECUTIVE SUMMARY

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EXECUTIVE SUMMARY

The City of Santa Cruz (City) needs a new water supply to maintain service to its customers during drought conditions. The Soquel Creek Water District (District) also has identified a need for a new supply to supplement its groundwater source and meet its “every day,” non-drought demands. The City and the District have jointly considered the possible viability of regional water supply alternatives that could meet their respective needs.

This executive summary discusses the findings from an evaluation of two water supply alternatives considered potentially viable: ocean-water desalination and wastewater reclamation.

INCREMENTAL ADDITIONAL SUPPLY

Several current and future drought and nondrought scenarios were examined to bracket the range of incremental supply needs for the City and the District. For the purposes of conceptual planning and cost estimating, the lower capacity was set at 2 million gallons per day (mgd), and the upper limit was set at 6 mgd (± 2,300 to 6,800 AF/yr). This range covers the expected supply needs for the City and the District.

DESALINATION PROJECT

Facility Siting

The following criteria were used to identify and screen potential sites for a new desalination treatment facility:

- Proximity to intake facilities and brine disposal sites.
- Proximity to distribution system infrastructure.
- Land requirement of 2 to 3 acres.

The following sites were considered:

- Terrace Point.
- City Industrial Park.
- Moss Landing.

Based on the reconnaissance level evaluation of the site alternatives, the Terrace Point and Industrial Park locations are considered viable. Land is potentially available at either location and both sites offer similar advantages with respect to proximity to intake, brine disposal, and the distribution system.

Additional engineering analyses will be required to confirm an actual site alternative if the project is implemented.

Facility and Infrastructure Requirements

Facility and infrastructure requirements for desalination include raw water intake; raw water pipelines and pumps; desalination treatment processes; brine disposal piping and appurtenant ocean discharge facilities; finished water pipelines and pumps; and power supply for the treatment facility.

Facility Sizing

Facilities must be sized to account for the fact that desalination treatment is not 100 percent efficient. For example, approximately 5 percent of influent water to the treatment process is "lost" as a nonrecoverable process waste stream, and an additional 55 percent is rejected as brine. Table ES.1 summarizes the sizing of each of the conveyance facilities taking account of process inefficiencies.

Table ES.1 Facility Sizing Requirements
Evaluation of Regional Water Supply Alternatives
City of Santa Cruz/Soquel Creek Water District

Facility	Capacity
Intake/Raw Water Pumps and Piping	4.7 to 14 mgd (5,300 to 15,800 AF/yr)
Brine Disposal Piping and Facilities	2.4 to 7.3 mgd (2,800 to 8,300 AF/yr)
Treated Water Pumps and Piping	2 to 6 mgd (2,300 to 6,800 AF/yr)

Intake Facilities

Beach wells and direct ocean intakes are common raw water intake systems. Beach well intakes are often preferred; however, this method was not selected because the beaches in the area are fine-grained materials that cannot provide sufficient raw water capacity. The direct-ocean intake was selected as the apparent most viable alternative.

For the purposes of this evaluation, it was assumed that the City's abandoned wastewater outfall could be modified and made suitable for use as an intake line. Based on a conceptual level evaluation of the existing facility, the following modifications will likely be required to use the intake:

- New baffle/screen to capture small particles and debris.
- New interior lining for existing pipe.
- Modifications to existing junction box.

Additional engineering analyses will be required to confirm these (or other) modifications, so a more detailed analysis of the pipeline and associated structures is recommended, if the project is implemented.

Raw Water Pipeline

Cost estimates are based on a 24-inch diameter raw water pipe, which has carrying capacity to meet the required range of raw water flows between 4.7 and 14.0 mgd (5,300 to 15,800 AF/yr). The length of the pipe will vary depending on site location and routing. Figure ES.1 shows potential pipe routing alternatives. Table ES.2 summarizes the pipeline lengths for two separate options considered in the evaluation.

Table ES.2 Pipeline Length
Evaluation of Regional Water Supply Alternatives
City of Santa Cruz/Soquel Creek Water District

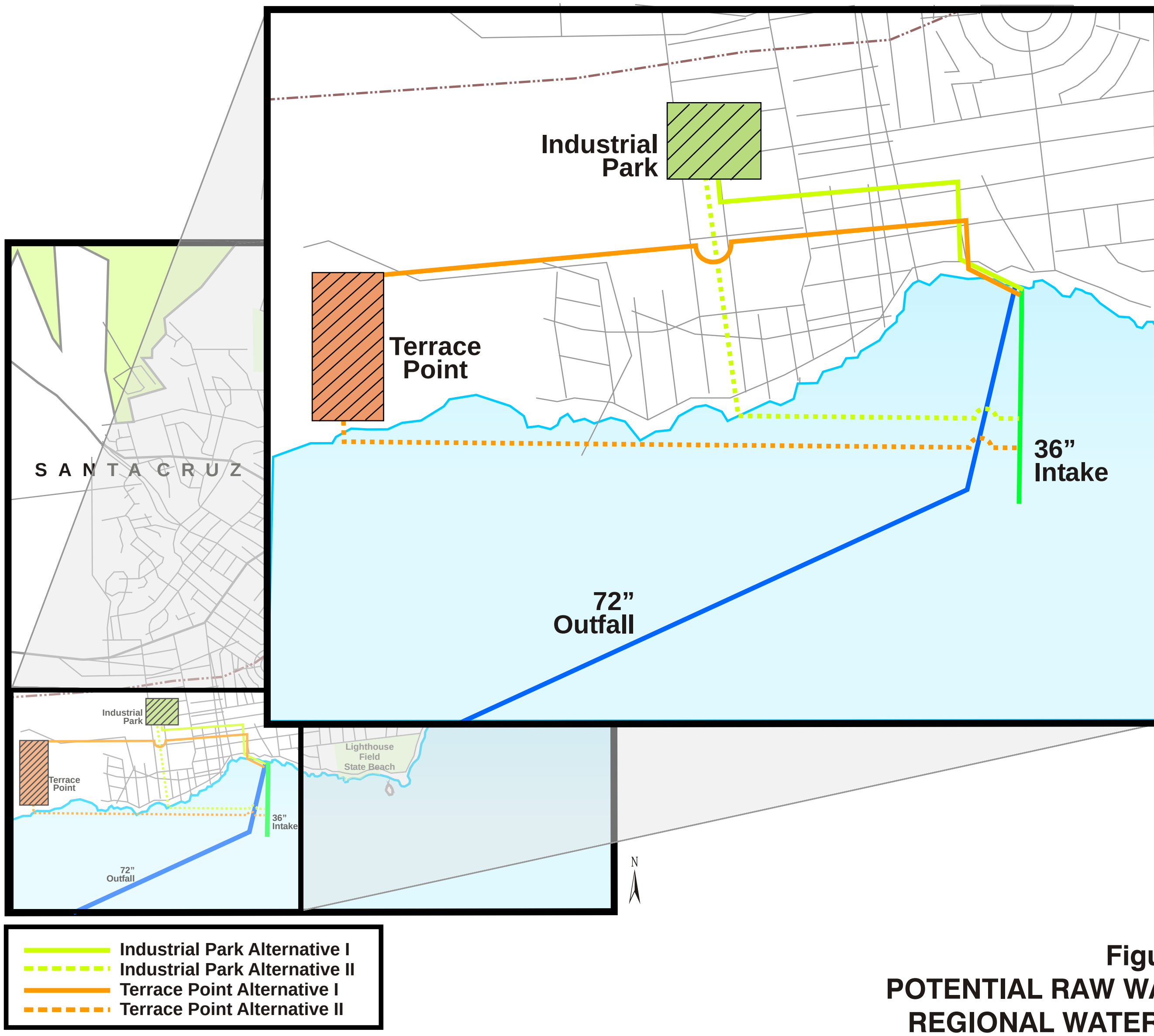
Option	Pipeline Length for Industrial Park Site	Pipeline Length for Terrance Point Site
Interconnection with landward end of outfall and routed through existing rights-of-way and easements.	5,000 feet	10,000 feet
Interconnection with outfall in the ocean, routing the pipeline along the ocean floor, and transitioning to new tunnel well below streets.	5,700 feet (1,600 feet installed on ocean floor and 3,100 feet of trenchless piping)	8,000 feet (6,700 feet installed on ocean floor and 1,300 feet of trenchless piping)

Treatment Process

Table ES.3 summarizes the desalination treatment process requirements. Figure ES.2 shows a schematic illustration of the desalination treatment processes.

Table ES.3 Treatment Process Requirements
Evaluation of Regional Water Supply Alternatives
City of Santa Cruz/Soquel Creek Water District

Process Parameter	Process Element
Pretreatment	• Membrane or Granular Media Filtration for Particulate Removal • UV Disinfection • Sulfuric Acid for pH & Scale Control • Anti-Scalant Chemical Addition
Treatment	• Reverse Osmosis for Desalination (1.0 to 1.5 mgd per train)
Post Treatment	• Disinfection • Corrosion Control Chemical Addition & pH adjustment



NOTES
1) Routes identified are two viable alternatives. Other similar routes are possible but are not shown because there are no appreciable differences in cost or implementation constraints.

2) Pipe size is 24 inches for delivered water flows of 2 to 6 mgd.

3) Pipe length varies from 5,000 to 10,000 feet for Alternative 1; 5,700 to 8,000 feet for Alternative 2.

Figure ES-1
POTENTIAL RAW WATER PIPELINE ROUTES
REGIONAL WATER SUPPLY EVALUATION (1,2,3)
CITY OF SANTA CRUZ/
SOQUEL CREEK WATER DISTRICT

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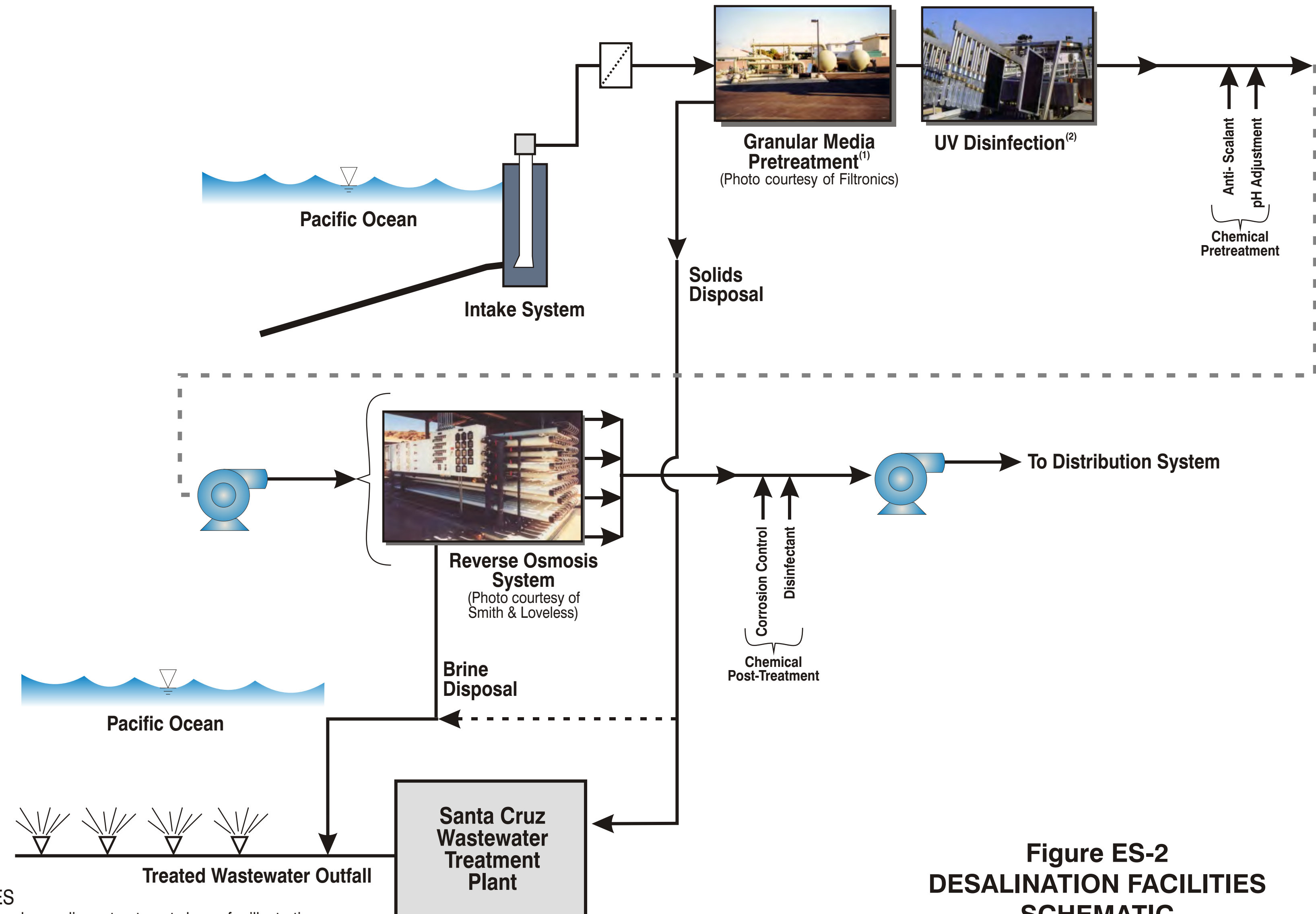

NOTES

(1) Granular media pretreatment shown for illustrative purposes. Membrane pretreatment may be a preferable alternative to be confirmed by additional engineering analysis, if project is implemented.

(2) UV disinfection shown for illustrative purpose. Chlorine disinfection may be a preferable alternative. To be confirmed by additional engineering analysis, if project is implemented.

Figure ES-2
DESALINATION FACILITIES SCHEMATIC
REGIONAL WATER SUPPLY EVALUATION
CITY OF SANTA CRUZ/
SOQUEL CREEK WATER DISTRICT

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Brine Disposal

Options for brine disposal include beach-well discharge and connection to the wastewater ocean outfall. Beach-well discharge is not viable due to insufficient hydraulic/dispersion capacity in the available beach areas. Discharge via the wastewater ocean outfall is viable. However, the amount of brine that can be discharged is strongly linked to the wastewater flow available for dilution, and the resulting density of the combined brine-wastewater solution. This is because the brine-wastewater results in a solution that is less buoyant than wastewater alone, and does not mix as effectively when discharged.

Based on preliminary modeling, equalization storage for brine may be required so that the brine can be discharged only at times when sufficient wastewater is available for dilution.

Finished Water Conveyance

Finished water system facility requirements include piping and pumping to the City distribution system, and distribution system upgrades (within the City system) for delivery to the District.

Preliminary model results indicate the following new facility requirements for City's distribution system:

- Dedicated pipeline from the facility to Escalone Drive at the intersection with Bay Street.
- Pipeline length of approximately 11,000 to 16,000 feet depending on site location.
- Pipeline diameter of 16 inches for 2-mgd system .
- Pipeline diameter of 20 inches for 6-mgd system.
- Three pumps (2 duty 1 standby) to pump water from the facility to the distribution system.

Preliminary analysis determined the following upgrades are required to deliver water to the District along 41st Avenue, at either Capitola or Soquel Avenue (or potentially both locations).

- New 16- or 20-inch diameter pipelines (based on 2- or 6- mgd capacity) ranging from 19,500 feet long to 24,500 feet long (depends on final routing).

In addition, it is likely that some modifications to the District's distribution system will be required to improve operational flexibility and reliability near the points of interconnection with the City system. Modifications could include upsizing of one or more main distribution pipes.

Cost Estimate for Desalination

The estimated costs for desalination are shown in Table ES.4. As shown in the table, the total annual costs have been calculated for three Case Conditions: 2, 4, and 6 mgd.

Table ES.4 Conceptual Costs for Desalination
Evaluation of Regional Water Supply Alternatives
City of Santa Cruz/Soquel Creek Water District

Delivered Water Capacity	Industrial Park		Terrace Point	
	Capital Cost(1,2,4,5)	Operating Cost(3,5)	Capital Cost(1,2,4,5)	Operating Cost(3,5)
2 mgd	\$26.1 - \$29.3	\$2.0 - \$2.1	\$27.2 - \$28.9	\$2.0 - \$2.1
4 mgd	\$37.7 - \$40.9	\$3.9 - \$4.0	\$39.2 - \$40.5	\$3.9 - \$4.0
6 mgd	\$49.4 - \$52.9	\$5.8 - \$5.9	\$51.3 - \$52.3	\$5.8 - \$5.9

Notes:
(1) Capital cost range reflects costs for piping Alternatives 1 and 2 in millions of dollars.
(2) Capital cost estimates include allowance for engineering, legal, construction administration.
(3) Operating cost range reflects costs for piping Alternatives 1 and 2. Operating cost assumes production at stated capacity, each day of the year.
(4) Capital cost estimates do not include costs for electrical distribution system upgrades that may be required.
(5) Cost in millions of dollars.

Treatment facilities sized within this range would provide additional supply to cover a wide range of potential supply deficits during a drought for the current and future demand conditions.

Implementation Analysis

The installation of seawater desalination facilities in coastal communities such as Pacifica and Santa Barbara and a planned installation in Cambria demonstrates that such facilities can be implemented with due consideration of technical, environmental, and institutional issues.

Technical Issues

The on-land facilities associated with a desalination system (i.e., pipelines, pump stations, and treatment systems) do not present any unusual engineering or constructability constraints. The engineering and construction of the seaward facilities present more challenges, and will require potentially complicated underwater construction.

Environmental Issues

Based on preliminary environmental review, there does not appear to be any significant environmental issues related to new infrastructure or construction. The most notable issues are construction related, although there are options for mitigation. Irrespective of the site location, there would be a substantial construction effort in potentially environmentally sensitive areas, including the ocean, so numerous permits will be required. The permitting effort and the associated environmental impact assessment/ documentation would require coordination with multiple agencies.

Institutional Issues

The primary institutional challenge for desalination is siting of the treatment facilities. Although land is potentially available at the Industrial Park and Terrace Point sites, considerable additional work is needed to confirm a site location and secure the land. Before land can be secured, environmental documentation for the project must be certified.

Summary of Implementation Issues

Potentially significant issues related to the implementation and viability of this project are:

- Due to the nature of construction of facilities in the ocean and planned discharge of brine into the ocean, there will be considerable coordination requirements with multiple agencies to complete the necessary environmental review and documentation. This process would likely take 18 to 24 months to complete, and could delay implementation of the project.
- Facility siting alternatives have been identified through initial screening. Additional work is needed to confirm a site location and secure the land. Final site selection would need to be determined based on feasibility of acquisition. Before land can be secured, environmental documentation and certification of the project concept must be completed.
- Sizing of the facility is critical to development of expected costs for construction and operation. Facility sizing would need to be confirmed and coordinated with planned conservation/curtailment efforts, and/or with planned development of alternate sources of supply (see also paragraph below regarding capacity and brine discharge limitations and considerations).
- The planned use of the abandoned outfall as a new intake structure and use of the existing wastewater outfall for brine disposal are based on conceptual engineering review of these facilities, including hydraulic capacity, age/condition, and ability to construct required modifications. Additional engineering at the preliminary design level will be required to more completely describe the engineering details for use of these facilities.
- Preliminary modeling analyses indicate that the plant capacity may be limited by available dilution capacity for the brine discharge, even if the brine discharge is equalized throughout the day. Additional analysis of future drought conditions, water supply demand and wastewater flows is needed to determine the maximum limitation of dilution. It is important to note that it may be possible to minimize the effect of limited dilution capacity by discharging brine only during times of peak diurnal wastewater flows (i.e., turn off or turn down the plant for 4 to 6 hours during the day so little or no brine is generated during periods of low wastewater flow). However, to do so would require that the desalination plant capacity be increased beyond the 6 mgd maximum assumed in this document in order to offset the “lost” production during the plant downtime during the day. The analysis of plant operation/capacity

relative to brine discharge and dilution needs to be considered concurrent to the final sizing analysis.

RECLAMATION PROJECT

Based on the findings of previous evaluations, the use of reclaimed water for irrigation within the City or District (e.g., golf courses, parks, cemetery, etc.) is not viable because it provided no appreciable additional supply benefit. Accordingly, the general project concept for a regional facility is to exchange reclaimed wastewater for new raw water supplies, rather than offset demand via domestic outside irrigation uses.

Available Supply from Reclamation

Santa Cruz

Based on previous findings, a reclaimed water exchange with the North Coast farmers is considered to be a viable project alternative for the City. Reclaimed water would be diverted to farmers for irrigation supply in exchange for groundwater that the farmers currently use. The water available for exchange is estimated from 500 to 700 million gallons (MG) per year.

Two other reclamation alternatives, in-stream exchange for surface water and groundwater recharge with reclaimed wastewater, were also examined for applicability to the City. In-stream exchange is nonviable, primarily because it provides no storage component for use during drought years. The City already has limited raw water storage capacity, so there is no benefit of diverting “excess” stream flow when it is available in the high runoff months. An in-stream exchange with reclaimed wastewater would be similarly constrained so it would be of no appreciable benefit, even in drought years.

Soquel Creek

Two project concepts for the District are considered potentially viable:

- ***Reclaimed Water for Agricultural Application.*** As noted above, a regional project would provide reclaimed water to the North Coast Farmers in all years. Under this project concept, the District would receive exchanged water in nondrought years. As previously detailed, the estimated additional supply from the North Coast ranges from approximately 400 MG/yr to 700 MG/yr (± 1,200 to 2,200 AF/yr) based on the estimated irrigation usage and groundwater basin yield.
- ***In-Stream Exchange.*** The District has previously evaluated a water supply project that would provide new supply via diversion from Soquel Creek. Although this project concept is potentially viable, there are seasonal/annual diversion constraints that could potentially limit diversions from the creek. The amount of supply from the project would be enhanced if the minimum stream flow downstream of the diversion point could be maintained, irrespective of diversion activity by the District.

The in-stream exchange concept would require up to 3.2 mgd (5 cfs) of reclaimed wastewater to augment stream flows during periods of diversion. By providing a source of supply to augment stream flows, the District would have more flexibility to divert water from the stream and increase its diversion up to its projected need of 650 MG/yr (± 2,000 AF/yr).

The in-stream exchange project would provide supplemental supply to the District only, and is not considered a "regional" project that would provide supply benefit to the City. It is considered in this document because it provides an opportunity to use reclaimed water during periods when it is not being used for irrigation. If this project was implemented in conjunction with a groundwater exchange project, there would be opportunity for cost sharing of capital and operating costs.

Siting

Treatment for would most likely be provided at the City's existing wastewater treatment plant. This is because location at a site other than the existing plant would require a duplication of all of the treatment processes at the existing plant prior to the tertiary treatment facilities.

Facility and Infrastructure Requirements

Facility and infrastructure requirements for reclamation include treatment systems and treated water pipelines and pumps.

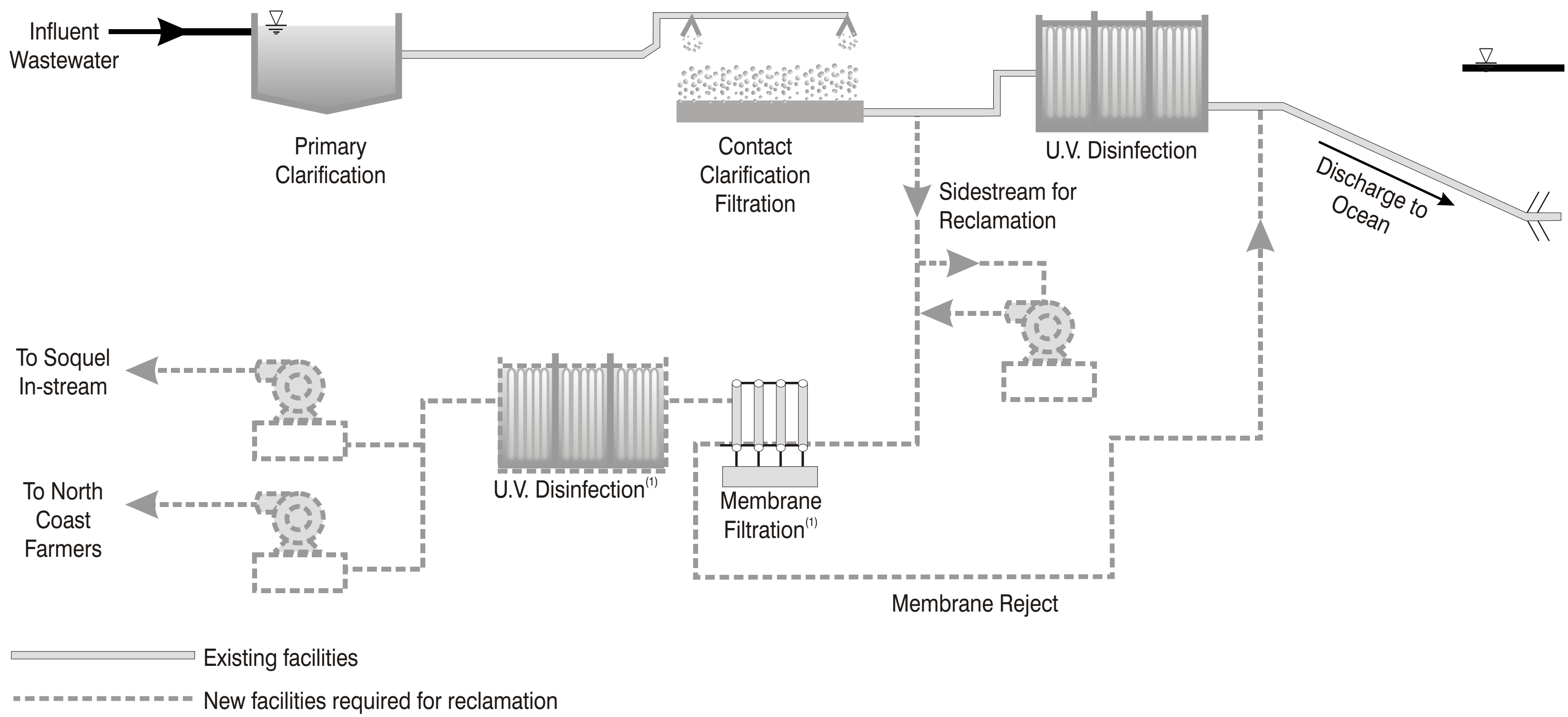
Treatment Systems

For the purposes of this analysis, it was assumed that tertiary treatment would be required for any reclamation project alternative. Figure ES.3 illustrates the treatment processes. The existing facilities provide secondary treatment and include the following process:

- Sedimentation.
- Aeration.
- Clarification.
- Disinfection.

Tertiary treatment would include the previously mentioned processes, additional filtration, and additional disinfection. The proposed methods of treatment would be:

- Filtration treatment via membranes.
- Post-disinfection (UV or sodium hypochlorite).



NOTES
(1) Membrane filtration and U.V. Disinfection shown. Granular media filtration and/or chlorine disinfection may be preferred alternatives. To be confirmed in design phase, if implemented.

Figure ES-3
RECLAMATION FACILITIES SCHEMATIC
REGIONAL WATER SUPPLY PROJECT
CITY OF SANTA CRUZ/
SOQUEL CREEK WATER DISTRICT

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For cost estimating, it was also assumed that desalting treatment via membranes would likely also be necessary to minimize the dissolved salts in the water used for irrigation, or water discharged for the in-stream exchange.

Conveyance

New conveyance piping and pumps will be required to transport water from the wastewater facility to the North Coast users or the District's stream diversion site.

Requirements for exchange with the North Coast users include:

- A main conveyance pipeline with various turnouts for the farms.
- Approximately 45,000 feet of 18-inch distribution piping.
- Three pumps (2 duty, 1 standby) for conveyance.
- Farmers would provide pipelines from the turnouts to their farms.
- Farmers would provide storage reservoirs totaling up to 3 MG.

Figure ES.4 illustrates the conveyance system requirements for groundwater exchange.

In-stream exchange would require the following:

- Approximately 35,000 feet of 16-inch piping.
- Three pumps (2 duty, 1 standby) for conveyance.

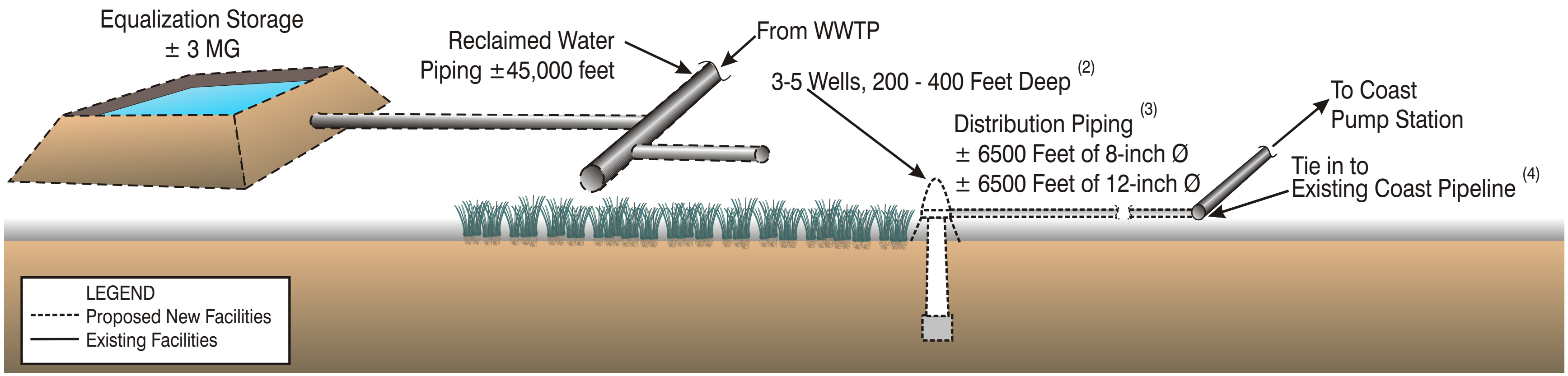
Figure ES.5 illustrates the conveyance system requirements for in-stream exchange.

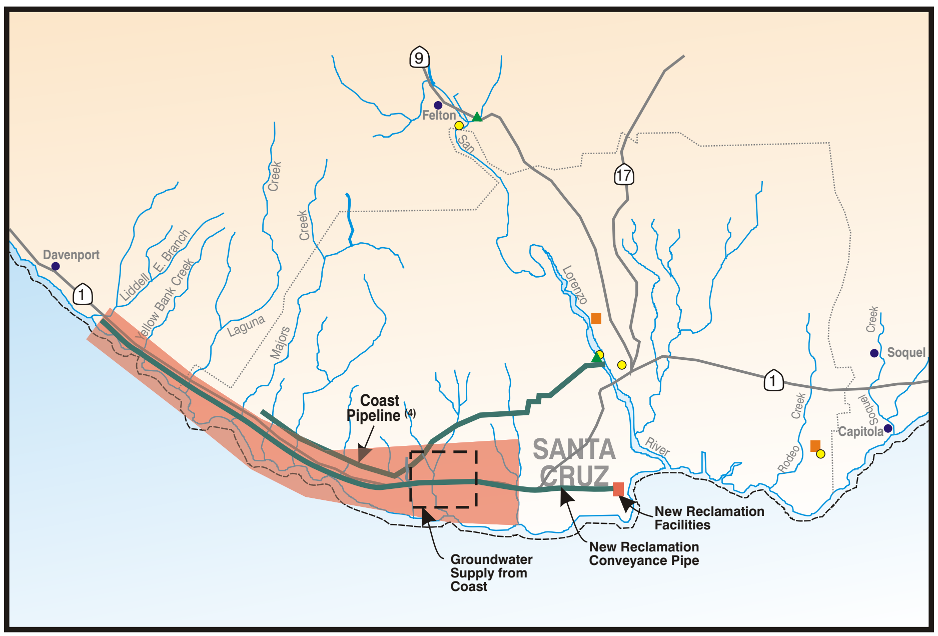
Groundwater Facilities and Conveyance

The following is required for groundwater exchange:

- Approximately 4 groundwater wells.
- 6,500 feet of 8-inch pipe and 6,500 feet of 12-inch pipe from the wells to the City's Coast Pipeline.

As shown on Figure ES.4, the project concept assumes that the groundwater would be delivered from the farms via the City's Coast Pipeline. The Coast Pipeline has hydraulic restrictions that limits flow to about 9 cfs. The groundwater supply could range from between 4 to 6 cfs, which would take up 40 to 60 percent of the capacity. Because of this the City may need to limit periods of groundwater pumpage to the summer months (when flow in the pipeline is less due to reduced diversion from the North Coast surface water sources). Alternatively, the City may need to increase the capacity of the lower reaches of the pipeline to accommodate the additional groundwater flow.





Potential Area for Reclamation/Groundwater Exchange Supply Project. (1)

NOTES

(1) The highlighted area represents the general location of reclamation along the coast.

(2) The depth of the wells may vary depending on location drilled because of varying surface features and depth to the aquifer along the coast.

(3) Assumes approximately one half mile piping at each of five well sites.

(4) The existing coast pipeline has hydraulic capacity limitations. Upgrades completed as part of a separate project are required to increase capacity to accommodate delivery of ground-water to the City.

Shaded area indicates general region/area of application for project alternative.

Figure ES-4
RECLAMATION FACILITY REQUIREMENTS FOR NORTH COAST AGRICULTURE REGIONAL WATER SUPPLY PROJECT
CITY OF SANTA CRUZ/
SOQUEL CREEK WATER DISTRICT

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NOTES

1) Route identified is one potential alternative to deliver water to the Soquel Creek WD project location.

2) Pipe length is approximately 35,000 feet.

3) Pipe size is 12 inches for delivered water flow of 3.2 mgd.

4) Location of Diversion with respect to project site is approximate. Final location to be downstream of Soquel Creek intake, if implemented.

Figure ES-5
RECLAIMED WATER CONVEYANCE PIPING FOR DELIVERY TO SOQUEL(1,2,3,4)
REGIONAL WATER SUPPLY EVALUATION
CITY OF SANTA CRUZ/
SOQUEL CREEK WATER DISTRICT

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Cost Estimate for Reclamation

The estimated costs for reclamation are shown in Table ES.5. The costs have been calculated based on a total annual supply of up to 700 MG/yr (± 2,200 AF/yr). Treatment and conveyance facilities sized within this range would provide additional supply required for both groundwater exchange and in-stream exchange project concepts.

Table ES.5 Conceptual Costs for Wastewater Reclamation
Evaluation of Regional Water Supply Alternatives
City of Santa Cruz/Soquel Creek Water District

	Capital Costs (in millions)(1,2,3)	Operating Costs (in millions)(4)
Reclamation for North Coast Groundwater Exchange	$49.3	$0.4
Reclamation for In-Stream Exchange in Soquel Creek	$31.0	$0.2 - $0.4

Notes:
(1) Capital costs assume 5-mgd (± 5,700 AF/yr) treatment capacity for both alternatives.
(2) Cost estimate includes allowances for engineering, construction administration, and contingencies.
(3) Cost estimate does not include allowances for distribution system upgrades that may be required to deliver water within the District.
(4) Operating cost has been decreased from full-year cost estimates assuming production at 5 mgd, 6 months of year for North Coast alternative, and operation at 3 mgd, 4 to 6 months per year for in-stream alternative.

As noted above, the in-stream exchange project for the District would be used to supplement the available supply from a separate surface water diversion and treatment project. Cost estimates for the surface water diversion and treatment facilities have been developed separately by the District and are not included in the cost estimates herein.

Implementation Analysis

The installation of numerous reclamation projects throughout California provides ample evidence that a regional project could be implemented with due consideration of technical, environmental, and institutional issues.

