Vista Grande Drainage Basin
Alternatives Analysis Report Project

Executive Summary (Draft)

Prepared by:

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Engineers/Consultants

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Table of Contents

1 Introduction

This report summarizes the alternatives evaluation that Jacobs Associates, Brown and Caldwell, and ESA (the Evaluation Team) provided for managing storm water and reducing flooding in the Vista Grande Drainage Basin in Daly City, California. It was prepared as part of the Vista Grande Drainage Basin Alternatives Analysis Project, commissioned by the City of Daly City (the City) to evaluate improvements to the Vista Grande Watershed storm water outfall system.

Storm-related flooding has recurred throughout the Vista Grande Watershed Drainage Basin, especially along the canal across John Muir Drive north of Lake Merced Boulevard. In this area, the storm water drainage system collects flows from a 2.5-square-mile basin in the City and conveys them via several underground culverts to the Vista Grande Canal, located adjacent to John Muir Drive in San Francisco. From there, the water flows to the Vista Grande Tunnel and Outfall Structure, through which it is discharged into the Pacific Ocean below Fort Funston, located in the Golden Gate National Recreation Area (GGNRA).

Sporadically, rainstorms produce storm runoff that exceeds the hydraulic capacities of the tunnel, estimated at 170 cubic feet per second (cfs), and the canal, estimated at 500 cfs. When storm water inflows exceed the tunnel's capacity, the water backs up into the canal and occasionally causes upstream flooding and overtopping of John Muir Drive in San Francisco. Excess water may flow from the canal across John Muir Drive into Lake Merced or into other areas at lower elevations. The resulting flooding adversely impacts the community and public resources.

The City commissioned the Vista Grande Drainage Basin Alternatives Analysis to develop feasible alternatives to:

- Manage storm water flows generated by the design storm event to improve public safety, flood protection, minimize property damage, and minimize public inconvenience.

- Encourage the diversion and reuse of storm water to reduce uncontrolled overflows into Lake Merced, improve storm water quality, and provide beneficial storm water uses to the community.

This executive summary reprises the alternative selection process and recaps the "top four" alternatives: three (5B, 6B, and 7) previously chosen by the City from an original selection of 17 alternatives as well as re-examining a fourth, the Lake Merced Alternative. This summary also outlines the next steps in the selection process.

2 Alternative Selection Process

For planning purposes, the City selected a design storm event with a four-hour duration and a 25-year recurrence interval.[1] As a part of the original evaluation process the Evaluation Team investigated seven initial alternative tunnel alignments from which the City selected five alternative tunnel alignments for further investigation. In addition, the City identified two non-tunnel alternatives—storm water storage basins and groundwater recharge—which could complement any of the tunnel alternatives or operate independently. The original alternatives reflect a range of potential solutions for addressing local flooding in the Vista Grande drainage basin. The details of the various alternatives are provided in Volume 1. The initial alternatives are summarized below:

- Alternatives 1A and 1B considered possible drainage tunnel alignments running from the beginning of the Vista Grande Canal, beneath the Olympic Club, to either a new outfall structure near Fort Funston (1A) or the existing outfall structure (1B).

- Alternative 2 considered a possible drainage tunnel alignment running from the north side of the Doelger Senior Center at Westlake Park, beneath the Olympic Club, to a new outfall structure near Thornton State Beach.

- Alternative 3 considered a possible drainage tunnel alignment running beneath John Daly Boulevard from the south side of Cliffside Drive, to a new outfall structure at Thornton State Beach.

- Alternative 4 considered a possible drainage tunnel alignment running from Westlake Park, beneath Northgate Avenue, to a new outfall structure near Thornton State Beach.

- Alternatives 5A and 5B considered possible drainage tunnel alignments running from a point approximately 800 feet downstream from the beginning of the Vista Grande Canal, beneath the Olympic Club, to either a new outfall structure near Fort Funston (5A) or the existing outfall structure (5B).

- Alternative 6A and 6B considered a possible drainage tunnel alignment running from a point approximately 2,100 feet from the beginning of the Vista Grande Canal, beneath the Olympic Club, to either a new outfall structure near Fort Funston (6A) or the existing outfall structure (6B).

- Alternatives 7A and 7B considered a possible drainage tunnel alignment running from a point approximately 3,500 feet from the beginning of the Vista Grande Canal, beneath the Olympic Club, to the existing outfall structure. Alternative 7A considered a large-diameter tunnel and Alternative 7B considered a small-diameter microtunnel sized to pass 330 cfs.

[1] RMC Water and Environment, Vista Grande Watershed Study, August 2006.

- Alternative 8, similar to Alternative 4, considered a possible microtunnel alignment running from Westlake Park, beneath a portion of Northgate Avenue and the Olympic Club, to a new outfall structure near Thornton State Beach.

- Alternative 9 considered the use of a storm water detention structure located beneath Westlake Park which, following the peak runoff flow, would pump temporary stored water back into the box culvert connected to the Vista Grande Canal. A storm water detention alternative can complement a tunnel alignment alternative to reduce discharges through an outfall structure.

- Alternative 10 considered the above alternatives in combination with a groundwater recharge feature.

The City compared these alternatives and selected Alternatives 1A, 4, 5B, 6B, 7, 9, and 10 for further investigation and development. The geologic assessment of a proposed outfall structure site at Thorton State Beach revealed a high potential for landslides and ground instability. Although constructable, it is likely a tunnel and outfall structure located at Thorton State Beach would be prone to long term ground control and maintenance costs. Accordingly, those alternatives which included constructing a outfall structure at Thorton State Beach were not as desirable as using the existing outfall site. This decision was formalized in a letter to JA dated August 9, 2007. Initially, 17 conceptual design alternatives were developed from these tunnel and nontunnel alternatives and presented to the City in the draft Alternatives Evaluation Report, dated December 12, 2007 (Volume 1, Table 9.2). The City and the project team selected these alternatives based on their potential for reducing flooding, operational viability, public impacts, environmental benefits, and constructability. The alternatives include three main elements: a drainage tunnel, a storage/detention structure, and storm water reuse opportunities.

From the original evaluation and input received during the public outreach activities, the City selected three of the 17 alternatives—Alternative 5B, Alternative 6B, and Alternative 7 (each of which can be combined with peak storm water flow storage at Westlake Park); see Figure 1. Subsequently, Jacobs Associates performed supplemental analyses to refine the three alternatives that would send storm water out of the basin directly into the Pacific Ocean. Jacobs Associates presented this analysis to the City in a memorandum titled "Vista Grande Drainage Basin Alternatives Analysis Project Supplemental Analyses (Final Draft)," dated August 29, 2008 (Volume 2).

On March 30, 2010, at the City's request, Jacobs Associates presented an evaluation of an additional alternative—the Lake Merced Alternative—for consideration along with Alternatives 5B, 6B, and 7. This evaluation was contained in a memorandum to the City titled "Vista Grande Drainage Basin Alternatives Analysis Project Lake Merced Alternative (Supplement)," (Volume 3).

3 Watershed Hydrology and Hydraulics

Vista Grande is a 2.5-square-mile, highly urbanized watershed located in Daly City, California. Most of the drainage area falls within the limits of Daly City; however a small portion of the northern part of the drainage area is located within the City and County of San Francisco (CCSF), and a central portion of the drainage area is within unincorporated San Mateo County. Figure 2 provides an overview of the study area.

Storm water runoff in the Vista Grande watershed is collected in an extensive storm drainage network that generally flows north and west toward the Vista Grande Canal in the northwest portion of the drainage basin. Storm water then flows northwest in the canal and discharges to the Vista Grande Tunnel, which conveys water west by gravity to an outfall to the Pacific Ocean. Figure 3 illustrates the distribution of simulated peak flows based on the historical rainfall record using a Rational Method hydraulic model. It is important to recognize that the existing capacities of the Vista Grande Canal and Tunnel are 500 cubic feet per second (cfs) and 170 cfs, respectively. Storm events with peak flows greater than the existing capacities may result in localized flooding within the Vista Grande Watershed.

The City worked jointly with the CCSF to develop the Vista Grande Watershed Plan (RMC, August 2006), to address flooding and drainage issues in the watershed. The plan describes frequent flooding problems at several locations in the Vista Grande watershed, including surcharging along major trunk lines of the storm drainage system and overflow flooding along the Vista Grande Canal. In addition, CCSF and stakeholder groups are concerned about managing and improving declining Lake Merced levels. Therefore, the City of Daly City and the CCSF want to identify whether enough storm water runoff is generated from the Vista Grande watershed to increase and sustain the water levels at Lake Merced. To maintain and improve the Lake Merced water quality, storm water runoff could be routed to the lake after passing through a natural processes wetland or after the initial watershed runoff was routed to the ocean. , Up to approximately 930 cfs of storm water could be routed to Lake Merced via four new box culverts beneath John Muir Drive. Runoff flows exceeding the capacity of the Lake Merced culverts would remain in the Vista Grande Canal and be routed to the ocean. The reader should note that this management scenario is conceptual and, because of the proposed system's versatility, different storm water management strategies could deliver various storm water re-use and flood protection benefits.

4 Summary of “Top Four” Alternatives

Below is a summary of the “top four alternatives” selected by the City: the three alternatives routing storm water directly to the Pacific Ocean (including a storm water storage facility) and the additional Lake Merced Alternative which routes storm water to Lake Merced and/or the Pacific Ocean.

A storm water storage basin, an option for use with Alternatives 5B, 6B, or 7, would be located within the city limits of Daly City, beneath Westlake Park. The underground storm water detention facility would include an intercept, a pneumatically controlled hydraulic diverter, a gross solid screening device, a 4-MG underground storage tank, pumps, and associated instrumentation and controls. This facility would provide peak storm water flow shaving capacity of up to 663 cfs and 4 MG. The pumps would be used to drain the stored storm water back to the storm drain system within 24 hours.

Each of the alternatives includes increasing the hydraulic capacity of the existing Vista Grande Tunnel and replacing the existing Daly City Outfall Structure with a new low profile outfall. The tunnel and outfall can be constructed from either John Muir Drive or via a construction shaft at Fort Funston. Construction access to the beach is extremely limited.

4.1 Alternative 5B

Alternative 5B, located within the city limits of San Francisco, would include a drop structure, a gross solid screening device, an 800-foot-long box culvert within the existing canal corridor, a new drainage tunnel that would be approximately 5,300 feet long, and a 4-million-gallon (MG) underground storm water storage tank beneath Westlake Park in Daly City. The existing tunnel outfall structure would be rehabilitated and modified to accommodate the new tunnel flow capacity.

A new drop structure, located at the canal inlet (John Muir and Lake Merced Blvd.), would collect the flows from the major culvert lines and direct the flows to the gross solid screening device. Assuming the screens were no more than 25% full, the screening device would have a capacity of 1,660 cubic feet per second (cfs). The transition between the screening device and the new box culvert would incorporate an overflow weir to split the flows. Flows up to 170 cfs would flow through a box culvert to the existing canal north of the new tunnel inlet and through the existing tunnel. Flows in excess of 170 cfs would flow over a weir into a separate double box culvert leading to the new tunnel inlet.

The new tunnel would run northwest from the wide section of the canal corridor, located approximately 800 feet downstream of the canal inlet, to the rehabilitated Vista Grande Outfall Structure. The tunnel would run under the Olympic Club, Highway 35, and Golden Gate National Recreation Area (GGNRA) lands.

4.2 Alternative 6B

Alternative 6B, located within the city limits of San Francisco, would include a drop structure, a gross solid screening device, a 2,100-foot-long box culvert within the existing canal corridor, a new drainage tunnel that would be approximately 4,200 feet long, and a 4-MG underground storm water storage tank beneath Westlake Park in Daly City. The existing tunnel outfall structure would be rehabilitated and modified to accommodate the new tunnel flow capacity.

A new drop structure, located at the canal inlet, would collect the flows from the major culvert lines and direct the flows to the gross solid screening device. Assuming the screens were no more than 25% full, the screening device would have a capacity of 1,660 cfs. The transition between the screening device and the new box culvert would incorporate an overflow weir to split the flows. Flows up to 170 cfs would flow through a box culvert to the existing canal north of the new tunnel inlet and through the existing tunnel. Flows in excess of 170 cfs would flow over a weir into a separate double culvert leading to the new tunnel inlet.

The new tunnel would run northwest from a wide section of the canal, located approximately 2,100 feet downstream of the canal inlet, to the rehabilitated Vista Grande Outfall Structure. The tunnel would run under the Olympic Club, Highway 35, and the GGNRA lands.

4.3 Alternative 7

Alternative 7, located within the city limits of San Francisco, would include a drop structure, a gross solid screening device, a 3,500-foot-long box culvert within the existing canal corridor, a new drainage tunnel that would be approximately 3,200 feet long, and a 4-MG underground storm water storage tank beneath Westlake Park in Daly City. The existing tunnel outfall structure would be rehabilitated and modified to accommodate the new tunnel flow capacity.

A new drop structure, located at the canal inlet, would collect the flows from the major culvert lines and direct the flows to a gross solid screening device. Assuming the screens were no more than 25% full, the screening device would have a capacity of 1,660 cfs. The transition between the screening device and the new box culvert would incorporate an overflow weir to split the flows. Flows up to 170 cfs would flow through a box culvert to the existing canal north of the new tunnel inlet and through the existing tunnel. Flows in excess of 170 cfs would flow over a weir into a separate double box culvert leading to the new tunnel inlet. The flows up to 170 cfs would flow through a separate box culvert to the existing canal north of the new tunnel inlet.

The enlarged Vista Grande tunnel would run west following the existing tunnel alignment, to the rehabilitated Vista Grande Outfall Structure. The tunnel would run beneath a small portion of the Olympic Club, Highway 35, and the GGNRA lands.

4.4 Lake Merced Alternative

Following discussions with the public and key stakeholders, CCSF and the City agreed to explore the potential benefits of using the existing infrastructure adjacent to Lake Merced and including Lake Merced as part of the overall system to reduce the localized flooding potential within the watershed while concurrently increasing and managing Lake Merced water levels. The analyses presented in Volume 3 integrate the Lake Merced Alternative into the ongoing alternatives study and address:

- Safely routing storm water from the Vista Grande Watershed to Lake Merced and the Pacific Ocean
- Improving storm water quality discharges
- Providing a nongroundwater source of water to assist the CCSF in managing enforceable Lake Merced lake levels
- Achieving desired operating water surface elevations for Lake Merced in a safe and environmentally acceptable manner
- Reducing uncontrolled canal overflows into Lake Merced
- Providing lake overflow capacity to minimize environmental and property damage associated with large storms and high lake levels

Implementing the Lake Merced Alternative would involve constructing facilities necessary to screen storm water; route flows to the existing canal, Lake Merced, or both; improve authorized non-storm water and storm water quality through natural treatment processes (surface flow wetland) adjacent to Lake Merced; control the Lake's water surface; and reduce the potential for localized flooding in the watershed. This alternative considers routing year round low-flow storm water and authorized non-storm water to Lake Merced via a surface flow wetland and a portion of the higher-flow screened storm water both to increase the lake's water volume and increase the lake level management flexibility. A new overflow outlet would be constructed in Lake Merced to provide a reliable outlet for managing the lake's water level. The balance of screened storm water flows would pass through the Canal, enlarged Vista Grade Tunnel, and new low-profile City outfall structure. (Volume 3, Figure 3 shows various outlet/overflow configurations from Lake Merced's South Lake and North Lake shorelines. Conceptual designs are included in Volume 3.)

Using one or more natural treatment processes, the Lake Merced Alternative could satisfy multiple project objectives. This alternative would provide CCSF with the facilities to operate Lake Merced within a desired water level range. The Lake's current water surface elevation is approximately 4.5 feet (CCSF Datum), and there is local interest in managing the Lake levels between a normal operating water surface elevation range of 5.0 feet and 9.5 feet. Routing all or a portion of the screened storm water and authorized non-storm water flows into the Lake under the Lake Merced Alternative would increase the Lake's water levels and volume, which would increase the flexibility for managing water levels and quality, and reduce flooding.

As part of the work to assess the feasibility for a storm water diversion into Lake Merced through a constructed natural treatment process (wetlands) adjacent to Lake Merced, the project team evaluated potential fluctuations in the Lake Merced water surface from such a diversion. First, Brown and Caldwell modified a box model provided by SFPUC to examine storm water diversion at a coarse scale. When modeled, applying a 60-year rainfall record and appropriate assumptions regarding rainfall resulting in runoff, such a diversion could sustain Lake Merced lake levels within the target water surface elevation range. Brown and Caldwell also carried out more detailed evaluation using actual rainfall data from the Oceanside/Richmond-Sunset gauge (near continuous data set from 1948 through 2005, 55 years of data). This gage was considered to be more appropriate for the Vista Grande drainage since it is located at the downstream end of the drainage basin. Comparison of the results from this work (see Attachment 1) with previous analyses show similar results. The average annual diversion to Lake Merced could be on the order of 1,100 acre feet (AF), more than sufficient to maintain lake levels, depending on the adopted storm water management strategy.

5 Regulatory Climate

The regulatory climate is receptive to a demonstration that the project alternatives satisfy the project objectives by following the NEPA/CEQA process. Private, local, state and federal entities own the lands needed to construct, operate and maintain the storm water improvements. The City would need to consult with relevant resource agencies and follow prescribed environmental review processes to evaluate project environmental effects and obtain construction permits for proposed components or improvements. The City would conduct environmental review processes under the California Environmental Quality Act (CEQA) and National Environmental Policy Act (NEPA). Generally, the key environmental issues requiring evaluation include:

- Storm water and authorized non-storm water quality;
- Lake Merced water quality;
- Public health and safety;
- Vegetation and wildlife habitat, including special-status species;
- Beach and coastal bluff erosion and sedimentation;
- Lake water level management, including effects on special-status species;
- Public access to the beach;
- Recreation activities and park resources;
- Private business enterprises;
- Aesthetics;
- Ocean resources; and,
- Short term construction-related traffic, road closures and noise.

A Permitting Workbook (Draft) is designed to outline the permitting process for the Daly City Vista Grande Drainage Basin project. The purpose of the workbook is to describe the permit and regulatory requirements for environmental compliance leading to the construction phase of the project. This will serve as a companion document to the Vista Grande Drainage Basin Alternatives Analysis Report by outlining the regulatory and permitting requirements for the project design alternatives. The Permitting Workbook is organized to guide the project team through the regulatory process and is included as Volume 4. A brief summary of the agencies and the associated permits required to construct and operate the project is presented in Tables 1 and 2, respectively.

Table 1
Potential Regulatory Permitting Requirements During Construction

Agency	Governing Regulation	Potential Requirements/ Permit
City of Daly City **Lead Agency,**	California Environmental Quality Act Municipal Code	Initial Study/ Environmental Assessment Environmental Impact Statement/Report Local building construction ordinances Compliance with SWPPP/ storm water control permit
State Water Resources Control Board	Clean Water Act	General Construction Permit/ Storm water Pollution Statewide General Waste Discharge Requirements Storm water Pollution Prevention Plan (SWPPP)
San Francisco Bay Regional Water Quality Control Board	Clean Water Act	Municipal Discharge Permit (likely) or coverage under "Storm water Permit"; Section 401 Water Quality Certification; Section 402 Policy on the Use of Constructed Wetlands for Urban Runoff (No. 94-102) Overflow discharge from the Lake Waste Discharge Requirements
California Department of Fish and Game	Fish and Game Code Section 1602	Lakebed Alteration Agreement
State of California State Historic Preservation Office	National Historic Preservation Act National Environmental Policy Act	Cultural Resources Inventory Section 106 Technical Report Tribal consultation Determination of Historical Significance Memorandum of Agreement
State of California State Lands Commission	California Public Resources Code, Division 6 Public Lands	General Lease Right-of-Way Evidence of right to use property Land and title documentation
California Coastal Commission	California Coastal Act	Coastal Development Permit CCSF LCP- Western Shoreline Plan Daly City LCP San Mateo LCP Local Coastal Plan compliance Public Works Plan Federal Consistency Determination
City and County of San Francisco	Local building ordinances Storm water control ordinance	Local building construction ordinances Compliance with SWPPP/ storm water control permit
County of San Mateo	Local building ordinances Storm water control ordinance	Local building construction ordinances Compliance with SWPPP/ storm water control permit
TBD	National Environmental Policy Act	Initial Study/ Environmental Assessment Environmental Impact Statement/Report
U.S. Army Corps of Engineers	Clean Water Act Section 404	Waters of the U.S. Delineation Report Participation at pre-application interagency meetings Section 404 Authorization Section 106 National Historic Preservation Act Nationwide Permit 7 Nationwide Permit 12 Nationwide Permit 33
U.S. Fish and Wildlife Service	Endangered Species Act Section 7	Biological Assessment Biological Opinion Incidental Take Statement
National Oceanic and Atmospheric Administration/ National Marine Fisheries Service	Endangered Species Act Section 7	Biological Assessment Biological Opinion Incidental Take Statement EFH Conservation Recommendations
Golden Gate National Recreation Area/ National Park Service		NEPA Special Use Permit Right-of-Way Permit

		State Lands Commission Lease Compliance
During Operation		
San Francisco Bay RWQCB	Basin Plan	Water Quality Objectives listed in this memo
Monitoring Plan and requirements that may be tied with the Municipal Regional Storm water Permit listed above		

Table 2
Potential Regulatory Permitting Requirements During Operation

Agency	Governing Regulation	Potential Requirements/ Permit
City of Daly City	Municipal Code	Compliance with Storm water quality permit(s)
State Water Resources Control Board	Clean Water Act	Statewide General Waste Discharge Requirements Storm water Pollution Prevention Plan (SWPPP)
San Francisco Bay Regional Water Quality Control Board	Clean Water Act	Basin Plan Municipal Discharge Permit (likely) or coverage under "Storm water Permit"; Section 401 Water Quality Certification; Section 402 Policy on the Use of Constructed Wetlands for Urban Runoff (No. 94-102) Overflow discharge from the Lake Waste Discharge Requirements
California Coastal Commission		Coastal Development Permit Local Coastal Plan compliance Public Works Plan Federal Consistency Determination
Golden Gate National Recreation Area/ National Park Service		Special Use Permit Right-of-Way Permit

6 Alternatives Analysis

6.1 Budget-Level Cost Estimates

Using the Association for the Advancement of Cost Engineering International classification system, budget-level cost estimates were prepared for the Alternatives employing unit costs developed from comparable projects, supplier quotes, and allowances. The opinions of probable project cost consider the contractor's direct and indirect costs, project professional services, an escalation estimate, and design contingency. Table 3 presents the opinion of probable project costs for the base estimate.

Table 3. Opinion of Probable Project Costs (Budget Level Accuracy)

Alternative	Tunnel Work Performed from John Muir Drive	Tunnel Work Performed from a Temporary Construction Shaft
5	$201.5 M	$180.2 M
6B	$209.8 M	$189.5 M
7	$219.4 M	$202.0 M
Lake Merced	$174.0 M	$153.5M

6.2 Project Objectives Evaluation

The evaluation methodology previously developed and used to prepare the Vista Grande Drainage Basin Alternatives Analysis Report (Draft) was applied to the Alternatives 5B, 6B, 7, and the Lake Merced Alternative. The results of the scoring suggested a preliminary ranking of alternatives summarized in Table 4 and presented in full detail in Attachment 2. The evaluation matrix incorporates criteria of providing public benefits within the basin; satisfying a functional operations criteria; complying with environmental regulations and processes; minimizing land acquisition costs; maximizing constructability; and, minimizing lifecycle costs. The evaluation results suggest that the Lake Merced Alternative, Alternative 7, Alternative 5B, and Alternative 6B strongly align with the project objectives. The Lake Merced Alternative provides significantly greater water re-use benefits than the other alternatives; however, permitting the storm water diversion will involve significant regulatory cooperation unique to this alternative.

Table 4. Preliminary Alternatives' Ranking based on the Project Evaluation Methodology		
Overall Rank	**Weighted Score[2]**	**Description**
1	5	Lake Merced Alternative with storm water screening, an inlet structure, a lake overflow structure, a wetlands treatment system, enlarged Vista Grande Tunnel constructed via a temporary construction access shaft, & low profile outfall structure
2	10	Alternative 7 with storm water screening, an enlarged Vista Grande Tunnel constructed via a temporary construction access shaft, & low profile outfall structure
3	13	Alternative 5B with storm water screening, an enlarged Vista Grande Tunnel constructed via a temporary construction access shaft, & low profile outfall structure
4	16	Alternative 6B with storm water screening, an enlarged Vista Grande Tunnel constructed via a temporary construction access shaft, & low profile outfall structure

[2] Weighted Score is the sum of weighted rank points where a lower sum suggests stronger alignment with project objectives and a higher sum suggests a weaker alignment with project objectives.

7 Next Steps

The City will follow its internal review processes and conduct a public hearing introducing the alternative(s) best serving the public's interests from Alternatives Analysis Report. Following the public hearing, the City will identify the alternatives which will be evaluated in the NEPA/CEQA framework and further developed in the preliminary design phase. The City should consider the following activities (see Timeline in Table 5) to continue the project development consistent with its objectives:

- Initiate the environmental and regulatory permitting process. The objective of this task is to assist the City with the NEPA/CEQA processes which will evaluate the Recommended Preferred Alternative relative to other alternatives. This process will identify the permits and agreements necessary to construct and operate the drainage basin improvements. This task also includes securing the permits and agreements.