Technical Issues

There are no significant engineering issues for either project; the treatment systems and the required infrastructure are typical of other water/wastewater facilities. However, there are a few key engineering issues that need to be investigated further as part of (or preferably prior to) implementation:

- ***Confirm Groundwater Conveyance via the North Coast Pipeline.*** This planning level concept is based on using the existing North Coast pipeline to convey water back to the City. The pipeline capacity is constrained under the existing hydraulic conditions, so modifications to this pipeline would likely be required to accommodate additional flow from the groundwater supply. Alternatively, a new pipeline would need to be constructed. Additional preliminary engineering work is needed to identify preferred options. In addition, it will also be important to identify the costs and scheduled implementation for the upgrade and/or new pipeline. For example, the City's current long-range plan is to complete rehabilitation/upgrades to the North Coast pipeline over the next 15 years. This time frame would not be consistent with the objectives to provide additional water supply in a timely manner.

- ***Confirm Groundwater Supply.*** Estimates of the groundwater yield along the North Coast vary. Although there is substantial published geologic/hydrogeologic information, there is very limited actual field data to confirm the aquifer characteristics. Additional fieldwork (i.e., test wells) is recommended to confirm the groundwater supply prior to final implementation of the project.

Environmental Issues

Based on preliminary environmental review, there does not appear to be any significant environmental issues. There are potential construction-related issues for pipeline routes in major City arterial streets, but these construction issues do not represent a "fatal flaw" for the project. Table 6 summarizes other potential environmental issues.

Institutional Issues

Even with a strong bias to implement the project there are several institutional issues that would need to be resolved:

- ***Confirm Project Concept with North Coast Farmers.*** There are several local examples of reclamation along the coast and Salinas Valley so there would not appear to be any significant implementation issues. Also, based on preliminary discussions with several North Coast farmers, there appears to be interest for the use of reclaimed water. However, more effort is needed to confirm that the interest is genuine, and that the interest is not limited to one or two crop types.

Confirm Groundwater Usage Entitlements on the North Coast. For reclamation to be viable on the North Coast there must be a guarantee that the groundwater would be available in exchange for the reclaimed supply. Based on preliminary review of the irrigated land along the coast it appears that much of the land currently irrigated with groundwater is owned by the State of California. As the owner of the land, the State also owns the rights to the underlying groundwater. To implement this project, rigorous contractual agreements with the State would need to be developed. There is no clear indication that the State would (or could) enter into such agreement for this project. In any case, to finalize an agreement would take time (perhaps years), and would have associated schedule implications. Although the agreements/entitlements could be developed in parallel to other project elements (e.g., EIR documentation, facility engineering, permitting), it would be preferable to have such agreements in place prior to pursuing/developing the project.

- ***Confirm In-Stream Exchange Concept.*** The in-stream exchange concept would require that the tertiary treated reclaimed water be discharged to Soquel Creek. Given that the water is highly treated, and that the discharge would only represent approximately one-sixth of the stream flow when operational, there are no apparent public health or habitat issues. For example, wastewater treated to lesser degrees is routinely discharged to waterways in the State with no obvious consequences to the fishery habitat. Previous precedents notwithstanding, there are several examples of public interest regarding discharge of reclaimed wastewater to streams - even if highly treated. As such, it would be preferable to have regulatory approval and public acceptance of the discharge in place prior to pursuing/developing this project.

Of these three broad issues, there is potential that two issues - need for project confirmation with the farmers and need for confirmation of groundwater usage entitlements - could potentially represent “fatal flaws.” As discussed above, there are several unanswered questions and unknowns for both of these issues. In particular, the permitting elements related to the groundwater usage are, at a minimum, very complex and would require involvement of multiple parties, including the Coastal Commission, State Water Resources Control Board, California State Parks Department, State Water Quality Control Board, State Division of Drinking Water, and local farmers. The City and District should consider that, even if the project is feasible in concept, the interagency involvement and related permitting elements could significantly impact the schedule for implementation.

City of Santa Cruz/Soquel Creek Water District
Alternative Water Supply Study

EVALUATION OF REGIONAL WATER SUPPLY ALTERNATIVES

FINAL
March 2002

In Association with
Black and Veatch Engineers and
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CITY OF SANTA CRUZ/SOQUEL CREEK WATER DISTRICT
ALTERNATIVE WATER SUPPLY STUDY

EVALUATION OF REGIONAL WATER SUPPLY ALTERNATIVES

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D - Brine Disposal Dilution Analysis

REGIONAL WATER SUPPLY ALTERNATIVES

The City of Santa Cruz (City) has completed a conceptual-level evaluation of water supply alternatives which identified ocean-water desalination and wastewater reclamation as potentially viable new water sources for the City (ref. Alternative Water Supply Study, November 2000). The evaluation also identified the potential viability of a "regional" facility to augment supplies of other neighboring communities. The Soquel Creek Water District (District) is a community that could benefit from, and participate in, a "regional" water supply project.

The purpose of this technical memorandum is to expand on the information developed previously in the Alternative Water Supply Study. Specifically, the memorandum includes an evaluation of "regional" desalination and wastewater reclamation facilities to augment water supplies for both the City and the District.

DESALINATION

The City's primary need for additional supply is a function of projected shortfalls during drought conditions. Conversely, the District's primary need is a new nondrought supply to reduce its "every day" groundwater pumpage. Accordingly, the general concept for a regional desalination facility is to provide water to the City during drought years, and to the District during nondrought, normal rainfall years.

Incremental Supply from Desalination

Santa Cruz

It is the City's intent to develop an overall water supply strategy that includes not only new water sources such as desalination, but also strategies to reduce demand. This overall water supply strategy - the Integrated Water Plan (IWP) - includes three elements:

- Reduced demand by conservation in all years.
- Reduced demand by usage curtailment in drought years.
- New sources of supply.

The IWP will compare and contrast new water supply alternatives to various growth and conservation/curtailment strategies to establish a most effective/preferred supply and demand combination.

The IWP is expected to be complete by mid-2002, so the amount of incremental supply that may be needed from a desalination facility has not yet been quantified. In the absence of an exact number, a range of possible incremental supply needs was examined for this analysis as set forth in Table 1.

Table 1 Estimated Range of Supply Shortfall and Treatment Capacity Needs (Santa Cruz Only) Evaluation of Regional Water Supply Alternatives City of Santa Cruz/Soquel Creek Water District

Year/Case Condition	Demand (MG)(1)	Total Projected Deficit (MG)(1)	Estimated Deficit Adjusted for Curtailment (MG)(2)			Treatment Capacity Required (MGD)(3)
			20%	30%	40%	
2005						
Summer Season(4)	2200	940	500	280	60	<0.4 - 3.3
Peak Month	450	280	190	145	60	2.0 - 6.3
2015						
Summer Season(4)	2300	1065	605	375	145	1.0 - 4.0
Peak Month	475	310	215	170	120	4.0 - 7.2
2030						
Summer Season(4)	2500	1340	840	590	340	2.3 - 5.6
Peak Month	515	350	250	195	145	4.8 - 8.3

Notes:
(1) Demand and deficit estimated from preliminary data developed for Santa Cruz Integrated Water Plan (FISKE, 10/2001)
(2) Estimated deficit adjustments calculated as [(demand * (1-curtailment percent)) - available $supply_{demand\text{-}deficit}$].
(3) Treatment capacity required calculated as projected deficit/number of days in deficit period.
(4) For purposes of this study, summer season assumed as May through September.

(***Note:*** The factors that influence the projected capacity needs include the amount of demand in a given year, the amount of curtailment, and the amount of seasonal vs. peak monthly shortfall. These factors are used to determine whether the plant should be sized to meet average seasonal shortfalls or peak monthly shortfalls. During severe drought conditions it is likely that the City will institute some level of curtailment to help offset demand. The net effect of curtailment is to reduce the need to meet peak monthly deficits. Therefore, for planning purposes, average supply shortfalls are used as the basis for sizing treatment capacity requirements rather than peak shortfalls.)

Soquel Creek

The District has a projected future demand at buildout of approximately 2,450 MG/yr (+/- 7,500 AF/yr). Adjusting this demand number for District-wide conservation of 10 percent, the District's demand at buildout is reduced to approximately 2,200 MG/yr (+/- 6,800 AF/yr). The District's perennial groundwater yield is less than its projected future supply need, so the District needs an additional supply source. A new supply will also help to restore groundwater levels in the basin, thereby providing a barrier against seawater intrusion.

The District pumps groundwater from two distinct aquifers, the Purisima and Aromas aquifers. The District currently pumps approximately 1200 MG/yr (+/- 3,600 AF/yr) from the Purisima aquifer, and approximately 600 MG/yr (+/- 1,800 AF/yr) from the Aromas aquifer. However, the District's groundwater management goal is to reduce annual pumping from the Purisima aquifer by approximately 200 MG/yr (+/- 600 AF/yr). Assuming a future demand of 2,200 MG/yr, the supply shortfall is approximately 600 - 650 MG/yr (+/- 2,000 AF/yr). This supply shortfall is used as the basis for evaluating supply alternatives.

Two case conditions were examined to bracket the range of potential nondrought year supply conditions:

Case Condition No. 1 - Future Demand Only. This case condition assumes that the Purisima aquifer would supply 950 MG/yr (+/- 2,900 AF/yr, the estimated perennial yield), and the Aromas aquifer would supply 600 MG/yr (+/- 1,800 AF/yr, the estimated perennial yield), for a total available groundwater supply of approximately 1,550 MG/yr (+/- 4,800 AF/yr). Accordingly, a new desalination facility would provide supply to offset the projected future demand of 650 MG/yr (2,200 MG/yr_{demand} - 1,550 MG/yr_{supply}) in nondrought years.

In drought years, the District's entire projected current and future demand of 2,200 MG/yr (+/- 6,800 AF/yr) would be supplied by its groundwater sources and water from the desalination facility would be diverted to the City. Under this condition, the aquifer would be stressed during the drought year(s), but it would be allowed to recharge during the subsequent nondrought years.

Case Condition No. 2 - Current and Future Demand. This case condition assumes that the District may choose to limit groundwater production in the year(s) after a prolonged drought, so that groundwater levels can recover. Limiting the production, or "resting" the aquifers, may be necessary because the water levels will likely decrease during a prolonged drought due to a combination of factors, including reduced recharge and increased pumping by the City, the District, and private wells. The production limits on the aquifer will vary depending on the severity/duration of the drought. For example, in the event of very low aquifer water levels after a prolonged drought, it may be necessary to completely rest the aquifer for some time. In this worst case condition the desalination facility would need to provide up to 2,200 MG/yr (current and future demand).

Table 2 summarizes the range of possible supply shortfalls (based on projected demand adjusted for expected savings from conservation), and the associated desalination capacity requirements for the two case conditions.

(***Note*****:** Although the District has storage reservoirs in its distribution system, it relies on its wells to cycle on/off as needed to meet peak demands in the system (ref. communication with District staff, September 2001). The desalination facility would need to act in a similar fashion as a well, providing supply as needed to meet varying seasonal demand conditions. This is important because it may affect the amount of supply provided by a desalination facility. For example, for

Table 2 Estimated Range of Supply Shortfall and Treatment Capacity Required (Soquel Creek Only) Evaluation of Regional Water Supply Alternatives City of Santa Cruz/Soquel Creek Water District					
Year/Case Condition	Total Projected Deficit (MG)[1]	Estimated Deficit Adjusted for Curtailment (MG)[1,2]			Treatment Capacity Required (mgd)[3]
		20%	30%	40%	
Case Condition No. 1 - Future Demand Only[4,5]					
	650	N/A	N/A	N/A	1.6
Case Condition No. 2 - Current and Future Demand[6]					
2005	1900	1515	1325	1135	3.2 - 5.2
Summer Season[7]	885	710	625	530	3.5 - 5.8
Peak Month	190	150	120	100	3.5 - 6.3
2015	2100	1680	1470	1260	3.5 - 8.1
Summer Season	1100	885	780	670	4.5 - 7.1
Peak Month	240	195	165	145	4.8 - 8.0
Buildout	2200	1760	1540	1320	3.7 - 6.0
Summer Season	1125	900	790	670	4.5 - 7.5
Peak Month	245	195	170	145	4.9 - 8.2

Notes:
(1) Deficit adjustment for Case Condition No. 2 assumes that the deficit will be reduced by demand reduction or supply offset (i.e. in year following drought the District may implement curtailment to offset demand or may continue to pump groundwater to meet a portion of its demand).
(2) Estimated deficit adjustments calculated as [(demand * (1-curtailment percent)) - available $\text{supply}_{\text{demand-deficit}}$].
(3) Treatment capacity required calculated as projected deficit/number of days in deficit period.
(4) Case Condition No. 1 assumes Soquel will need new supply to meet its projected future demand only.
(5) Case Condition No. 1 assumes supply during nondrought years only; accordingly, no adjustment for curtailment is assumed.
(6) Case Condition No. 2 assumes the District will cease all or a portion of its groundwater production following a prolonged drought so that the aquifer water levels can recover.
(7) For purposes of this study, summer season assumed as May through September.

Case Condition No. 1 the average annual supply required from the desalination facility is approximately 650 MG/year, or about 1.8 mgd. However, the peak monthly demands will likely be higher than the average annual demands by a factor of 1.5 to 1.7. The difference between the average and peak demand will need to be made up with either storage, and/or new supply (i.e. groundwater or additional “peak” desalination capacity). As shown in Table 2, the capacity

required from a desalination facility could range between 1.6 mgd (Case Condition No. 1 - Future Demand Only) and 4 to 8 mgd (Case Condition No. 2 - Current and Future

Demand), depending on whether it is sized for average annual versus peak monthly/seasonal demand offset).

Summary – Incremental Supply from Desalination

As shown in Tables 1 and 2, there is considerable range in the possible supply needs for both the City and the District. The actual amount of supply required depends on a variety of factors, and will be strongly influenced by the amount of supply provided from other new (or existing) sources and assumptions for demand offset. There is no absolute "answer" to fit the needs of both the City and the District, but there is sufficient overlap among the potential case conditions to establish a reasonable capacity range for a desalination facility. For the purposes of conceptual planning and cost estimating, the lower capacity limit is set at 2 mgd, and the upper limit is set at 6 mgd.

(***Note:*** As noted earlier in this document, it is assumed that a new desalination facility would provide supply to the City during drought years, and to the District in nondrought years. Accordingly, the supply requirements included in Tables 1 and 2 are not additive).

Facility Siting Analysis

Siting Criteria

Four criteria were considered to identify and screen potential sites for a new treatment facility:

Proximity to Intake Facilities and Brine Disposal. New infrastructure - including pumps, piping, etc. - will make up a substantial portion of the capital costs of a new facility. Accordingly, it is advantageous to locate a new facility as close to the intake and brine disposal facilities as possible to minimize costs.

Proximity to Distribution System Infrastructure. The operation of a new desalination facility should result in minimum change to water distribution operations to the extent practical. To minimize impacts to operations, it is desirable to locate a new facility as close as possible to an existing distribution system "hub" or "backbone." Close proximity is desirable because it reduces the need for new distribution system infrastructure. For the City, this means locating the facility as close as possible to Bay Street Reservoir, since a majority of the City system is in a distribution system zone served by the reservoir. For the District, this means locating the facility near the center of the zone served by the Purisima aquifer. Ideally, to meet both objectives for the City and the District, the facility would be located somewhere between the City's Bay Street Reservoir and the District.

Land Requirement of 2 to 3 Acres for the Treatment Plant*.* Land requirements for a new facility will vary depending upon its production capacity. For planning purposes, a size range of 2 to 3 acres is sufficient to accommodate a plant in the desired capacity range. Ideally, the facility should be located in an area with other light industrial/heavy commercial facilities to minimize any potential impact to the surrounding land uses.

Proximity to Power Supply. Sea water desalination requires high operating pressures and is energy intensive. It is highly desirable to locate a new facility in relative close proximity to a high voltage power supply in order to minimize the costs for new electrical system infrastructure (i.e. substation, transmission lines, etc.).

Site Alternatives

Reconnaissance level surveys completed for this evaluation included a review of aerial maps and photos, supplemented with field visits. This siting evaluation survey indicated that there are no viable locations in the east/southeast area of the City, or in/around the District that meet the aforementioned siting criteria. The primary reason is distance from critical facilities (i.e., proximity to the intake structure, ocean outfall for brine disposal, and treated water storage in Bay Street Reservoir). Equally important, much of the City and District land area is established to the point that there is no readily available land. The land that is available is typically small parcels with surrounding uses such as residential and commercial that are not ideally compatible with a treatment facility.

The sites that were considered for this evaluation are:

Terrace Point. The University of California at Santa Cruz (UCSC) currently owns land at Terrace Point, much of which remains undeveloped. The site is considered for the following reasons:

- There is undeveloped land that could be used to site the new facilities.
- The site is relatively close to the critical facilities (intake and outfall facilities and Bay Street Reservoir).
- The site includes nonresidential research facilities. A new treatment facility would not conflict with the existing land use in/around the area.
- UCSC has an agreement with the City that it will assist with water system infrastructure upgrades that are (will be) necessary to support increasing demands as the campus population grows. One way that UCSC could provide assistance to the City is to make land available for new facilities.

City Industrial Park. The City's industrial park includes land that is undeveloped and/or unoccupied. The industrial park area is considered for the following reasons:

- There are large areas of undeveloped land.
- The area is relatively close to the critical facilities (intake and outfall facilities and Bay Street Reservoir).
- The park area includes light industry and other large commercial facilities. A desalination facility in/around this area would not conflict with surrounding land uses.

Moss Landing. The Moss Landing site is a 180-acre industrial parcel approximately 25 miles south/southeast of the City. On the parcel is an industrial complex for a brick manufacturing facility that ceased production several years ago. The site is now available for sale.

A primary benefit of this site is that it contains significant infrastructure that would be ideally suited for a new desalination facility, including:

- 48-inch diameter seawater intake pipe (previously used to provide cooling), 36-inch diameter discharge pipe extending two miles into the ocean (previously used for discharge of spent cooling water).
- Two fresh water wells at approximately 2.5 mgd capacity (previously used to supply potable water to the facility).
- Seven concrete ponds, approximately 3 MG capacity each (previously used for holding the bricks/cooling water).
- Pumping station at approximately 9 mgd capacity (previously used for seawater intake and spent cooling water discharge).

The intake and discharge lines are sized with ample capacity for the range of projected supply needs. It may also be possible to retrofit other existing facilities on the site to support the desalination treatment process which would reduce capital costs (i.e. the pump station may be suitable for finished water delivery with little or no modification, the ponds may be suitable for waste stream solids handling, etc.). All facilities are reportedly in good working condition. The site is also immediately adjacent to the Moss Landing power generation facility, so there is ready access to power.

Discussion. Based on the reconnaissance-level evaluation of site alternatives, the Terrace Point and Industrial Park areas are considered to be the most viable. Land is potentially available at either location, and both sites offer similar advantages with respect to proximity to critical facilities. Figure 1 shows the approximate location of these two site alternatives.

Although the Moss Landing area has several potential advantages, it also has two notable drawbacks: high cost and uncertainty of implementation. The high cost results from the need for 25 miles of new finished water pipeline from Moss Landing northward to the District and the City. Conceptual-level cost estimates show that the pipeline cost alone could range from $25 million to $38 million (assuming 3 mgd and 6 mgd capacity, respectively, a possible range given the City's and District's potential supply needs). The pipeline cost, added to the cost of the land and facilities ($18 million per the property broker) would result in baseline costs ranging from $43 million to $56 million, <u>not including</u>

NOTES

(1) Areas indicated represent potential general locations only. Site specific locations (lot, parcel, etc.) depend on size of facilities, size of land available, etc.

Figure 1
SITE ALTERNATIVES FOR DESALINATION TREATMENT FACILITIES(1)
REGIONAL WATER SUPPLY EVALUATION
CITY OF SANTA CRUZ/
SOQUEL CREEK WATER DISTRICT

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the cost of the treatment systems and appurtenant facilities. This site also presents several substantive implementation issues.

The site is not located in either the City or the District service area, nor is it located in Santa Cruz County, so there would be numerous cross jurisdictional elements to resolve to approve and implement the project. For similar reasons, pipeline routing and the associated easement and property access approval would require multi-agency involvement. The implementation issues do not necessarily represent a “fatal flaw” for the site, but they would add considerable time - perhaps years - to the project schedule. The high cost and implementation issues make this site less viable than either of the two potential sites in the City. For these reasons, this site is not considered further in this report.

Facility Requirements for Desalination

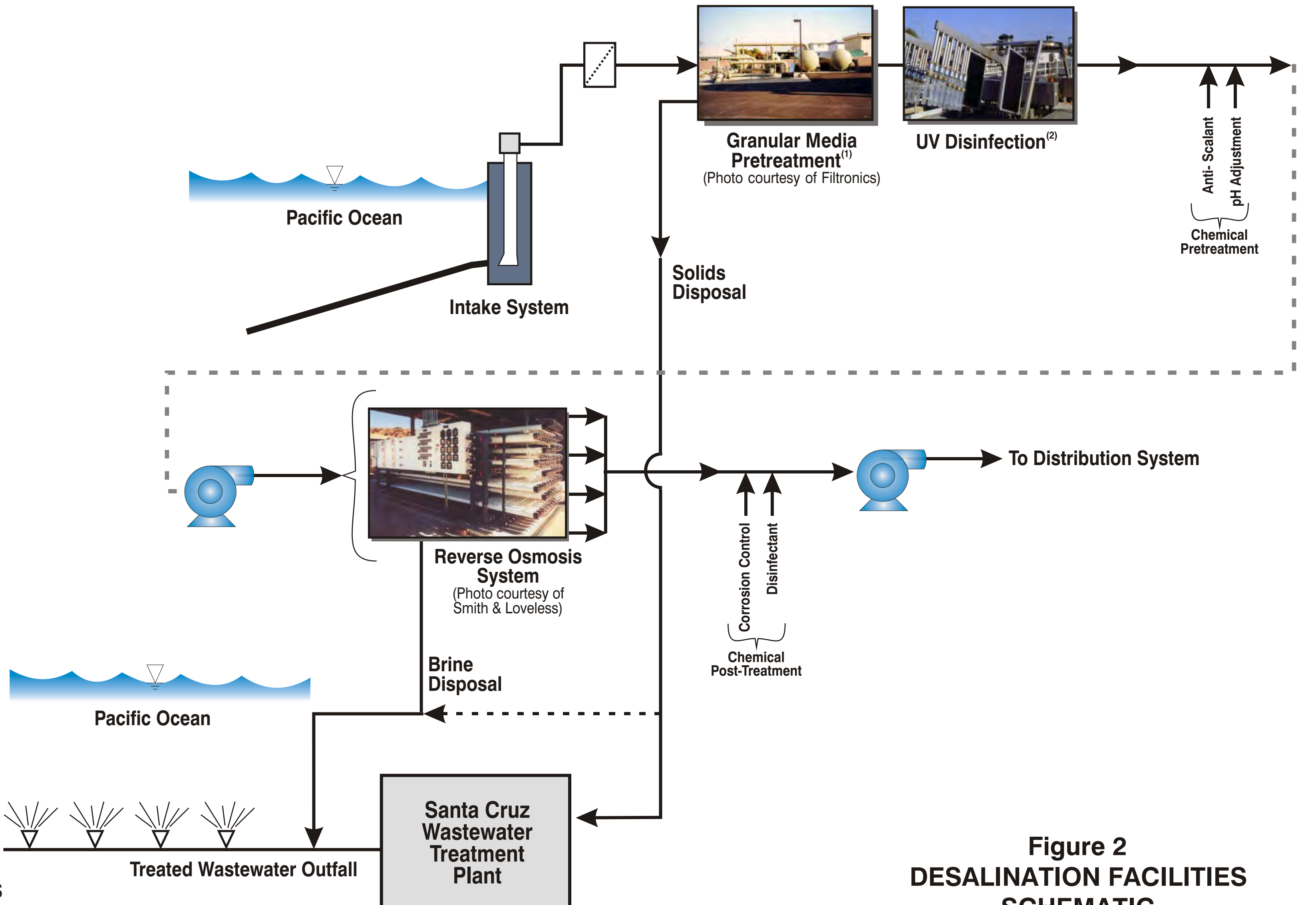
New desalination facility requirements are shown schematically in Figure 2. The conveyance facilities include raw water pumps and piping, treated water pumps and piping within the distribution system, and brine disposal piping. The treatment facilities consist of the desalination process equipment and ancillary support systems, pretreatment facilities, disinfection, chemical pretreatment for pH control and anti-scale control, reverse osmosis (RO) membrane units, chemical post-treatment for pH and corrosion control, and a new building.

Engineering Evaluation

Facility Sizing. For seawater desalination, approximately 5 percent of the raw water is lost during pretreatment, and approximately 55 percent is rejected as brine. Accordingly, raw water conveyance facilities must be sized for 4.7 to 14 mgd, or approximately 2.4 times the treated water supply needs. Brine and disposal conveyance facilities must be sized for 2.4 to 7.3 mgd.

The treated water conveyance facilities from the treatment plant to the distribution system must be sized for a treated water demand ranging from 2 mgd to 6 mgd to bracket the expected capacity needs for the City and the District. To identify distribution system upgrade requirements within the City system (to deliver water to the District via the City’s distribution system), a demand of 2 mgd to 6 mgd, as detailed in Table 2, was used as the range of possible delivery capacity.

Intake Facilities. Two alternatives for intake systems were considered: beach wells and direct ocean intake. Beach well intake systems are often preferred because the beach sands serve as a natural filter, removing solids from the raw water and providing “pretreatment” that minimizes solids loading/fouling of the reverse osmosis membranes (the desalting membranes). However, the drawback of this intake system is that it may require a large beach area, depending on the capacity needs and specific beach area hydrogeologic characteristics. Direct ocean intakes are also used, particularly when the hydrogeologic characteristics of coastal areas cannot provide enough capacity for larger facilities. While



NOTES

(1) Granular media pretreatment shown for illustrative purposes. Membrane pretreatment may be a preferable alternative to be confirmed by additional engineering analysis, if project is implemented.

(2) UV disinfection shown for illustrative purpose. Chlorine disinfection may be a preferable alternative. To be confirmed by additional engineering analysis, if project is implemented.

Figure 2
DESALINATION FACILITIES SCHEMATIC
REGIONAL WATER SUPPLY EVALUATION
CITY OF SANTA CRUZ/
SOQUEL CREEK WATER DISTRICT

easier to site, direct ocean intakes do not filter the ocean water as well as beach wells. This can lead to various water quality and treatment issues at the plant. Additionally, the open ocean intakes tend to be higher maintenance facilities.

Beach Intake Systems. Figure 3 shows the beach areas that were investigated as potential intake locations. The beach areas included the alluvial plain at the mouth of the San Lorenzo River, the Santa Cruz Boardwalk beachfront, beach areas between the Boardwalk and Pleasure Point, the Capitola beachfront, and New Brighton and Rio Del Mar beaches. Each beach area was evaluated to determine its capability to support beach intake and well systems, including vertical wells, horizontal/radial wells, and infiltration galleries. The summary information regarding the beach hydrogeologic characteristics are as follows (Hopkins Groundwater Consultants, October 2001):

- Beach areas typically comprise fine to coarse grained materials ranging from 10 to 20 feet thick.
- Seasonal beachfront dynamics (e.g., wave action) periodically alter the average beach profile.

Much of the beach areas have been equipped with protection systems (e.g., rip rap and/or seawalls) to minimize impacts of beachfront dynamics and associated erosion. These protection systems are generally not conducive to beach intake systems because they can either increase the potential for wave damage (if the intake system is placed on the seaward side of the protection system), or can impede landward flow of water (if the intake system is placed on the landward side of the protection system).

Based on these general characteristics, the production capacities for the various beach areas are estimated to range between 0.3 and 3 mgd (Hopkins, October 2001). The production capacity of the beach areas is constrained by a combination of factors. The beaches typically have small geometry – long, narrow and shallow – which results in a limited saturated thickness. The beaches also contain a relatively high percentage of fine-grained material, which limits production capacity for sustained periods. Given the typical beach conditions a single intake system is not considered viable (e.g., one system located at the Santa Cruz Boardwalk beachfront). A combination of beach intake systems (e.g., multiple systems at different beaches) could potentially provide increased capacity, but is similarly considered to be nonviable. The available hydrogeologic information confirms that the local beach areas are not well suited for intake systems even for low capacities, so a multiple intake system would not be practical or cost effective.

- ***Direct Ocean Intake.*** The concept for a direct ocean intake is to use the City's abandoned wastewater outfall as a new intake line. The abandoned 36-inch diameter outfall extends approximately 2,300 feet into the Pacific Ocean and has a final depth of approximately 40 feet below mean sea level. Conversion of the existing ocean outfall to an intake facility would require the following modifications:

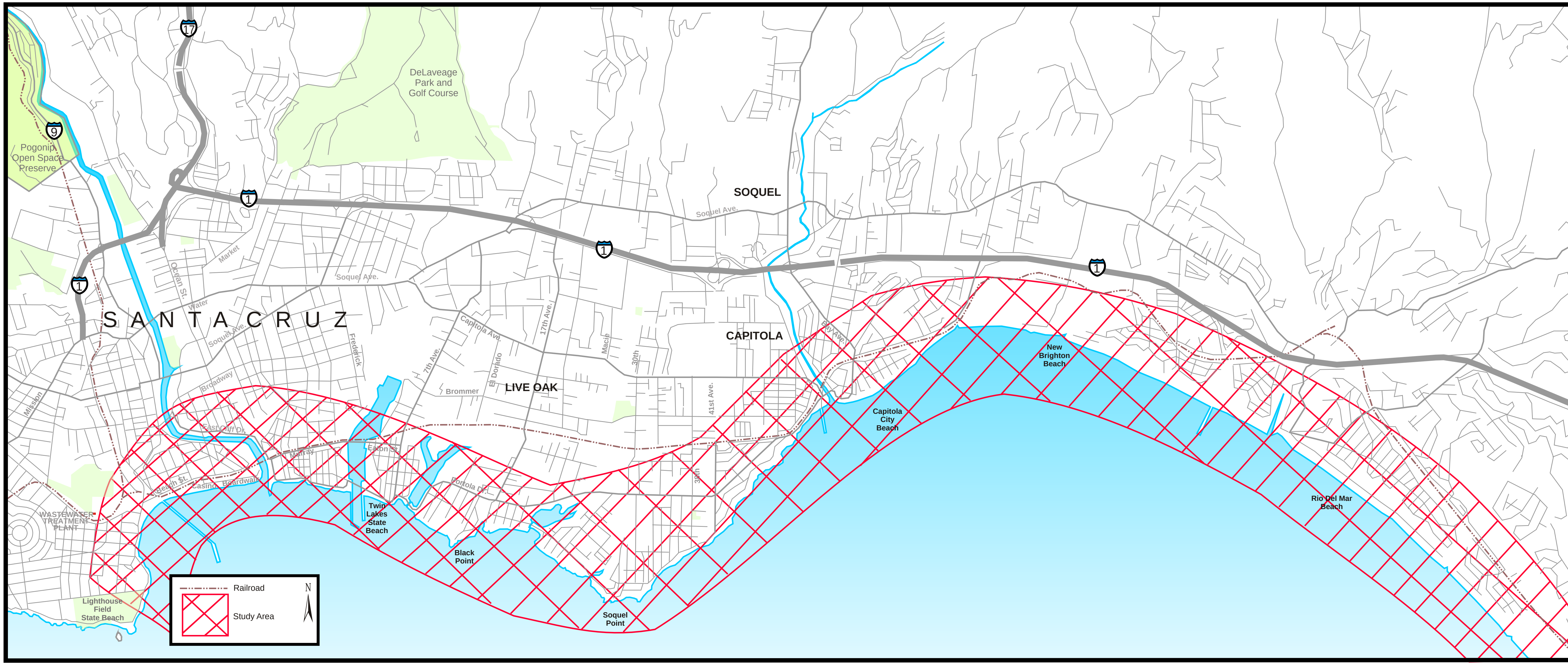


Figure 3
PROJECT STUDY AREA
FOR BEACH INTAKE AND DISPOSAL SITES
REGIONAL WATER SUPPLY EVALUATION
CITY OF SANTA CRUZ/
SOQUEL CREEK WATER DISTRICT

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- Piping modification at the end of the outfall to allow for water intake, including new baffle/screen to minimize capture of small particles and debris.
- New interior lining for the existing pipe.

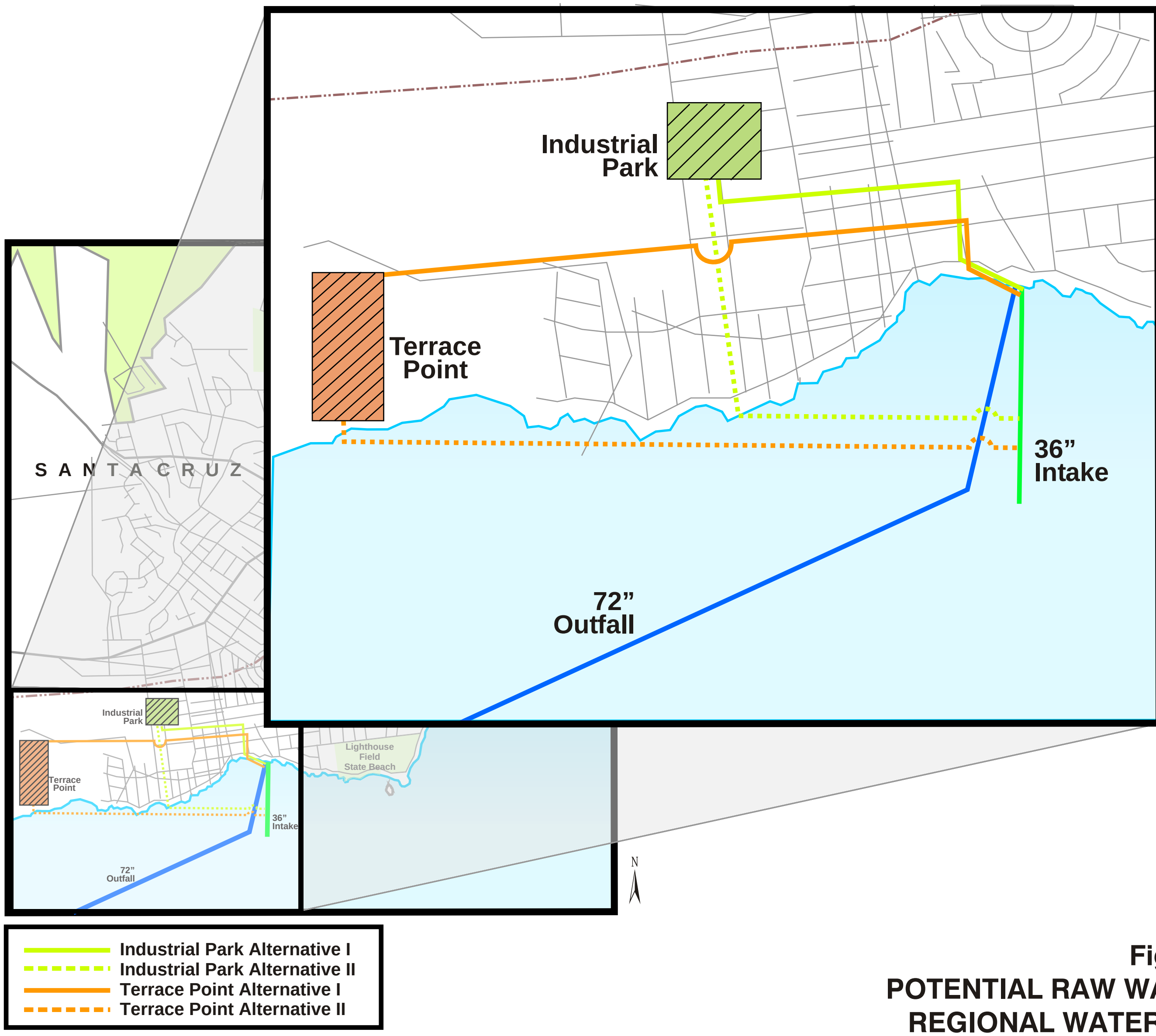
Modification to the existing onshore junction box to convert use from water outfall to water intake. With modifications in these areas, it is possible that the abandoned outfall line can be made suitable for an intake facility. However, a detailed engineering analysis of the pipeline and associated onshore structures is beyond the scope of this work, so it will be necessary to complete a more detailed evaluation of the outfall line in subsequent phases of the design. It is also important to note that the outfall line, although abandoned, was used within the last two years as an emergency outfall line during extreme and unusual operating conditions. The emergency conditions included a prolonged extreme winter storm event coupled with a failure of new effluent discharge pumps at the WWTP. The operating conditions that required use of the abandoned line were quite unusual and are not expected to recur. However, in the event that the abandoned line is converted to an intake, emergency use for wastewater outfall would not be possible without piping modifications for dual use application. This issue should also be explored in more detail during subsequent engineering analyses.

For the purposes of this study, conversion of the existing outfall line to a direct intake system is used as the basis for conceptual engineering and cost estimates presented in the remainder of this document. If subsequent engineering analyses identify that the abandoned line is not suitable (for engineering or WWTP operational reasons), costs for the intake system will need to be modified accordingly.

- ***Raw Water Conveyance.*** Raw water conveyance facilities would need to be sized to provide approximately 4.7 to 14.0 mgd of raw water, corresponding to about 2 to 6 mgd treated water.

- ***Raw Water Pipeline.*** A 24-inch diameter raw water conveyance pipe will work for the range of raw water conveyance needs.

  The length of the pipeline is dependent on two factors: the final location of the treatment facility (Terrace Point or the Industrial Park), and the pipe routing. Two options were considered for pipe routing, as shown in Figure 4. The first option would include interconnection to the landward end of the abandoned outfall, and routing the pipeline to the treatment facility in existing rights-of-way/easements in City streets. For this option, the length of pipe ranges between 5,000 feet and 10,000 feet for the Industrial Park and Terrace Point sites, respectively. The second option would include interconnection to the abandoned outfall in the ocean, and routing the pipeline to the



NOTES

1) Routes identified are two viable alternatives. Other similar routes are possible but are not shown because there are no appreciable differences in cost or implementation constraints.

2) Pipe size varies from approximately 24 inches to 30 inches for delivered water flows of 2 to 6 mgd.

3) Pipe length varies from 5,000 to 10,000 feet for Alternative 1; 5,700 to 8,000 feet for Alternative 2.

Figure 4

POTENTIAL RAW WATER PIPELINE ROUTES
REGIONAL WATER SUPPLY EVALUATION (1,2,3)
CITY OF SANTA CRUZ/
SOQUEL CREEK WATER DISTRICT

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treatment facility along the ocean floor, then transitioning to a new tunnel section well below City streets and infrastructure on land. For this option, the length of pipe ranges between 5,700 feet (1,600 feet installed on the ocean floor and 3,100 feet installed using trenchless installation techniques) and 8,000 feet (6,700 feet installed on the ocean floor and 1,300 feet installed using trenchless installation techniques), for the Industrial Park and Terrace Point sites, respectively.

Field reconnaissance in the area between the two treatment plant sites and the intake indicates that there are no "ideal" pipe routes for Option 1. Although feasible, installation of traditional "cut and cover" piping is generally not well-suited to the existing narrow residential streets. If possible, it is desirable to minimize (or avoid) construction through the residential area. Option 2 would include trenchless installation techniques for the landward portion of the pipe, which would avoid construction impacts compared to the more routine cut and cover installation. However, trenchless techniques are typically more costly per linear foot than the cut and cover method. A rough comparison of costs for the two options indicates that Pipe Option 2 would cost approximately three times more than Pipe Option 1.

- ***Raw Water Pumping.*** The pumping requirements (i.e., static head and friction losses) for either of the pipe route and site alternative options are not identical, but are similar. For the purposes of this analysis, it was assumed that a minimum of three pumps (two duty and one standby, each with variable speed drives) would be provided. For a treated water capacity of 2 mgd (requires 4.7 mgd raw water, minimum) three 1,600 gpm pumps at 90 hp each are required, based on an assumed head of 150 feet. For a treated water capacity of 6 mgd (requires 14.0 mgd raw water, minimum) four 3,200 gpm pumps at 175 hp each are required, based on the same head.

***Treatment and Support Systems*.** The RO process desalinates water using a semi-permeable membrane that allows water to pass through when pressure is applied, while not permitting dissolved ions to pass through. The RO membrane is rolled up into spiral wound elements to yield a relatively high membrane surface area in a compact space. The pressurized sea water flows along the surface of the membrane where a portion of the water - the permeate or "product" - and a small amount of the ionic impurities diffuse through the membrane. The remaining water, and most of the dissolved ions such as sodium and chloride is rejected as concentrate or "brine."

A high level of pretreatment is required upstream of the RO process to minimize maintenance problems and associated costs, and to maximize productivity. Particulate material in the raw water stream should be removed upstream of the RO process. This could be accomplished through a variety of pretreatment processes. For the purposes of this conceptual level design two pretreatment processes were considered - pretreatment with membrane filters or with a more conventional treatment approach (coagulation followed by gravity filters). In addition, it was assumed that small amounts of sulfuric acid and an anti-scalant would be added prior to the RO membrane filters to prevent scaling on the membrane surface. Equipment would also be provided for disinfection and chemical post-treatment. Post-treatment of the permeate water is required to control the corrosiveness of the water and to ensure that it has a palatable taste.

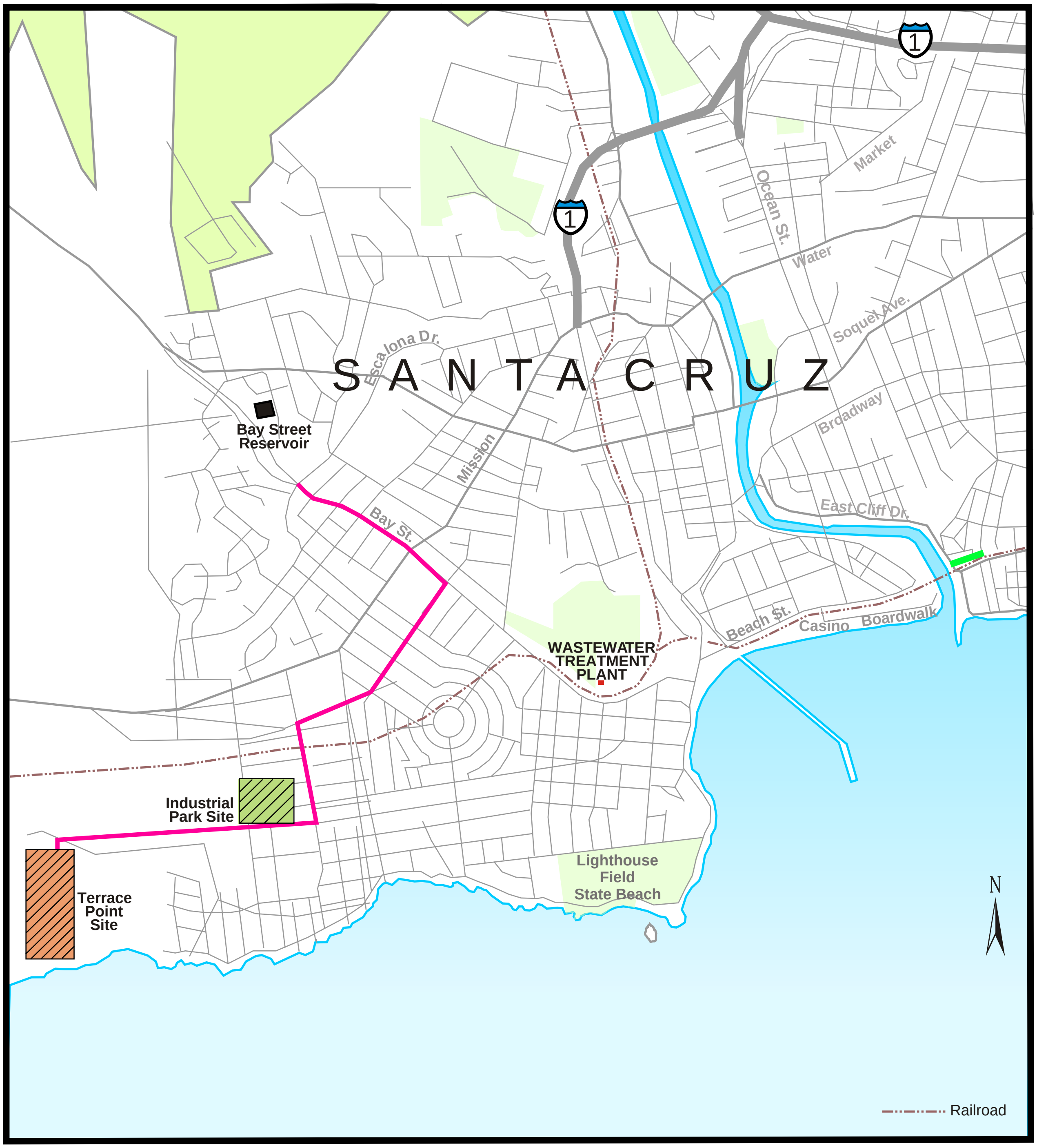
As previously mentioned, with conventional RO equipment the maximum product water recovery for seawater desalination is approximately 40 to 50 percent. A recovery factor of 45 percent was used as the basis for this report. The main limitation for this is the operating pressure; recovery greater than 45 percent would require more pressure than the conventional maximum of 1,000 psi. It is important to note that as membrane technology continues to improve an economically obtained recovery factor will continue to rise. However, for the purposes of this report RO units with higher recovery factors were not considered. An energy recovery system would be installed on the brine system downstream of the RO units to improve the overall energy efficiency of the operation.

The number of membrane RO units and the layout of those units varies for each of the flows evaluated. For this analysis it was assumed that the RO units would have a design flow of 1.0 to 1.5 mgd. When one of the units is out of service, the remaining units could be operated at a higher pressure and a higher flux to yield a larger percentage of the nominal flow rate. This would occur if a unit was down for maintenance or backwash. Following filtration and post-treatment, the finished water would leave the RO desalination facility for subsequent disinfection, storage, and distribution.

(***Note:*** Detailed selection and design of reverse osmosis desalination processes requires knowledge of the raw water composition. Since the seawater composition at the proposed site was not available, the composition of typical seawater with a total dissolved solids (TDS) concentration of approximately 34,400 mg/L was used for the conceptual level design presented in this study. For this conceptual level design it was assumed that dissolved iron, manganese, and hydrogen sulfide gas would not be present in the raw water. This is a reasonable assumption for a planning-level design, because the presence of these materials would not significantly affect the overall project cost. Although assuming a seawater composition allows for the conceptual evaluation and selection of a desalination process, it is recommended that seawater samples be obtained and analyzed prior to further design of the treatment facilities.)

Finished Water Conveyance. The finished water conveyance includes two elements: pumping and piping for delivery to the City's distribution system and distribution system upgrades within the City system for delivery to the District.

- ***Piping and Pumping to the City Distribution System.*** Finished water piping and pumping facilities were determined based on distribution system model simulations for water delivery scenarios ranging from 2 to 6 mgd. Preliminary model results indicate that it is necessary to have a dedicated pipeline from the facility to Escalona Drive at the intersection of Bay Street, rather than all the way to Bay Street Reservoir. Figure 5 shows a potential finished water pipeline route, which would include approximately 11,000 to 16,000 feet, depending on the site alternative. The pipeline size ranges from 16 inches for 2-mgd capacity, to 20 inches for 6-mgd capacity.



NOTES

1) Route identified is one potentially viable alternative. Other similar routes are possible but are not shown because there are no appreciable differences in costs or implementation constraints to the route shown.
2) Pipe length varies fromapproximately 11,000 feet to 16,000 feet for Industrial Park and Terrace Points site, respectively.
3) Pipe size ranges from 16 to 20 inches for treated water delivery capacity range of 2 to 6 mgd.

Figure 5
POTENTIAL FINISHED WATER PIPELINE ROUTE(1,2,3)
REGIONAL WATER SUPPLY EVALUATION
CITY OF SANTA CRUZ/
SOQUEL CREEK WATER DISTRICT

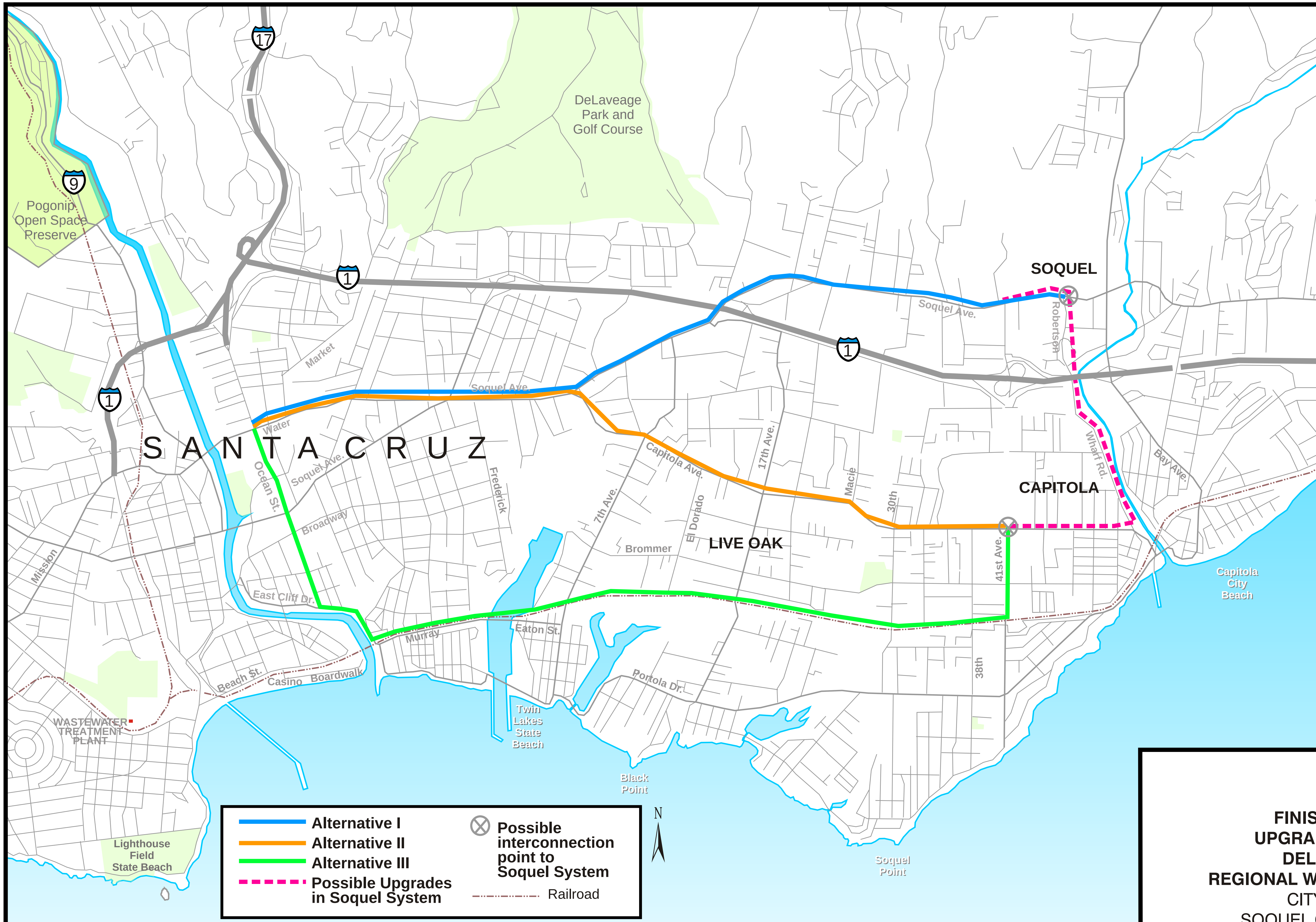
For the purposes of this analysis it is assumed that pumping and piping would be provided by a minimum of three pumps (two duty and one standby, each with variable speed drives) and approximately 11,000 feet of distribution piping. The number, size and capacity of the pumps will vary depending on the flow requirements. For example, for 2-mgd capacity three 50 hp pumps at approximately 700 gpm each would be required. For 6-mgd treated water delivery capacity, three 150 hp pumps at approximately 2,100 gpm each.

- ***Distribution System Upgrades*.** The District's interconnection points to City's system are located in an area that includes predominantly old, undersized piping. The existing piping was not designed with the intent to serve as a hydraulic "backbone" to provide water to the District. To identify possible upgrade needs, a preliminary hydraulic analysis of the City's system identified piping upgrades for a range of possible delivery conditions to the District. The modeling identified the three potential piping alternatives, as shown on Figure 6. Pipe sizes range from 16- to 20-inch for a delivered capacity of 2 to 6 mgd. Pipe lengths range from approximately 19,500 feet for Alternatives 1 and 2 to approximately 24,500 feet for Alternative 3. Of the three alternatives, Alternative 3 is considered to be the least likely because it requires easement access along the railway, which, based on past experience, is very difficult to obtain. For the purposes of this analysis, cost estimates are developed assuming 19,500 feet of 16-inch diameter piping.

The hydraulic analysis did not include an evaluation of distribution capacity within the District. A cursory investigation of the District's system indicates that interconnection at the intersection of Capitola Avenue and 41st for Alternative 2 would require District-side upgrades, at least up to the intersection of Soquel and Robertson Avenue, as shown on Figure 6. It is also important to note that it would be preferable to have more than one connection point between the City and District systems, which would increase piping requirements and costs. A detailed analysis of distribution capacity within the District is beyond the scope of work of this project, but should be conducted as part of preliminary engineering of a new desalination project, if implemented. Multiple interties would increase operating reliability and flexibility.

The hydraulic analysis assumes that the City would deliver 2 to 6 mgd to the District, but does not assume that the water would be 100 percent desalinated water. As shown on Figure 6, the City-side distribution system upgrades do not extend all the way to treatment plant. Without a direct interconnection to the desalination plant, it is likely that the water delivered to the District would be primarily treated surface water from the City's Graham Hill WTP and groundwater from the City's Beltz wells.

Brine Disposal. Under normal operation, reject brine from the RO units will pass first through an energy recovery turbine (which provides supplemental power to minimize the raw input power required for the high pressure pumps) and then to the brine disposal pipeline. Two alternatives for the brine disposal were considered:



NOTES

1) Routes identified are three most viable alternatives considering existing distribution system constraints.

2) Pipe length varies from approximately 19,500 feet for Alternatives 1 and 2 to 24,500 feet for Alternative 3.

3) Pipe size varies from approximately 16 inches to 20 inches for delivered water flows of 2 to 6 mgd.

4) Pipe rates and pipe sizing shown are based on installation of new parallel pipes, not upsizing of existing infrastructure (see discussion in text).

Figure 6
FINISHED WATER PIPING UPGRADE ALTERNATIVE FOR DELIVERY TO SOQUEL[1,2,3,4]
REGIONAL WATER SUPPLY EVALUATION
CITY OF SANTA CRUZ/
SOQUEL CREEK WATER DISTRICT

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- ***Beach Well Discharge.*** The same beach areas that were investigated as potential intake locations were also evaluated as potential discharge locations (see Figure 4). The various beach areas were evaluated to determine their capability to support beach discharge for brine.

  The capacity of the beach areas to support brine discharge is generally considered to be constrained for similar reasons that make beach intake systems infeasible. These factors include small geometry (long, narrow beach areas resulting in a limited saturated thickness), and relatively high percentage of fine-grained material in the beach sands (limits ability to produce significant quantities for a sustained period of time). Given these conditions, a beach well discharge of brine is not considered viable.

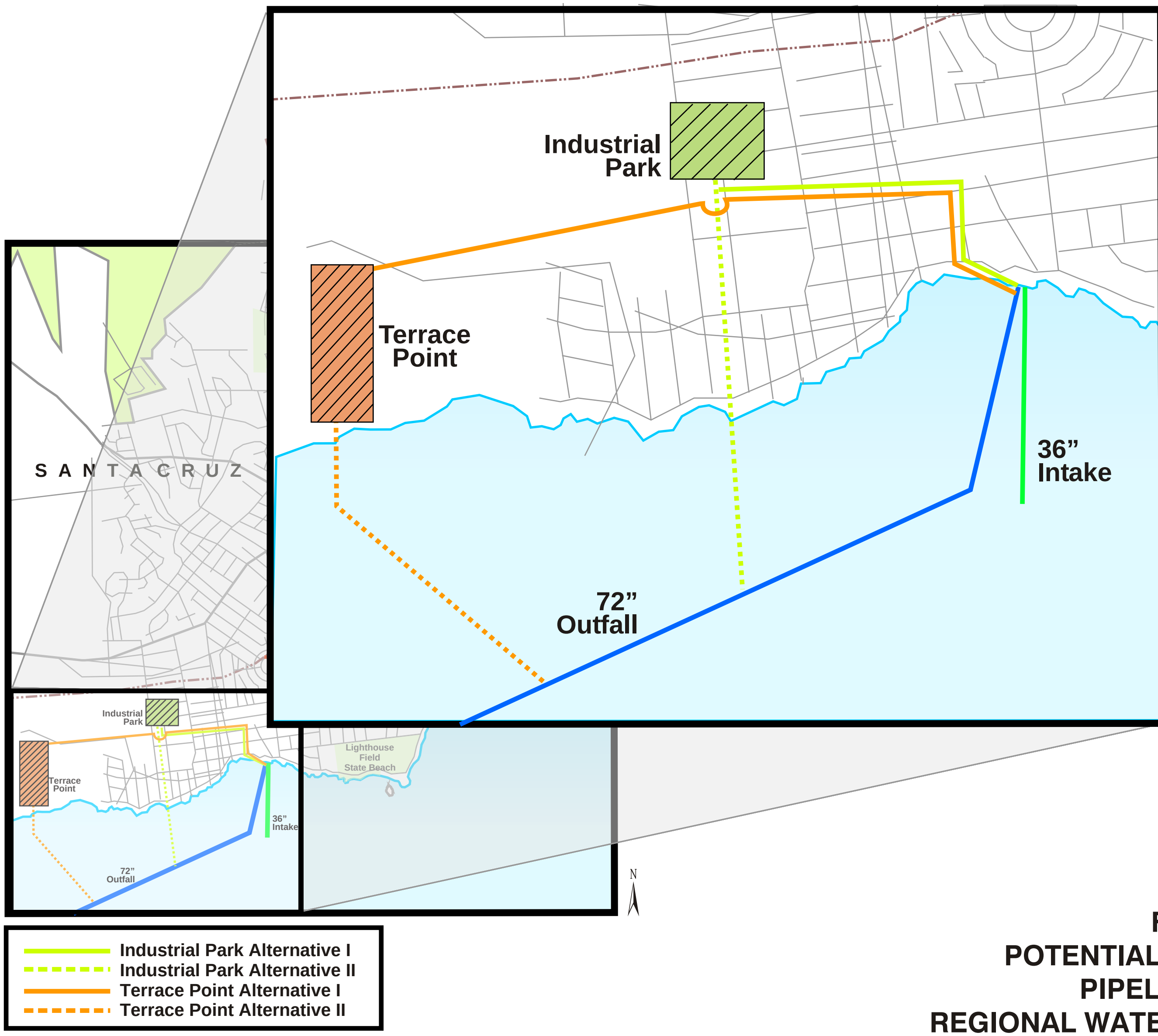
- ***Connection to the Wastewater Ocean Outfall.*** The capacity of the ocean outfall by gravity flow is approximately 31 mgd at mean tide level and 20 mgd at extreme high water. Additional capacity up to 80 mgd is available via pumping. The current average dry weather flow (ADWF) from the wastewater treatment plant is approximately 10 mgd, although the plant has a design capacity of approximately 17 mgd, and has space for up to 23 mgd total capacity if needed in the future. For the City's operation, it is expected that the desalination facility would operate primarily during the high-demand summer months when wastewater flows are lowest. For supply to the District, the plant could be operational each day in nondrought years.

  A comparison of average wastewater flows and a range of brine discharge ranging from 2.4 to 7.3 mgd (expected worst case) confirms that there is sufficient capacity in the outfall. However, modeling runs indicate that dilution capacity at the outfall discharge, not hydraulic capacity, is a limiting factor. This is because the addition of brine to wastewater results in a composite solution of lower density. The lower density solution is less buoyant, and does not mix as effectively with the surrounding and overhead water at the outfall discharge. Flow equalization of the brine will be required under summer operating conditions, during which wastewater flows are typically the lowest, concurrent to maximum brine discharge from a desalination plant. Flow equalization is required to maintain maximum dilution of the combined wastewater discharge to the ocean. For treated water deliveries of 2 and 6 mgd, respectively, the required brine flow equalization storage ranges from approximately 0.2 to 1.4 MG (ref. Appendix D - Brine Disposal Dilution Analysis).

  (Note: The modeling analysis indicates that dilution for brine discharge could be a significant limiting parameter during drought conditions, even if flow equalization is provided. For example, simulated drought conditions for year 2002 projected wastewater flows indicate that the maximum desalination operating capacity would be approximately 4 mgd, <u>even if 1MG flow equalization is provided.</u> This is due to the fact that wastewater flows will be reduced during a drought (due in part to conservation and curtailment) so there is less available wastewater flow for dilution. It is reasonable to project that wastewater flows will increase in the future, even during drought conditions, so it is possible that the desalination plant could operate at up to 6 mgd. Additional assessment of drought operating scenarios, with due consideration for future water demands and diurnal wastewater flow patterns during summer conditions, is beyond the scope of this document. Such an evaluation is required to determine the maximum desalination capacity based on available dilution, and should be investigated further during preliminary design, if this project is implemented.)

The size of this brine disposal pipe ranges from 16 to 24 inches, depending on capacity requirements. The length of the pipeline will depend on two factors: the final location of the treatment facility (e.g., Terrace Point or the Industrial Park), and the pipe routing. Two options were considered for pipe routing similar to the options for raw water piping, as shown in Figure 7. The first option would include interconnection to the landward end of the existing outfall, and would be routed to the outfall in existing rights-of-way/easements in City streets. The length of pipe ranges between 5,000 feet and 10,000 feet for the Industrial Park and Terrace Point sites, respectively. The second option includes interconnection to the existing outfall in the ocean, with routing in a new tunnel section well below city streets and infrastructure, then transitioning to the ocean floor for the seaward portion. For this option, the length of pipe ranges between 4,700 feet (1,700 feet installed on the ocean floor and 3,000 feet installed using trenchless installation techniques) and 3,700 feet (400 feet installed on the ocean floor and 3,300 feet installed using trenchless installation techniques), for the Industrial Park and Terrace Point sites, respectively. As discussed above for the raw water pipes, there are no “ideal” pipe routes. If possible, it is desirable to minimize (or avoid) construction through the residential area. Option 2 would include tunneling for the landward portion of the pipe, which would avoid construction impacts of a “cut and cover” installation. However, trenchless techniques are typically more costly per linear foot than the cut and cover method. A rough comparison of costs for the two options indicates that Pipe Option 2 is approximately 3.5 times more than Pipe Option 1 for the Industrial Park site. However, for the Terrace Point site, Pipe Option 2 is approximately 5 times as expensive as Pipe Option 2.

Power Supply. Seawater desalination requires approximately one megawatt of power per mgd of treatment plant finished water production. This is equivalent to 18 to 20 kilowatt-hours per 1,000 gallons of water produced.



NOTES

1) Routes identified are two viable alternatives. Other similar routes are possible but are not shown because there are no appreciable differences in cost or implementation constraints.

2) Pipe size varies from approximately 16 inches to 20 inches for brine flow water flows of 2.5 to 7.5 mgd.

3) Pipe length varies from 5,000 to 10,000 feet for Alternative 1, and 3,700 to 4,700 feet for Alternative 2.

Figure 7
POTENTIAL BRINE DISPOSAL PIPELINE ROUTES
REGIONAL WATER SUPPLY EVALUATION(1,2)
CITY OF SANTA CRUZ/
SOQUEL CREEK WATER DISTRICT

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Although this is a large power requirement, preliminary review of power availability confirmed that there is a power substation near the industrial park area that could provide the necessary power. However, in order to provide power to the treatment plant from the substation a new high voltage line would need to be installed and connected to a transformer on the treatment plant site. The new transformer would in turn deliver low voltage power to the treatment plant.

Two alternatives to conventional power supplies, photovoltaic and fuel cells, were previously evaluated (Alternative Water Supply Study, November 2000). The evaluation concluded that alternative power sources, although promising, are not feasible at this time for power requirements typical of many large-scale, industrial-type applications. Although it is possible that alternative energy sources may be developed to the point where they can produce larger quantities of power at lower costs, this change does not appear to be forthcoming in the foreseeable future. Accordingly, the project concept for the desalination facilities assumes that power would be provided by conventional sources. The project concepts do assume, however, that the facilities would be designed to minimize energy use to the extent practical. For example, energy recovery turbines would be used to minimize the power requirements for the high-pressure pumps.

Cost Estimate for Desalination

The estimated costs for desalination are shown in Table 3. As shown in the table, the total annual costs have been calculated for three Case Conditions: 2, 4, and 6 mgd. Treatment facilities sized within this range would provide additional supply to cover a wide range of potential supply deficits during a drought for the current and future demand conditions.

Implementation Analysis

The installation of seawater desalination facilities in coastal communities such as Pacifica and Santa Barbara and a planned installation in Cambria demonstrates that such facilities can be implemented with due consideration of technical, environmental, and institutional issues.

Technical Issues. The on-land facilities associated with a desalination system (i.e., pipelines, pump stations, and treatment systems) do not present any unusual engineering or constructability constraints. The engineering and construction of the seaward facilities present more challenges, and will require potentially complicated underwater construction.

Table 3 Conceptual Costs for Desalination
Evaluation of Regional Water Supply Alternatives
City of Santa Cruz/Soquel Creek Water District

Delivered Water Capacity	Industrial Park		Terrace Point	
	Capital Cost(1,2,4,5)	Operating Cost(3,5)	Capital Cost(1,2,4,5)	Operating Cost(3,5)
2 mgd	$26.1 - $29.3	$2.0 - $2.1	$27.2 - $28.9	$2.0 - $2.1
4 mgd	$37.7 - $40.9	$3.9 - $4.0	$39.2 - $40.5	$3.9 - $4.0
6 mgd	$49.4 - $52.9	$5.8 - $5.9	$51.3 - $52.3	$5.8 - $5.9

Notes:
(1) Capital cost range reflects costs for piping Alternatives 1 and 2 in millions of dollars.
(2) Capital cost estimates include allowance for engineering, legal, construction administration (see Appendix B).
(3) Operating cost range reflects costs for piping Alternatives 1 and 2. Operating cost assumes production at stated capacity, each day of the year.
(4) Capital cost estimates do not include costs for electrical distribution system upgrades that may be required.
(5) Cost in millions of dollars.

Environmental Issues. Based on preliminary environmental review, there do not appear to be any significant environmental issues related to new infrastructure or construction. The most notable issues are construction related, although there are options for mitigation. Irrespective of the site location, there would be a substantial construction effort in potentially environmentally sensitive areas, including the ocean, so numerous permits will be required. The permitting effort and the associated environmental impact assessment/ documentation would require coordination with multiple agencies. Table 4 summarizes potential environmental issues.

Institutional Issues. The primary institutional challenge for desalination is siting of the treatment facilities. Although land is potentially available at the Industrial Park and Terrace Point sites, considerable additional work is needed to confirm a site location and secure the land. Before land can be secured, environmental documentation for the project must be certified.

Coastal Commission and Monterey Bay Marine Sanctuary. Based on preliminary discussions with representatives from the Coastal Commission, there are several potential issues related to coastal siting and operation of the desalination facility near the marine sanctuary boundaries. These issues include habitat impact (e.g., potential for impingement on ocean intake screens, species susceptibility to brine discharge, etc.), construction-related issues near the coast and in the ocean, and growth-related issues. These issues are all prominent; however, none are “new” or outside the realm of what would typically be addressed via the environmental review and permitting process.

Table 4 Summary of Environmental Issues for Desalination
Evaluation of Regional Water Supply Alternatives
City of Santa Cruz/Soquel Creek Water District

Issue	Impact at City Industrial Park Site[(1)]	Impact at Terrace Point Site[(1)]	Comments
Construction Related Impacts	Minor	Minor-Moderate	• Visitor uses may be sensitive at Terrace Point Site
Compatibility with Adjacent Land Uses	Minor	Minor-Moderate	• Terrace Point site aesthetics would need to be compatible with current and future uses
Visual Impacts	Minor	Minor-Moderate	• Terrace Point site aesthetics would need to be compatible with current and future uses
Potential for Cultural Resources	Minor-Moderate	Minor	• Excavation activities could uncover cultural, archaeological, historical or paleontological resources
Potential for Public Controversy	Moderate-Major	Moderate-Major	• General Plan requires a specific plan for Terrace Point prior to development • Desalination facility would be located within area designated for "coastal-dependent" uses • High potential for public/regulatory concern regarding impacts to marine life (i.e., impingement on intake screens, effect from brine discharge, etc.)
Potential to Disrupt Traffic From Pipeline	Major	Major	• Potential disruption of traffic on Mission Street Boulevard and Bay Avenue during construction
Growth Inducement	Major	Major	• Size to accommodate growth consistent with the City and County's General Plans • Growth inducement is a potential impact of any project increasing water supplies • Potential impact may be mitigated if used as a regional supply project, serving the City only in drought conditions and other users in nondrought years
Energy Usage	Major	Major	• Significant energy requirements • Coastal Act requires minimizing energy consumption

Notes:
(1) Minor/Major/Moderate represents the anticipated severity of the issue and is based on a conceptual-level environmental and regulatory constraints analysis.

Summary of Implementation Issues

Potentially significant issues related to the implementation and viability of this project are:

- Due to the nature of construction of facilities in the ocean and planned discharge of brine into the ocean, there will be considerable coordination requirements with multiple agencies to complete the necessary environmental review and documentation. This process would likely take 18 to 24 months to complete, and could delay implementation of the project.
- Facility siting alternatives have been identified through initial screening. Additional work is needed to confirm a site location and secure the land. Final site selection would need to be determined based on feasibility of acquisition. Before land can be secured, environmental documentation and certification of the project concept must be completed.
- Sizing of the facility is critical to development of expected costs for construction and operation. Facility sizing would need to be confirmed and coordinated with planned conservation/curtailment efforts, and/or with planned development of alternate sources of supply (see also paragraph below regarding capacity and brine discharge limitations and considerations).
- The planned use of the abandoned outfall as a new intake structure and use of the existing wastewater outfall for brine disposal are based on conceptual engineering review of these facilities, including hydraulic capacity, age/condition, and ability to construct required modifications. Additional engineering at the preliminary design level will be required to more completely describe the engineering details for use of these facilities.
- Preliminary modeling analyses indicate that the plant capacity may be limited by available dilution capacity for the brine discharge, even if the brine discharge is equalized throughout the day. Additional analysis of future drought conditions, water supply demand and wastewater flows is needed to determine the maximum limitation of dilution. It is important to note that it may be possible to minimize the effect of limited dilution capacity by discharging brine only during times of peak diurnal wastewater flows (i.e., turn off or turn down the plant for 4 to 6 hours during the day so little or no brine is generated during periods of low wastewater flow). However, to do so would require that the desalination plant capacity be increased beyond the 6 mgd maximum assumed in this document in order to offset the “lost” production during the plant downtime during the day. The analysis of plant operation/capacity relative to brine discharge and dilution needs to be considered concurrent to the final sizing analysis.)