- Develop the 35 percent Preliminary Design documents including a condition assessment of the existing Vista Grande Tunnel, engineering drawings, a specification outline, and cost estimate of the Preferred Alternative. The objective of this task is to define the project features in sufficient detail to support the funding, permitting, and land management efforts.

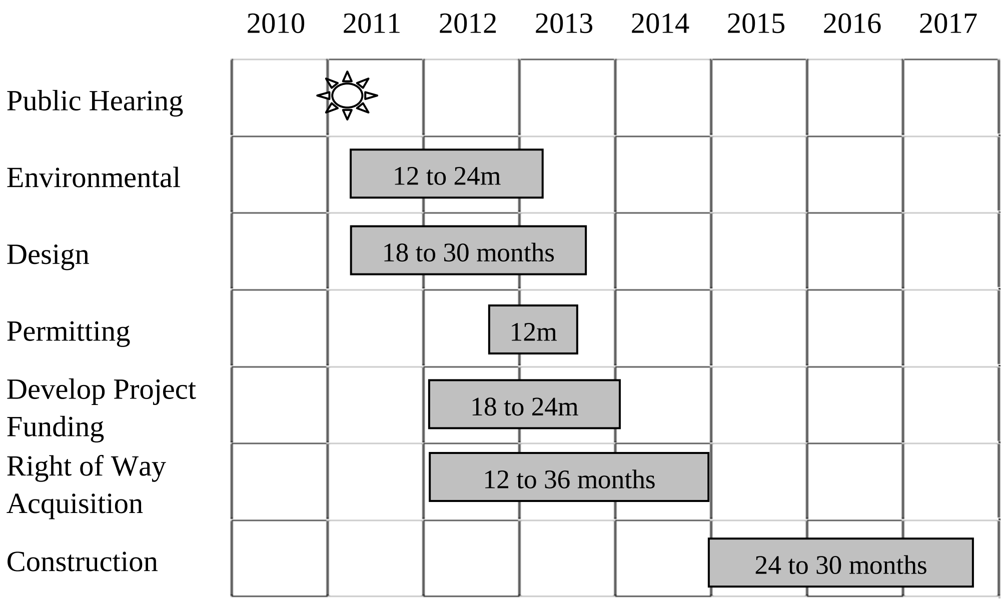
- Daly City would continue developing its Vista Grande Watershed Storm water Management Strategic Plan.

- Develop and pursue a public funding strategy. The task objective is to assist the City secure public funding for the drainage basin improvements.

- Initiate the land acquisitions (as required) and easement process. The objectives of this task are to: (a) identify the necessary easements and land acquisitions; and (b) assist the City with the processes to access the required lands for the project.

- Continue the public outreach and communication efforts. The objectives of this task are to: (a) facilitate the City's decision-making processes; (b) participate in the public outreach; and (c) assist the City in its response to comments from the public and other stakeholders.

Table 5. Conceptual Project Timeline



8 Figures

Figure 1: Screened Alternatives 5B, 6, and 7 adjacent to the Lake Merced Alternative.

Figure 2: Vista Grande Watershed Study Area outlined in yellow
other demarcations refer to city and county boundaries

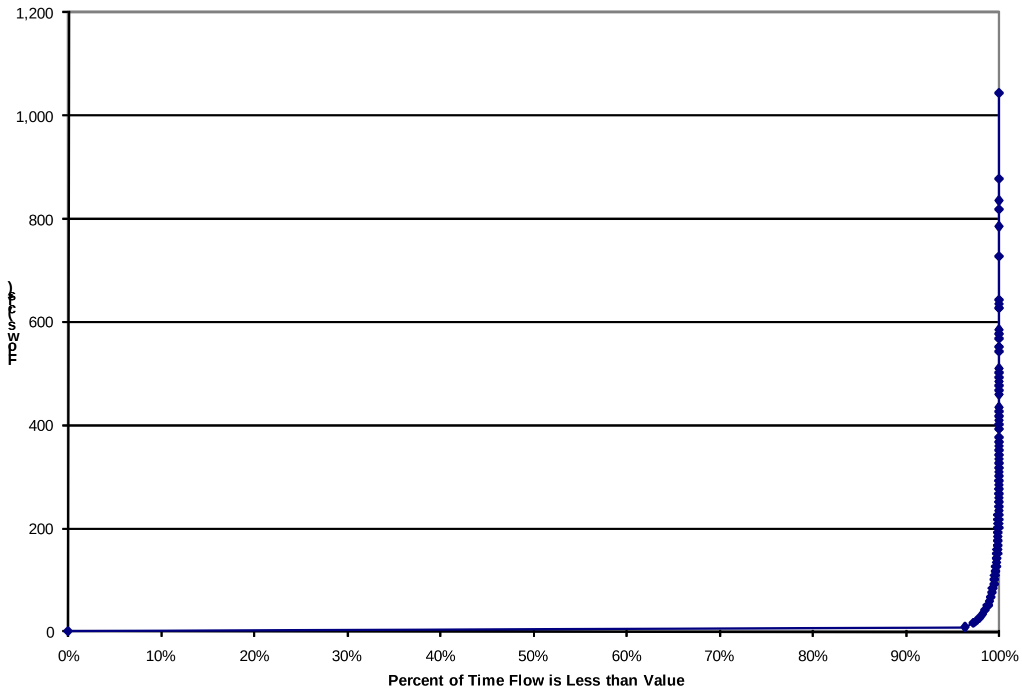


Figure 3a: Vista Grande Watershed Flow Duration Curve
Distribution of simulated peak flows, full range

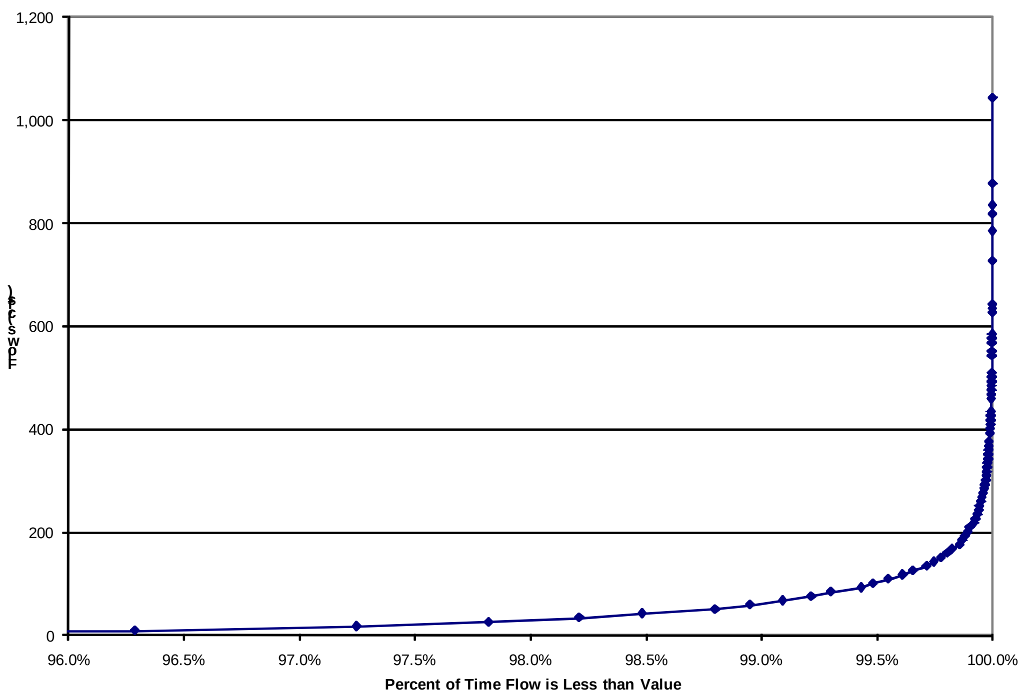


Figure 3b: Vista Grande Watershed Flow Duration Curve
Distribution of simulated peak flows, *96 to 100% of range*

ATTACHMENT 1 Historic Rain Data and Flows Evaluation Technical Memorandum

BROWN AND CALDWELL

Technical Memorandum

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Prepared for: City of Daly City

Project Title: Vista Grande Drainage Basin Improvement Project

Project No: 137576

Technical Memorandum No. 1

Subject: Historical Rain Data and Flows Evaluation

Date: February 12, 2010
Revised April 13, 2010

To: Patrick Sweetland, City of Daly City
Robert Ovadia, PE, City of Daly City

Copy to: Blake Rothfuss, PE, Jacobs Associates
File - 137576

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TABLE OF CONTENTS

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PURPOSE AND OBJECTIVES

The purpose of the analysis described herein was to evaluate and the historical record of rain data from the National Climactic Data Center (NCDC) San Francisco-Oceanside rain gauge to identify the magnitude and frequency of flows and associated volumes that could reach Lake Merced via the Vista Grande Canal. For this evaluation, the following objectives were achieved:

- Build on previous work; in particular, use the recently-developed XP-SWMM hydraulic model of the Vista Grande storm drainage system as a baseline for modification.
- Evaluate the historical rain data record to identify trends in event peak intensities, flows, volumes, and durations
- Estimate the volume of flows that could typically reach Lake Merced

BACKGROUND

Vista Grande is a 2.5-square-mile, highly urbanized watershed located in Daly City, California. Most of the drainage area falls within the limits of the Daly City; however a small portion of the northern part of the drainage area is located within the City and County of San Francisco (CCSF), and a central portion of the drainage area is unincorporated San Mateo County. Figure 1 provides an overview of the study area.

Stormwater runoff in the Vista Grande watershed is collected in an extensive storm drainage network that generally flows north and west toward the Vista Grande Canal in the northwest portion of the drainage basin. Stormwater then flows northwest in the canal and discharges to the Vista Grande Tunnel, which conveys water west by gravity to an outfall to the Pacific Ocean.

The City of Daly City worked jointly with the CCSF to develop the Vista Grande Watershed Plan (RMC, August 2006) to address flooding and drainage issues in the watershed. The plan describes frequent flooding problems at several locations in the Vista Grande watershed, including surcharging along major trunk lines of the storm drainage system and overflow flooding along the Vista Grande Canal. In addition, CCSF and interest groups are concerned that the water level of Lake Merced is dropping. Therefore, the City of Daly City and the CCSF want to identify whether enough stormwater runoff is generated from the Vista Grande watershed to increase and sustain the water levels at Lake Merced. To protect the quality of the lake, stormwater runoff from the initial runoff (estimated operationally as runoff from the first 1-inch of rainfall) for each year would be diverted to the ocean, capturing the "first flush". Runoff from the remaining events would be diverted to the Impound Lake of Lake Merced, up to the capacity of four box culverts or 930 cubic feet per second (cfs). Any runoff flows greater than 930 cfs would then be diverted to the ocean. The reader should note that this management scenario is preliminary. Any proposed system would have great flexibility regarding which flows to divert into Lake Merced and which flows to bypass to the Pacific Ocean.