WASTEWATER RECLAMATION

The City's previous evaluation of water supply alternatives identified reclamation as potentially viable, provided that the reclaimed water could be "exchanged" for an alternative supply source such as groundwater (ref. Alternative Water Supply Study, November 2000). The use of reclaimed water for irrigation within the City (e.g., golf courses, parks, cemetery, etc.) was also evaluated, and was determined to be nonviable because it provided no appreciable additional supply benefit. Accordingly, the general project concept for a regional facility is to exchange reclaimed wastewater for new raw water supplies, rather than offset demand via domestic outside irrigation uses (nonagricultural irrigation).

Other reclaimed water uses for the City, including groundwater recharge and direct reuse, were previously determined to be nonviable and are not considered herein (ref. Alternative Water Supply Study, November 2000). A review of other similar reclaimed water uses for the District is beyond the scope of this document. The use of reclaimed water for "in-District" uses (i.e., greenscape irrigation) has been evaluated by the District and it was determined that any reclaimed application of this type would be of minimal benefit with respect to demand reduction (ref. communication with District staff, December 2001).

Incremental Supply from Reclamation

Santa Cruz

Based on previous findings, a reclaimed water exchange with the North Coast farmers is considered to be a viable project alternative for the City. Reclaimed water would be delivered to farmers for irrigation supply in exchange for groundwater that the farmers currently use.

The amount of water that could be exchanged with the North Coast farmers is unknown because there are no accurate records to indicate the volume of groundwater actually used for irrigation. A review of aerial photographs and crop maps indicates that there are approximately 1,400 to 1,500 acres of irrigated land between the City and Lidell Creek. The irrigated land northward of Lidell Creek is believed to be irrigated with surface water, which could not be exchanged due to water-right constraints (ref. discussion with City staff, October 2001). Approximately 800 to 850 acres are believed to be irrigated predominantly with groundwater, which could potentially be exchanged for use by the City and the District. Assuming an irrigation of 1.5 to 2 acre-ft of water per acre (a reasonable range for crop types along the North Coast), a rough estimate of the groundwater usage is 400 to 500 MG per year (+/- 1,200 to 1,500 AFY). The estimated available groundwater yield based on review of coastal hydrogeology ranges from 500 to 700 MG/yr (1,500 to 2,000 AFY) (ref. Alternative Water Supply Study, November 2000).

(***Note***: Two other reclamation alternatives, in-stream exchange for surface water and groundwater recharge with reclaimed wastewater, were also examined for applicability to the City. In-stream exchange is nonviable, primarily because it provides no storage component for use during drought years. The City already has limited raw water storage capacity, so there is no benefit of diverting "excess" stream flow when it is available in the high runoff months. An in-stream exchange with reclaimed wastewater would be similarly constrained so it would be of no appreciable benefit, even in drought years. For example, in drought years the City's stream sources have little or no flow, so there is no appreciable water to "exchange." Other technical and implementation issues could further constrain the viability of an in-stream exchange. A discharge of wastewater to the stream – even if treated to a high degree – would only be viable if it occurred downstream of the City's diversion points, so as not to conflict with the City's diversion for potable supply. There does not appear to be any substantive advantage to pursue a change in the water rights that would allow a modification to the City's existing diversion locations or conditions for an in-stream discharge of reclaimed water. Groundwater recharge with reclaimed wastewater was previously examined and determined to be nonviable because it provided no appreciable supply during the drought periods (ref. Preliminary Investigation of Water Supply Alternatives, Technical Memorandum 4 – "Alternative Screening," February 2000)).

Soquel Creek Water District

The District's need for additional supply is less than the City's, so it is possible that a water exchange alternative could provide all of the projected supply shortfall in a nondrought year. Two project concepts for the District are considered potentially viable:

- ***Reclaimed Water for Agricultural Application.*** As noted above, a regional project would provide reclaimed water to the North Coast Farmers in all years, and the District would receive exchanged water in nondrought years, similar to the project concept for a regional desalination facility. As previously detailed, the estimated additional supply from the North Coast ranges from approximately 400 MG/yr to 700 MG/yr based on the estimated irrigation usage and groundwater basin yield.

- ***In Stream Exchange.*** The District has previously evaluated a water supply project that would provide new supply via diversion from Soquel Creek. Although this project concept is potentially viable, there are seasonal/annual diversion constraints that could potentially limit diversions from the creek. The final analysis of diversion related constraint is not yet complete, but the constants could limit the supply availability to approximately 400 MG/yr to 450 MG/yr (1200 to 1400 AF/yr), or approximately 200 MG/yr to 250 MG/yr less than the District's projected need of 650 MG/yr. The amount of supply from the project would be enhanced if the minimum stream flow downstream of the diversion point could be maintained, irrespective of diversion activity by the District.

  The in-stream exchange concept would provide up to 3.2 mgd (5 cfs) of reclaimed wastewater to augment stream flows during periods of diversion. By providing a source of supply to augment stream flows, the District would have more flexibility to divert water from the stream and increase its diversion up to its projected need of 650 MG/yr.

  (***Note:*** The in-stream exchange project would provide supplemental supply to the District only, and is not considered a "regional" project that would provide supply benefit to the City. It is considered in this document because it provides an opportunity to use reclaimed water during periods when it is not being used for irrigation. If this project was implemented in conjunction with a groundwater exchange project, there would be opportunity for cost sharing of capital and operating costs.)

Summary - Incremental Supply from Reclamation

The possible supply available from the North Coast project is bracketed by the amount of irrigation exchange potential and/or the groundwater basin yield. The range of available supply from this alternative is approximately 400 MG/yr to 650 MG/yr, based on irrigation exchange potential and groundwater basin yield, respectively. The supply available from the in-stream project is essentially fixed at 650 MG/yr based on the District's needs.

The supply needs for both projects are similar, but are not additive. For example, the irrigation needs along the North Coast occur during the summer months (May through August), whereas the stream diversion for the District would occur during the winter months (terminating in April). For the purposes of conceptual planning, the supply to be provided by the reclamation facilities is assumed to be up to 700 MG/yr, which covers the range of needs for both project concepts.

Facility Siting Analysis

For planning purposes, it is assumed that the reclaimed water supply for both the irrigation and in-stream project concepts would be provided by new tertiary treatment systems at the City's existing wastewater treatment plant. Because there is very little available land at the existing wastewater treatment plant, it was assumed that membrane filtration was the only treatment process considered for the proposed reclaimed water facilities. This is mainly due to the relatively small footprint that these facilities typically require. As-built drawings of the plant were reviewed and a site visit was conducted to confirm this assumption. A portion of the available site area has been earmarked for possible future facilities (either new primary treatment or odor control not related to reclamation). However, based on the recent track record of operation it does not appear that the future facilities will be needed, so the available space could be used instead for new reclamation treatment processes (ref. communication with Brown and Caldwell Engineers, September 2001).

(***Note:*** It is also possible that treatment facilities could be sited elsewhere at a "satellite" location in the City or in the District. While this would raise the capital cost of the facility, an off-plant location would reduce some of the aforementioned siting and congestion issues at the site. However, based on a cursory review of areas within the City, there do not appear to be sites that are clearly preferable/viable for satellite treatment plants. For example, the City wastewater facilities are already located more toward the north end of the City, so there is no apparent advantage of a satellite facility further northward, closer to the coastal farms. It is also important to note that satellite facilities would require duplication of the basic treatment process at the City's plant, and additional processes to provide a higher quality effluent. The need for a completely new set of treatment processes increases the site area requirements, and also increases costs of a satellite facility. The need for increased site area requirements is a particular constraint for the District's in-stream project location, which is already limited with respect to available site area. For these reasons satellite facilities are considered to be marginally viable at best, and were not considered in detail for this analysis.)

Facility Requirements for Wastewater Reclamation

New reclamation facility requirements are shown schematically in Figures 8, 9, and 10. The facilities include:

- ***Treatment System.*** The City's wastewater treatment plant produces water that is suitable for some agricultural applications (indirect irrigation of nontable crops), and for limited-public-access irrigation. Based on experience with other reclamation projects in the state, a reclaimed supply that has no restrictions on use is most likely to be implemented. Per the state's regulatory requirements (Title 22, Division 4, Chapter 3 Reclamation Criteria) such "unrestricted use" of reclaimed water requires additional treatment to that currently provided at the City's treatment plant. The current treatment facilities provide "secondary" treatment (sedimentation, aeration/clarification, and disinfection) whereas unrestricted use would require "tertiary" treatment (secondary treatment plus filtration and additional disinfection). New filtration treatment and modifications to the disinfection system would be required to upgrade treatment for unrestricted use applications.

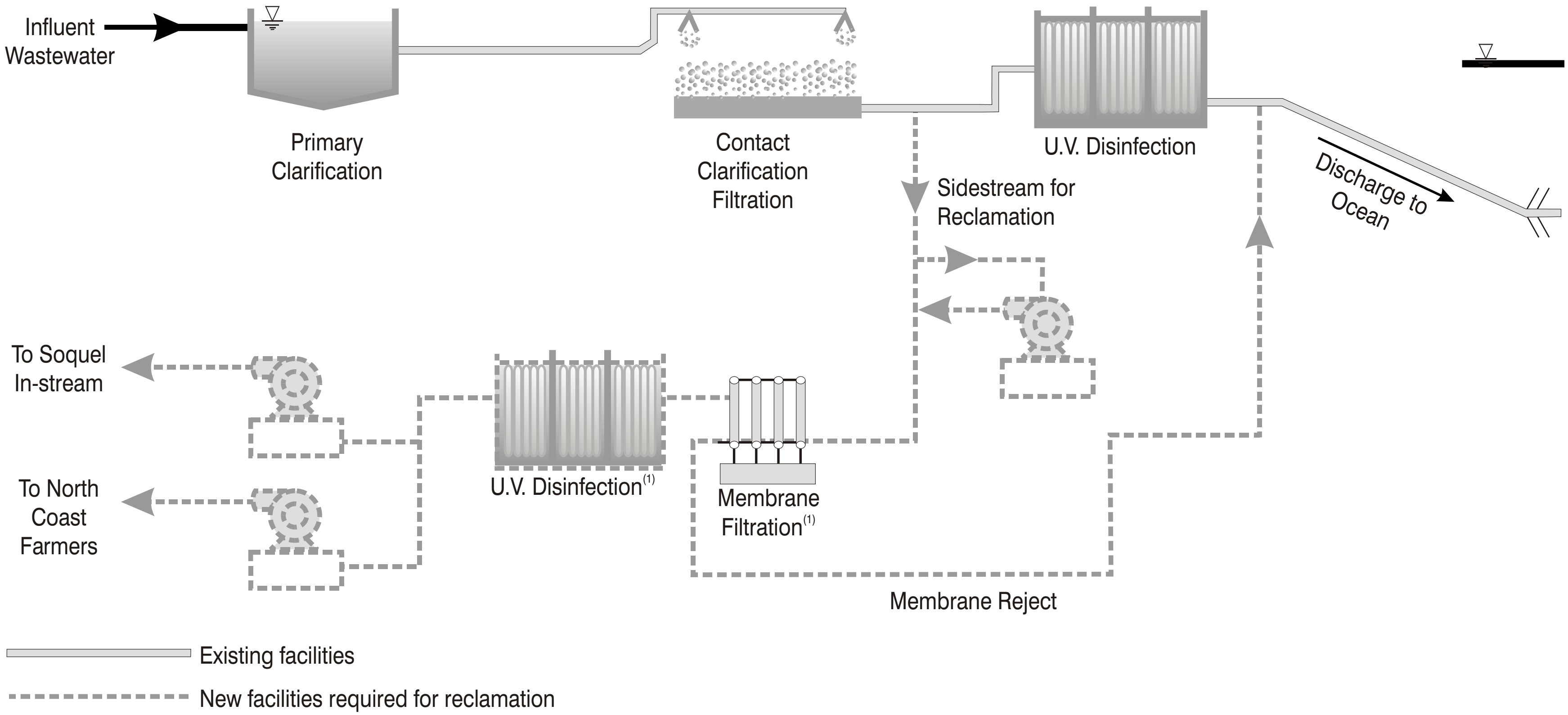
  (***Note***: For the purposes of developing costs for this evaluation, it is assumed that a desalting process such as RO membranes may also be required. Desalting may be required to minimize dissolved salts in the water used for irrigation, or water discharged for the in-stream exchange.)

- ***Reclaimed Water Conveyance System.*** As shown in Figures 9 and 10, the conveyance system includes pumps and new reclaimed water conveyance piping, either to the North Coast users or to the District's stream diversion site.

- ***Groundwater Facilities and Conveyance System.*** As shown in Figure 9, new facilities to pump and distribute the groundwater from the North Coast include new groundwater wells and new distribution piping from the wells to the City's Coast Pipeline. For this analysis, it is assumed that groundwater would be conveyed to the City via the Coast Pipeline.

  (***Note****:* The Coast Pipeline has hydraulic capacity restrictions that would limit the ability to convey the groundwater supply during some periods of the year. The City has plans to make pipeline improvements that could provide the capacity required for the additional groundwater flows. Pipeline capacity requirements for the groundwater supply should be evaluated as part of preliminary engineering for the Coast Pipeline improvements (scheduled for completion in summer 2002)).

- ***City Distribution System Upgrades.*** The project concept for groundwater exchange provides water to the District in nondrought years. The water would be delivered to the District via the City system, similar to the project concepts for delivery of desalted



NOTES

(1) Membrane filtration and U.V. Disinfection shown. Granular media filtration and/or chlorine disinfection may be preferred alternatives. To be confirmed in design phase, if implemented.

Figure 8
RECLAMATION FACILITIES SCHEMATIC
REGIONAL WATER SUPPLY PROJECT
CITY OF SANTA CRUZ/
SOQUEL CREEK WATER DISTRICT

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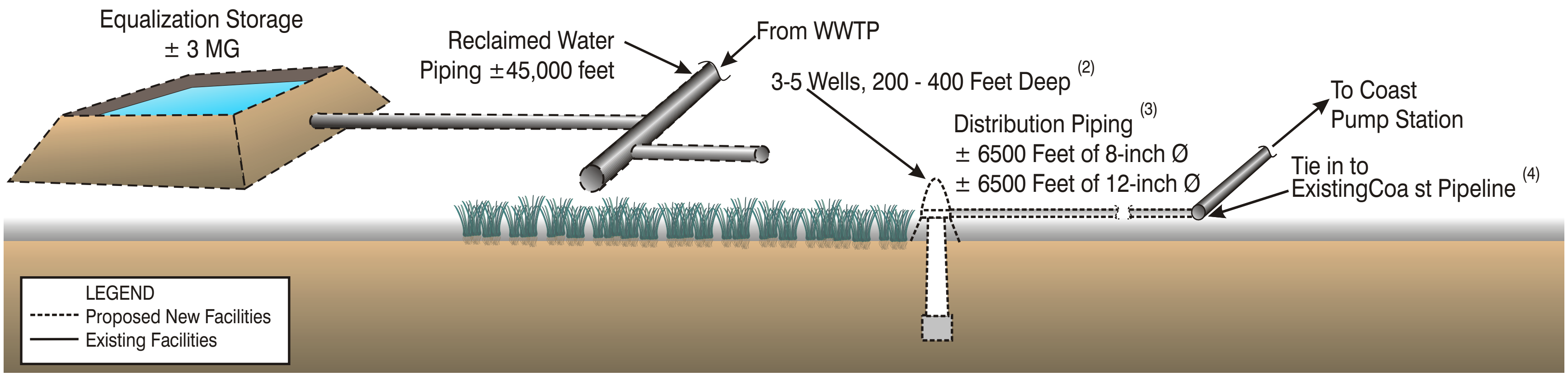

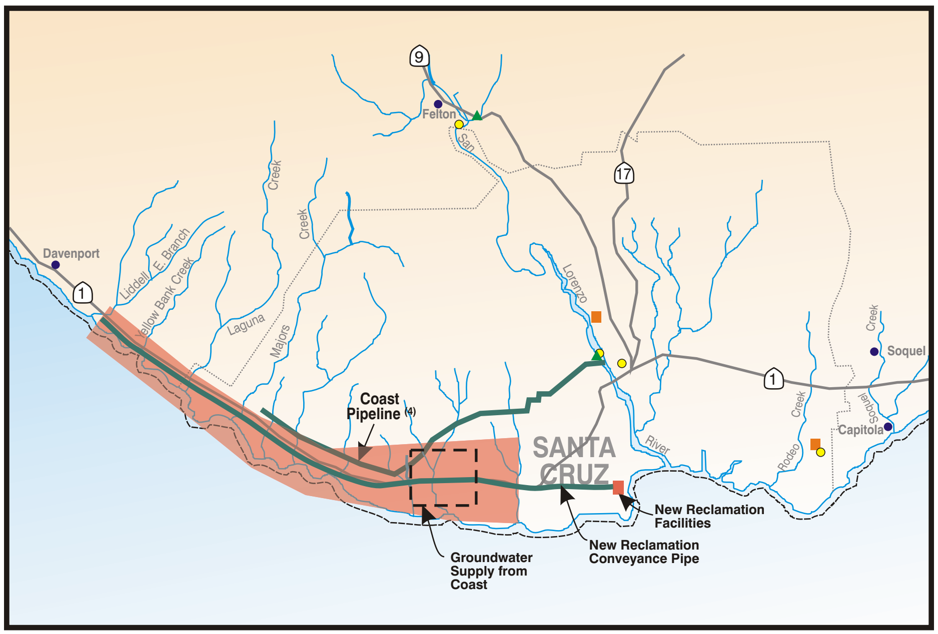

Potential Area for Reclamation/Groundwater Exchange Supply Project P5B [1]

NOTES

(1) The highlighted area represents the general location of reclamation along the coast.

(2) The depth of the wells may vary depending on location drilled because of varying surface features and depth to the aquifer along the coast.

(3) Assumes approximately one half mile piping at each of five well sites.

(4) The existing coast pipeline has hydraulic capacity limitations. Upgrades completed as part of a separate project are required to increase capacity to accommodate delivery of ground-water to the City.

Shaded area indicates general region/area of application for project alternative.

Figure 9
RECLAMATION FACILITY REQUIREMENTS FOR NORTH COAST AGRICULTURE REGIONAL WATER SUPPLY PROJECT
CITY OF SANTA CRUZ/
SOQUEL CREEK WATER DISTRICT

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NOTES

1) Route identified is one potential alternative to deliver water to the Soquel Creek WD project location.

2) Pipe length is approximately 35,000 feet.

3) Pipe size is 12 inches for delivered water flow of 3.2 mgd.

4) Location of Diversion with respect to project site is approximate. Final location to be downstream of Soquel Creek intake, if implemented.

Figure 10
RECLAIMED WATER CONVEYANCE PIPING FOR DELIVERY TO SOQUEL[1,2,3,4]
REGIONAL WATER SUPPLY EVALUATION
CITY OF SANTA CRUZ/
SOQUEL CREEK WATER DISTRICT

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water. Portions of the City's system would need to be upgraded to deliver the water to the District, similar to upgrades required for delivery of desalinated water. During drought years the water would be distributed to the City, while the District pumped additional water from its aquifer.

Engineering Evaluation

Facility Sizing. The seasonal irrigation needs for farms along the North Coast are estimated to range between 400 and 500 MG/yr. The irrigation demands are not "steady" during the summer season, and tend to increase toward the end of the summer. Accordingly, treatment facility upgrades would need to be sized to accommodate peak month demands which are estimated to range from 125 to 150 MG (about 4 to 5 mgd). Conversely, the seasonal needs for the in-stream project would be steady at approximately 3.2 mgd (5 cfs). These supply needs are similar, so for the purposes of this study the treatment facilities are assumed to provide up to 5 mgd, even though it may be possible that not all of the 5 mgd capacity would be used for a specific alternative. Based on these potential irrigation demands the conceptual facility requirements are as follows:

Treatment Facility Upgrades. Facilities to provide up to 5 mgd of tertiary treatment would include membrane filtration and disinfection. The project concept includes membrane filtration followed by either ultraviolet irradiation or chlorination with sodium hypochlorite. Associated chemical feed and storage equipment for chemical systems (e.g., sodium hypochlorite) may also be needed depending on type of disinfection. Figure 11 shows the potential layout location for the new treatment facilities.

Conveyance System Infrastructure. Conveyance system infrastructure would include pumps and piping as follows:

North Coast Groundwater Exchange. It is assumed that the City would provide a main conveyance pipeline to the farmers, from which water would be delivered through various "turnouts," as shown on Figure 9. The project concept assumes that the new pipeline would generally follow the Highway 1 alignment, and that the distribution piping to the various farms would be provided by the farmers (i.e., the City would provide the main supply, and the farmers would be responsible for delivery downstream of the connection). In addition to distribution piping, it is assumed that farmers would provide a small storage reservoir (or multiple reservoirs). A small storage facility of approximately 3 MG is required to provide equalization storage since farmers irrigate only several hours per day, whereas the reclaimed facilities would operate 24 hours per day.

A minimum of approximately 45,000 feet of 18-inch piping would be required for the main distribution header to the farms. Pumping would be accomplished by three (3) 150 hp pumps, each with a capacity of 2.5 mgd (2 duty and 1 standby).

NOTES
(1) Site alternatives represent potential general locations only. Final location would depend on siting with respect to current/planned space utilization at the plant.

Figure 11
SITE ALTERNATIVES FOR RECLAMATION TREATMENT FACILITIES AT WWTP (1)
REGIONAL WATER SUPPLY EVALUATION
CITY OF SANTA CRUZ/
SOQUEL CREEK WATER DISTRICT

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Soquel In-Stream Exchange. In-stream exchange is equivalent to an isolated reclaimed water user, so for this evaluation it is assumed that the distribution facilities to the District's proposed diversion structure campus would include a dedicated pumping and conveyance system. The diversion location is located approximately one hundred feet above sea level, so dedicated pumps would be required to meet the pumping requirements.

A minimum of approximately 35,000 feet of 16-inch piping would be required for conveyance. Pumping would be accomplished by three 200 hp pumps, two duty and one standby, each with a capacity of 1.75 mgd.

(Note: The in-stream exchange would require only 3.2 mgd. If the project was sized for up to 5 mgd (for North Coast flow requirements), there would be approximately 1.8 mgd of "unused" capacity that could be used for other applications in the City or District. If this project is implemented, it is recommended other potential users of the reclaimed supply be identified.)

Groundwater Facilities and Conveyance System. The estimated groundwater yield from the North Coast is 500 to 700 MG/yr. Depending on demand needs, this supply could be developed all year round, or perhaps only during the higher demand summer months. To estimate the number of groundwater wells required, it is assumed that the wells may need to provide 75 to 125 MG/month during the summer months, and that each well could be capable of producing 0.5 to 1 mgd. To meet these conditions 3 to 5 wells would be required. For the purposes of this report it was assumed that four new groundwater wells would be installed.

The exact siting of the wells is unknown, so for the purposes of this study it is assumed that the wells would be located within 1/2 mile of the Highway 1 corridor. Piping requirements to deliver water from the wells to the main conveyance pipeline (for delivery to the City) is assumed as 6,500 feet of 8-inch, and 6,500 feet of 12-inch pipe. It is assumed that the existing North Coast pipeline would be used to deliver the pumped groundwater from the coast to the City.

(***Note****:* The North Coast Pipeline currently has hydraulic capacity constraints which limit the flow to about 9 cfs. The groundwater pumpage could range between 4 to 6 cfs, which would take up about 40 to 60 percent of the pipeline capacity. Because of this the City may need to limit periods of groundwater pumpage to the summer months (when flow in the pipeline is less due to reduced diversion from the North Coast surface water sources). Alternatively, the City may need to increase the capacity of the lower reaches of the pipeline to accommodate the additional groundwater flow. The City plans to evaluate pipeline rehabilitation and upgrade options, and complete preliminary design of the preferred option during the summer of 2002.)

City Distribution System Upgrades. The North Coast groundwater exchange project could provide water to the District in nondrought years, up to approximately 2 mgd. As discussed above, the District's interconnection points to City's system are located in an area that includes predominantly old, undersized piping. The City's existing piping in the area was not designed with the intent to serve as a hydraulic "backbone" to provide water to the District. A distribution system upgrade with a 16-inch pipe would be required for a delivered capacity of 2 mgd to the District (see Figure 6 for alternative pipe routes). For the purposes of this evaluation, a 16-inch pipeline at approximately 19,500 feet was used for cost estimates.

Cost Estimate for Reclamation

The estimated costs for reclamation are shown in Table 5. The costs have been calculated based on a total annual supply of up to 700 MG/yr. Treatment and conveyance facilities sized within this range would provide additional supply required for both groundwater exchange and in-stream exchange project concepts.

(***Note***: As noted above, the in-stream exchange project for the District would be used to supplement the available supply from a separate surface water diversion and treatment project. Cost estimates for the surface water diversion and treatment facilities have been developed separately by the District and are not included in the cost estimates herein.)

Table 5 Conceptual Costs for Wastewater Reclamation
Evaluation of Regional Water Supply Alternatives
City of Santa Cruz/Soquel Creek Water District

	Capital Costs (in millions)(1,2,3)	Operating Costs (in millions)(4)
Reclamation for North Coast Groundwater Exchange	$49.3	$0.4
Reclamation for In-Stream Exchange in Soquel Creek	$31.0	$0.2 - $0.4

Notes:

(1) Capital costs assume 5-mgd treatment capacity for both alternatives.

(2) Cost estimate includes allowances for engineering, construction administration, and contingencies (see Appendix C).

(3) Cost estimate does not include allowances for distribution system upgrades that may be required to deliver water within the District.

(4) Operating cost has been decreased from full-year cost estimates assuming production at 5 mgd, 6 months of year for North Coast alternative, and operation at 3 mgd, 4 to 6 months per year for in-stream alternative.

Implementation Analysis for Reclamation

The installation of numerous reclamation projects throughout California provides ample evidence that a regional project could be implemented with due consideration of technical, environmental, and institutional issues.

Technical Issues. There are no significant engineering issues for either project; the treatment systems and the required infrastructure are typical of other water/wastewater facilities. However, there are a few key engineering issues that need to be investigated further as part of (or preferably prior to) implementation:

- ***Confirm Groundwater Conveyance via the North Coast Pipeline.*** This planning level concept is based on using the existing North Coast pipeline to convey water back to the City. The pipeline capacity is constrained under the existing hydraulic conditions, so modifications to this pipeline would likely be required to accommodate additional flow from the groundwater supply. Alternatively, a new pipeline would need to be constructed. Additional preliminary engineering work is needed to identify preferred options. In addition, it will also be important to identify the costs and scheduled implementation for the upgrade and/or new pipeline. For example, the City's current long-range plan is to complete rehabilitation/upgrades to the North Coast pipeline over the next 15 years. This time frame would not be consistent with the objectives to provide additional water supply in a timely manner.

- ***Confirm Groundwater Supply.*** Estimates of the groundwater yield along the North Coast vary. Although there is substantial published geologic/hydrogeologic information, there is very limited actual field data to confirm the aquifer characteristics. Additional fieldwork (i.e. test wells) is recommended to confirm the groundwater supply prior to final implementation of the project.

Environmental Issues. Based on preliminary environmental review, there do not appear to be any significant environmental issues. There are potential construction-related issues for pipeline routes in major City arterial streets, but these construction issues do not represent a "fatal flaw" for the project. Table 6 summarizes other potential environmental issues.

Institutional Issues. Even with a strong bias to implement the project there are several institutional issues that would need to be resolved:

- ***Confirm Project Concept with North Coast Farmers*. **There are several local examples of reclamation along the coast and Salinas Valley so there would not appear to be a significant implementation issues. Also, based on preliminary discussions with several North Coast farmers, there appears to be interest for the use of reclaimed water. However, more effort is needed to confirm that the interest is genuine, and that the interest is not limited to one or two crop types.

- ***Confirm Groundwater Usage Entitlements on the North Coast.*** For reclamation to be viable on the North Coast there must be a guarantee that the groundwater would be available in exchange for the reclaimed supply. Based on preliminary review of the irrigated land along the coast it appears that much of the land currently irrigated with groundwater is owned by the State of California. As the owner of the land, the State also owns the rights to the underlying groundwater. To implement this project, rigorous contractual agreements with the State would need to be developed. There is no clear indication that the State would (or could) enter into such agreement for this project. In any case, to finalize an agreement would take time (perhaps years), and would have

Table 6 Summary of Environmental Issues for Wastewater Reclamation
Evaluation of Regional Water Supply Alternatives
City of Santa Cruz/Soquel Creek Water District

Issue	Groundwater Exchange Project(1)	In-Stream Exchange Project (1)	Comments
Construction Related Impacts	Minor-Moderate	Minor-Moderate	• Construction at the WWTP should pose only minor impacts
Compatibility with Adjacent Land Uses	Minor	Minor	• Treatment facilities would be constructed at the WWTP
Visual Impacts	Minor	Minor	• No apparent impact
Potential for Cultural Resources	Minor-Moderate	Minor	• Excavation activities along the North Coast could uncover cultural, archaeological, historical resources
Potential for Public Controversy	Minor-Moderate	Moderate-Major	• Reclaim pipeline for North Coast is within area designated for "coastal-dependent" uses • Potential for public controversy regarding application of reclaimed water for irrigation to land overlying groundwater to be used for supply • Potential for public controversy regarding direct in-stream application of reclaimed water
Potential to Disrupt Traffic From Pipeline	Major	Major	• New piping along City streets (Capitola and Soquel Avenue) could present significant traffic-related construction impacts
Growth Inducement	Minor-Moderate	Moderate-Major	• Growth inducement is a potential impact of any project increasing water supplies(2) • Potential impact may be mitigated with the regional supply project concept, serving the City only in drought conditions and other users in nondrought years
Energy Usage	Moderate	Moderate	• Reclamation requires substantial pumping to route water to points of delivery; Coastal Act requires minimizing energy consumption

Notes:
(1) Minor/Major/Moderate represents the anticipated severity of the issue and is based on a conceptual-level review of potential environmental and regulatory constraints.
(2) The future demand projections are consistent with the City and County General Plans.

associated schedule implications. Although the agreements/entitlements could be developed in parallel to other project elements (e.g., EIR documentation, facility engineering, permitting), it would be preferable to have such agreements in place prior to pursuing/developing the project.

- ***Confirm In-Stream Exchange Concept.*** The in-stream exchange concept would require that the tertiary treated reclaimed water be discharged to Soquel Creek. Given that the water is highly treated, and that the discharge would only represent approximately one-sixth of the stream flow when operational, there are no apparent public health or habitat issues. For example, wastewater treated to lesser degrees is routinely discharged to waterways in the State with no obvious consequences to the fishery habitat. Previous precedents notwithstanding, there are several examples of public interest regarding discharge of reclaimed wastewater to streams - even if highly treated. As such, it would be preferable to have regulatory approval and public acceptance of the discharge in place prior to pursuing/developing this project.

Of these three broad issues, there is potential that two issues - need for project confirmation with the farmers and need for confirmation of groundwater usage entitlements - could potentially represent “fatal flaws.” As discussed above, there are several unanswered questions and unknowns for both of these issues. In particular, the permitting elements related to the groundwater usage are, at a minimum, very complex and would require involvement of multiple parties, including the Coastal Commission, State Water Resources Control Board, California State Parks Department, State Water Quality Control Board, State Division of Drinking Water, and local farmers. The City and District should consider that, even if the project is feasible in concept, the interagency involvement and related permitting elements could significantly impact the schedule for implementation.

APPENDIX A - FEASIBILITY EVALUATION OF BEACH WELLS FOR SEAWATER INTAKE AND BRINE DISCHARGE

INTRODUCTION

The City of Santa Cruz (City) is currently studying the feasibility of developing a new municipal water supply by desalinizing seawater. The City has retained Carollo Engineers (Carollo) to conduct the study and develop potential desalting project alternatives. Hopkins Groundwater Consultants, Inc. (Hopkins) conducted a reconnaissance-level hydrogeological review study to support Carollo in project alternative development and evaluation. Presented in this report are the findings, conclusions, and recommendations developed from the Hopkins review study of the option to produce saline groundwater from an intake system being considered for construction along the Santa Cruz coastline in the vicinity of the City. The saline groundwater would be used as a feedwater supply to the proposed desalination facility in lieu of an open water intake structure. If this option is feasible, the location of subsurface production facilities may influence the ultimate location of the proposed processing facility and project costs. The coastal area that was defined for this study lies between the City of Santa Cruz Boardwalk and the beachfront along Rio Del Mar. The study area is shown on Plate 1 – Study Area Location Map.

It is understood that the City is concurrently reevaluating future water supply projections to refine estimates of future demand and potential water system deficiencies. Past projections indicate that future demands will require a new source to provide 2 to 10 million gallons per day (mgd) of additional water supply. Approximately 4 to 25 mgd of raw saltwater will be required to for the plant to produce such a supply from the proposed desalination treatment processes being considered. Assuming plant operation will be maintained at a constant rate, the well field supply rate will range from approximately 4 to 25 mgd (2,800 to 17,500 gallons per minute [gpm]), and the post-treatment brine discharge rates will be approximately 2 to 15 mgd (1,400 to 10,500 gpm).

BACKGROUND

In response to projected shortfalls in future water supplies, the City is evaluating the feasibility of constructing an ocean water desalination facility as a supplemental supply source. Saline water for the plant could be derived from either an offshore open water intake structure or a subsurface intake facility that would produce saline water from the shallow sediments present along the coastline. The saline groundwater production option, where feasible, has several advantages over the open intake supply method, the primary advantage of which is a potentially significant reduction in required pretreatment before desalination due to the natural filtering capacity of the aquifer sands. B&V (1994) estimated that cost savings derived from this prefiltration benefit could range up to $200 per acre-foot of produced water. Additional potential benefits include reduced permitting requirements, improved reliability (particularly during winter storm events), improved public perception, and the virtual elimination of potential harm to marine life (CCC, 1992).

The City also is considering options for the disposal of reject brine generated from the desalination process. These options include the construction of a new ocean outfall, utilization of an existing ocean outfall, or injection into coastal sediments. The disposal of brine through injection could have advantages over direct ocean discharge if dispersion of the concentrate occurred within the aquifer prior to its offshore discharge. Additional potential benefits include reduced permitting requirements, lowered construction costs, and reduced environmental impact.

Saline groundwater intake and disposal alternatives have been evaluated by numerous California water supply agencies that are faced with resource development within districts that boarder the coastline. This study combines the knowledge and experience gained by Hopkins staff while working on similar projects with available information on local coastal hydrogeology to develop a planning level assessment of project feasibility.

PURPOSE AND SCOPE

The purpose of the study was to provide a general review of the coastal hydrogeology and develop a professional opinion on the feasibility of developing a subsurface seawater intake system and/or a reject brine disposal system in the study area. The scope of work was developed through discussions with Mr. Ken Wilkins, Associate Engineer with Carollo, and presented as Task Order No. 1 in the professional services agreement dated August 21, 2001. The scope of work as performed for this study included:

- Discussing project details regarding operational capacities and proposed intake locations with project team members.
- Collecting and reviewing hydrogeologic data on coastal conditions within the study area from sources including the City, County of Santa Cruz, Santa Cruz Port District, and the University of California Santa Cruz.
- Conducting field surveys of the potential beach sites to observe their present condition and logistical constraints, and comparing these observations with historical documentation.
- Preparing this letter report, which specifically discusses the findings of the review study and local beach geology as it relates to its potential capacity for use as a saltwater intake/discharge area.

Included in this report are various plates and appendices. Appendix A – Regional Geology, provides the general geology of the coastline in the study area. Appendix B – Plates From Shoreline Erosion Study, provides pertinent details from a shoreline study that was conducted by the California State Department of Navigation and Ocean Development in 1977 (Habel and Armstrong, 1977), which provides a qualitative description of the beachfront and adjacent developments. Appendix C – Shoreline Protection Structures and Cliff Failure Hazard, contains valuable information about the present shoreline protective structures, recent sea cliff erosion rates, and a qualitative assessment of potential hazard to structures on the cliffs (Griggs and Savoy, 1985).

STUDY AREA AND METHODS OF EVALUATION

As considered, a saline groundwater supply would be produced from land-based collection facilities. These facilities could be comprised of a series of conventional (vertical) wells, horizontally drilled wells, and radial collector wells, or a shoreline infiltration gallery. The basic geometry of the beach and underlying aquifer systems vary along the coast in the study area. Data collection and evaluation for this study was essentially categorized by changes in apparent hydrogeologic conditions. The specific areas of study include: a) the alluvial plain at the mouth of the San Lorenzo River, b) beachfront adjacent to the Santa Cruz Boardwalk, c) beaches between the Boardwalk and Pleasure Point, d) Capitola Beach, and e) the beach stretching from New Brighton to Rio Del Mar.

The data collection and review methods used for this study included literature and aerial photograph review, a beach reconnaissance survey, and personal communication. Literature review provided specific information on nearshore geology and hydrogeology, littoral transport of coastal sediments, shoreline erosion rates, and man-made structures that have been constructed along the shoreline in the study area. A list of these references is provided in this report. A review of aerial photographs was conducted to obtain a qualitative sense of how the beachfront has changed seasonally, after large storm events, and after man-made structures were emplaced. Historical studies of shoreline erosion also utilized aerial photographs as a means to calculate annual rates of sea cliff erosion (USACOE, 1957). On September 10, 2001, Hopkins conducted a reconnaissance-level survey of existing shoreline conditions within the study area. Field reconnaissance was useful for confirming historical documentation and conceptually developing an awareness of the constraints and challenges that could impact the proposed project. During the survey, Hopkins' staff talked to people who have worked along the beach for many years and can recall normal conditions and the effects of large storm events (i.e., staff with the Santa Cruz Harbor and California State Parks Department).

FINDINGS

GEOLOGY

Regional Geologic Setting

The geology of the northern Monterey Bay coastline within the study area provides evidence that the present shoreline was elevated well above sea level in the recent geologic past (around 10,000 years ago). During that time, erosion caused by local runoff incised steep-sided drainage courses into the exposed bedrock. Subsequent submergence caused deposition of alluvial materials in these coastal stream- and river-cut valleys. Available data from geotechnical borings indicate that along the present-day coastline at the mouth of the streams (or lagoons) and the San Lorenzo River, the alluvium-filled paleochannels extend to depths in the ranging from 30 to 60 feet below ground surface. The fill was likely much deeper at the time when ocean waves were carving the adjacent uplifted terrace features. Subsequent uplift, caused by the current tectonic forces in the region, has caused erosion of much of the alluvial fill in these channels, resulting in relatively shallow and narrow alluvial basins.

Geologic mapping studies indicate that upper Miocene- to Pliocene-age Purisima Formation underlies the alluvial and marine deposits found along the coastline within the study area. The siltstone, sandstone, and mudstone beds of the Purisima bedrock gently dip south to southeast between 2 and 5 degrees across the study area (Clark, 1981). The regional geology within and adjacent to the study area has been mapped in detail by numerous investigators. Appendix A contains a regional map of the study area that was cropped from a comprehensive United States Geological Survey (USGS) map of Santa Cruz County (Brabb, 1997). This map is a compilation of numerous geologic maps that were constructed by several investigators, a list of which is included in Appendix A. As indicated in the following sections of this report, the regional geology has little significance to the proposed project along the shoreline. The primary geologic focus of this report is on the shallow, localized, coastal materials.

Coastal Geology

Most available data on the northern Monterey Bay coastal environment were historically collected for studies that were conducted to understand and subsequently abate shoreline erosion. Initial studies included cooperative efforts between State and Federal agencies that primarily focused on the rate of sea cliff retreat (USACOE, 1957; Habel and Armstrong, 1977), in an effort to prevent the loss of developed property. Although the focus of this study on the beach environment is fundamentally different from these earlier evaluations, the active natural processes and conditions documented by those investigations provide a sound basis for this assessment.

With the exception of the San Lorenzo River mouth and the locations where smaller drainage channels (lagoons) discharge into the bay, bedrock bluffs border the entire coastline of the study area. The bluffs are comprised of Purisima Formation sediments that offer varying degrees of resistance to erosion. The bedrock strength and the degree of fracturing along the coastal bluffs are primary geologic factors that determine the susceptibility of the sea cliff to wave erosion and control the changing shape of the coastline. On top of the coastal bluffs is a thin 10- to 20-foot-thick, Quaternary-age marine terrace deposit (Dupre', 1990). Many areas along the shore have accumulated fine- to coarse-grained beach sand deposits that range from 10 to 20 feet thick. In other areas, there are talus deposits that identify recent failure of the undermined bedrock cliffs. The shoreline conditions have been summarized in detail by past studies (USACOE, 1957; Habel and Armstrong, 1977; Griggs and Savoy, 1985). Based on the Hopkins' field reconnaissance, the present shoreline conditions compare closely to the description of conditions provided in Habel and Armstrong, (1977). For this reason select plates from Habel and Armstrong, (1977) (which include a detailed description of discrete reaches of the beach and adjacent bluffs) have been included in Appendix B of this report.

The geomorphology of the northern Monterey Bay shoreline has been documented through observations and data generated by past studies. A number of these studies conclude that shallow bedrock lines the ocean bottom along the Santa Cruz coast. The rapid rate of shoreline retreat has resulted in the formation of a wide littoral shelf (Chin and Wolf, 1988). The nearshore depths are determined by the prevailing size and direction of the winter waves that cause a majority of the scour. Offshore studies have concluded that the shallow ocean floor is seasonally blanketed by and periodically stripped of the littoral drift deposits. Observations of kelp beds in these shallow waters also suggest the presence of a rocky, unprotected, ocean bottom environment.

SHORELINE GEOMORPHOLOGY

As is typical of a cliff-lined coast, the beach profile in the study area changes seasonally primarily because of high-energy waves (storm waves) that typically reflect off the cliff base and erode the neasrshore sediment. The energy provided by an average winter storm typically is absorbed in the swash zone of a sandy beach. During the summer months, an abundant accumulation of san forms a wide beach that is capable of withstanding "normal" winter erosion rates, thus giving the beach an appearance of stability (see Plate 2 – Average Seasonal Changes of the Beach Profile). Unfortunately, beach profile in the study area is dominated by periodic storm events that eliminate the sand buffer and case severe seas cliff erosion (see Plate 3 – Large Storm Effects on the Beach Profile). It is this type of periodic occurrence that could negatively impact the proposed subsurface intake system the most.

As man has developed the shoreline properties through the years, coastal erosion-related damage to and/or loss of structures has become a cause of public concern. Sea cliff retreat at rates of up to 1 to 2 feet per year was considered intolerable along these sections of coastline. In the study area, much of the coastline has been developed and protective measures emplaced to prevent wave damage to coastal structures (e.g., the construction of seawalls or the placement of riprap at the base of threatened slopes). Seawalls and riprap tend to stabilize the beach landward of the protective structures, but also create greater erosion forces on the seaward side. The results of these two protective measures are illustrated in Plate 4 – Typical Cliff Protection Methods. If the proposed extraction facilities were placed on the seaward side of such protective structures, they would be highly susceptible to wave damage. If an intake system were placed on the landward side of such protective structures, the structures themselves would then impede the landward flow of saline groundwater.

Another method of beach protection that is utilized in the study area is the construction of jetties or riprap groins that extend perpendicularly out from the shoreline. These structures stall the longshore current and serve as a trap to littoral drift, which builds a wider beach on the up current side. The widened beach then provides a larger buffer against the erosion of winter waves. (The direction of littoral currents within the study area is provided in Appendix B.) This type of shoreline protection is the method that is most compatible with the proposed construction of a subsurface intake system. The accumulation of sand provides a thicker aquifer as the beach builds seaward and provides greater protection from winter waves. However, available data indicate that the existing riprap groins along this section of coast provide neither a large enough beach area nor a sufficiently thick sand section to support the proposed saltwater extraction rates. Appendix C contains plates from a study conducted in 1985 that provide detailed information on coastal protection structures in place at that time. Our reconnaissance-level field survey concluded that the conditions at the present time are much the same as those summarized by the previous study (Griggs and Savoy, 1985). Riprap groins that extend seaward from the beach are not indicated on the summary plates contained in Appendix C. However, these structures are described in the following sections of this report.

HYDROGEOLOGY

Groundwater geology in the vicinity of the City has been defined by numerous studies. There are two shallow coastal aquifer systems in the study area that historically have provided groundwater to the overlying users: the Purisima Formation bedrock aquifer and the San Lorenzo River alluvial basin. Both the City and the Soquel Creek Water District (SCWD), which adjoins the City service area southward along the coast, rely heavily on these freshwater sources.

Purisima Formation Bedrock Aquifer

Historical documentation of pumping patterns along the coast indicates that groundwater levels in some of the bedrock aquifer zones of the Purisima Formation have fallen below sea level. Because of this condition, the focus of past studies and the monitoring of present basin conditions are to determine if seawater intrusion is actively occurring. The SCWD, who relies entirely on groundwater for their supply, has relocated several coastal wells to sites that are inland and away from the coastline so as to minimize the potential threat of seawater intrusion.

Use of this aquifer system is not considered a viable alternative for saltwater production. Although additional production from this aquifer along the coastline would create a landward gradient that would induce the flow of seawater, many factors preclude the ability to consider this option.

Physical limitations to pumping are intrinsic of the relatively thin aquifer zones of fine- to medium-grained sand that comprise this leaky, confined aquifer system. The offshore outcrop area is not well defined and, while saltwater entry poses a threat to freshwater uses, infiltration rates are likely inadequate to support the proposed project demands. Groundwater produced from this aquifer would derive a large component from onshore recharge and, as a result, would decrease the available freshwater supply.

Institutional limitations that would affect this alternative are believed sufficient to eliminate further consideration of this option. Inducing an onshore flow of saltwater for the proposed project would be contrary to the non-degradation policy that has been established to protect freshwater aquifers in the state.

San Lorenzo River Alluvial Basin

Previous studies divided the San Lorenzo River alluvial groundwater basin into upstream and downstream subbasins. The separation of these subbasins is defined by the bedrock ridge trending east-west in the vicinity of Water Street (B&C, 1984). Available data on the subsurface geology indicate that the alluvial aquifer is likely hydraulically continuous. Historical data generated by the recent San Lorenzo River Flood Control Project assessment conducted by the U.S. Army Corps of Engineers provides geotechnical information on the soils underlying the flood control levees that were constructed to contain the active river channel (USACOE, 1995). This assessment was conducted between the Highway 1 Bridge crossing to the north and the Southern Pacific Railroad Bridge that crosses the mouth of the river at the shoreline to the south. The 1995 study used cone penetrometer and test hole boring methods to generate geologic data. These data indicated there were laterally discontinuous layers of alluvial deposits (lenticular deposits) and suggested that roughly 40 percent of the alluvial material beneath the active channel was comprised of fine-grained silt and clay. The logs suggest that the alluvium located up river (above Water Street) contains a greater abundance of coarser sand and gravel deposits.

Investigative studies conducted for the seismic retrofit analysis of the Soquel Avenue and Water Street Bridge sites provide additional geotechnical data. These data indicate that the alluvial materials at these two locations extend to depths of up to approximately 90 feet below mean seal level (MSL). The deepest section is found at the Soquel Avenue Bridge. Although the alluvium is reported to range in depth down to approximately 90 feet below MSL, the average saturated thickness for the entire alluvial basin is considerably less than perhaps 40 feet. Because of the relatively shallow average basin depth, it has a small groundwater storage capacity. The total groundwater storage capacity of the basin has been estimated at about 2,000 acre-feet, with approximately 90 percent of the groundwater stored below MSL (Fugro, 1999).

The river channel elevations at the Soquel Avenue and Water Street crossings are reportedly at MSL and at an elevation about 3 feet above MSL, respectively. The Soquel Avenue

Bridge crossing is approximately 4,300 feet below the Highway 1 Bridge, where the riverbed elevation is approximately 10 feet above MSL. The gentle slope of the river channel profile can allow high tidal surges to move saltwater rapidly up river. Use of the aquifer system along the river for a freshwater supply is therefore limited to the upper reaches (i.e., Tait Street), where the water table depression caused by pumping has a reduced potential to induce saltwater flow into the aquifer system. Historical documentation has indicated that saltwater was pumped from the Tait Street wells on at least one occasion and that the basin is susceptible to seawater intrusion (DWR, 1975).

Over the past 20 years, the City has conducted several studies to assess the potential of using the San Lorenzo River alluvial aquifer system to expand the City water supply (ESA, 1979; Ranney, 1981; B&C, 1984; L&S, 1990; Fugro, 1998). The common conclusion reached by these studies is that there is a potential to increase alluvial groundwater production, but well location must be selective and operational constraints must be seasonally imposed to avoid seawater intrusion.

The production of groundwater from the lower portion of the alluvial basin will most certainly induce seawater intrusion. Substantial extractions would cumulatively add to the current amount of water removed from the basin and create a pumping depression that induces infiltration from within the river channel and the nearshore sediments. Absent of significant river flows, tidal surges that push seawater upriver will promote saltwater recharge further inland into the freshwater aquifer. The production of a brackish groundwater supply from the San Lorenzo River alluvial aquifer would derive its source both from freshwater recharge inland and saltwater recharge from the coastline. The cumulative production capacity of a well field constructed in the coarse-grained sediments located between Tait Street and Water Street was estimated at up to 3 to 6 mgd supply (2,000 to 4,000 gpm) and only when there was significant flow in the river during the winter months (Fugro, 1998).

Between Water Street and the coastline, the data indicate a greater abundance of silt and clay material is contained within the alluvial section (USACOE, 1995). These materials will impede vertical infiltration, and will ultimately limit the amount of induced flow that can be obtained from the lower river reach and the coastline through either conventional wells or lateral collector systems. We estimate that the San Lorenzo River alluvial basin may be capable of sustaining a steady year-round supply of saline groundwater on the order of 3 to 4 mgd. Available data do not indicate that the alluvial groundwater basin at the river mouth would be capable of sustaining a reliable supply of saline groundwater in the amounts required by the proposed project (i.e., over 4 mgd).

Brine disposal in the alluvial groundwater system is not feasible primarily due to its shallow depth and narrow configuration, high groundwater conditions, and proximity to developed properties. The unconfined and unconsolidated nature of the river alluvium could subject overlying uses to conditions that cause local structural impacts if the basin was overfilled. These conditions include raising the groundwater levels near or above the ground surface during injection of brines to create nuisance water issues or liquefaction potential.

The physical limitations on production of groundwater from this aquifer will include: a) aquifer yield, b) the potential impacts to existing freshwater production located up river, and c) the potential for land surface subsidence that could be caused by dewatering the unconsolidated clay and silt layers. These issues will need to be addressed along with other potential regulatory concerns if this supply is considered for further study. Site-specific evaluation through field-testing will be required to further refine aquifer yield estimates.

Coastal Beach Sections

Successful saline groundwater production facilities located in the project area must be capable of establishing hydraulic communication with the ocean to form the primary source of recharge. In addition, to obtain the required flow rates, extraction facilities must be constructed in materials of adequate saturated thickness and permeability. As previously discussed, the shoreline geomorphology of the northern Monterey Bay coastline does not naturally accommodate these conditions. Man-made structures placed for beach stabilization have provided limited sections of shoreline where littoral accretion of any significance is occurring. Based on the information gathered for this study and to facilitate discussion of existing conditions, the shoreline has been divided into the following four coastal segments:

a. City of Santa Cruz Boardwalk - The coastline adjacent the City of Santa Cruz Boardwalk is oriented east and west and is located up coast of the San Lorenzo River mouth. Both riprap and a seawall are being utilized to protect the shoreline. The beachfront is relatively flat with beach sand thickness ranging between 10 and 20 feet. Because the beach area seaward of the protection structures is unprotected, high-energy storm events remove most of the sand cover and expose the underlying bedrock. However, the beach is semi-stable as a result of sand replenishment from the San Lorenzo River and the natural bedrock jetty that protrudes seaward and is located down coast (east) of the river mouth.

b. Shoreline Between San Lorenzo River and Pleasure Point - Proceeding down coast from the San Lorenzo River mouth, the cliffs backing Seabright Beach were rapidly eroding until construction of the Santa Cruz Harbor in 1963 (Griggs and Savoy, 1985). A wide semi-permanent beach now buffers the area and lies on top of a shallow bedrock shelf. The depth of the beach sand material that would comprise the potential seawater aquifer ranges between 10 and 25 feet and is deepest near the Santa Cruz Harbor jetty. This section of beach is mostly unprotected and relies almost solely on the sand reserve accumulated in the summer months to protect the shoreline. East of the Santa

Cruz Harbor, the low cliffs consist of weak siltstone and sandstone that are being removed by active erosion. The erosion rates were estimated in 1985 to be between 7 and 25 inches per year. Today most of the coastline between the harbor and Pleasure point is protected by riprap that is placed directly at the base of the cliffs. There is no room behind the riprap for beach wells and there is a high risk of wave damage if facilities are placed seaward of the protective materials.

c. Capitola Beach - The coastal bluffs, in the area adjacent the Capitola Creek outlet, historically have been subjected to severe erosion. The current shoreline configuration along this segment of beach trends in a northeast/southwest direction. Accelerated erosion rates of up to 32 inches per year have resulted in routine removal of littoral sands. Since the construction of protective seawalls and the Capitola jetty (east of Capitola Creek), the erosion has primarily stopped in the vicinity of Capitola creek and a wide but shallow beach has developed. Although this beach may provide protection to the landward development, it is not of sufficient depth to support significant saline groundwater production. South of the jetty, erosion continues along Capitola State Beach between the creek outlet and New Brighton Beach at rates of between 8 and 31 inches per year. Past and present high erosion rates have resulted in scouring a shallow bedrock shelf where little if any sand is seasonally deposited in this area.

d. Beach Between New Brighton and Rio Del Mar - Between New Brighton State Beach and the community of Rio Del Mar, the shoreline is oriented in a southeast/northwesterly direction. This configuration allows for a seasonal accumulation of littoral sands greater than what generally accumulates up coast of New Brighton. As documented by historical studies, the amount of erosion caused by large storm events was sufficient to clear the beach of sand and cause significant sea cliff erosion (USACOE, 1957). At the present time, the combination of riprap and an almost continuous seawall protect the bluffs along the entire section of beach between New Brighton and Rio Del Mar. As discussed previously these protective structures promote sea cliff protection, but do not benefit beach well production.

BEACH INTAKE CAPACITY

The intake capacity of subsurface facilities located along the shoreline in the study area is anticipated to range from 0.3 to 3 mgd, (200 to 2,000 gpm) at best. The most capacity with the least risk of wave damage would be available from San Lorenzo alluvium. However, the estimated production from this area is still short of the project demand. Testing conducted for the City of Ventura desalination project evaluated production from beach sands similar in grain size to the sands found along the Santa Cruz coast. The Ventura Beach sediment extends to depths of up to 40 feet. Field-test results indicated wells placed at that location would be capable of producing approximately 200 gpm, but would require a 600-foot spacing to limit interference

(SGD, 1994a). Production constraints found in the study area that would severely limit production capacity are: a) the set back distance from the ocean water source, b) thin aquifers with a small saturated thickness, c) relatively fine-grained aquifer materials, d) the seacliff and underlying bedrock (which are no-flow boundaries), e) local seawalls or riprap that impede flow, and f) potential decrease in the local freshwater supply to existing wells due to extraction from potable aquifers.

Desalination projects that have successfully developed a saline groundwater supply are located in areas where coastal sediments provide sufficient supply to meet the plant's requirements. Most of these facilities are significantly smaller than the proposed project (CCC, 1992). Table 1 below provides a listing of operating plants along the California coast that use subsurface intake facilities. As shown in Table 1, the output capacity of these plants is considerably lower than the proposed project. The largest project, which is operated by the Marina County Water District, derives its supply from a well-sorted, coarse-to very coarse-grained sand aquifer that has a saturated thickness of more than double the thickness of beach sand along the North Monterey Bay coastline.

Table 1 – Coastal Desalination Plants in California Using Subsurface Intake

Plant Identification	Operational Capacity	Estimated Required Groundwater
San Nicolas Island, U.S. Navy	0.03 mgd	0.07 mgd
Santa Catalina Island	0.132 mgd	0.30 mgd
Marina County Water District	1.0 mgd	2.5 mgd

BEACH DISPOSAL CAPACITIES

Beach disposal capacities are severely constrained by the lack of aquifer thickness along the shoreline, the shallow depth to water (which would result in mounding above ground surface), low injection specific-capacity values, the potential for liquefaction and loss of shoreline materials, and the absence of freshwater offshore flow to blend and reduce the reject brine salinity. The lack of fresh water for blending will result in salinity increases in the surf zone, where emergent groundwater will blend. These combined constraints make the shoreline disposal alternative for reject brines infeasible.

CONCLUSIONS AND RECOMMENDATIONS

Erosional forces are the dominant factor in the sediment balance equation for the beachfront between the Santa Cruz Boardwalk and Rio Del Mar. Although beach sand can accumulate for a number of consecutive years, the combination of a large tidal surge with a severe storm event can strip it away. The entire section of coastline in the study area is underlain by moderately indurated siltstone, sandstone, and mudstone of the Purisima Formation. Active erosion of these bedrock units has resulted in creating a shallow wave-cut platform along the shoreline that generally ranges in depth from 10 to 20 feet below mean sea level. Littoral transfer of fine- to coarse-grained sands southward along the coastline creates a transient nearshore blanket of material that is thin to non-existent. Seasonal onshore sand deposits, which create beaches, can widen during the calmer summer months and then be completely removed during episodic events in the winter.

Sections of the coastline where man-made structures disrupt the currents and absorb the wave energy have resulted in maintaining a thicker section of beach sand during winter seasons. Existing structures do not provide sufficient protection for enough beach area to support a non-interruptible subsurface intake system. Although it is technically possible to design and build such structures in the surf zone (e.g., a series of riprap groins that extend seaward a sufficient distance to develop a permanent beach of adequate thickness), it may not be feasible either from a public perception or regulatory perspective.

Larger alluvial deposits contained in the infilled drainage channels along the coast provide a greater opportunity to produce an uninterrupted supply of saline water. However, available data indicate the total thickness and lateral extent of these deposits severely limit the production potential. Additionally, deposits at the mouth of the present-day lagoons have been found to contain deposits of abundant organic material, which would further impact the suitability of these sites. The location of the current shoreline with respect to these landforms was once part of the ancient backwater lagoon. Groundwater produced from these deposits would likely contain elevated concentrations of nitrates, phosphorus, bacteria, and hydrogen sulfide along with organic compounds that could make the water quality unsuitable or too costly to condition for a feed water supply.

The larger alluvial deposits in the vicinity of the San Lorenzo River mouth and the Capitola Creek outlet are again restricted by the predominantly fine-grained nature of the materials and the basin geometry (shallow and narrow). Because virtually no flow is available from the underlying bedrock, a linear pumping trough would be developed during groundwater extraction that would extend both landward beneath the existing flow channel and seaward beneath the bay. The silt and clay beds within the alluvium would impede the vertical infiltration of saltwater, thereby potentially preventing sufficient flow to sustain the required production rates. Furthermore, groundwater extraction from the alluvial plain at the San Lorenzo River mouth would likely jeopardize the reliability of the existing City water supply wells located inland (the Tait Wells). Additional depletion of freshwater in the aquifer would contribute to a lower water table, which in turn would induce saltwater infiltration during tidal surges that flow up river. The City's Tait wells produce groundwater from elevations well below sea level, and there is at least one historical account of saltwater production that interrupted the supply from these wells (DWR, 1975). Although this condition may not occur during the winter when high river flows are present, it would likely occur when river flows subside.

It is not feasible to dispose of the saline brine reject fluids within the beach sands because there is no appreciable seaward flow of fresh groundwater for blending. The brines would emerge at the beach with virtually the same concentration of salts as when it was injected. Although wave action would cause mixing, the salinity of the nearshore seawater would be elevated in the vicinity of a beach disposal system. Even if fresh or brackish water was found to be flowing through the alluvium in the vicinity of the coastal lagoons, streams or rivers, the shallow depth to groundwater does not allow for the groundwater mounding that results from injection operations. This would prevent even the lowest projected flow rates from being accommodated without creating adverse conditions around the injection facilities.

CLOSURE

This report was prepared for the exclusive use of the City of Santa Cruz and its agents for specific application to saline groundwater production and brine injection in the shallow aquifer systems along the coastline between the Santa Cruz Boardwalk and Rio Del Mar. The findings conclusions and recommendations presented herein were prepared in accordance with generally accepted hydrogeolgic engineering practices. No other warranty, express or implied, is made.

□

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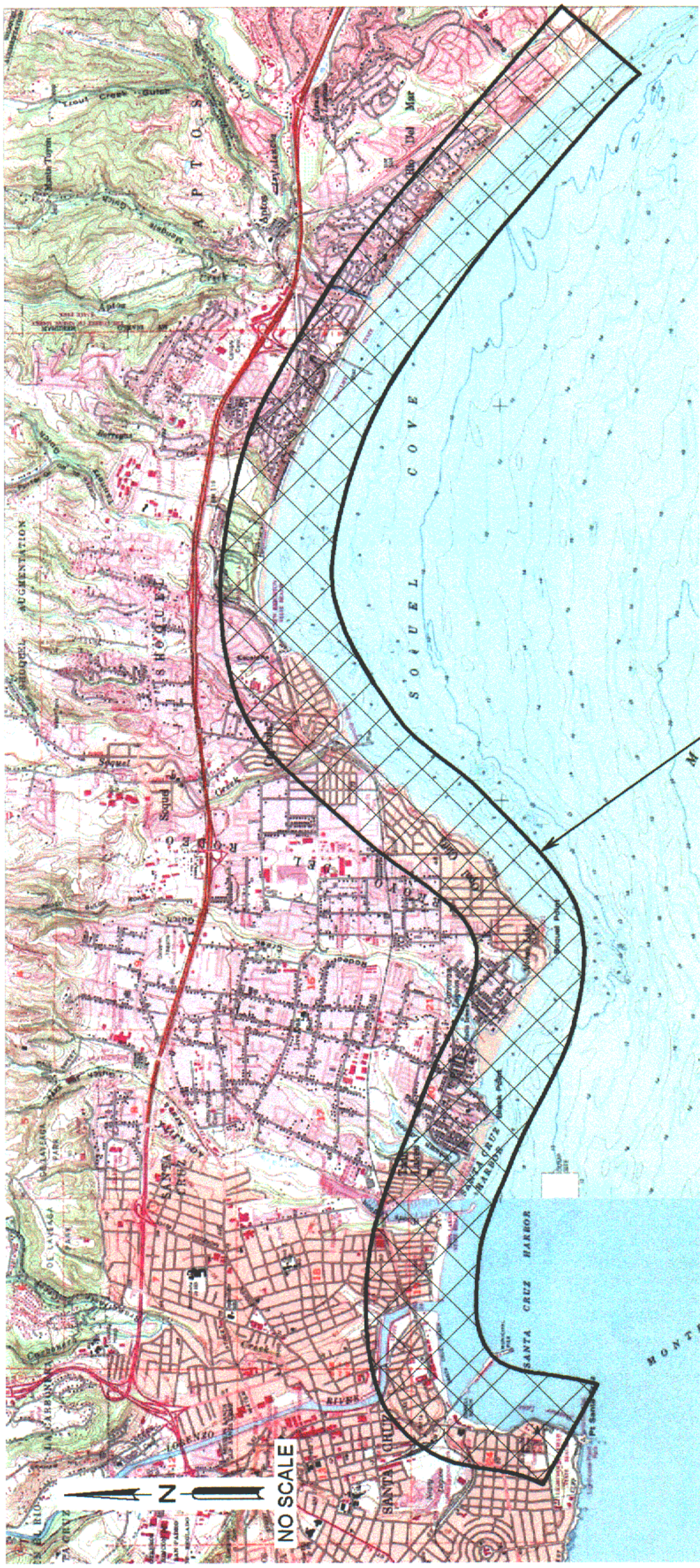
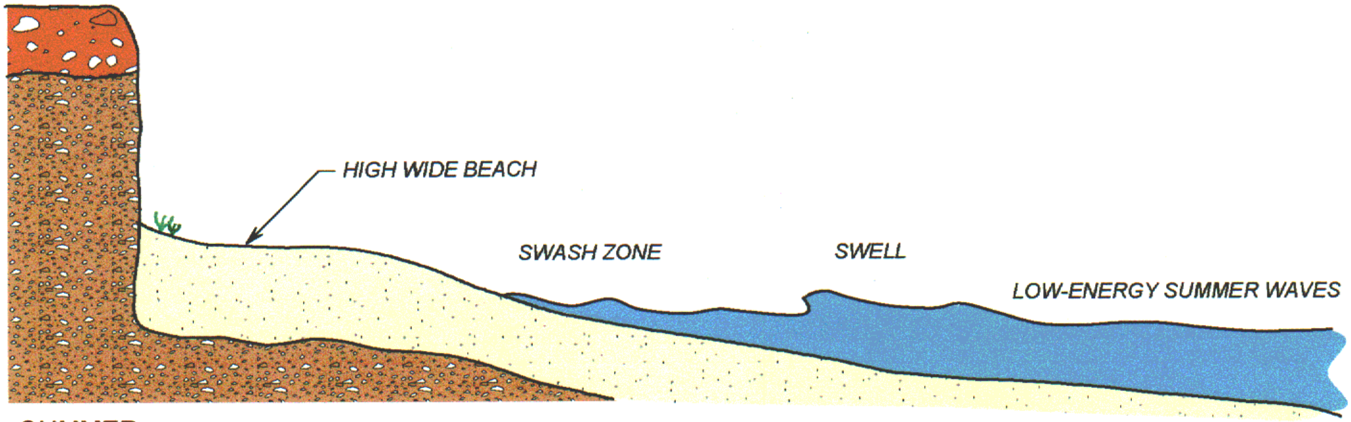
PLATES

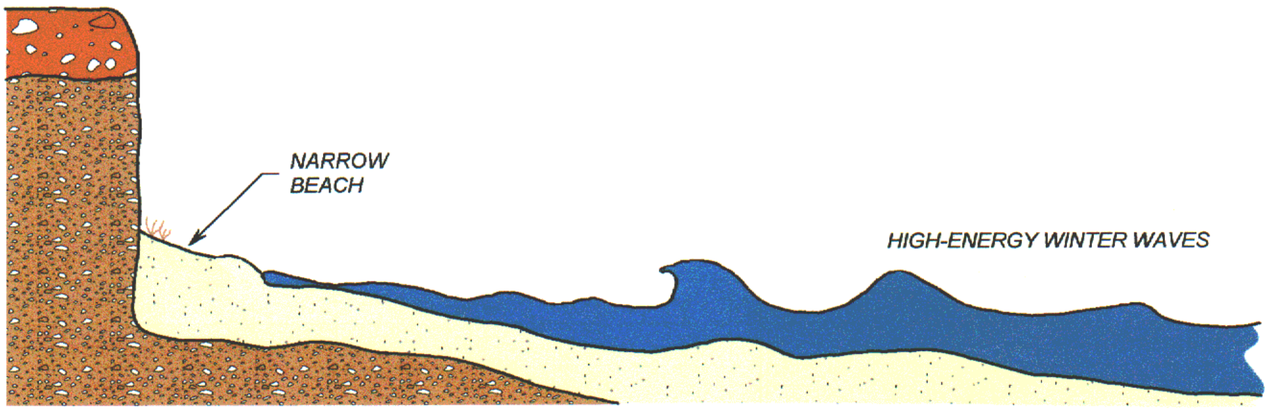


SOURCE: USGS SOQUEL & SANTA CRUZ QUADRANGLES, 1954, REVISED 1994.