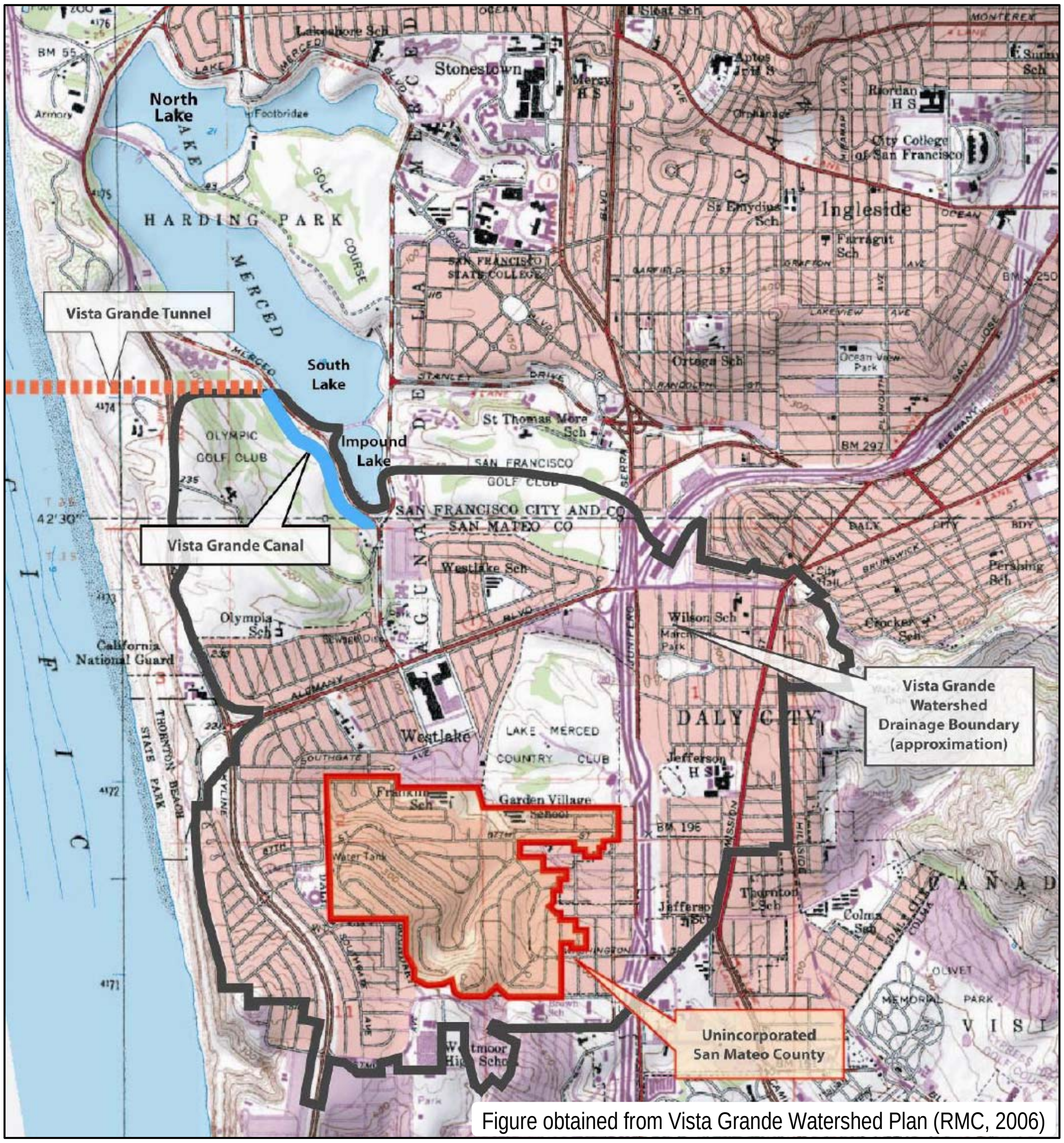

Figure obtained from Vista Grande Watershed Plan (RMC, 2006)

Figure 1. Vista Grande Watershed and Vicinity

MODELING METHODOLOGY

The City of Daly City is currently working with Jacobs Associates and Brown and Caldwell on further evaluation and analysis of proposed improvements to the Vista Grande Canal and Tunnel. One objective is to identify the quantity and frequency of flows to the Impound Lake of Lake Merced. To reach this objective, Brown and Caldwell performed a(n):

- Review of Historical Rain Data
- Estimation of Flows to the Project Area
- Estimation of the Runoff Volume that could reach Lake Merced

Historical Rain Data Review

Brown and Caldwell obtained the historical rain data record from the NCDC website for the San Francisco-Oceanside rain gauge, formerly known as the Richmond-Sunset gauge. Rain data is available from this website in an hourly timestep, from July 1948 through April 2003. Data from October 1951 through December 1952 is missing; therefore the historical record contains approximately 55 years of data. Figure 2 provides an overview of the entire rainfall record for the NCDC San Francisco-Oceanside rain gauge, which will be referred to for the remainder of this memorandum as the historical rain data.

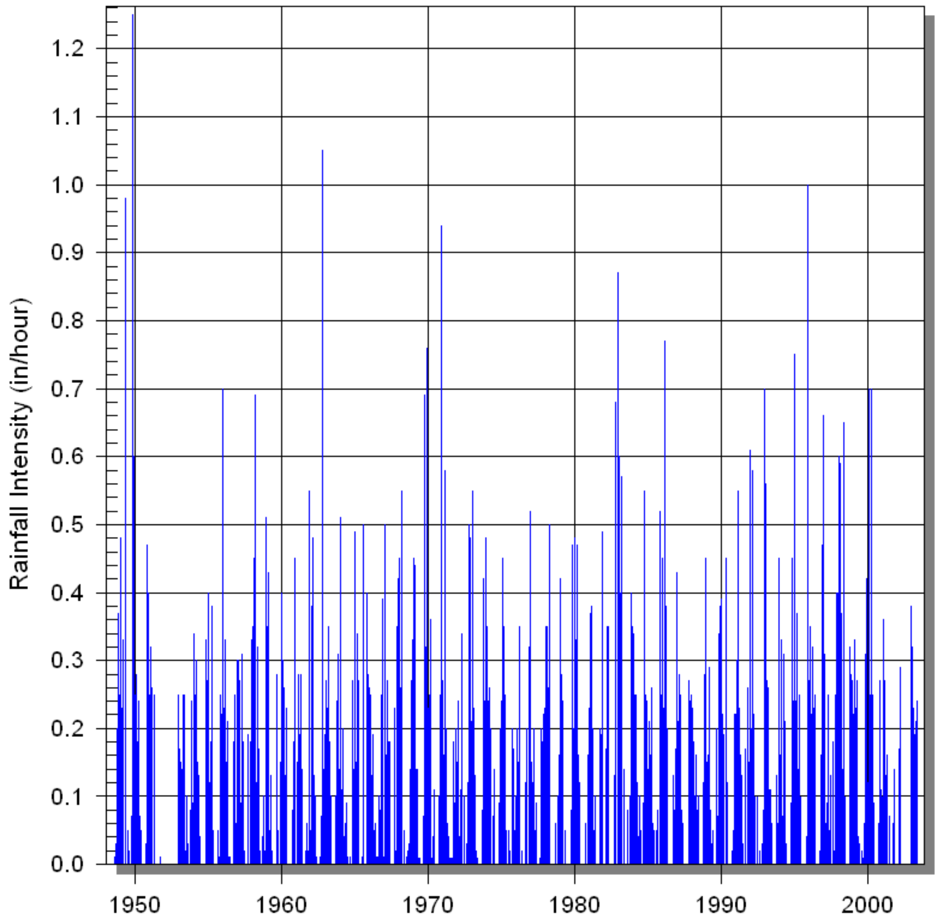

Figure 2. Rain Data Record for NCDC San Francisco-Oceanside Rain Gauge

Geosyntec provided Brown and Caldwell with a summary of the historical rain events, which included a chronological listing of the total rain depth, duration, and peak rain intensity by event. Events were defined as having at least 0.15 inches in total rainfall with an inter-event duration of at least 6 hours. Brown and Caldwell then reviewed the data from Geosyntec and ranked each event by peak rain intensity and by total rain depth. In addition, Brown and Caldwell also identified the maximum rainfall intensity for each year of the historical record and developed rainfall recurrence intervals using a Log Pearson Type 3 statistical analysis, a typical statistical approach applied to such data.

Flows Estimation

Flows were calculated for the entire 55-year historical rainfall record using the Rational Method equation of

$$Q = CiA,$$

Where:

Q = Flow (cfs)

C = Runoff Coefficient

i = Rainfall Intensity (inches/hour)

A = Area (acres)

Brown and Caldwell calculated the area for the Rational Method equation by taking the sum total of the area of any subcatchments in the XP-SWMM model that are tributary to the area of interest. This total area was estimated to be 1,670 acres.

In determining an appropriate runoff coefficient value, Brown and Caldwell first calculated the average percent impervious of the subcatchments in the XP-SWMM model and identified the overall average to be 57%, or a C value of 0.57. This value was then compared to the *Vista Grande Watershed Study* (RMC, August 2006) C value of 0.71. Next, Brown and Caldwell developed an independent estimate of the runoff coefficient (C) by simulating several historical events in XP-SWMM and then dividing the sum of the total simulated flows for each event by the sum of the total rain for the event. Table 1 shows the events that were included in this evaluation and the calculated runoff coefficients for each event. Brown and Caldwell received and reviewed modeling files from the XP-SWMM model developed by RMC (RMC, September 2007 – February 2009) and found that these modeling files were sufficient for simulating specific historical events. The model named *ModelA_25yrWithImprovements*, which represents proposed conditions and includes the future drainage improvements as recommended in the RMC memorandum (February 2009), was used for all of the event simulations. The Rational Method was used to estimate flows for this evaluation because simulation times in excess of one year would be needed to run the 55-year historical rainfall record through this XP-SWMM model, making it an impractical method for generating long-term flows and calculating corresponding flow recurrences.

Finally, the runoff coefficient was further adjusted (calibrated) to a value of 0.50 to provide the best overall fit between the XP-SWMM and the Rational Method peak hour flows. Table 2 and Figures 3 through 6 compare the peak hour flows for the XP-SWMM model versus the Rational Method for each of the evaluated events.

Table 1. Summary of Evaluated Events (SF-Oceanside Gauge)

Event	Approximate Peak Hour Recurrence Interval (yr)	Peak Hour Rain Intensity (in/hr)	Approximate Event Duration (hrs)	Initial Estimated Runoff Coefficient - C
August 11, 1965	2	0.50	12	0.5
February 18-19, 1986	10	0.77	9	0.5
May 14, 1949	25	0.98	3	0.5
October 11-14, 1962	50	1.05	32	0.5

Table 2. Comparison of XP-SWMM and Rational Method Simulated Flows for Selected Events		
Event	Simulated or Estimated Peak Flow at Downstream End of Project Area [2084-C] (cfs)	Calculated Total Volume (cf)
August 1965 Historical Event (2-year, 1-hour)		
XP-SWMM Model	420	3,200,000
Rational Method	420	3,600,000
Ratio of XP-SWMM Model Peak Flow to Rational Method Peak Flow	1.00	0.89
February 1986 Historical Event (10-year, 1-hour)		
XP-SWMM Model	640	5,200,000
Rational Method	640	6,100,000
Ratio of XP-SWMM Model Peak Flow to Rational Method Peak Flow	1.00	0.85
May 1949 Historical Event (25-year, 1-hour)		
XP-SWMM Model	810	3,600,000
Rational Method	820	3,900,000
Ratio of XP-SWMM Model Peak Flow to Rational Method Peak Flow	0.99	0.92
October 1962 Historical Event (50-year, 1-hour)		
XP-SWMM Model	870	22,300,000
Rational Method	880	22,700,000
Ratio of XP-SWMM Model Peak Flow to Rational Method Peak Flow	0.99	0.98

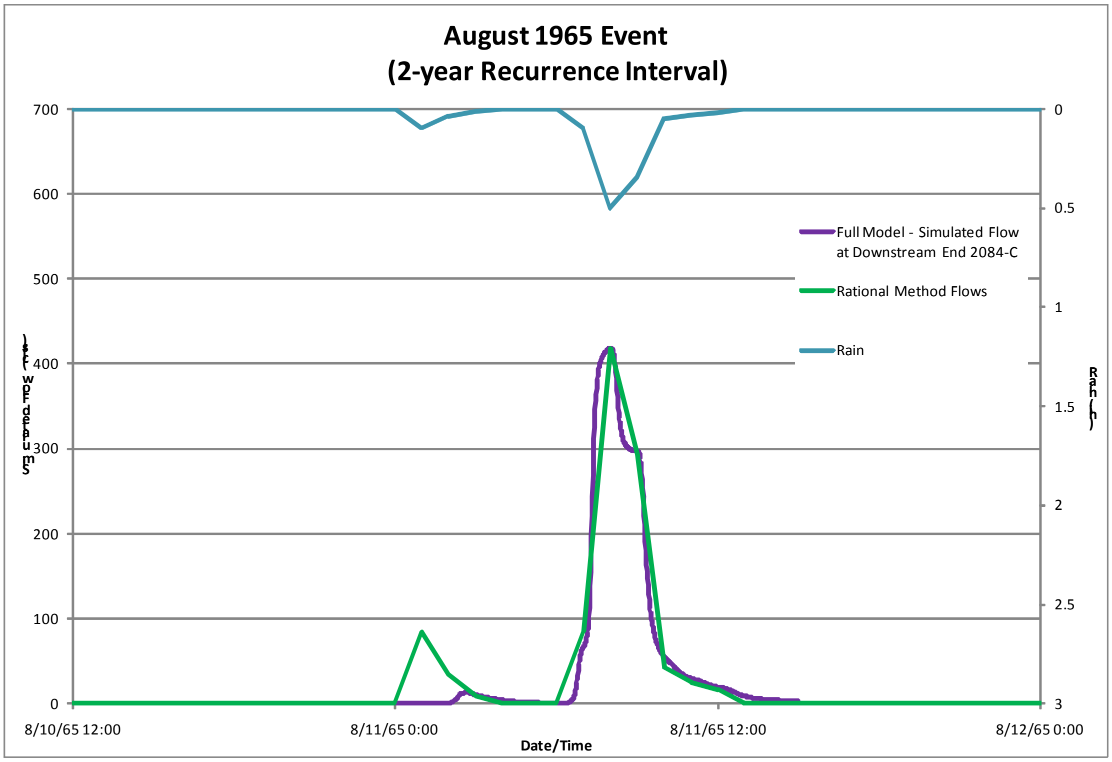

Figure 3. Comparison of Peak Hour Flows for August 11, 1965

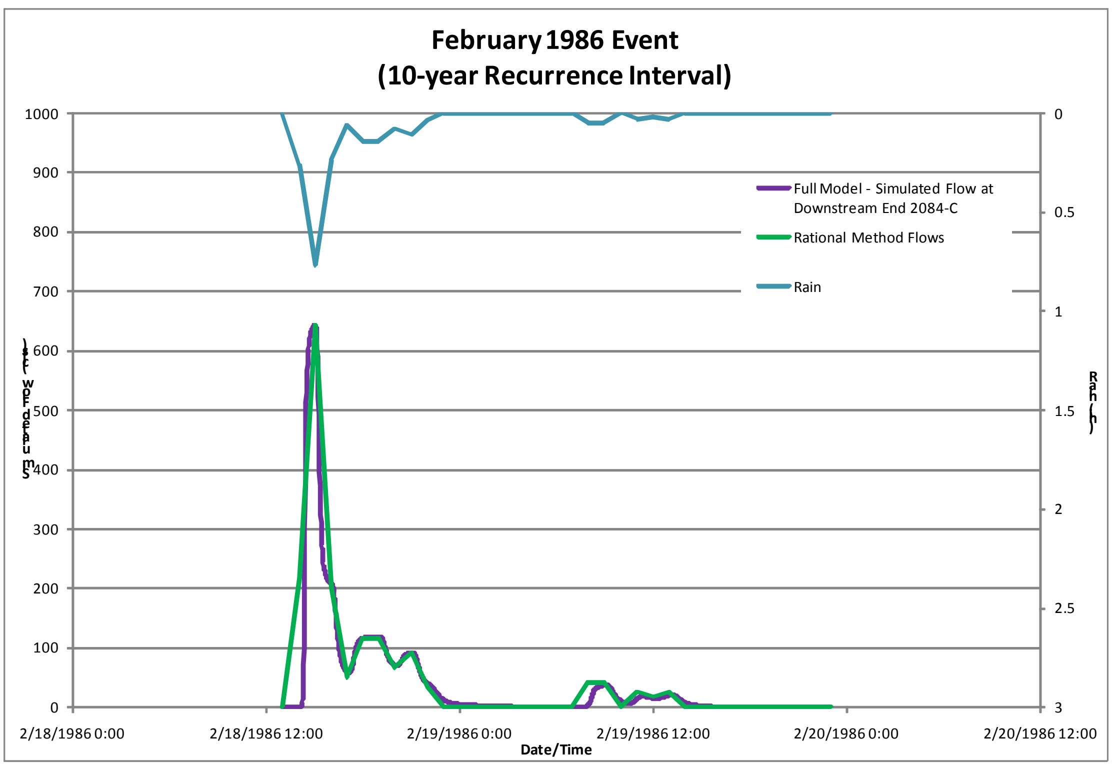

Figure 4. Comparison of Peak Hour Flows for February 18-19, 1986

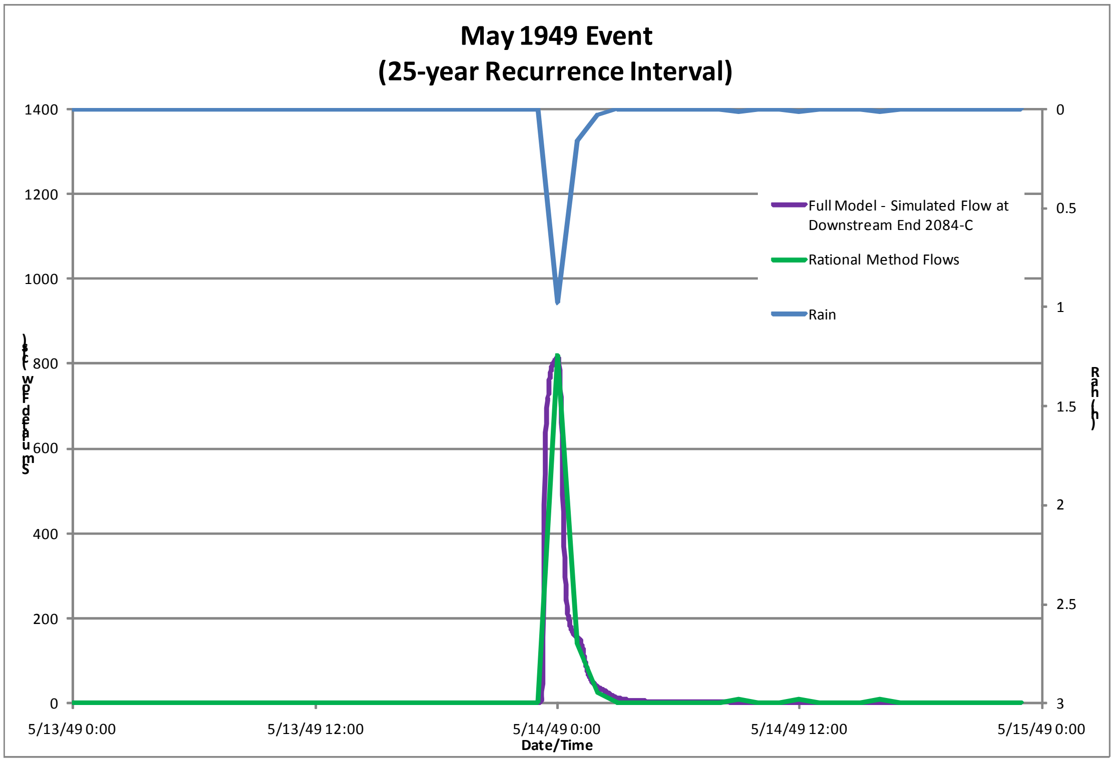

Figure 5. Comparison of Peak Hour Flows for May 14, 1949

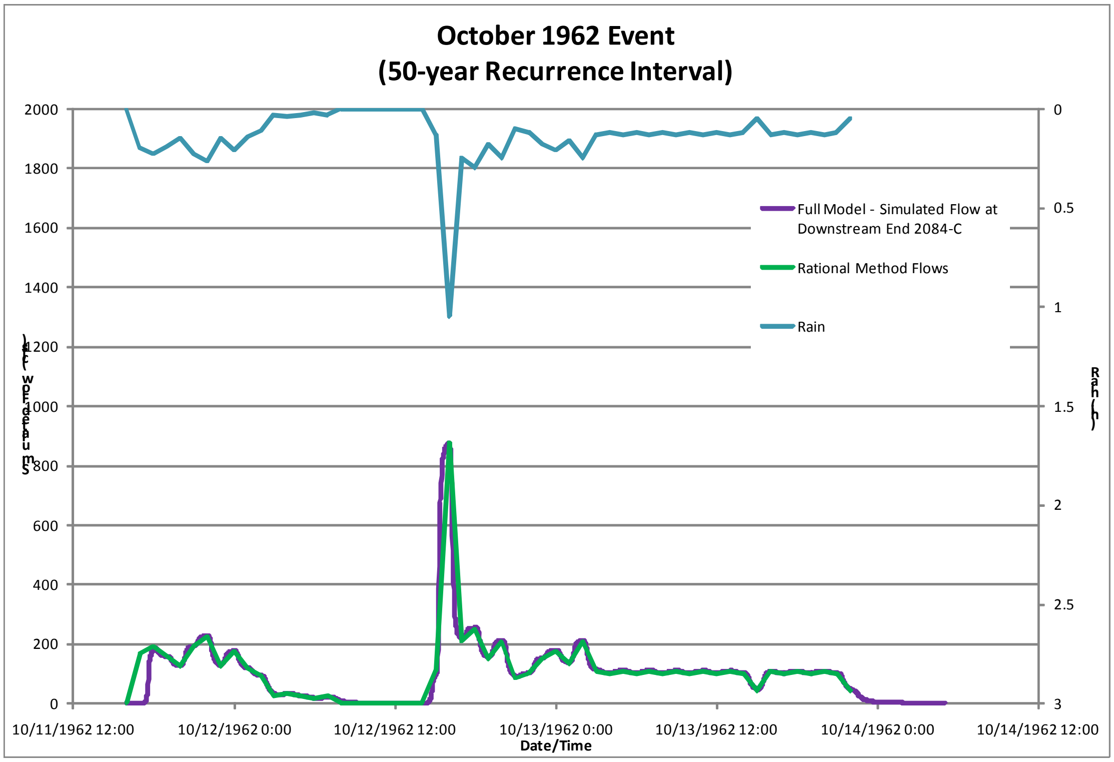

Figure 6. Comparison of Peak Hour Flows for October 11-14, 1962

Volume Estimation

The volume of runoff was calculated by multiplying the rainfall depth for each event by the runoff coefficient and the total drainage area of 1,670 acres.

MODELING RESULTS

Modeling results include:

- Rainfall Recurrence Intervals
- Event Distribution
- Flow Occurrences
- Estimated Volume to Lake Merced

Rainfall Recurrence Intervals

Figure 7 shows the statistical evaluation of the annual maximum peak rain intensities for the 55-year historical rain record and Table 3 provides the recurrence intervals of these events. Table 4 shows the recurrence intervals as determined from the National Oceanic and Atmospheric Administration (NOAA) publication Atlas 2, Volume 11 (NOAA, 1973). The values in Table 4 were generated either by identifying the appropriate isopluvial line for Daly City, using the equations identified in NOAA Atlas-Table 11, or visually estimating the values from the nomograph on NOAA. Atlas- Figure 6. As seen in Table 4, the recurrence intervals that were developed using the historical data compare well with the 1-hour recurrence intervals computed using the NOAA Atlas. Table 5 shows the recurrence intervals of the precipitation depths as identified in the RMC report (RMC, June 2008a). RMC's total rainfall depths for the 10-year, 1-hour and 25-year, 1-hour events (Table 5) are different than those calculated by Brown and Caldwell using the NOAA Atlas (Table 2) because the RMC values appear to have been developed by multiplying the peak 10-minute rain intensity for the event by six to estimate the 1-hour value (e.g. 0.21 in/10min * 6 = 1.3 in/hr). In addition, RMC applied the rainfall distribution that is indicated in Figure 8 and this distribution estimates a larger rainfall volume than a normal distribution curve. .

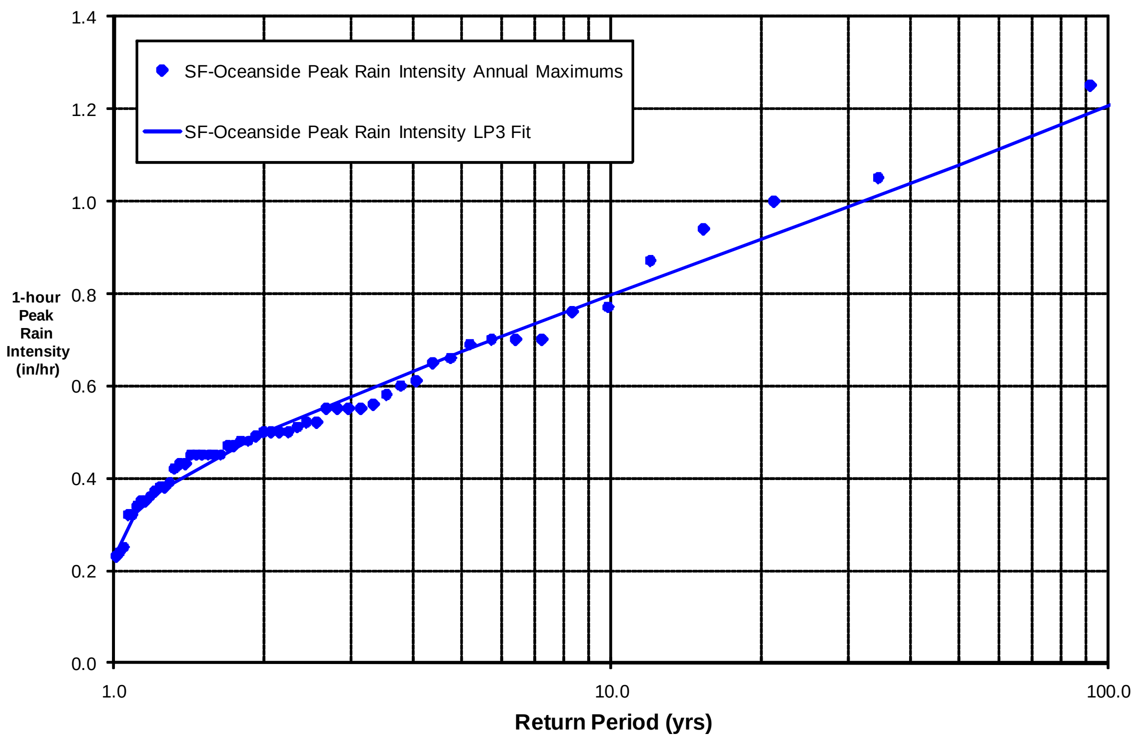

Figure 7. NCDC San Francisco-Oceanside Rain Data – Statistical Evaluation

Table 3. Rainfall Recurrence Intervals for NCDC SF-Oceanside Gauge		
Annual Probability	Return Period (year)	Peak 1-Hour Rain Intensity (in/hr)
0.5	2	0.50
0.2	5	0.67
0.1	10	0.80
0.04	25	0.96
0.02	50	1.08
0.01	100	1.20