STUDY AREA LOCATION MAP



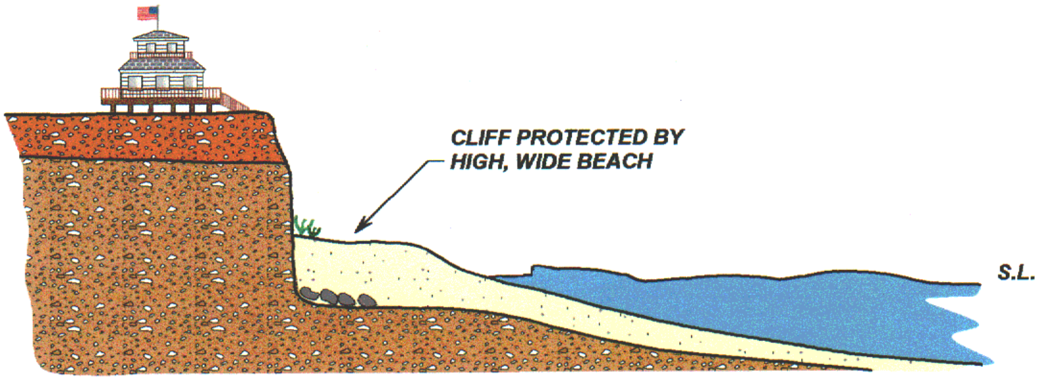
SUMMER



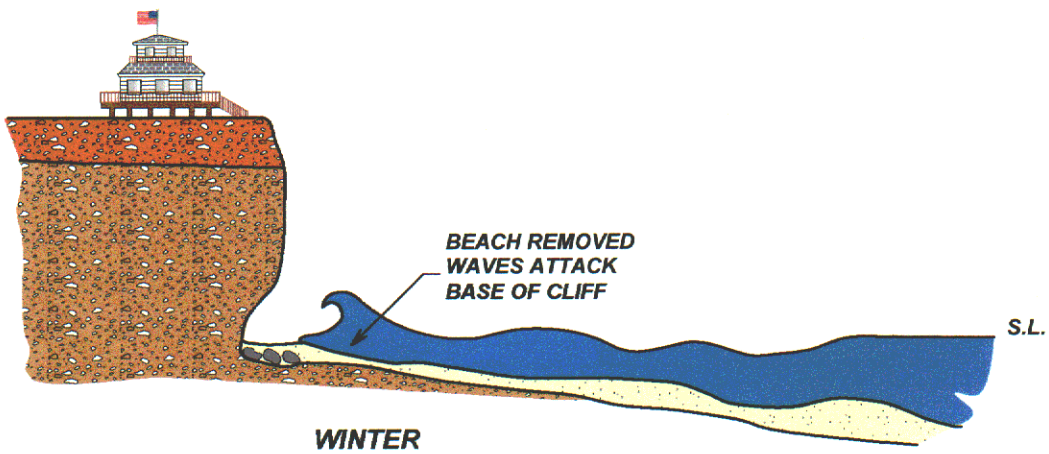
WINTER

AVERAGE SEASONAL CHANGES OF THE BEACH PROFILE

PLATE 2

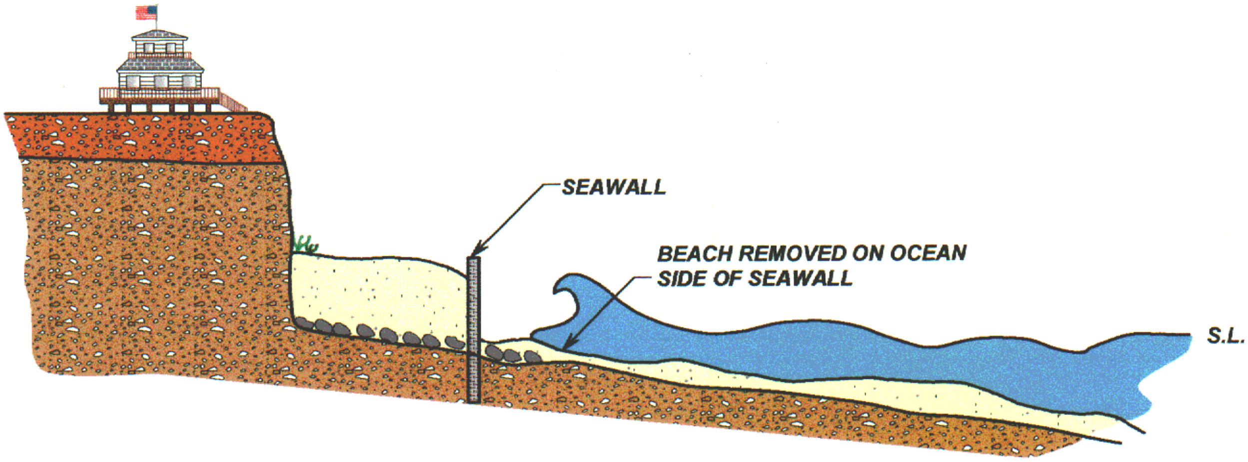


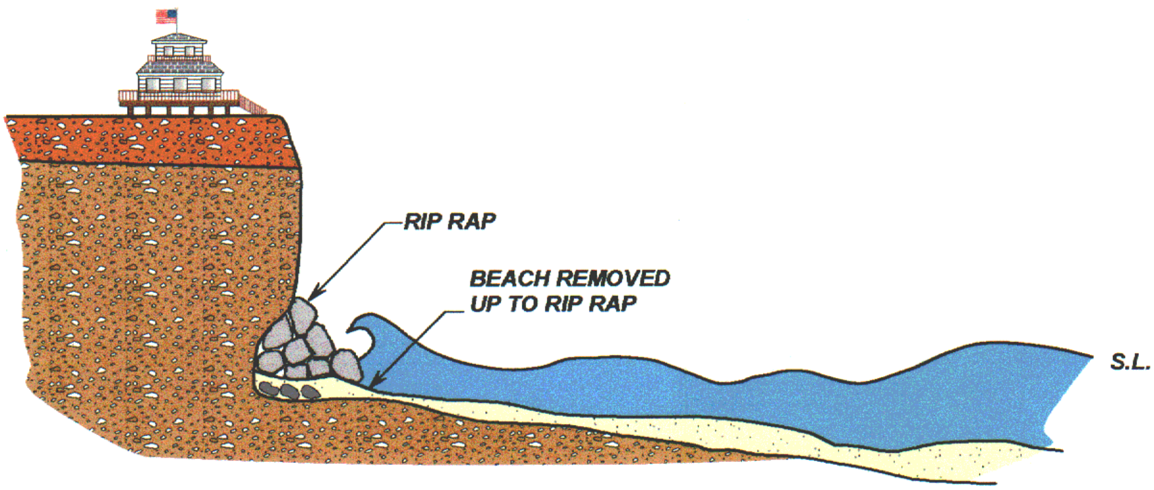
SUMMER



WINTER

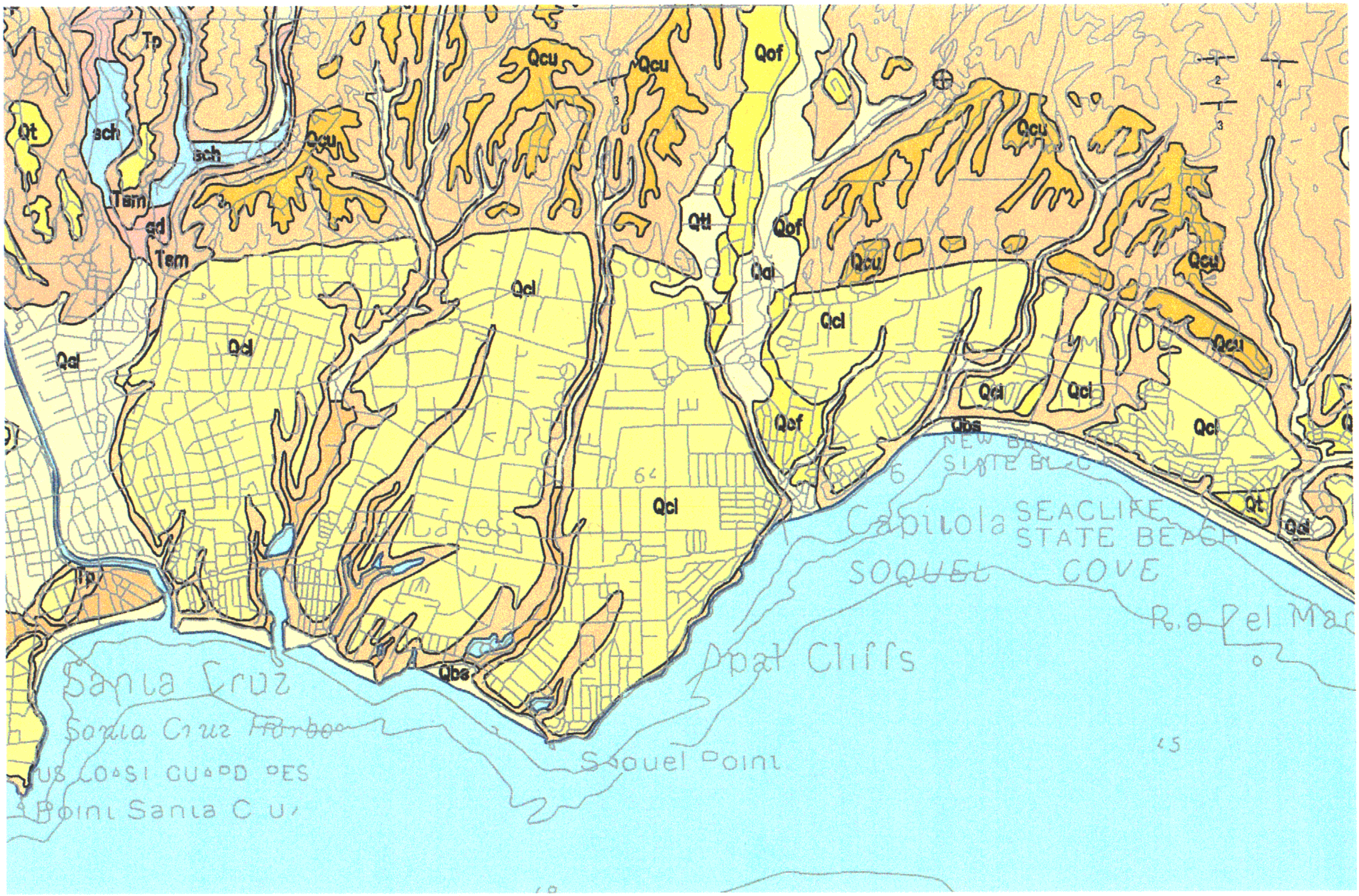
LARGE STORM EFFECTS ON THE BEACH PROFILE





TYPICAL CLIFF PROTECTION METHODS

APPENDIX A
REGIONAL GEOLOGY
(from U.S. Geological Survey Open-File Report 97-489, [1997])



Tp
Qt
sch
sch
Tsm
gd
Tsm
Qcu
Qcu
Qcu
Qof
Qtl
Qof
Qal
Qcu
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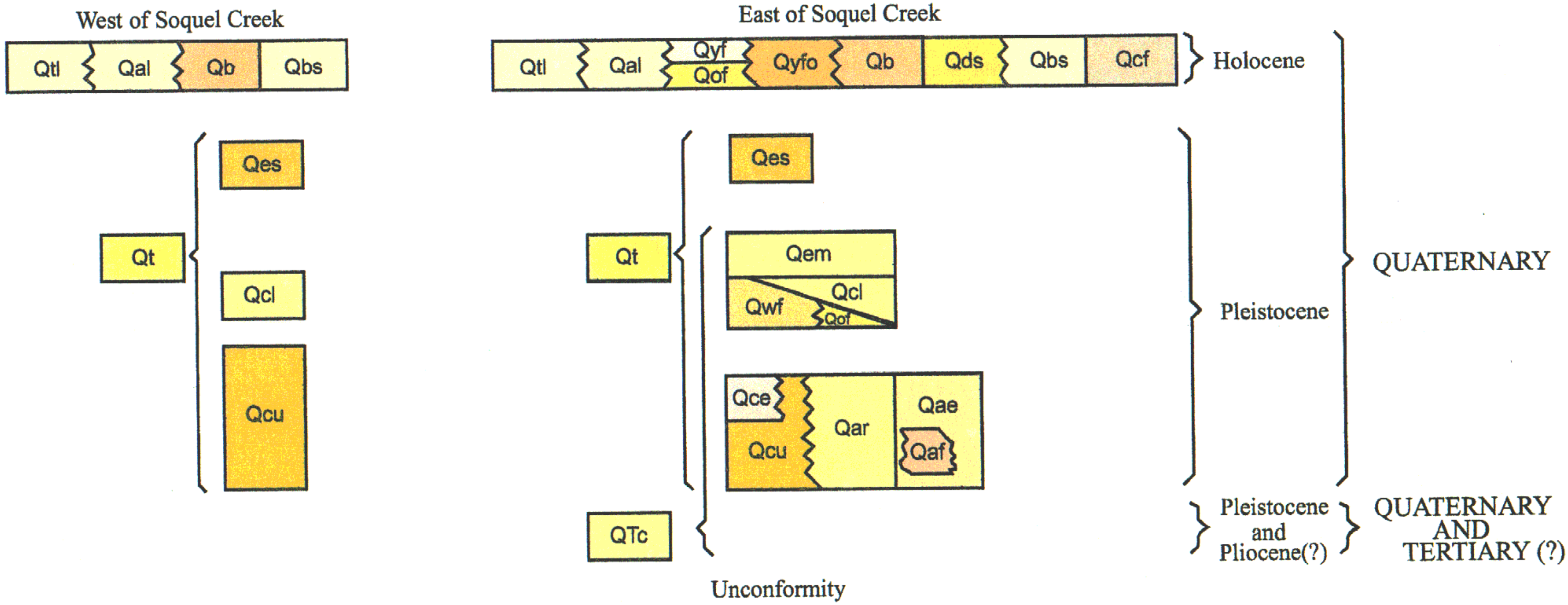
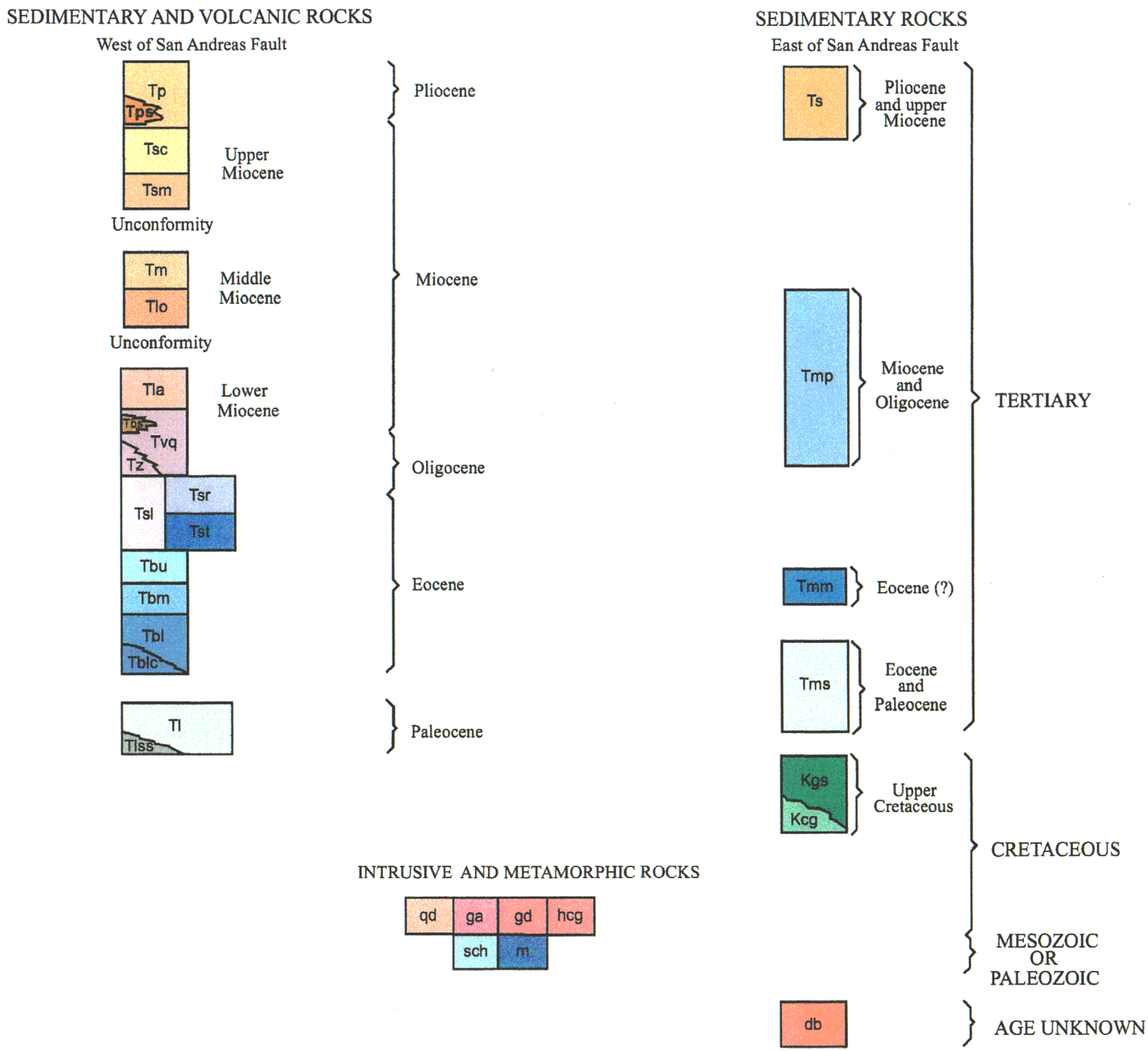
Qcu
Qal
Qcl
Qcl
Qcl
Qcl
Qcl
Qcl
Qcl
Qcf
Qbs
Qt
Qal
Tp
Qbs
Capitola
SEACLIFE
STATE BEACH
SOQUEL
COVE
Opal Cliffs
Santa Cruz
Soquel Point
Point Santa Cruz

CORRELATION OF MAP UNITS

SURFICIAL DEPOSITS




SEDIMENTARY AND VOLCANIC ROCKS
West of San Andreas Fault
Tp
Tps
Tsc
Tsm
Upper Miocene
Unconformity
Tm
Tlo
Middle Miocene
Unconformity
Tla
Lower Miocene
Tbs
Tvq
Tz
Tsl
Tsr
Tst
Tbu
Tbm
Tbl
Tblc
Tl
Tlss
Pliocene
Miocene
Oligocene
Eocene
Paleocene
SEDIMENTARY ROCKS
East of San Andreas Fault
Ts
Pliocene and upper Miocene
Tmp
Miocene and Oligocene
TERTIARY
Tmm
Eocene (?)
Tms
Eocene and Paleocene
Kgs
Kcg
Upper Cretaceous
CRETACEOUS
MESOZOIC OR PALEOZOIC
db
AGE UNKNOWN
INTRUSIVE AND METAMORPHIC ROCKS
qd
ga
gd
hcg
sch
m

Contact - Dashed where approximately located; queried where uncertain

Fault - Dashed where approximately located; dotted where concealed; queried where uncertain. Arrows indicate direction of relative movement

Anticline - Dashed where approximately located; queried where uncertain

Syncline - Dashed where approximately located; queried where uncertain

Strike and dip of beds

80 **Inclined**

50 **Approximate dip of bedding**

Vertical

Horizontal

60 **Overturned**

20 **Strike and dip of foliation**

SOURCES OF DATA

Quaternary deposits nearly entirely from Dupré (1975). Landslides intentionally omitted (see map by Cooper-Clark and Associates, 1975). See maps by Hall and others (1974) and Sarna-Wojcicki and others (1975) for fault hazards. See map by Dupré (1975) for liquefaction hazard and map by McCrory and others (1977) for earthquake intensity zonation. For areas offshore, see McCulloch and others (1977), Hoskins and Griffiths (1971), Greene (1977), and Greene and others (1973). For an interpretation of subsurface basement rocks in the Monterey Bay area, see Ross and Brabb (1973). For analyses of middle and late Tertiary sedimentation and tectonics, see Stanley (1984), and Phillips (1983, 1984a, b).

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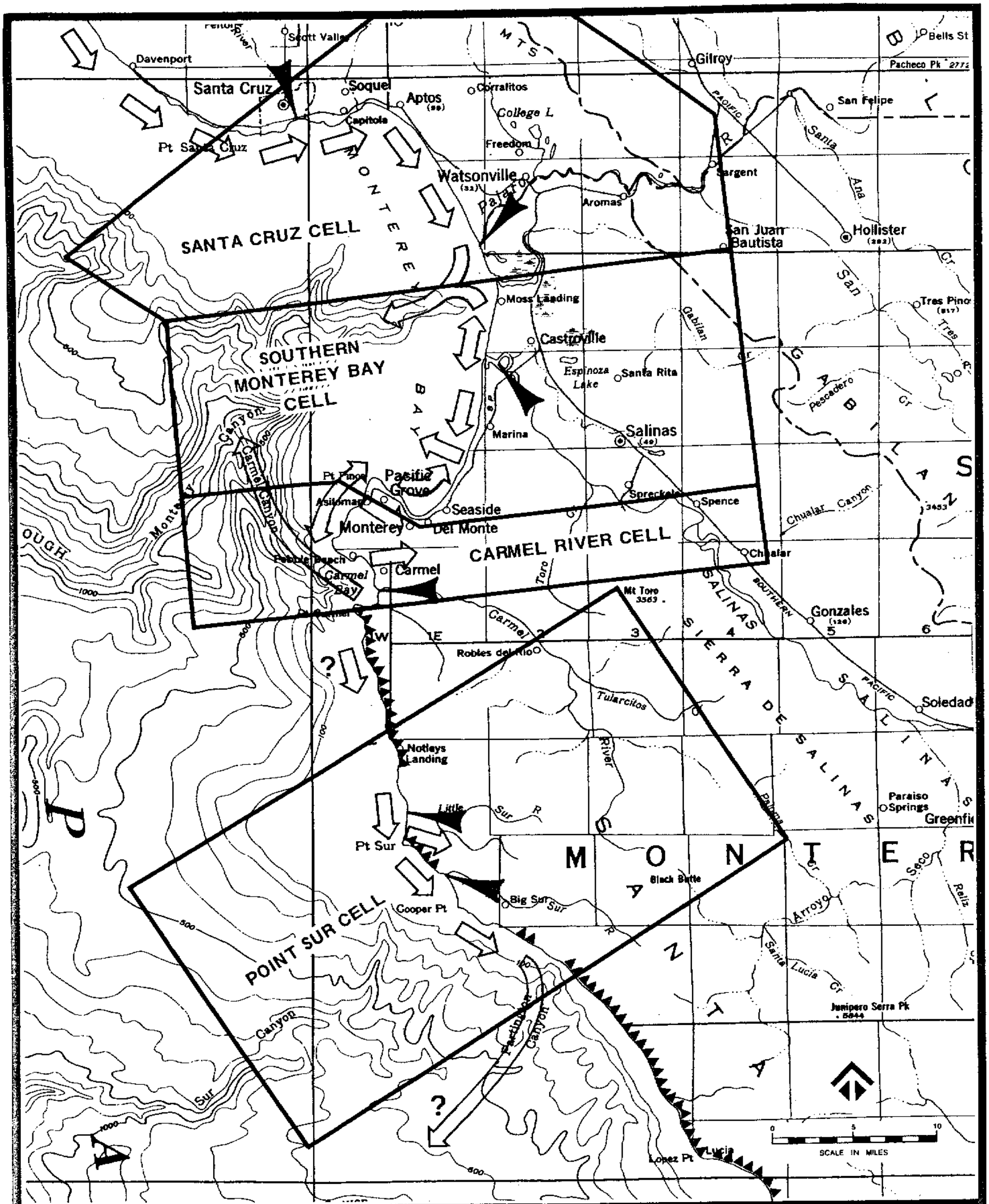
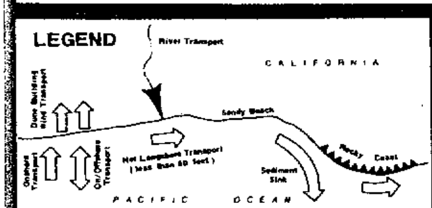
2. Brabb (1960) and Cummings and others (1962). See also report by McJunken (1984).
3. Dibblee (1966).
4. Brabb and Dibblee (1979), Burchfiel (1958), and Hector (1976).
5. Dibblee and Brabb (1978) and Ross (1970).
6. Clark (1981), Leo (1961, 1967). Ben Lomond fault extended from map by Stanley and McCaffrey (1983, fig. 3).
7. Dibblee and others (1978).
8. Dibblee and Brabb (1980), McLauhlin and others (1971), and McLaughlin (1971, 1973).
9. Dibblee (1973).
10. Dupre (1975).

APPENDIX B
PLATES FROM SHORELINE EROSION STUDY
(from Assessment and Atlas of Shoreline Erosion Along the California Coast, [Habel and Armstrong, 1977])





LITTORAL CELLS

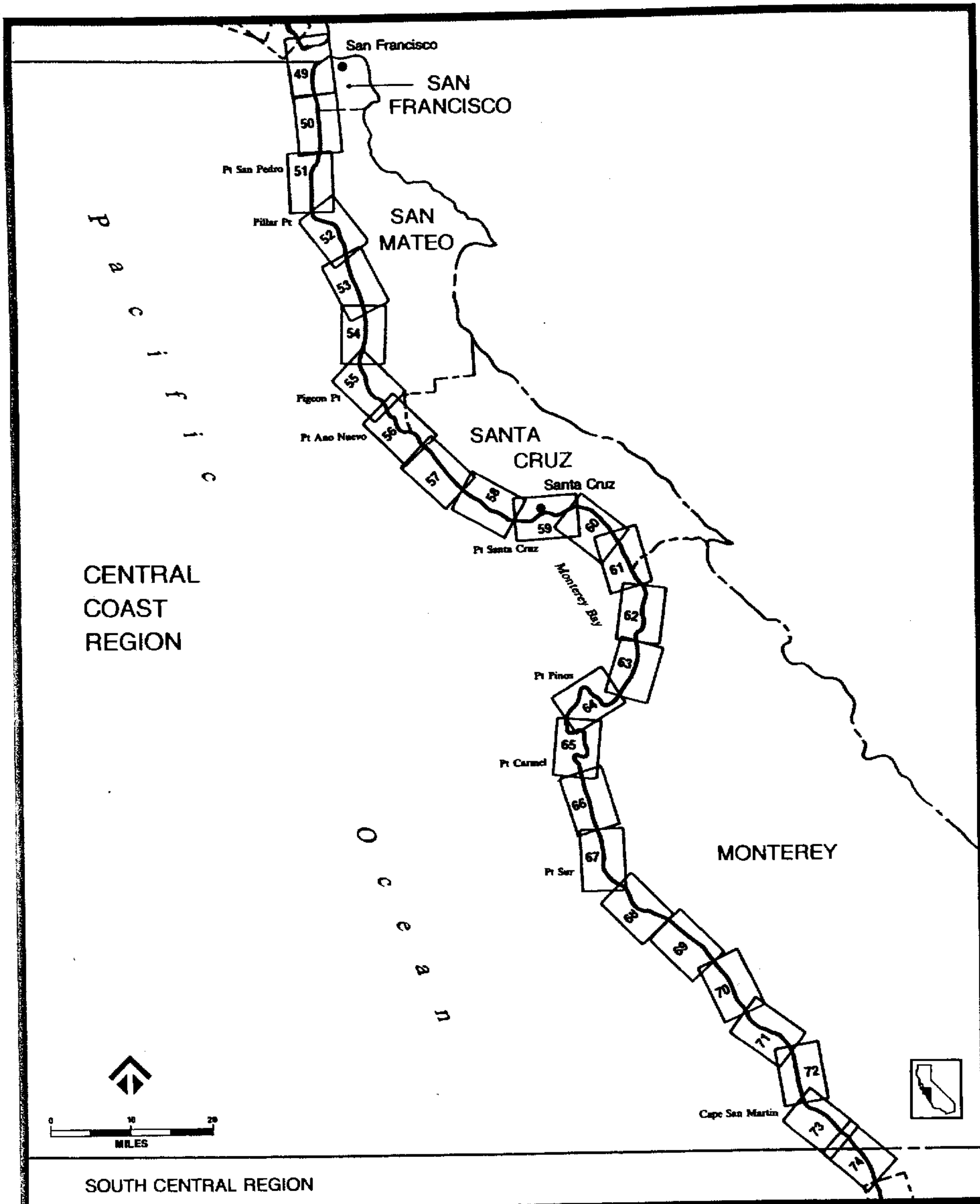
MAP NUMBER 6

STATE OF CALIFORNIA — THE RESOURCES AGENCY

DEPARTMENT OF NAVIGATION & OCEAN DEVELOPMENT

23

1977



MAP INDEX

MAP NUMBER 3

STATE OF CALIFORNIA — THE RESOURCES AGENCY

DEPARTMENT OF NAVIGATION & OCEAN DEVELOPMENT

A-13

DNOD

1977

Rugged rocky shore with offshore rocks and reef backed by nearly vertical, wave cut, sinuous cliff with flat rock shelf and narrow rock protrusions along rim of wide coastal terrace. Small sandy pocket beaches formed between protrusions and at mouth of creeks. Creek mouth backed by low active dunes and lagoon.

Low wave cut cliff with flat rock shelf and flat offshore rock reef and protrusions with arches. Small sandy pocket beach at creek mouth backed by mobile home park and park facilities subject to damage during high wave conditions.

Wave cut rocky cliff with flat offshore rock reef and sea stacks. Narrow rock protrusions act as groins to form small pocket beaches. Road and overlooks protected by rock rubble seawall and revetment. Road and houses subject to damage during high wave conditions. Cliff undercut with frequent slides in non-protected stretches.

Rocky point with offshore rock reef, sea stack, sea caves, and arch backed by park, lighthouse, and road.

High, rocky, wave cut cliff with offshore rock reef backed by road, park, and motels. Base of cliff protected by rock rubble revetment.

Wide sandy beach with offshore piers backed by concrete seawall, beach, and recreation facilities.

Sand bar across river mouth backed by flood control channel.

Wide sandy beach formed between natural rock groin at river mouth and jetty backed by wave cut eroded cliff with road and houses along rim. Beach stabilized by downcoast jetty.

Santa Cruz Harbor entrance jetties backed by small-craft harbor.

Sandy beach formed between jetty and rock point backed by low bluff bisected by creeks and lagoon. Houses along rim of bluff endangered during high wave condiions. Beach periodically nourished by sand dredged from harbor entrance.

Sandy pocket beaches between rocky points with flat rock shelf and sea caves backed by wave cut cliff bisected by lagoons. Houses protected by rock rubble seawalls. Houses endangered during high wave conditions and by cliff erosion.

Rocky point with flat offshore rock reef and flat rock shelf backed by wave cut cliff. Houses along rim endangered during high wave conditions protected by concrete and rock rubble seawalls.

Rocky wave cut cliff with flat offshore rock reef backed by houses, road, and railroad. Cliff undercut with slides along base, face, and top with rock protection along toe. Houses and road endangered by slides.

Wide sandy beach backed by motels, commercial buildings, and creek. Beach formed by down-coast rock groin.

Rocky wave-undercut cliff with flat offshore rock reef with apartments, houses, and road along rim. Active slides along base, face, and top. Buildings and road endangered by slides.

• SHORELINE CONDITION •

PRESENT DEVELOPMENT CRITICAL	ARTIFICIAL PROTECTION
PRESENT DEVELOPMENT NON-CRITICAL	PROTECTIVE BEACH
FUTURE DEVELOPMENT CRITICAL	STABLE ROCK

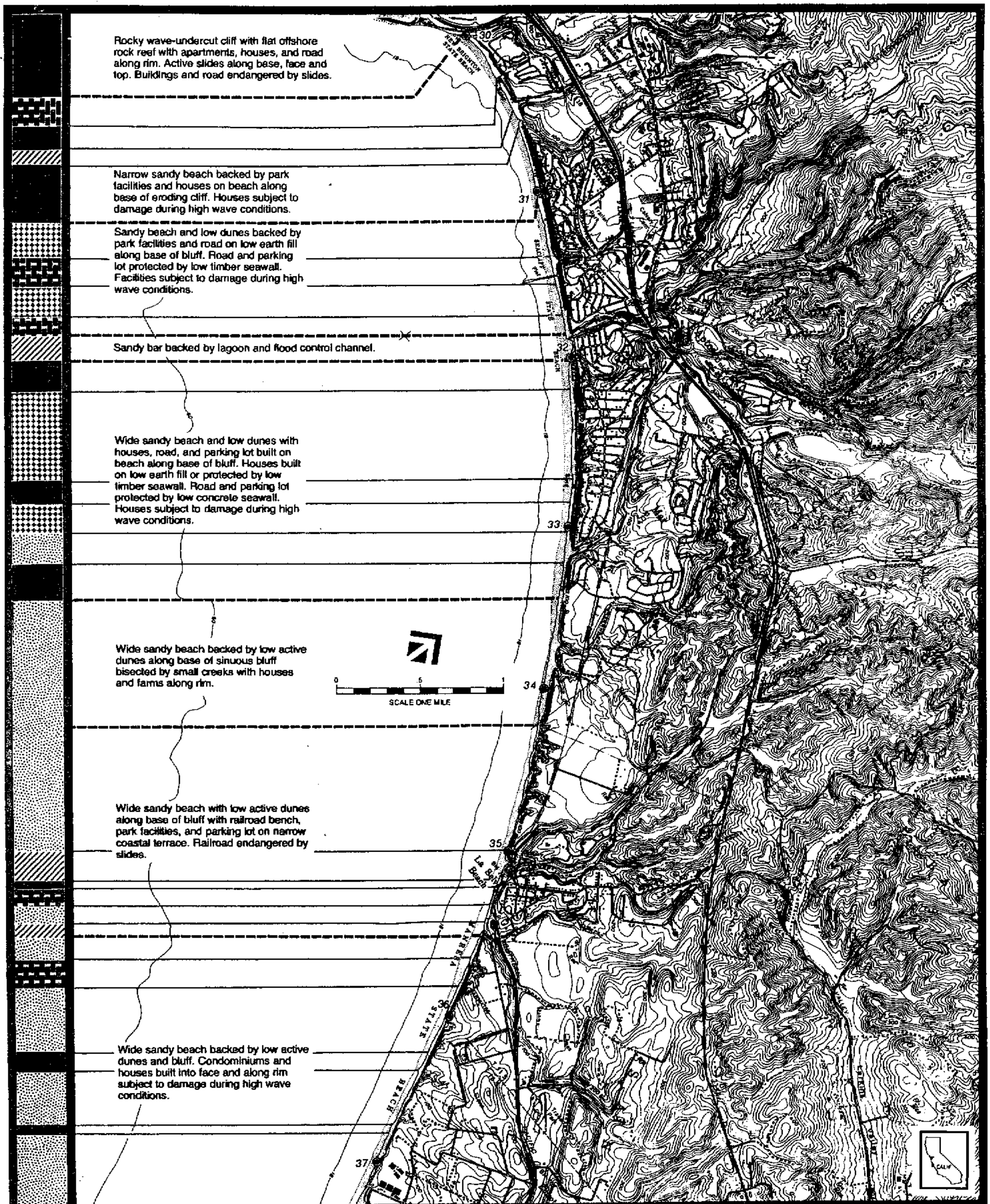
SANTA CRUZ CO. — MI. 21-29

MAP NUMBER 59

STATE OF CALIFORNIA — THE RESOURCES AGENCY

DEPARTMENT OF NAVIGATION & OCEAN DEVELOPMENT

1977



- SHORELINE CONDITION -

PRESENT DEVELOPMENT CRITICAL	ARTIFICIAL PROTECTION		
PRESENT DEVELOPMENT NON-CRITICAL	PROTECTIVE BEACH		
FUTURE DEVELOPMENT CRITICAL	STABLE ROCK		

SANTA CRUZ CO. — ML 30 - 37

MAP NUMBER **60**

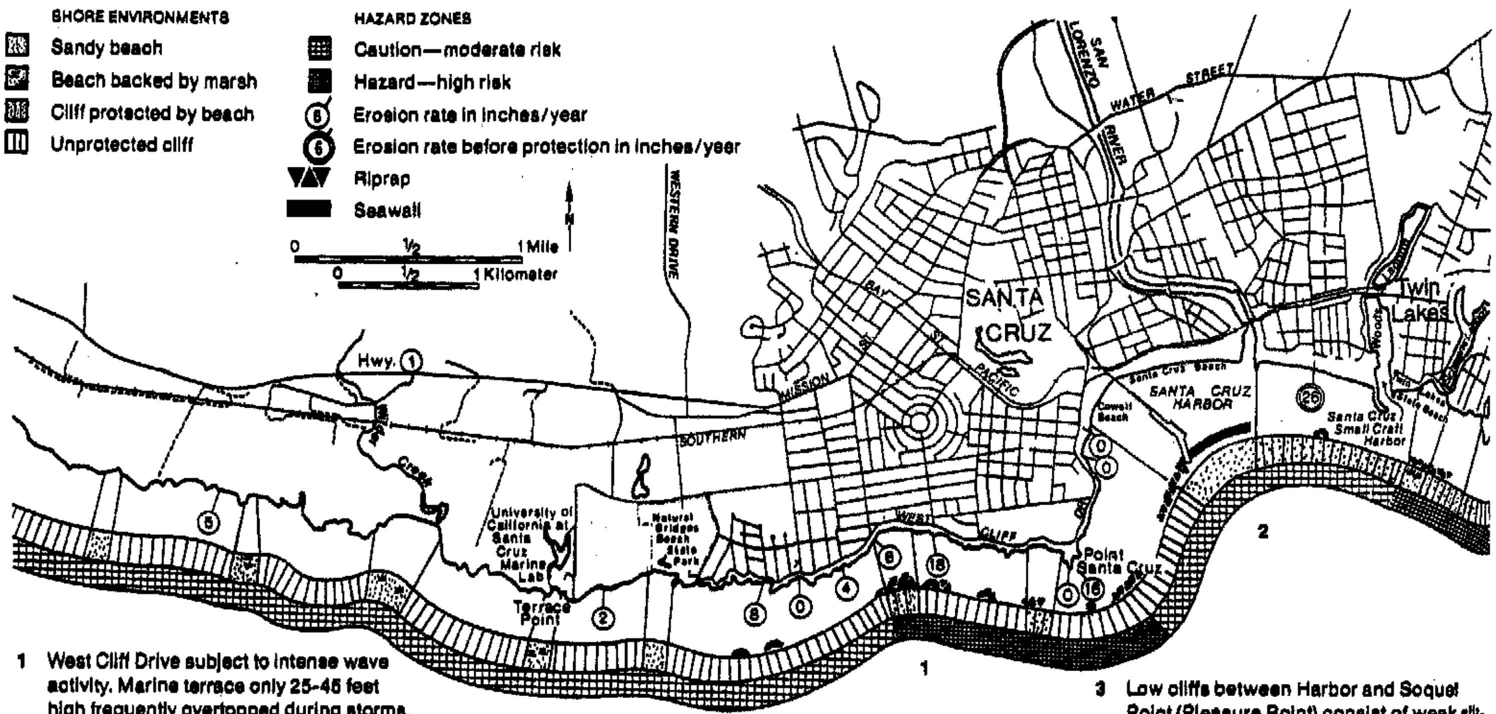
STATE OF CALIFORNIA — THE RESOURCES AGENCY