Table 4. NOAA Rain Depths in inches for Various Recurrence Intervals[1]			
Recurrence Interval (year)	Storm Duration		
	1-hour	6-hour	24-hour
2	0.58	1.40	2.30
5	0.70	1.70	3.00
10	0.85	1.90	3.60
25	1.00	2.10	4.00
50	1.10	2.40	4.50
100	1.17	2.60	4.70

[1]NOAA Atlas 2, Volume 11

Table 5. RMC Precipitation Depths for 1-, 6-, and 4-hour Storms		
Duration	Precipitation Depth (inches) for Various Frequencies	
	10-year	25-year
1-hour	1.20	1.30
6-hour	1.90	2.10
4-hour (interpolated)	1.65	1.81

[1]*Technical Memorandum: Vista Grande Watershed Storm Drain Evaluation - Design Storm Comparison (RMC, June 2008a)*

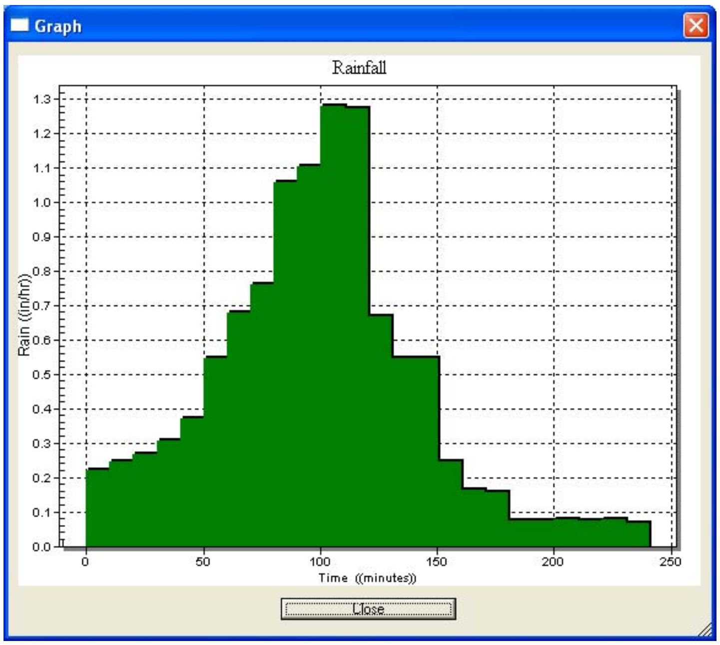

Figure 8. RMC Rainfall Distribution in the XP-SWMM Model for the 25-year, 4-hour Design Event

For comparison, Brown and Caldwell also evaluated the rainfall recurrence intervals for the NCDC SF-Downtown rain gauge (also known as the Mission-Delores gauge). This is the next closest NCDC gauge to the project site and has a more complete rainfall record, with data from 1948-2009. Table 6 and Figure 9 compare the SF-Downtown gauge to the SF-Oceanside gauge and RMC values. Overall, the NCDC gauges compare well with the SF-Downtown gauge predicting higher peak 1-hour rain intensities with the larger recurrence intervals. Figure 9 also shows that the historical data generate lower peak 1-hour rain intensities than the RMC 10-year, 4-hour and 25-year, 4-hour design events.

Table 6. Comparison of Rainfall Recurrence Intervals for NCDC Gauges							
Annual Probability	Recurrence Interval (yr)	Peak 1-hour Rain Intensity: SF-Oceanside Gauge (in/hr)	Peak 1-hour Rain Intensity: SF-Downtown Gauge (in/hr)	Ratio of SF-Downtown and SF-Oceanside Gauges Data	Peak 1-hour Rain Intensity - RMC Design Events (in/hr)	Ratio of SF-Oceanside Gauge to RMC Design Event Data	Ratio of SF-Downtown Gauge to RMC Design Event Data
0.5	2	0.50	0.50	1.0	---	---	---
0.2	5	0.67	0.71	1.1	---	---	---
0.1	10	0.80	0.88	1.1	1.2	0.7	1.5
0.04	25	0.96	1.12	1.2	1.3	0.9	1.4
0.02	50	1.08	1.32	1.2	---	---	---
0.01	100	1.20	1.54	1.3	---	---	---

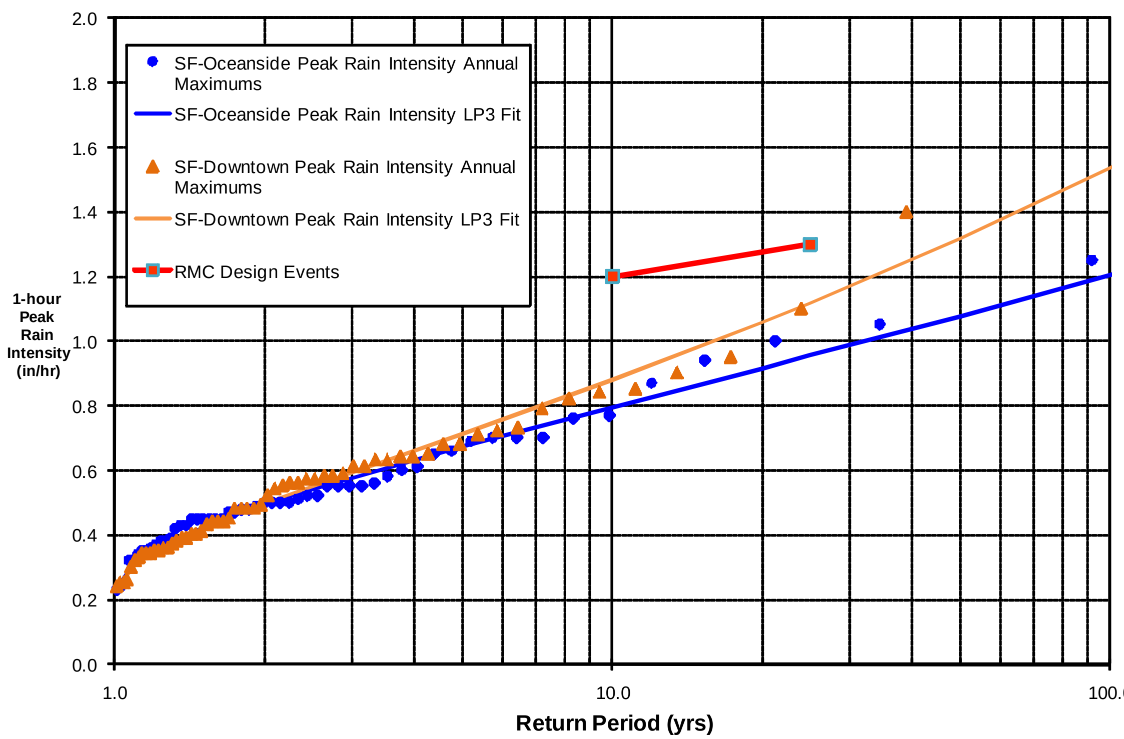

Figure 9. Comparison of SF-Oceanside and SF-Downtown Statistical Evaluations

Comparison of Design and Historical Events

Brown and Caldwell also compared two historical events from the SF-Oceanside data record to the RMC design events. Figure 10 and Table 7 compare the February 18-19, 1986 historical event to the 10-year, 4-hour design event, and Figure 11 and Table 8 compare the May 14, 1949 event to the 25-year, 4-hour event. These events were selected because they had the most comparable peak hour intensities and event durations. In both cases the rain and flows are larger for the design events than for the historical events.

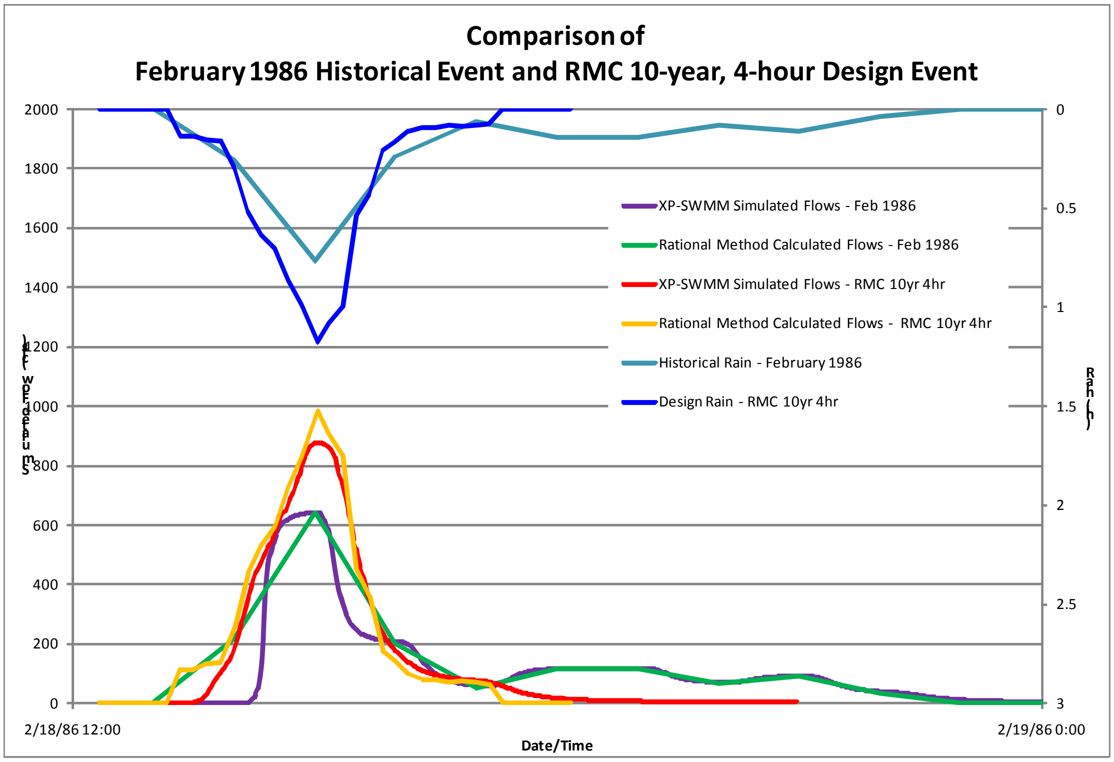

Figure 10. Comparison of SF-Oceanside February 1986 Historical Event and RMC 10-year, 4-hour Design Event

Table 7. Comparison of Simulated Flows for RMC 10-year, 4-hour Design Event vs. February 1986 Historical Event	
Event	Simulated or Estimated Peak Flow at Downstream End of Project Area [2084-C] (cfs)
RMC 10-year, 4-hour Design Event	
XP-SWMM Model: Feb 1986 Historical Event	640
Rational Method: Feb 1986 Historical Event	640
XP-SWMM Model: 10-year, 4-hour Design Event	880
Rational Method with 10-year, 4-hour Design Event (Peak 10-minute Intensity)	980
Ratio of XP-SWMM Model and Rational Method Peak Flows for Feb 1986 Historical Event	1.00
Ratio of XP-SWMM Model and Rational Method Peak Flows for 10-year, 4-hour Design Event	0.90
Ratio of XP-SWMM Model Peak Flows for 10-year, 4-hour Design Event and Feb 1986 Historical Event	1.38
Ratio of Rational Method Peak Flows for 10-year, 4-hour Design Event and Feb 1986 Historical Event	1.53