DEPARTMENT OF NAVIGATION & OCEAN DEVELOPMENT

DNOD

1977

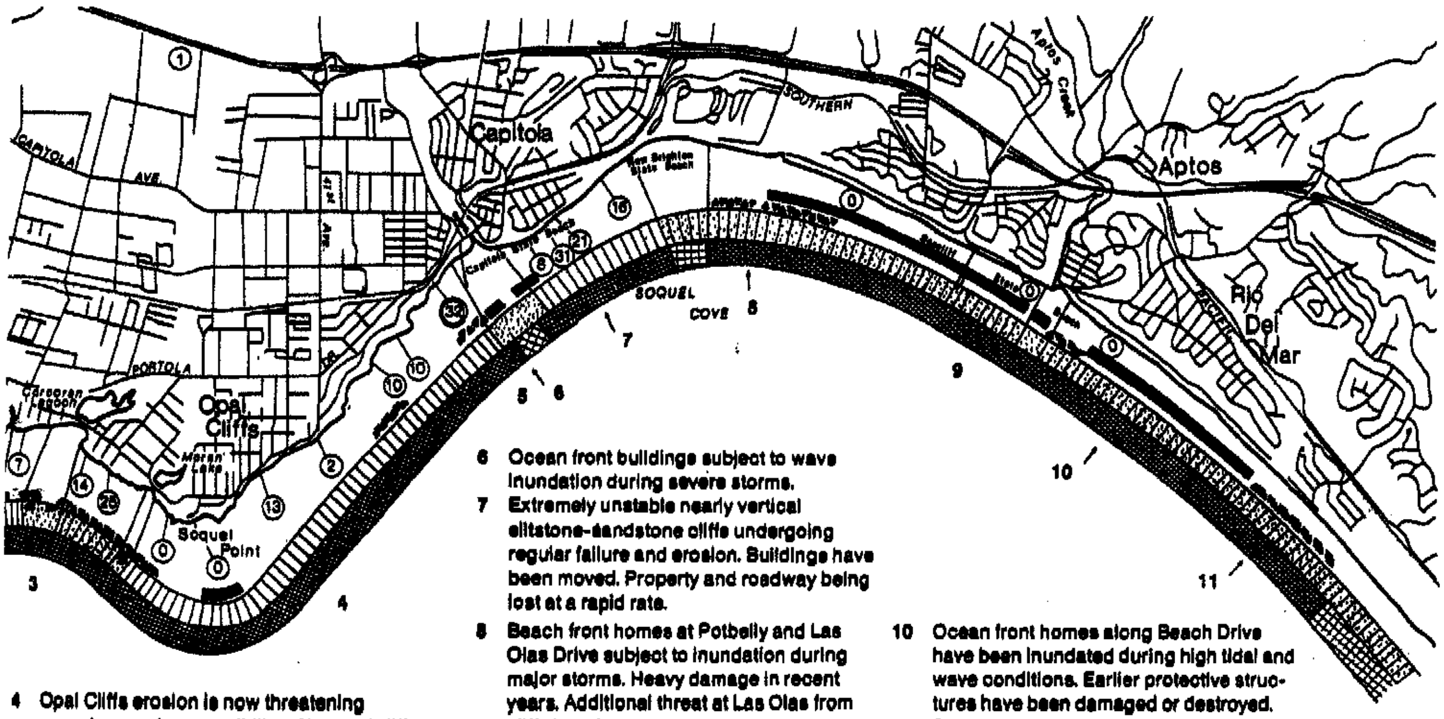
APPENDIX C
SHORELINE PROTECTION STRUCTURES AND CLIFF FAILURE HAZARDS
(from Living With The California Coast, [Griggs and Savoy,1985])



1 West Cliff Drive subject to intense wave activity. Marine terrace only 25-45 feet high frequently overtopped during storms. Undercut and collapse of bike path and roadway is recurring problem. Winter 1983 storms produced up to 40 feet of erosion of unconsolidated material lying atop the low bedrock terrace. Severe erosion at west side of Lighthouse Point.

2 Cliffs backing Seabright Beach were rapidly eroding until harbor construction in 1963. Wide permanent beach now buffers the area.

3 Low cliffs between Harbor and Soquel Point (Pleasure Point) consist of weak siltstone and sandstone characterized by active erosion. Property, home, and road damage common during severe storms. Much of the public and private property now protected with riprap.



6 Ocean front buildings subject to wave inundation during severe storms.

7 Extremely unstable nearly vertical siltstone-sandstone cliffs undergoing regular failure and erosion. Buildings have been moved. Property and roadway being lost at a rapid rate.

8 Beach front homes at Potbelly and Las Olas Drive subject to inundation during major storms. Heavy damage in recent years. Additional threat at Las Olas from cliff slumping.

9 Seacliff State Beach facilities and protective bulkhead repeatedly damaged or destroyed during major storms from southwest.

10 Ocean front homes along Beach Drive have been inundated during high tidal and wave conditions. Earlier protective structures have been damaged or destroyed. Present seawall overtopped but not destroyed during 1983 storms. Additional threat from failure of unconsolidated material in steep seacliff behind homes.

11 Homes on beach fill seriously damaged as waves overtopped and damaged protective riprap during 1983 storms.

4 Opal Cliffs erosion is now threatening many homes. Inaccessibility of base of cliff has made the installation of any protective works very difficult.

5 Severe erosion immediately upcoast from Capitola now reduced with riprap.

Site analysis: Santa Cruz through Rio Del Mar.

APPENDIX B - COST ESTIMATES - DESALINATION

City of Santa Cruz / Soquel Creek Water District
Regional Water Supply Study - Desalination
DRAFT Summary of Conceptual Level Opinion of Probable Costs

Amortization Period	20 yrs
Interest Rate	8.0%

The user should only modify these yellow cells.

Range of Desalination Plant Costs

	2 MGD	4 MGD	6 MGD
Capital Costs Paid by Santa Cruz	50%	50%	50%
Capital Costs Paid by Soquel Creek	50%	50%	50%
O& M Costs Paid by Santa Cruz *	50%	50%	50%
O&M Costs Paid by Soquel Creek *	50%	50%	50%
Number of Months Plant Operates	12	6	4
Percentage of Water Delivered to Santa Cruz	50%	50%	50%
Percentage of Water Delivered to Soquel Creek	50%	50%	50%
Estimated Capital Costs	**$26 M - $29 M**	**$38 M - $41 M**	**$49 M - $53 M**
Annualized Costs	$2.7 M - $3.0 M	$3.8 M - $4.2 M	$5.0 M - $5.4 M
Annual O&M Costs	$2.1 M - $2.1 M	$2.2 M - $2.2 M	$2.3 M - $2.5 M
Total Annual Costs	**$4.7 M - $5.1 M**	**$6.0 M - $6.4 M**	**$7.3 M - $7.8 M**
Annual Volume of Water Produced	730 MG or 2239 AF	730 MG or 2239 AF	730 MG or 2239 AF
Price Per MG Of Water Produced	$6,500 - $7,003	$8,239 - $8,734	$10,041 - $10,731
Price Per AF Of Water Produced	$2,118 - $2,282	$2,685 - $2,846	$3,272 - $3,497
Price Per 100 CF Of Water Produced	**$4.86 - $5.24**	**$6.16 - $6.53**	**$7.51 - $8.03**

Average Estimated Annual Costs

Santa Cruz's Annual Costs			
Current Santa Cruz Water Rate	$1.97 / 100 CF		
Annual Expenses	$2.5 M	$3.1 M	$3.8 M
Annual Volume of Water Received	365 MG or 1120 AF	365 MG or 1120 AF	365 MG or 1120 AF
Price Per MG Of Water Produced *	$6,751	$8,487	$10,386
Price Per AF Of Water Produced *	$2,200	$2,765	$3,384
Price Per 100 CF Of Water Produced *	$5.05	$6.35	$7.77
Revenue from Desalinated Water Sales	$1.0 M	$1.0 M	$1.0 M
Annual Budget Impact **	-$1.5 M	-$2.1 M	-$2.8 M

Soquel Creek's Annual Costs			
Current Soquel Creek Water Rate	$2.00 / 100 CF		
Annual Expenses	$2.5 M	$3.1 M	$3.8 M
Annual Volume of Water Received	365 MG or 1120 AF	365 MG or 1120 AF	365 MG or 1120 AF
Price Per MG Of Water Produced *	$6,751	$8,487	$10,386
Price Per AF Of Water Produced *	$2,200	$2,765	$3,384
Price Per 100 CF Of Water Produced *	$5.05	$6.35	$7.77
Revenue from Desalinated Water Sales	$1.0 M	$1.0 M	$1.0 M
Annual Budget Impact **	-$1.5 M	-$2.1 M	-$2.8 M

* Note: The percentage of annual O&M costs paid by a utility is equal to the percentage of desalinated water delivered to the utility during that year. If a utility does not receive water in a given year (i.e., O&M costs equal 0%) then a unit based water price cannot be calculated because the utility did not receive any water.

** Note: Negative number indicates a (potential) deficit.

APPENDIX C - COST ESTIMATES - WATER RECLAMATION

City of Santa Cruz / Soquel Creek Water District
Regional Water Supply Study
DRAFT Summary of Conceptual Level Opinion of Probable Costs for Reclaimed Water

Capital Costs Paid by Santa Cruz	50%
O& M Costs Paid by Santa Cruz	50%
Number of Months Plant Operates	5
Ammoritization Period	20 years
Interest Rate	8.0%

Capital Costs Paid by Soquel Creek	50%
O& M Costs Paid by Soquel Creek	50%

The user should only modify these yellow cells.

Reclaimed Plant to North Coast

	5 MGD
Estimated Capital Costs	$49 M
Annualized Costs	$5.0 M
Annual O&M Costs	$0.4 M
Total Annual Costs	$5.4 M
O&M Costs as % of Annual Costs	7%
Annual Volume of Water Produced	760 MG
Price Per MG Of Water Produced	$7,068
Price Per AF Of Water Produced	$2,303
Price Per 100 CF Of Water Produced	$5.29

Reclaimed Plant to Diversion Structure

	5 MGD
Estimated Capital Costs	$31 M
Annualized Costs	$3.2 M
Annual O&M Costs	$0.4 M
Total Annual Costs	$3.5 M
O&M Costs as % of Annual Costs	10%
Annual Volume of Water Produced	760 MG
Price Per MG Of Water Produced	$4,615
Price Per AF Of Water Produced	$1,504
Price Per 100 CF Of Water Produced	$3.45

Cost Sharing Summary

Santa Cruz's Annual Costs	
Price Per MG Of Water Produced	$3,534
Price Per AF Of Water Produced	$1,151
Price Per 100 CF Of Water Produced	$2.64
Soquel Creek's Annual Costs	
Price Per MG Of Water Produced	$3,534
Price Per AF Of Water Produced	$1,151
Price Per 100 CF Of Water Produced	$2.64

Santa Cruz's Annual Costs	
Price Per MG Of Water Produced	$2,308
Price Per AF Of Water Produced	$752
Price Per 100 CF Of Water Produced	$1.73
Soquel Creek's Annual Costs	
Price Per MG Of Water Produced	$2,308
Price Per AF Of Water Produced	$752
Price Per 100 CF Of Water Produced	$1.73

APPENDIX D - Brine Disposal Dilution Analysis

February 15, 2002

Mr. Brian Jordan
Black and Veatch Corporation
800 Wilshire Blvd., Suite 600
Los Angeles, California 90017

Mr. Ken Wilkins
Carollo Engineers
2700 Ygnacio Valley Road, Suite 300
Walnut Creek, California 94598 11-22202-005/5

Subject: Soquel Creek Water District Alternative Water Supply Project–Brine Disposal

Dear Mr. Jordan and Mr. Wilkins:

We have completed a desktop evaluation of diffuser hydraulics, effluent dilution and flow equalization requirements for the disposal of brine produced from the proposed desalination plant through the existing effluent outfall operated by the City of Santa Cruz (City). Our analysis was performed in accordance with Brown and Caldwell's proposal to the Soquel Creek Water District dated May 15, 2001, and the services agreement with Black and Veatch dated December 20, 2001.

Executive Summary

We have reached several conclusions regarding the potential to add brine from a proposed desalination plant to the City's existing effluent discharge.

Dilution Factor. The addition of brine to effluent caused the dilution achieved at the diffuser to decrease substantially. The main mechanism for this decrease was the high salinity of the brine, which resulted in a higher density in the composite effluent. Since the majority of the dilution process occurs during the rise of a buoyant plume, this loss of buoyancy translates directly into a decrease in dilution. Higher effluent temperatures during the summer were found to increase the buoyancy of the composite effluent and enhance dilution. The maximum brine flow that can be added to effluent while still meeting a minimum required dilution of 114:1 ranges from 85 to 115 percent of the effluent flow.

Flow Equalization. Flow equalization of brine discharge will be required when it exceeds the maximum brine flow permitted to meet a dilution of 114:1. Based on a worst-case drought scenario with average, minimum, and peak daily effluent flows of 5.0, 1.0 and 10.5 mgd, respectively, recommended equalization storage is 0.2 million gallons for a 2-mgd desalination plant and 1 million gallons for a 4-mgd desalination plant. Under drought conditions, effluent flow is too low to allow for the complete discharge of stored brine over a 24-hour period for a 6-mgd desalination plant. To dispose of brine on a daily basis from a 6-mgd desalination plant requires a minimum average daily effluent flow of approximately 8.1 mgd and an equalization basin of 1.4 million gallons. Rather than relaying solely on brine storage capacity, the rate of desalination could be lowered during times of low effluent flow so that a dilution of 114:1 is maintained. In addition, brine disposal will need to be curtailed during brief (4-6 hours) episodes of extreme peak wet weather flow that meet or exceed the hydraulic capacity of the outfall.

Trace Metal Concentrations. Brine concentrations of trace metals (arsenic, copper, mercury, silver, and zinc) will be less than effluent concentrations, and concentrations in the composite effluent will remain far below effluent limits.

Brine Addition to Effluent. On a conceptual basis, we have identified two locations where brine can be added to effluent: the tunnel portal box located at the City's treatment plant or the tunnel gate box located near the beach. In both cases, the brine must be added in a way that promotes complete mixing between the brine and the effluent to avoid two-phase, stratified flow in the effluent outfall.

Effluent Monitoring. The City will have to modify its current monitoring of effluent in the tunnel portal box if brine is added at this point.

Corrosion. With addition of brine, a number of structures will be exposed to waters with a higher level of salinity, and thus be more susceptible to corrosion. Of particular concern are the 36-inch and 72-inch sluice gates in the outfall gate box.

Objectives

The objectives of the study were to:

- Examine effluent flow and temperature records to establish reasonable seasonal and diurnal patterns on which to base subsequent dilution analysis.

- Evaluate diffuser hydraulics and effluent dilution under various brine disposal scenarios.

- Estimate brine-flow equalization requirements under various brine disposal scenarios.

- Identify possible locations for brine addition to the effluent outfall system.

- Compare projected concentrations of trace metal pollutants in composite effluent to discharge permit limitations.

- Identify any other issues of concern regarding brine addition.

Methods And Assumptions

Diffuser hydraulics and dilution are a function of a number of variables including outfall and diffuser characteristics, effluent density, effluent flow rate, and the density in the ambient water. Density in turn is a function of both temperature and salinity. High dilution rates are achieved when: 1) effluent is much more buoyant than the ambient fluid (e.g., fresh water discharged into seawater, warm water discharged into cool water), 2) when the ambient fluid is not density stratified (e.g., winter versus summer conditions), and 3) during times of low flow rates. Thus, the addition of cool, high-salinity brine to effluent is anticipated to lower dilution by increasing the density, lowering the temperature and increasing the flow rate of the composite effluent.

Composite Effluent

The first step in evaluating the impact of brine addition on diffuser performance was to estimate the temperature and salinity of various mixtures of brine and effluent (Table 1). Here we assumed the complete mixing of the brine and effluent prior to discharge from the outfall (see “Brine Addition to Effluent” section below). Brine flows for the 2, 4 and 6 million gallon per day (mgd) plant were 2.44, 4.89, and 7.33 mgd, respectively, and are based on a rejection rate of 55 percent at the reverse osmosis (RO) membranes of the desalination plant. Brine salinity and temperature were estimated using coastal water quality data from Brown and Caldwell’s Oceanographic Predesign Phase Report–Santa Cruz Effluent Facilities Planning Study (1978), assuming that the intake for the desalination plant was 12 meter (m) deep. Winter conditions are based on the averages of six profiles collected on February 23-24, 1977. Summer/fall conditions are the average of four profiles collected on September 22, 1976. These profiles were measured in 45 and 60 feet of water roughly one-third of a mile off of Terrence Point, Santa Cruz. Brine was concentrated by a factor of 1.82 based on an RO rejection rate of 55 percent. No change in brine temperature was assumed through the treatment process. Effluent temperature was based on 2001 effluent data in February (winter case) and October (summer/fall case). Effluent salinity was assumed to be 0.5 parts per thousand (ppt).

Table 1. Brine and Effluent Water Quality

	Brine		Effluent	
Parameter	Winter	Summer/Fall	Winter	Summer/Fall
Temperature, °C	12.32	12.80	18.0	23.0
Salinity, ppt	61.38	61.42	0.5	0.5

Ambient Water Quality

Seasonal changes in density stratification in the ambient fluid (receiving waters) can affect dilution. For example, strong thermal stratification in the summer and fall can inhibit dilution since it hinders the upward momentum of the buoyant plume, thereby limiting mixing between the rising plume and the ambient water. In addition, as shown in the table above, brine and effluent quality changes with season, particularly effluent temperature. Thus, two seasonal scenarios were examined (Table 2). They include wintertime when ambient coastal waters are isothermal, and summer/fall conditions when the waters are thermally stratified. Winter conditions are based on the averages of three profiles collected on February 23-24, 1977. Summer/fall conditions are the average of two profiles collected on September 22, 1976. These profiles were measured in 120 feet of water roughly 1.2 miles off of Terrence Point, Santa Cruz. Note the summertime thermocline at a depth of around 10 m where the water temperature drops from 13.7 to 12.7 °C with decreasing depth.

Table 2. Winter and Summer/Fall Ambient Water Quality

Depth, m	Winter Temperature, °C	Winter Salinity, ppt	Summer/Fall Temperature, °C	Summer/Fall Salinity, ppt
0	12.48	33.76	14.87	33.69
4	12.47	33.77	14.13	33.70
8	12.44	33.78	13.70	33.73
12	12.38	33.78	12.70	33.76
16	12.31	33.78	12.52	33.78
20	12.18	33.78	12.32	33.78
24	11.98	33.79	12.16	33.81
28	11.79	33.77	12.07	33.80
32	11.56	33.79	11.94	33.80

Diffuser Hydraulics

We evaluated the flow distribution along the outfall using Brown and Caldwell's proprietary diffuser hydraulics program DIFF$$. Model inputs included diffuser characteristics, flow rate, density of the composite effluent, and density of the ambient fluid. The outfall consists of a 72-inch-diameter pipe with three major sections and an end gate structure (Table 3). Its average downward slope is 0.0076 feet per foot. The number and diameter of the ports were designed to maintain fairly constant discharge velocity along the length of the diffuser under a wide range of flow conditions. Over half the ports are now closed since the outfall is currently operated under design capacity. In this analysis we assumed no change in the current diffuser configuration of the outfall.

Table 3. Diffuser Characteristics

Diffuser section	Port diameter, inches	Number of ports	Ports currently open	Section length, feet
End gate	4.25	2	2	NA
Offshore	3.7	50	20	606
Middle	2.5	64	25	706
Nearshore	2.0	60	25	714

Dilution Calculation

We estimated the near-field or initial dilution factor (DF) using PLUMES, a computer interface which supports two United States Environmental Protection Agency dilution models for effluent discharge. The first of these models is UM, an integral model that solves the equations of conservation of mass and energy as a buoyant plume moves away from a diffuser. It can estimate dilution from a single port and was used to model dilution at the end gate. The second model, RSB, is an empirical model based on years of field and laboratory experiments. RSB is applicable only for multiple-port diffusers and was used to analyze the offshore, middle and nearshore sections of the diffuser. For a given flow rate into the outfall, each diffuser section was modeled individually using the flow rate in that diffuser section estimated from the hydraulic model. Dilution model inputs included flow rate, diffuser section characteristics, temperature and salinity of the composite effluent, and temperature and salinity of the receiving waters (ambient fluid).

Preliminary modeling showed that the lowest DF was consistently observed at the end gate. However, since this section accounts for only 5 to 6 percent of the total effluent flow, and since UM tends to predict lower DF values than the more conservative RSB model, we used the diffuser section with the next lowest DF as the base-line dilution value. This was observed in the offshore section of the outfall. This diffuser section discharges roughly 50 percent of the total effluent flow. Thus, this section represents the largest relative flow out of the outfall with a minimal dilution. All DF values discussed below are based on dilution estimates from the offshore section of the diffuser.

Note that the City's current minimum dilution requirements are based on a flow-weighted DF encompassing all four diffuser sections. However, this report focuses only on the minimum DF from the offshore section of the diffuser. We used this approach because the Central Coast Regional Water Quality Control Board has indicated to the City that future decisions regarding the outfall will be based on the poorest performing section of the diffuser.

DILUTION MODELING RESULTS

Factors Controlling Dilution Factor

The addition of brine to effluent had a substantial impact on the DF achieved at the outfall (Figure 1). With no brine discharge, DFs ranged from around 300:1 to 700:1. However, as the relative amount of brine to effluent increases (moving to the left on the curves in Figure 1), DF drops dramatically from 200:1-300:1 under dilute conditions to less than 100:1 at ratios of around one part brine to one part effluent. The extremely high salinity of the brine is a major cause of the observed drop in dilution with increasing brine to effluent ratio. As this ratio increases, the composite effluent becomes less buoyant. Since the majority of the dilution process occurs during the rise of a buoyant plume, this loss of buoyancy translates directly into a decrease in dilution.

The relatively high summer/fall temperature of the effluent (23^{o}C) plays a minor role in elevating the buoyancy of the composite effluent, and thereby enhancing dilution at low brine to effluent ratios. For example, at low ratios summer/fall DFs range from 60:1-100:1 while winter DFs range from 10:1-20:1. The level of density stratification in the receiving waters also affects the magnitude of dilution. Stratification tends to limit dilution since the upward momentum of a buoyant plume is inhibited by the density gradient in the ambient water column. The lower DF values under summer versus winter conditions at high effluent flows are a result of the buoyant rise and subsequent dilution of the plume being impeded by summertime density stratification. Results of summer/fall and winter dilution modeling are summarized in Appendix A-1 and A-2.

Acceptable Brine Flows

Based on substantial hydraulic and dilution modeling, we developed a relationship between effluent flow and the maximum brine flow that can be added to effluent while still meeting a DF of 114:1 (Figure 2). The target of 114:1 is the minimum DF, flow-weighted for all four diffuser sections, allowed under the City's current NPDES Permit (CA 0048194). The relationship was developed for both winter (cool effluent and no thermal stratification in receiving waters) and summer conditions (warmer effluent and thermal stratification in receiving waters). The best-fit second-order quadratic curves of maximum brine flow (Q_b) as a function of effluent flow (Q_e) are:

$$\text{Summer/Fall } Q_b = -0.0207Q_e^2 + 1.114Q_e + 0.125 \quad (R^2 = 0.999)$$

$$\text{Winter } Q_b = -0.0128Q_e^2 + 0.957Q_e + 0.181 \quad (R^2 = 0.999)$$

The magnitude of maximum brine flow is roughly equivalent to the effluent flow. Slightly higher brine flows are permitted in summer versus winter because higher dilution is achieved in the summer. Effluent is substantially warmer in the summer (23^{o}C) versus the winter (18^{o}C), and this warm, more buoyant water enhances dilution upon discharge even though the ambient water is density stratified and likely inhibits dilution to some extent. See Appendix A-3 for a summary of dilution calculations.

Brine Flow Equalization Requirements

Flow equalization, the controlled decrease in the rate of brine discharge, will be required when the actual brine flow exceeds the maximum brine flow permitted to meet a DF of 114:1. This condition is likely to occur during times of low effluent flow. Based on the relationships developed above, we estimated daily flow equalization requirements under typical summer/fall low-flow conditions (7.5 mgd average daily flow, 14.1 mgd peak flow) (Figure 3). With a 2-mgd desalination plant, flow equalization will be required for 3-4 hours during early morning low-flow conditions. With a 4-mgd plant, this duration increases to roughly 6-8 hours. With a 6-mgd plant flow equalization will be required throughout the evening and during afternoon low flows. However, as noted below for the 6-mgd scenario, there is not enough effluent flow to allow for the complete discharge of stored brine over a 24-hour period.

Since the amount of brine that can be disposed of is proportional to the effluent flow, the worse case scenario from a brine disposal perspective is when daily average effluent flows are lowest. Based on data from the plant for 2001 (Figure 4) the average daily flow on the two lowest days averaged 6.25 mgd with a peak flow of 14 mgd. Assuming a typical daily variation in effluent flow, we estimated the storage requirement to equalize brine flow to maintain a dilution factor of 114:1 (Appendix B-1). Results are tabulated in Table 4. Under this scenario, a 6-mgd plant would result in a brine flow too high to equalize and discharge over the same 24-our period. The largest desalination plant capacity that could still discharge brine over a 24-hour period is 5.7 mgd and would require an equalization basin of 1.2 million gallons. Minimum effluent flows at the treatment plant have been known to drop to around 1 mgd during drought conditions as a result of lower water usage due to conservation, rationing and lower rates of infiltration. Assuming a worst case scenario in which the minimum daily flow is 1 mgd, and the average and peak daily flows are 5.0 and 10.5 mgd, storage requirements would increase substantially (Appendix B-2).

Table 4. Brine Equalization Storage Requirements

Average daily effluent flow, mgd	Comment	Storage requirements, million gallons[1,2]			Maximum size of desal plant, mgd/required storage
		2-mgd desal plant	4-mgd desal plant	6-mgd desal plant	
5.0	Estimated drought conditions. See Appendix B-2.	0.2	1.0	NA	4.1/1.1
6.25	2001 low flow conditions. See Appendix B-1.	0.06	0.74	NA	5.7/1.2
8.1	Minimum required effluent flow for 6-mgd desal plant. See Appendix B-3.	0	0.52	1.4	NA

[1]Values are storage required to dispose of brine within a 24-hour period while maintaining dilution of 114:1.
[2]NA–not applicable, effluent flow to low to dilute brine within a 24-hour period.

To dispose of brine on a daily basis for the 6-mgd desalination option requires a minimum average daily effluent flow of approximately 8.1 mgd (Appendix B-3), and an equalization basin of 1.42 million gallons. In 2002, flow rates below this threshold occurred for only 23 days: 4 days in July, 7 days in August, 4 days in September, 5 days in November and 1 day in December. With large enough brine storage, these episodes of low brine flow capacity could be overcome. For example, for the 6-mgd desalination option an estimated 6 million gallons of storage would be needed to allow for adequate equalization during the six consecutive low-flow days that occurred from August 24 to 29. Conversely, rather than relaying on excessive brine storage capacity, the rate of desalination could be lowered during times of low effluent flow. However, this may be difficult to achieve since times of low effluent flow are likely to coincide with times of greatest freshwater demand (e.g., hot summer months).

ADDITIONAL CONSIDERATION

The City will need to address several additional potential issues if brine disposal through the effluent outfall goes forward.

Trace Metal Concentrations

The concentration of trace metals in brine can be estimated by multiplying ocean levels by the concentration factor of 1.82. This factor is based on an RO membrane rejection rate of 55 percent. As shown in Table 5, trace metal concentrations in brine are expected to be less than effluent trace metal concentrations. Thus, the addition of brine to effluent will result in lower concentrations of trace metals discharged to the ocean, and concentrations in the composite effluent will remain far below effluent limits.

Table 5. Trace Metal Concentrations in ug/L

Parameter	Effluent[1]	Brine[2]	Effluent limit[3]
Arsenic	< 100	5.5	578
Copper	< 60	3.7	117
Mercury	< 0.8	0.0009	4.543
Silver	< 40	0.3	62.26
Zinc	43.9	14.6	1388

[1]Effluent concentration based on highest reported value or, if never detected, on the highest reported detection limit for 2000 and 2001.
[2]Brine based on background seawater concentrations cited in the California State Water Resources Control Board's California Ocean Plan (1997).
[3]Reported effluent limit is 6-month median reported in City's NPDES permit.

Brine Addition to Effluent

We identified two locations where brine can be added to effluent. They include the tunnel portal box located roughly 40 feet southeast of the WWTF administration building and the tunnel gate box located near the beach just south of West Cliff Drive. In both cases, the brine must be added in a way that promotes complete mixing between the brine and the effluent to avoid two-phase, stratified flow. Two-phase flow is the phenomena in which a distinct layer of low-density freshwater flows on top of heavier saline water. If this was to occur, heavier brine could potentially fill the outfall and limit the flow out the diffusers as well as the achieved dilution. To ensure adequate mixing of the brine and effluent, the brine will need to be discharged into the portal or gate box through a system of jet diffusers that face into the oncoming effluent flow.

Note that if brine is added to the tunnel portal box, modification in the City's effluent monitoring may be required since they currently monitor effluent through a sampling port just upstream of the box. In addition, combining brine with effluent will expose a number of structures to high-salinity water. As a result the structures will be more susceptible to corrosion. Of particular concern are the 36-inch and 72-inch sluice gates in the outfall gate box.

Brown and Caldwell has appreciated the chance to work with you on this project. Please do not hesitate to contact us with any questions.

Very truly yours,

BROWN AND CALDWELL

Marc W. Beutel
Project Manager

William K. Faisst
Vice President

MB:jw

Enclosures

Appendix A-1. Summer/Fall Outfall Modeling

Parameter	INPUT
Ambient Temp (C)[1]	11.98
Ambient Density (g/ml)[1]	1.02567
Desal Intake Temp (C)[2]	12.8
Desal Intake Salinity (ppt)[2]	33.78
Effluent Temp (C)[3]	23
Effluent Salinity (ppt)[3]	0.5

Desal Plant[4]				Wastewater Effluent			Composite Effluent					
Prod. Water (mgd)	Brine Flow (mgd)	Brine Salinity (ppt)	Brine Temp (C)	Flow (mgd)	TDS (ppt)	Temp (C)	Flow[5,6] (mgd)	Salinity[5] (ppt)	Temp[5] (C)	Final density[7] (g/ml)	Density ratio[6,8]	Resulting Dilution Factor
0	0.00	0	0	4	0.5	23	4.00	0.50	23.00	0.99798	0.02700	608.2
0	0.00	0	0	6	0.5	23	6.00	0.50	23.00	0.99798	0.02700	549.1
0	0.00	0	0	10	0.5	23	10.00	0.50	23.00	0.99798	0.02700	376.0
0	0.00	0	0	15	0.5	23	15.00	0.50	23.00	0.99798	0.02700	297.9

Desal Plant[4]				Wastewater Effluent			Composite Effluent					
Prod. Water (mgd)	Brine Flow (mgd)	Brine Salinity (ppt)	Brine Temp (C)	Flow (mgd)	TDS (ppt)	Temp (C)	Flow[5,6] (mgd)	Salinity[5] (ppt)	Temp[5] (C)	Final density[7] (g/ml)	Density ratio[6,8]	Resulting Dilution Factor
2	2.44	61.42	12.8	2	0.5	23	4.44	34.01	17.39	1.02468	0.00096	94.1
2	2.44	61.42	12.8	3	0.5	23	5.44	27.85	18.42	1.01975	0.00577	216.9
2	2.44	61.42	12.8	4	0.5	23	6.44	23.61	19.13	1.01636	0.00907	268.6
2	2.44	61.42	12.8	6	0.5	23	8.44	18.14	20.05	1.01200	0.01333	292.8
2	2.44	61.42	12.8	10	0.5	23	12.44	12.47	21.00	1.00749	0.01772	265.5
2	2.44	61.42	12.8	15	0.5	23	17.44	9.04	21.57	1.00476	0.02038	237.7

Desal Plant[4]				Wastewater Effluent			Composite Effluent					
Prod. Water (mgd)	Brine Flow (mgd)	Brine Salinity (ppt)	Brine Temp (C)	Flow (mgd)	TDS (ppt)	Temp (C)	Flow[5,6] (mgd)	Salinity[5] (ppt)	Temp[5] (C)	Final density[7] (g/ml)	Density ratio[6,8]	Resulting Dilution Factor
4	4.89	61.42	12.8	4	0.5	23	8.89	34.01	17.39	1.02468	0.00096	69.7
4	4.89	61.42	12.8	5	0.5	23	9.89	30.62	17.96	1.02195	0.00363	129.6
4	4.89	61.42	12.8	6	0.5	23	10.89	27.85	18.42	1.01975	0.00577	162.7
4	4.89	61.42	12.8	8	0.5	23	12.89	23.61	19.13	1.01636	0.00907	192.1
4	4.89	61.42	12.8	10	0.5	23	14.89	20.51	19.65	1.01389	0.01149	195.8
4	4.89	61.42	12.8	15	0.5	23	19.89	15.48	20.49	1.00988	0.01539	194.7

Desal Plant[4]				Wastewater Effluent			Composite Effluent					
Prod. Water (mgd)	Brine Flow (mgd)	Brine Salinity (ppt)	Brine Temp (C)	WWTP Flow (mgd)	WWTP TDS (ppt)	WWTP Temp (C)	Flow[5,6] (mgd)	Salinity[5] (ppt)	Temp[5] (C)	Final density[7] (g/ml)	Density ratio[6,8]	Resulting Dilution Factor
6	7.33	61.42	12.8	6	0.5	23	13.33	34.01	17.39	1.02468	0.00096	57.3
6	7.33	61.42	12.8	8	0.5	23	15.33	29.64	18.12	1.02118	0.00438	125.9
6	7.33	61.42	12.8	10	0.5	23	17.33	26.28	18.68	1.01848	0.00701	155.6
6	7.33	61.42	12.8	15	0.5	23	22.33	20.51	19.65	1.01389	0.01149	167.9

[1]September 1976, 30 m depth, 120' deep station (Brown and Caldwell, 1978).
[2]September 1976, 12 m depth, 60' and 45' deep station (Brown and Caldwell, 1978).
[3]October 2001 wastewater effluent temperature (Dave Sasser, personal comm.), assumed salinity of 0.5 ppt.
[4]Brine flow and salinity assumes 5% of ocean inflow lost to pretreatment and 55% rejected at RO membranes (Brian Jordan, personal comm.).
[5]Key inputs into PLUMES model to determine dilution factor.
[6]Key inputs to diffuser hydraulics model.
[7]Estimated based on effluent temp and salinity using PLUMES model.
[8]Density ratio equals (ambient density-effluent density)/ambient density.