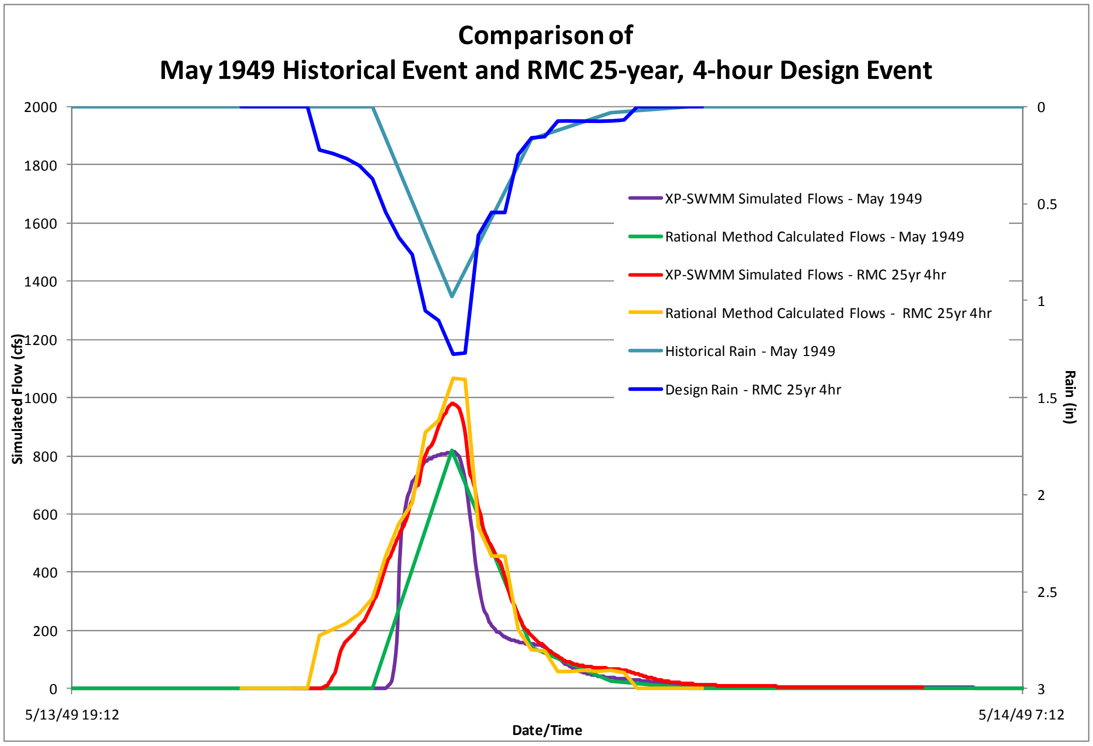

Figure 11. Comparison of SF-Oceanside May 1949 Historical Event and RMC 25-year, 4-hour Design Event

Table 8. Comparison of Simulated Flows for RMC 25-year, 4-hour Design Event vs. May 1949 Historical Event	
Event	Simulated or Estimated Peak Flow at Downstream End of Project Area [2084-C] (cfs)
RMC 25-year, 4-hour Design Event	
XP-SWMM Model: May 1949 Historical Event	810
Rational Method: May1949 Historical Event	820
XP-SWMM Model: 25-year, 4-hour Design Event	980
Rational Method with 25-year, 4-hour Design Event (Peak 10-minute Intensity)	1,070
RMC Results for Constrained Model: 25-year, 4-hour Design Event	930
RMC Results for Unconstrained Model: 25-year, 4-hour Design Event	1,660
Ratio of XP-SWMM Model and Rational Method Peak Flows for May1949 Historical Event	0.99
Ratio of XP-SWMM Model and Rational Method Peak Flows for 25-year, 4-hour Design Event	0.92

Table 8. Comparison of Simulated Flows for RMC 25-year, 4-hour Design Event vs. May 1949 Historical Event	
Event	Simulated or Estimated Peak Flow at Downstream End of Project Area [2084-C] (cfs)
RMC 25-year, 4-hour Design Event	
Ratio of Rational Method and RMC Results for Constrained Model Peak Flows for 25-year, 4-hour Design Event	1.15
Ratio of XP-SWMM Model and RMC Results for Unconstrained Model Peak Flows for 25-year, 4-hour Design Event	0.64
Ratio of XP-SWMM Model and RMC Reported Peak Flows (Constrained Model) for 25-year, 4-hour Design Event	1.05
Ratio of RMC Results for Constrained Model and Unconstrained Model Peak Flows for 25-year, 4-hour Design Event	0.56
Ratio of XP-SWMM Model Peak Flows for 25-year, 4-hour Design Event and May1949 Historical Event	1.21
Ratio of Rational Method Peak Flows for 25-year, 4-hour Design Event and May1949 Historical Event	1.30

Event Distributions

Tables 9 through 12 show which events during the historical record represent the highest 10 percent and lowest 10 percent in peak rain intensities and total rain depths. An event is defined as one in which the rainfall depth was at least 0.15 inches over six or more hours. As indicated in these tables, the October 12, 1962 event was ranked in the top 10 percent for both event peak intensity and total rain depth.

Also indicated in Tables 9 through 12 are the estimated rainfall volumes and peak flows for each event. The flows in these figures were calculated using the Rational Method, with a C value of 0.50 and an area of 1,670 acres. Figure 12 provides a distribution of the peak flows associated with each of the historical rain events. As indicated in this figure, the peak flows are less than 200 cfs for about 77 percent of the time. Figure 13 shows the distribution of the historical rain event volumes. Again, for 77 percent of the time the event runoff volumes are 20 MG or less. The average peak intensity was 0.19 inches per hour (in/hr), which resulted in an average total maximum flow of about 157 cfs. The average total rain depth was 0.68 inches, which resulted in an average runoff volume of 395 million gallons (MG). On average, the event duration was just over 14 hours.

For comparison, Table 13 summarizes the flows and volumes that were reported by RMC for the 10-year and 25-year, 4-hour design events (RMC, August 2006). Flows represented in Table 13 were calculated using the Rational Method, with a C value of 0.71 and an area of 1,670 acres. Note that the events from the historical rain data record have longer storm duration compared to the 4-hour design event that was used by RMC; as a result, the peak flows from the events listed in Tables 9 through 12 are much lower than the “theoretical” RMC predictions, such as is shown in Table 9 with the October 1962 storm.

Table 9. Ranking of Historical Rain Events by Total Event Intensity					
Rank	Storm Date	Rain Depth (in)	Duration (hrs)	Peak Intensity (in/hr)	Estimated Peak Flow (cfs)[1]
1	11/8/1949	1.46	72	1.25	1,044
2	10/12/1962	5.53	32	1.05	877
3	12/11/1995	4.40	36	1.00	835
4	5/14/1949	1.18	3	0.98	818
5	11/27/1970	1.90	20	0.94	785
6	12/22/1982	1.62	32	0.87	726

[1]Peak flow estimated using the Rational Method, Q = CiA, where i is the peak intensity of the event, A is 1,670 acres, and C is 0.5

Table 10 Historical Rain Events with the Lowest Peak Intensity					
Rank	Storm Date	Rain Depth (in)	Duration (hrs)	Peak Intensity (in/hr)	Estimated Peak Flow (cfs)[1]
1410	11/22/1961	0.20	9	0.03	25
1411	10/30/1974	0.17	13	0.03	25
1412	1/8/1989	0.16	18	0.03	25
1413	12/3/1995	0.20	15	0.03	25
1414	4/24/1963	0.18	30	0.02	17
1410	11/22/1961	0.20	9	0.03	25

[1]Peak flow estimated using the Rational Method, Q = CiA, where I is the peak intensity of the event, A is 1,670 acres, and C is 0.5

Table 11. Historical Rain Events with the Highest Total Rain Depth					
Rank	Storm Date	Rain Depth (in)	Duration	Peak Intensity (in/hr)	Estimated Runoff Volume (MG)
1	10/12/1962	5.53	32	1.05	251
2	1/20/1967	5.15	41	0.50	234
3	12/21/1955	4.75	58	0.45	215
4	2/2/1998	4.71	30	0.59	214
5	1/29/1963	4.68	67	0.27	212

Table 12. Historical Rain Events with the Lowest Total Rain Depth					
Rank	Storm Date	Rain Depth (in)	Duration	Peak Intensity (in/hr)	Estimated Runoff Volume (MG)
1410	12/20/1957	0.16	9	0.05	7
1411	12/22/1986	0.16	6	0.05	7
1412	10/10/1969	0.16	6	0.04	7
1413	11/12/1971	0.16	9	0.04	7
1414	1/8/1989	0.16	18	0.03	7

Table 13. Design Storm Comparison for Discharges to Vista Grande Canal[1]		
Parameter	10-year, 4-hour Design Storm	25-year, 4-hour Design Storm
Peak Flow (cfs)[1]	1,520	1,660
Total Volume (MG)	57.7	63.8

[1]Technical Memorandum: Vista Grande Watershed Storm Drain Evaluation - Design Storm Comparison (RMC, June 2008a); peak flow is estimated using a C value of 0.71 and an area of 1,670 acres.

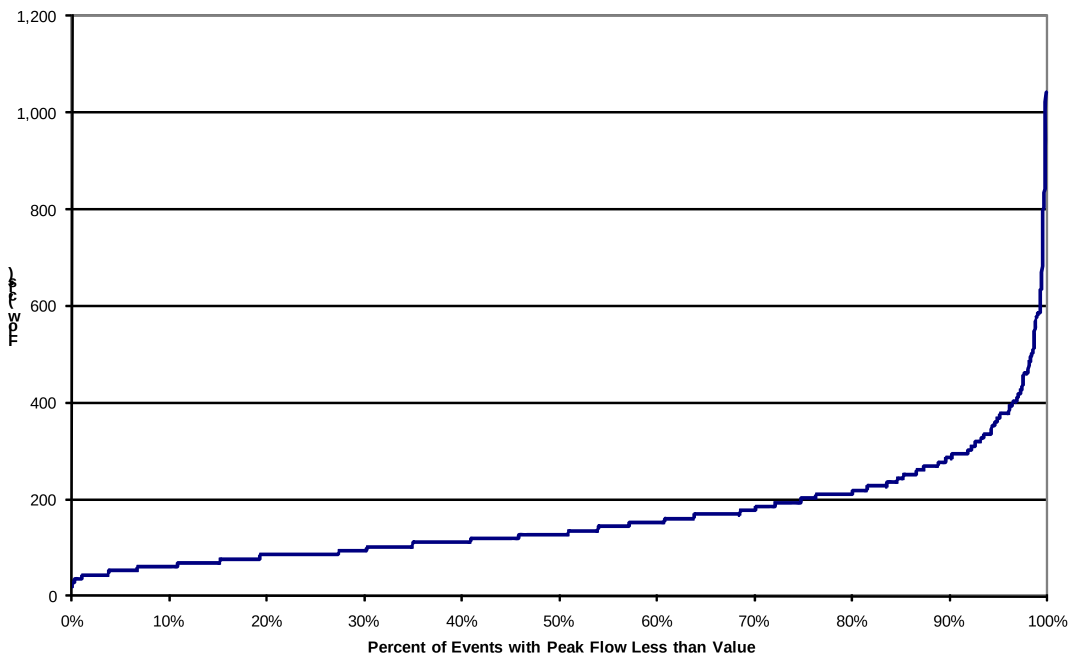

Figure 12. Event Peak Flow Distribution

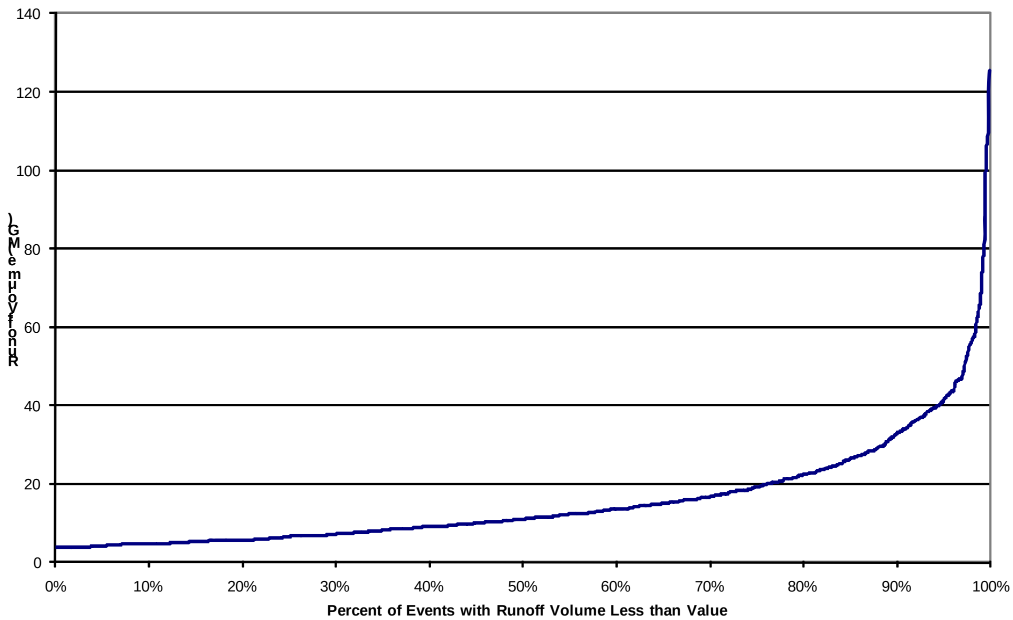

Figure 13. Event Volume Distribution

Flow Distribution

Figure 14 is a flow duration curve developed using the estimated flows for the historical record. As indicated in Figure 15, which provides a close-up view of Figure 14, flows of 10 cfs or less are estimated to occur 97 percent of the time.

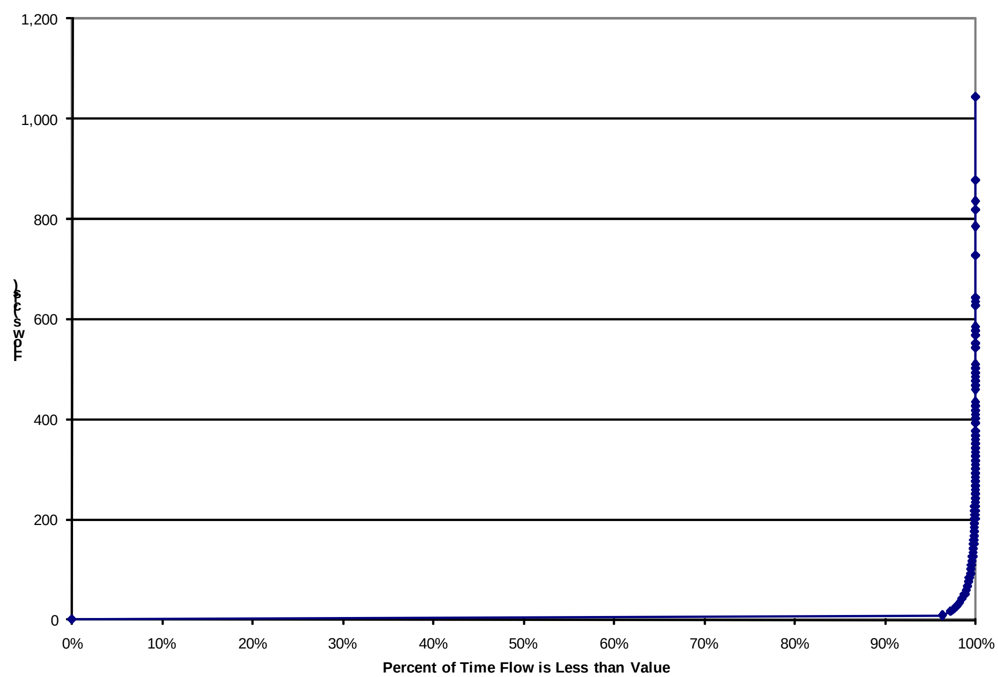

Figure 14. Flow Duration Curve – Overall

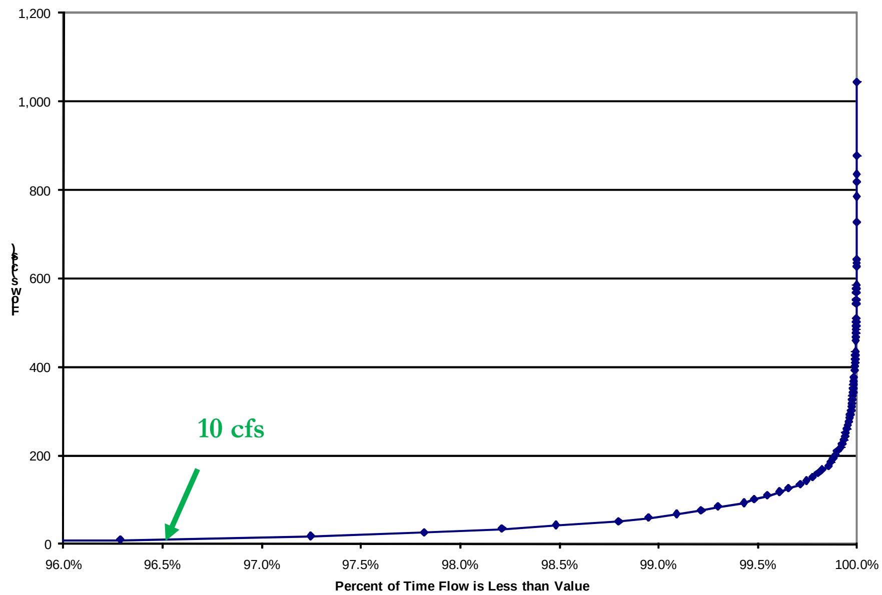

Figure 15. Flow Duration Curve –96 Percent to 100 Percent

Estimated Volume to Lake Merced

The average annual volume of flow estimated for the Vista Grande project area during the 55-year historical record of rain from the NCDC San Francisco-Oceanside was approximately 395 MG. According to the proposed design, the first 1-inch of rainfall for the year will be diverted to the ocean for water quality purposes ("first flush"); a 1-inch rainfall generates approximately 23 MG of runoff volume. As a result, the average annual volume of runoff that can be anticipated to be diverted to Lake Merced is approximately 372 MG or 1,142 acre feet.

Conclusions

The results presented above lead to several key conclusions:

- The actual rainfall data compare well to the values reported in the NOAA Atlas, but the values from RMC's analyses are higher. For example, the RMC 10-year, 1-hour recurrence interval is comparable to the NOAA 100-year, 1-hour recurrence interval, a more conservative assumption. Therefore the actual rain data may predict lower peak flows compared to those from RMC's analyses.
- The work presented in this technical memorandum builds on RMC's analyses for a "constrained" watershed, or one with improvements to drain water from the watershed more rapidly.
- The XP-SWMM model has prohibitively long simulation times and therefore is not recommended for simulating flows for a long-term historical rainfall record.
- Flows that were calculated using the Rational Method, calibrated against historical event flows that were simulated in XP-SWMM, are similar to those generated by the XP-SWMM model and therefore the Rational Method can provide a reasonable estimate of long-term simulated flows (e.g those based on the 40 or more years of recorded rain data).
- With the reconstructed Vista Grande tunnel (estimated capacity of 500 cfs) operating, Daly City could route the vast majority of flow directly to the Pacific Ocean.

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ATTACHMENT 2 Preliminary Project Alternatives Evaluation

Attachment 2 Preliminary Project Alternatives Evaluation

Project Alternatives Evaluation

		Rating Points	Alternative 5B-4 MG: Tunnel driven from canal	Alternative 5B-4 MG: Tunnel driven from Ft. Funston temporary construction shaft	Alternative 6-4 MG: Tunnel driven from canal	Alternative 6-4 MG: Tunnel driven from Ft. Funston temporary construction shaft	Alternative 7- 4 MG: Tunnel driven from canal	Alternative 7- 4 MG: Tunnel driven from Ft. Funston temporary construction shaft	Lk Merced Alternative: Tunnel driven from Ft. Funston temporary construction shaft
Deliver Public Benefits									
	Community benefits	Satisfaction Rating: 1 (satisfied) to 5 (dissatisfied)	3	2	2.5	1.5	2	1	1
	Public inconvenience (temporary, interim, & permanent)		4	3	4.5	3.5	5	4.5	3
	Water Re-Use Opportunities		4	4	4	4	4	4	1
	Flood protection	Satisfaction Rating: 1 (completely) to 5 (minimal)	1	1	1	1	1	1	2
	Reduce potential for overflow into Lake Merced		2	2	2	2	2	2	1
	Debris screening		1	1	1	1	1	1	1
	Wetlands enhancement	Satisfaction Rating: 1 (complements) to 3 (supports) to 5 (no support)	3.5	3	2.5	2.5	2.5	2	2
	Storing & Managing Lake Merced Lake Levels		3	3	3	3	3	3	1
	Groundwater recharge potential		5	5	5	5	5	5	2
See Sensitivity Matrix below	**Score (sum of ratings)**		**26.5**	**24**	**25.5**	**23.5**	**25.5**	**23.5**	14
Operability	Facility operations	Operability Rating: 1 (convenient) to 5 (inconvenient)	1.5	1.5	1.5	1.5	1.5	1.5	3
	Stormwater screening effectiveness	Operability Rating: 1 (completely) to 5 (minimal)	2.5	2.5	2	2	1.5	1.5	3
	Stormwater screening maintainability	Operability Rating: 1 (convenient) to 5 (inconvenient)	2	2	2	2	2	2	2
See Sensitivity Matrix below	**Score (sum of ratings)**		**6**	**6**	**5.5**	**5.5**	**5**	**5**	**8**
Environmental Compliance	Impacts on the environment	Environmental Impact Rating: 1 (minimal) to 5 (significant)	4	3	4.5	3.5	5	4	3
	Effects on sensitive species		3	3	4	3	4	3	3.5
	NEPA/CEQA requirements	Permitting Rating: 1 (simple and well understood) to 5 (complex and time consuming)	4	3	4	3	4	3	4
	Water Quality Permit requirements (RWQCB)		3	3	3	3	3	3	5
See Sensitivity Matrix below	**Score (sum of ratings)**		**14**	**12**	**15.5**	**12.5**	**16**	**13**	**15.5**
Minimize Land Acquisition Costs	Land acquisition and right-of-way requirements	Land Use Rating: 1 (simple and well understood) to 5 (complex and time consuming)	5	5	5	5	3	3	3
	Temporary easement requirements		5	5	5	5	3	3	3
	Utility interference issues and relocation requirements		3	2.5	3	2.5	3	2.5	3.5
See Sensitivity Matrix below	**Score (sum of ratings)**		**13**	**12.5**	**13**	**12.5**	**9**	**8.5**	**9.5**
Maximize Constructability	Construction working space and access	Constructability Rating: 1 (simple) to 5 (complex)	4	3.5	4	3.5	4	3.5	2.5
	Spoils management		4	1	4	1	4	1	3
	Constructability		5	3	5	3	3.5	3	3.5
	Construction Duration		4	4.5	3.5	4	3	3.5	2.5
	SW Intercepts incl force main		2	2.5	2	2.5	2	2.5	2.5
	Anticipated Ground Conditions		2.5	3	2.5	3	2.5	3	3
See Sensitivity Matrix below	**Score (sum of ratings)**		**21.5**	**17.5**	**21**	**17**	**19**	**16.5**	**17**
Minimze Lifecycle Costs	Relative construction costs from relative cost sheet	Cost Ranking: 1 (lowest cost-risk) to n (highest cost-risk) x (ratio of Alternative cost to least cost alternative)	18	16	19	17	20	18	10
	Relative O&M costs-- debris removal & disposal, water treatment, pump maintenance & pumping costs	O&M Rating: 1 (low cost-risk) to 5 (high cost-risk)	3	3	3	3	3	3	4
See Sensitivity Matrix below	**Score (sum of ratings)**		**21**	**19**	**22**	**20**	**23**	**21**	**14**
	Overall Score (x10)		See Sensitivity Matrix below	See Sensitivity Matrix below	See Sensitivity Matrix below	See Sensitivity Matrix below	See Sensitivity Matrix below	See Sensitivity Matrix below	See Sensitivity Matrix below
			4	2	6	3	7	5	1
		Estimated cost w/ contingencies ($M)	**201.5**	**180.2**	**209.8**	**189.4**	**219.4**	**202**	**110.4**

Weighting Sensitivity Matrix

	Deliver Public Benefits	Operability	Environmental Compliance	Minimize Land Acquisition Costs	Maximize Constructability	Minimize Lifecycle Costs	Description		5B canal	5B Ft. Funston	6 canal	6 Ft. Funston	7 canal	7 Ft. Funston	Lk Merced
									4						
A	100%	100%	100%	100%	100%	100%	Equal weight distribution	**Weighted Overall Score**	1023	913	1025	912	974	878	780
								Rank	**6**	**4**	**7**	**3**	**5**	**2**	**1**
B	15%	5%	10%	15%	5%	50%	65% cost + 35% non-cost	**Weighted Overall Score**	193	175	197	179	194	178	133
								Rank	**5**	**2**	**7**	**4**	**6**	**3**	**1**
C	33%	5%	10%	15%	5%	33%	48% cost + 52% non-cost	**Weighted Overall Score**	202	183	203	184	199	182	133
								Rank	**6**	**3**	**7**	**4**	**5**	**2**	**1**
D	33%	10%	10%	17%	5%	25%	35% cost + 65% non-cost	**Weighted Overall Score**	193	176	193	176	187	171	129
								Rank	**7**	**3**	**6**	**4**	**5**	**2**	**1**
E	0%	0%	0%	0%	0%	0%		**Weighted Overall Score**	0	0	0	0	0	0	0
								Rank	**1**	**1**	**1**	**1**	**1**	**1**	**1**
								Consolidated Rank	25.0	13.0	28.0	16.0	22.0	10.0	5.0
								Overall Rank	**6**	**3**	**7**	**4**	**5**	**2**	**1**