Appendix A-2. Winter Outfall Modeling

Parameter	INPUT
Ambient Temp (C)[1]	11.68
Ambient Density (g/ml)[1]	1.02572
Desal Intake Temp (C)[2]	12.32
Desal Intake Salinity (ppt)[2]	33.76
Effluent Temp (C)[3]	18
Effluent Salinity (ppt)[3]	0.5

Desal Plant[4]				Wastewater Effluent			Composite Effluent					
Prod. Water (mgd)	Brine Flow (mgd)	Brine Salinity (ppt)	Brine Temp (C)	Flow (mgd)	TDS (ppt)	Temp (C)	Flow[5,6] (mgd)	Salinity[5] (ppt)	Temp[5] (C)	Final density[7] (g/ml)	Density ratio[6,8]	Resulting Dilution Factor
0	0.00	0	0	4	0.5	18	4.00	0.50	18.00	0.99904	0.02601	706.7
0	0.00	0	0	6	0.5	18	6.00	0.50	18.00	0.99904	0.02601	682.9
0	0.00	0	0	10	0.5	18	10.00	0.50	18.00	0.99904	0.02601	525.8
0	0.00	0	0	15	0.5	18	15.00	0.50	18.00	0.99904	0.02601	386.1

Desal Plant[4]				Wastewater Effluent			Composite Effluent					
Prod. Water (mgd)	Brine Flow (mgd)	Brine Salinity (ppt)	Brine Temp (C)	Flow (mgd)	TDS (ppt)	Temp (C)	Flow[5,6] (mgd)	Salinity[5] (ppt)	Temp[5] (C)	Final density[7] (g/ml)	Density ratio[6,8]	Resulting Dilution Factor
2	2.44	61.38	12.317	2	0.5	18	4.44	33.98	14.87	1.02524	0.00047	34.0
2	2.44	61.38	12.317	3	0.5	18	5.44	27.84	15.45	1.02040	0.00518	186.9
2	2.44	61.38	12.317	4	0.5	18	6.44	23.59	15.84	1.01707	0.00843	258.6
2	2.44	61.38	12.317	6	0.5	18	8.44	18.12	16.35	1.01280	0.01259	342.7
2	2.44	61.38	12.317	10	0.5	18	12.44	12.46	16.88	1.00838	0.01690	378.3
2	2.44	61.38	12.317	15	0.5	18	17.44	9.03	17.20	1.00571	0.01951	313.1

Desal Plant[4]				Wastewater Effluent			Composite Effluent					
Prod. Water (mgd)	Brine Flow (mgd)	Brine Salinity (ppt)	Brine Temp (C)	Flow (mgd)	TDS (ppt)	Temp (C)	Flow[5,6] (mgd)	Salinity[5] (ppt)	Temp[5] (C)	Final density[7] (g/ml)	Density ratio[6,8]	Resulting Dilution Factor
4	4.89	61.38	12.317	4	0.5	18	8.89	33.98	14.87	1.02524	0.00047	29.2
4	4.89	61.38	12.317	5	0.5	18	9.89	30.60	15.19	1.02257	0.00307	103.2
4	4.89	61.38	12.317	6	0.5	18	10.89	27.84	15.45	1.02040	0.00518	149.1
4	4.89	61.38	12.317	8	0.5	18	12.89	23.59	15.84	1.01707	0.00843	226.2
4	4.89	61.38	12.317	10	0.5	18	14.89	20.49	16.13	1.01465	0.01079	265.6
4	4.89	61.38	12.317	15	0.5	18	19.89	15.47	16.60	1.01073	0.01461	258.9

Desal Plant[4]				Wastewater Effluent			Composite Effluent					
Prod. Water (mgd)	Brine Flow (mgd)	Brine Salinity (ppt)	Brine Temp (C)	WWTP Flow (mgd)	WWTP TDS (ppt)	WWTP Temp (C)	Flow[5,6] (mgd)	Salinity[5] (ppt)	Temp[5] (C)	Final density[7] (g/ml)	Density ratio[6,8]	Resulting Dilution Factor
6	7.33	61.38	12.317	6	0.5	18	13.33	33.98	14.87	1.02524	0.00047	26.6
6	7.33	61.38	12.317	8	0.5	18	15.33	29.62	15.28	1.02180	0.00382	101.4
6	7.33	61.38	12.317	10	0.5	18	17.33	26.26	15.60	1.01916	0.00639	159.9
6	7.33	61.38	12.317	15	0.5	18	22.33	20.49	16.13	1.01465	0.01079	226.7

[1]February 1977, 30 m depth, 120' deep station (Brown and Caldwell, 1978).
[2]February 1977, 12 m depth, 60' and 45' deep station (Brown and Caldwell, 1978).
[3]February 2001 wastewater effluent temperature (Dave Sasser, personal comm.), assumed salinity of 0.5 ppt.
[4]Brine flow and salinity assumes 5% of ocean inflow lost to pretreatment and 55% rejected at RO membranes (Brian Jordan, personal comm.).
[5]Key inputs into PLUMES model to determine dilution factor.
[6]Key inputs to diffuser hydraulics model.
[7]Estimated based on effluent temp and salinity using PLUMES model.
[8]Density ratio equals (ambient density-effluent density)/ambient density.

Appendix A-3. Relative Flow Rates Required to Maintain DF of 114

Summer/Fall Outfall Modeling

Parameter	INPUT
Ambient Temp (C)[1]	11.98
Ambient Density (g/ml)[1]	1.02567
Desal Intake Temp (C)[2]	12.8
Desal Intake Salinity (ppt)[2]	33.78
Effluent Temp (C)[3]	23
Effluent Salinity (ppt)[3]	0.5

Diffuser		Brine	Desal Plant[4]			Wastewater Effluent			Composite Effluent					
Section B Flow (mgd)	Total Flow (mgd)	Flow: WWTP Flow	Brine Flow (mgd)	Brine Salinity (ppt)	Brine Temp (C)	Flow (mgd)	TDS (ppt)	Temp (C)	Flow[5,6] (mgd)	Salinity[5] (ppt)	Temp[5] (C)	Final density[7] (g/ml)	Density ratio[6,8]	Resulting Dilution Factor
0.50	1.25	1.250	0.69	61.42	12.8	0.56	0.5	23	1.25	34.34	17.33	1.02495	0.00070	117.2
1.00	2.25	1.230	1.24	61.42	12.8	1.01	0.5	23	2.25	34.10	17.37	1.02476	0.00089	115.3
2.00	4.25	1.155	2.28	61.42	12.8	1.97	0.5	23	4.25	33.15	17.53	1.02399	0.00163	116.9
4.00	8.40	1.065	4.33	61.42	12.8	4.07	0.5	23	8.40	31.92	17.74	1.02300	0.00260	115.5
6.00	12.50	1.000	6.25	61.42	12.8	6.25	0.5	23	12.50	30.96	17.90	1.02223	0.00335	115.7
8.00	16.65	0.950	8.11	61.42	12.8	8.54	0.5	23	16.65	30.18	18.03	1.02161	0.00396	115.4
10.00	20.80	0.900	9.85	61.42	12.8	10.95	0.5	23	20.80	29.36	18.17	1.02095	0.00460	116.3

See Appendix A-1 for notes.

Winter Outfall Modeling

Parameter	INPUT
Ambient Temp (C)[1]	11.68
Ambient Density (g/ml)[1]	1.02572
Desal Intake Temp (C)[2]	12.32
Desal Intake Salinity (ppt)[2]	33.76
Effluent Temp (C)[3]	18
Effluent Salinity (ppt)[3]	0.5

Diffuser		Brine	Desal Plant[4]			Wastewater Effluent			Composite Effluent					
Section B Flow (mgd)	Total Flow (mgd)	Flow: WWTP Flow	Brine Flow (mgd)	Brine Salinity (ppt)	Brine Temp (C)	Flow (mgd)	TDS (ppt)	Temp (C)	Flow[5,6] (mgd)	Salinity[5] (ppt)	Temp[5] (C)	Final density[7] (g/ml)	Density ratio[6,8]	Resulting Dilution Factor
0.50	1.40	1.138	0.75	61.38	12.32	0.65	0.5	18	1.40	32.91	14.98	1.02439	0.00125	114.6
1.00	2.30	1.108	1.21	61.38	12.32	1.09	0.5	18	2.30	32.50	15.01	1.02407	0.00156	114.5
2.00	4.35	1.045	2.22	61.38	12.32	2.13	0.5	18	4.35	31.61	15.10	1.02337	0.00225	114.6
4.00	8.45	0.953	4.12	61.38	12.32	4.33	0.5	18	8.45	30.21	15.23	1.02226	0.00332	114.7
6.00	12.55	0.895	5.93	61.38	12.32	6.62	0.5	18	12.55	29.25	15.32	1.02151	0.00406	114.4
8.00	16.70	0.855	7.70	61.38	12.32	9.00	0.5	18	16.70	28.56	15.38	1.02097	0.00458	114.7
10.00	20.85	0.830	9.46	61.38	12.32	11.39	0.5	18	20.85	28.11	15.42	1.02062	0.00493	114.0

See Appendix A-2 for notes.

Appendix B-1. Estimated Equalization Basin Storage Requirements for 2001 Low Flow Conditions

			2 mgd desal Plant			4 mgd desal Plant			6 mgd desal Plant			5.7 mgd desal Plant		
Time	Effluent flow (mgd)	Max. brine flow (mgd)	Brine flow (mgd)	Excess capacity (mgd)	Cumulative required storage (gal)	Brine flow (mgd)	Excess capacity (mgd)	Cumulative required storage (gal)	Brine flow (mgd)	Excess capacity (mgd)	Cumulative required storage (gal)	Brine flow (mgd)	Excess capacity (mgd)	Cumulative required storage (gal)
10:00 PM	6.59	6.57							7.33	-0.76	31,862			
11:00 PM	4.39	4.62				4.89	-0.27	11,319	7.33	-2.71	144,847	6.00	-1.38	57,569
12:00 AM	4.39	4.62	2.44	2.18		4.89	-0.27	22,637	7.33	-2.71	257,832	6.00	-1.38	115,137
1:00 AM	2.93	3.21	2.44	0.77		4.89	-1.68	92,665	7.33	-4.12	429,527	6.00	-2.79	231,415
2:00 AM	2.20	2.47	2.44	0.03		4.89	-2.42	193,435	7.33	-4.86	631,964	6.00	-3.53	378,435
3:00 AM	1.90	2.17	2.44	-0.27	11,243	4.89	-2.72	306,761	7.33	-5.16	846,956	6.00	-3.83	538,011
4:00 AM	1.76	2.02	2.44	-0.42	28,819	4.89	-2.87	426,421	7.33	-5.31	1,068,283	6.00	-3.98	703,921
5:00 AM	1.76	2.02	2.44	-0.42	46,396	4.89	-2.87	546,081	7.33	-5.31	1,289,610	6.00	-3.98	869,831
6:00 AM	1.90	2.17	2.44	-0.27	**57,639**	4.89	-2.72	659,407	7.33	-5.16	1,504,602	6.00	-3.83	1,029,407
7:00 AM	2.93	3.21	2.44	0.77	25,583	4.89	-1.68	729,435	7.33	-4.12	1,676,297	6.00	-2.79	1,145,685
8:00 AM	4.39	4.62	2.44	2.18	*-65,182*	4.89	-0.27	**740,753**	7.33	-2.71	1,789,282	6.00	-1.38	**1,203,253**
9:00 AM	6.88	6.81	2.44	4.37		4.89	1.92	660,761	7.33	-0.52	1,810,957	6.00	0.81	1,169,511
10:00 AM	8.79	8.31	2.44	5.87		4.89	3.42	518,162	7.33	0.98	1,770,024	6.00	2.31	1,073,162
11:00 AM	11.72	10.33	2.44	7.89		4.89	5.44	291,457	7.33	3.00	1,644,985	6.00	4.33	892,707
12:00 PM	13.92	11.61	2.44	9.17		4.89	6.72	11,390	7.33	4.28	1,466,585	6.00	5.61	658,890
1:00 PM	13.92	11.61	2.44	9.17		4.89	6.72	*-268,678*	7.33	4.28	1,288,184	6.00	5.61	425,072
2:00 PM	10.99	9.86	2.44	7.42		4.89	4.97		7.33	2.53	1,182,784	6.00	3.86	264,256
3:00 PM	7.32	7.17	2.44	4.73		4.89	2.28		7.33	-0.16	1,189,456	6.00	1.17	215,511
4:00 PM	5.86	5.94	2.44	3.50		4.89	1.05		7.33	-1.39	1,247,433	6.00	-0.06	218,072
5:00 PM	5.86	5.94	2.44	3.50		4.89	1.05		7.33	-1.39	1,305,411	6.00	-0.06	220,633
6:00 PM	6.59	6.57	2.44	4.13		4.89	1.68		7.33	-0.76	1,337,273	6.00	0.57	197,078
7:00 PM	7.32	7.17	2.44	4.73		4.89	2.28		7.33	-0.16	1,343,945	6.00	1.17	148,333
8:00 PM	8.06	7.75	2.44	5.31		4.89	2.86		7.33	0.42	1,326,351	6.00	1.75	75,323
9:00 PM	7.62	7.41	2.44	4.97		4.89	2.52		7.33	0.08	1,323,206	6.00	1.41	16,760
10:00 PM	6.59	6.57	2.44	4.13		4.89	1.68		7.33	-0.76	1,355,068	6.00	0.57	*-6,794*
11:00 PM	4.39	4.62	2.44	2.18		4.89	-0.27	11,319	7.33	-2.71	1,468,053	6.00	-1.38	57,569
Average	6.25	6.01												
Peak	13.92													
Storage Required					**57,639**			**740,753**			**NA**			**1,203,253**

Notes:
Wastewater effluent flow based on average of the two minimum flows observed in 2001, average daily flow was 6.25 mgd and maximum daily flow was 14 mgd.
Daily variation in wastewater effluent flow estimated from "typical" patterns reported in Metcalf and Eddy (Wastewater Engineering).
Brine equalization assumed to begin on the previous night for the 4-mgd, 6-mgd and 5.7-mgd plant scenarios.
Shaded areas show times of storage accumulation.
Negative cumulative storage (values in italics) indicate that equalization basin is empty.
NA - not applicable, not enough wastewater flow to discharge all stored brine.

Appendix B-2. Estimated Equalization Basin Storage Requirements for Drought Conditions

			2 mgd desal Plant			4 mgd desal Plant			6 mgd desal Plant			4.1 mgd desal Plant		
Time	Effluent flow (mgd)	Max. brine flow (mgd)	Brine flow (mgd)	Excess capacity (mgd)	Cumulative required storage (gal)	Brine flow (mgd)	Excess capacity (mgd)	Cumulative required storage (gal)	Brine flow (mgd)	Excess capacity (mgd)	Cumulative required storage (gal)	Brine flow (mgd)	Excess capacity (mgd)	Cumulative required storage (gal)
10:00 PM	5.58	5.69							7.33	-1.64	68,308			
11:00 PM	3.72	3.98				4.89	-0.91	37,984	7.33	-3.35	207,959	5.00	-1.02	42,567
12:00 AM	3.72	3.98	2.44	1.54		4.89	-0.91	75,968	7.33	-3.35	347,610	5.00	-1.02	85,135
1:00 AM	2.48	2.76	2.44	0.32		4.89	-2.13	164,825	7.33	-4.57	538,133	5.00	-2.24	178,575
2:00 AM	1.65	1.91	2.44	-0.53	22,161	4.89	-2.98	289,069	7.33	-5.42	764,044	5.00	-3.09	307,402
3:00 AM	1.24	1.47	2.44	-0.97	62,456	4.89	-3.42	431,447	7.33	-5.86	1,008,089	5.00	-3.53	454,364
4:00 AM	0.99	1.21	2.44	-1.23	113,774	4.89	-3.68	584,849	7.33	-6.12	1,263,157	5.00	-3.79	612,349
5:00 AM	0.99	1.21	2.44	-1.23	165,092	4.89	-3.68	738,250	7.33	-6.12	1,518,225	5.00	-3.79	770,333
6:00 AM	1.24	1.47	2.44	-0.97	**205,387**	4.89	-3.42	880,629	7.33	-5.86	1,762,270	5.00	-3.53	917,295
7:00 AM	2.48	2.76	2.44	0.32	192,161	4.89	-2.13	969,485	7.33	-4.57	1,952,794	5.00	-2.24	1,010,735
8:00 AM	3.72	3.98	2.44	1.54	128,061	4.89	-0.91	**1,007,469**	7.33	-3.35	2,092,445	5.00	-1.02	**1,053,303**
9:00 AM	5.82	5.91	2.44	3.47	*-16,442*	4.89	1.02	965,049	7.33	-1.42	2,151,691	5.00	0.91	1,015,466
10:00 AM	7.44	7.26	2.44	4.82		4.89	2.37	866,309	7.33	-0.07	2,154,618	5.00	2.26	921,309
11:00 AM	9.91	9.13	2.44	6.69		4.89	4.24	689,666	7.33	1.80	2,079,641	5.00	4.13	749,249
12:00 PM	10.53	9.56	2.44	7.12		4.89	4.67	495,202	7.33	2.23	1,986,843	5.00	4.56	559,368
1:00 PM	9.91	9.13	2.44	6.69		4.89	4.24	318,558	7.33	1.80	1,911,866	5.00	4.13	387,308
2:00 PM	7.44	7.26	2.44	4.82		4.89	2.37	219,818	7.33	-0.07	1,914,793	5.00	2.26	293,151
3:00 PM	6.20	6.23	2.44	3.79		4.89	1.34	164,003	7.33	-1.10	1,960,645	5.00	1.23	241,920
4:00 PM	4.96	5.14	2.44	2.70		4.89	0.25	153,764	7.33	-2.19	2,052,072	5.00	0.14	236,264
5:00 PM	4.96	5.14	2.44	2.70		4.89	0.25	143,524	7.33	-2.19	2,143,499	5.00	0.14	230,607
6:00 PM	5.58	5.69	2.44	3.25		4.89	0.80	110,166	7.33	-1.64	2,211,807	5.00	0.69	201,832
7:00 PM	6.20	6.23	2.44	3.79		4.89	1.34	54,351	7.33	-1.10	2,257,659	5.00	1.23	150,601
8:00 PM	6.82	6.75	2.44	4.31		4.89	1.86	*-23,257*	7.33	-0.58	2,281,718	5.00	1.75	77,576
9:00 PM	6.44	6.44	2.44	4.00		4.89	1.55		7.33	-0.89	2,318,773	5.00	1.44	17,548
10:00 PM	5.58	5.69	2.44	3.25		4.89	0.80		7.33	-1.64	2,387,081	5.00	0.69	*-11,227*
11:00 PM	3.72	3.98	2.44	1.54		4.89	-0.91	37,984	7.33	-3.35	2,526,732	5.00	-1.02	42,567
Average	5.00	5.01												
Peak	10.53													
Storage Required					**205,387**			**1,007,469**			**NA**			**1,053,303**

Notes:
Wastewater effluent flow based on drought scenario with average, minimum and maximum daily flows of 5.0, 1.0 and 10.5 mgd.
Daily variation in wastewater effluent flow estimated from "typical" patterns reported in Metcalf and Eddy (Wastewater Engineering).
Brine equalization assumed to begin on the previous night for the 4-mgd, 6-mgd and 4.1-mgd plant scenarios.
Shaded areas show times of storage accumulation.
Negative cumulative storage (values in italics) indicate that equalization basin is empty.
NA - not applicable, not enough wastewater flow to discharge all stored brine.

Appendix B-3. Estimated Equalization Basin Storage Requirements at Average Daily Flow of 8.1 mgd

			2 mgd desal Plant			4 mgd desal Plant			6 mgd desal Plant		
Time	Effluent flow (mgd)	Max. brine flow (mgd)	Brine flow (mgd)	Excess capacity (mgd)	Cumulative required storage (gal)	Brine flow (mgd)	Excess capacity (mgd)	Cumulative required storage (gal)	Brine flow (mgd)	Excess capacity (mgd)	Cumulative required storage (gal)
11:00 PM	5.70	5.80							7.33	-1.53	63,954
12:00 AM	5.70	5.80	2.44	3.36		4.89	0.91		7.33	-1.53	127,909
1:00 AM	3.80	4.05	2.44	1.61		4.89	-0.84	34,825	7.33	-3.28	264,400
2:00 AM	2.85	3.13	2.44	0.69		4.89	-1.76	108,250	7.33	-4.20	439,492
3:00 AM	2.47	2.75	2.44	0.31		4.89	-2.14	197,550	7.33	-4.58	630,458
4:00 AM	2.28	2.55	2.44	0.11		4.89	-2.34	294,880	7.33	-4.78	829,456
5:00 AM	2.28	2.55	2.44	0.11		4.89	-2.34	392,211	7.33	-4.78	1,028,453
6:00 AM	2.47	2.75	2.44	0.31		4.89	-2.14	481,511	7.33	-4.58	1,219,420
7:00 AM	3.80	4.05	2.44	1.61		4.89	-0.84	**516,336**	7.33	-3.28	1,355,911
8:00 AM	5.70	5.80	2.44	3.36		4.89	0.91	478,624	7.33	-1.53	**1,419,866**
9:00 AM	8.92	8.41	2.44	5.97		4.89	3.52	331,866	7.33	1.08	1,374,775
10:00 AM	11.39	10.12	2.44	7.68		4.89	5.23	113,845	7.33	2.79	1,258,421
11:00 AM	15.19	12.26	2.44	9.82		4.89	7.37	*-193,297*	7.33	4.93	1,052,945
12:00 PM	18.04	13.47	2.44	11.03		4.89	8.58		7.33	6.14	796,948
1:00 PM	18.04	13.47	2.44	11.03		4.89	8.58		7.33	6.14	540,951
2:00 PM	14.24	11.78	2.44	9.34		4.89	6.89		7.33	4.45	355,425
3:00 PM	9.49	8.83	2.44	6.39		4.89	3.94		7.33	1.50	292,956
4:00 PM	7.59	7.39	2.44	4.95		4.89	2.50		7.33	0.06	290,590
5:00 PM	7.59	7.39	2.44	4.95		4.89	2.50		7.33	0.06	288,225
6:00 PM	8.54	8.13	2.44	5.69		4.89	3.24		7.33	0.80	255,031
7:00 PM	9.49	8.83	2.44	6.39		4.89	3.94		7.33	1.50	192,562
8:00 PM	10.44	9.49	2.44	7.05		4.89	4.60		7.33	2.16	102,373
9:00 PM	9.87	9.10	2.44	6.66		4.89	4.21		7.33	1.77	28,630
10:00 PM	8.54	8.13	2.44	5.69		4.89	3.24		7.33	0.80	*-4,564*
11:00 PM	5.70	5.80	2.44	3.36		4.89	0.91		7.33	-1.53	63,954
Average	8.10	7.33									
Peak	18.04										
Storage Required					**0**			**516,336**			**1,419,866**

Notes:
Wastewater effluent flow based on average daily flow of 8.1 mgd, the minimum wastewater flow required for the 6-mgd desalination scenario.
Daily variation in wastewater effluent flow estimated from "typical" patterns reported in Metcalf and Eddy (Wastewater Engineering).
Brine equalization assumed to begin on the previous night for the 6-mgd plant scenario.
Shaded areas show times of storage accumulation.
Negative cumulative storage (values in italics) indicate that equalization basin is empty.

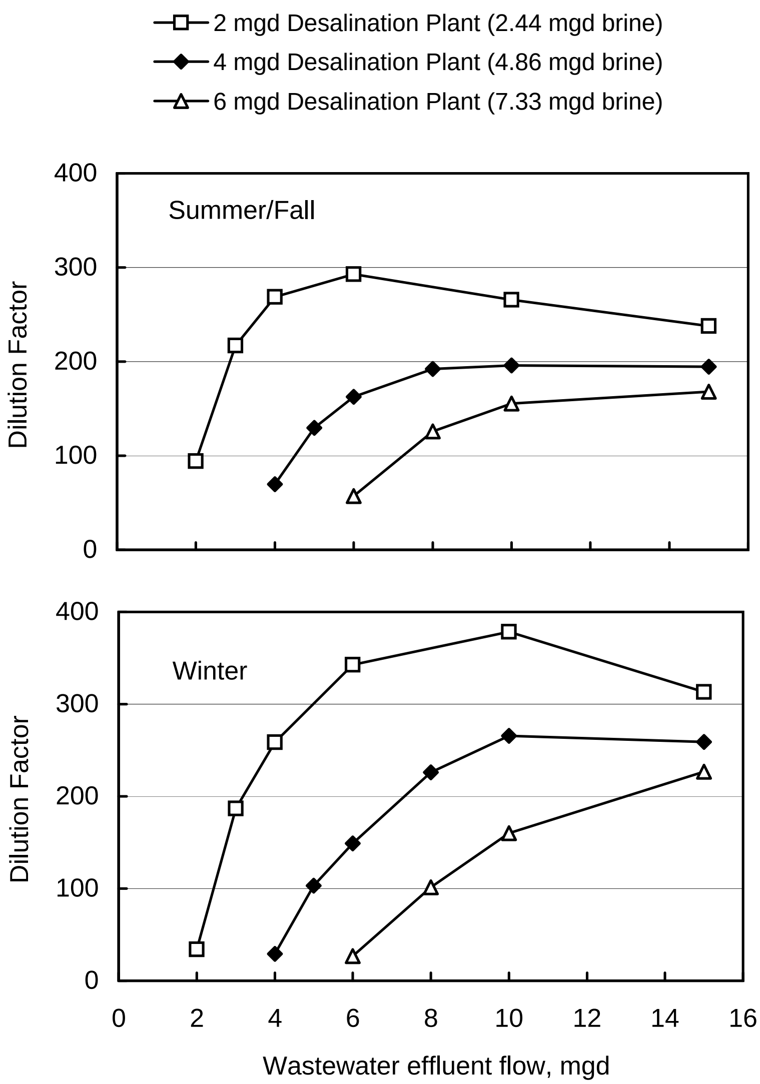


Figure 1 - Dilution factors under three desalination alternative as a function of increasing wastewater effluent flow rates.

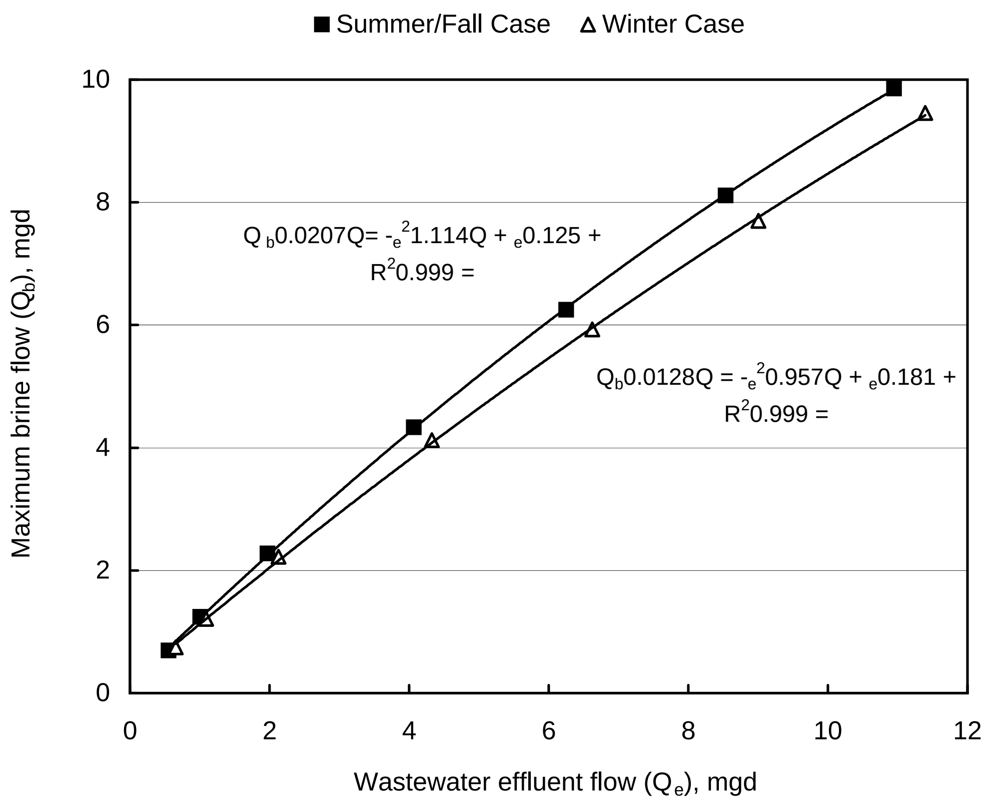


Figure 2 - Maximum brine flow as a function of wastewater effluent flow to maintain a dilution factor of 114. Line through points shows best-fit second order quadratic.

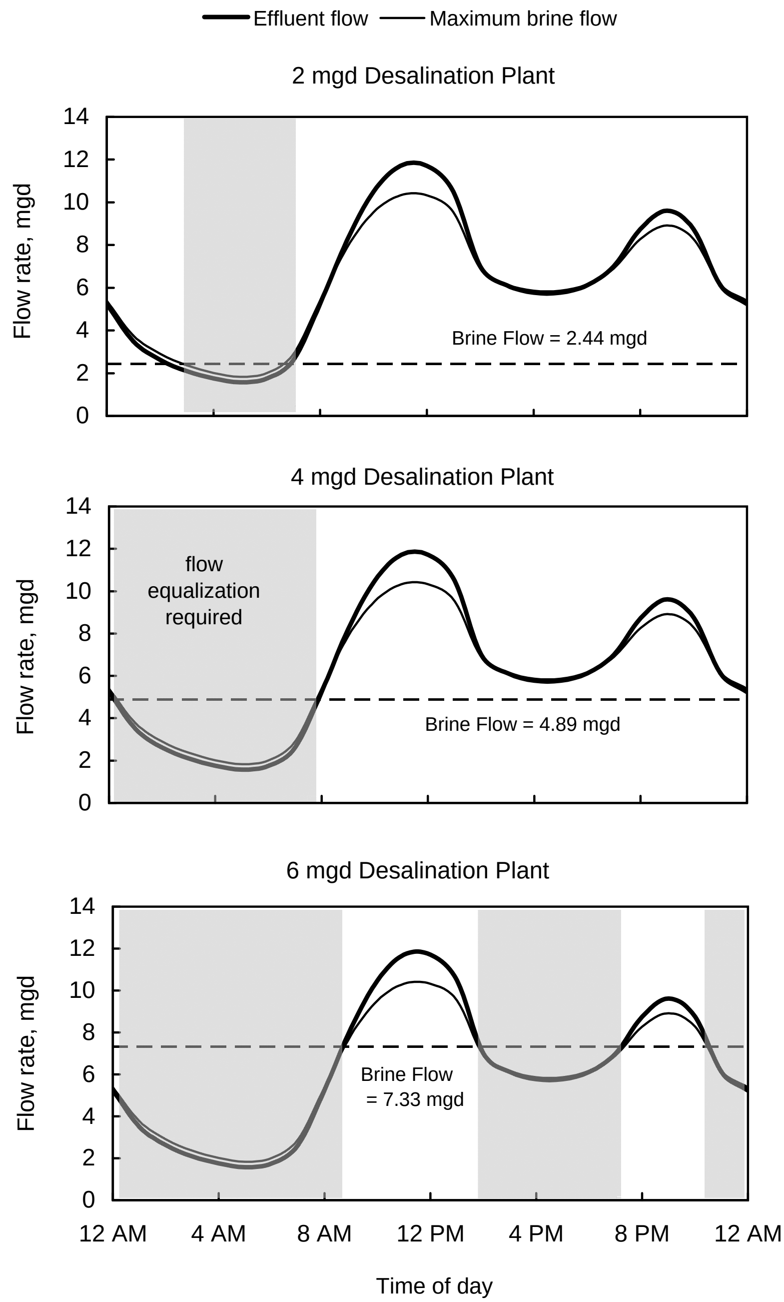


Figure 3 - Summer/Fall equalization requirements under different desalination scenarios. Heavy line shows hypothetical effluent flow regime based on typical low flow conditions in September (7.5 mgd average, 14.1 mgd maximum). Light line shows maximum acceptable brine flow to maintain dilution factor of 114. When this brine flow falls below the actual brine flow, flow equalization will be required. Shaded area shows estimated duration of flow equalization.

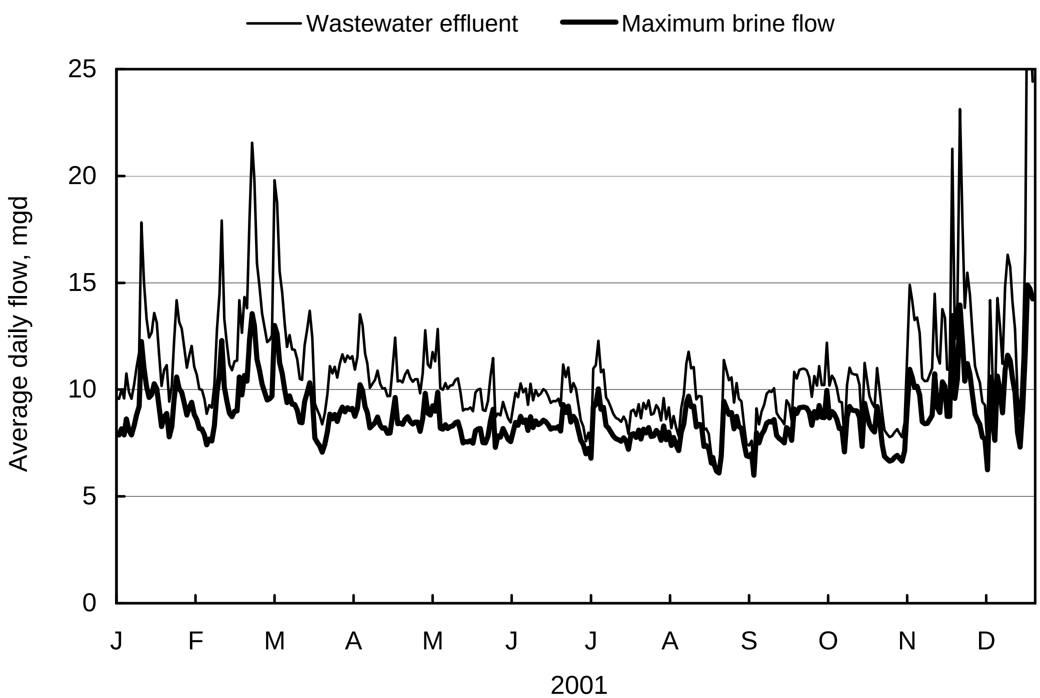


Figure 4 - Average daily flows of wastewater effluent and maximum brine permitted to still maintain dilution factor of 114. Note minimum flows in mid August and early September.