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SAUSALITO FLOOD STUDY

Prepared for

The City of Sausalito

Prepared by

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PWA REF. # 1598

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1. INTRODUCTION

1.1 BACKGROUND

This report documents a study to analyze the flood potential and alternative means of reducing flood potential in the area from Harbor Drive to Gate 5 road in Sausalito, Marin County, California. Figure 1-1 shows a map of the study area.

Flooding due to high tides and rainfall runoff has been a problem for the property owners in the area from Harbor Drive to the Gate 5 road for some time. The flood zones seem to be moving landward and could be aggravated in the future. In an attempt to improve the situation, many property owners are proposing to fill their property, raise building elevations and other approaches to reduce the flood impacts. The City of Sausalito decided to evaluate the flood problem and remedial measures before any steps for flood reduction are taken. Philip Williams & Associates (PWA) were contracted to carry out this study.

The present study is aimed at evaluating potential flood control elements to reduce the flooding problem. Flood control elements studied are the following:

1. One-way tide gate to prevent tidal flow into the storm drains
2. Pump station to pump extreme runoff to the bay
3. Levee system to prevent coastal flooding

These alternatives were reviewed based on their flood reduction performance and in consideration of the Federal Emergency Management Agency (FEMA) regulations.

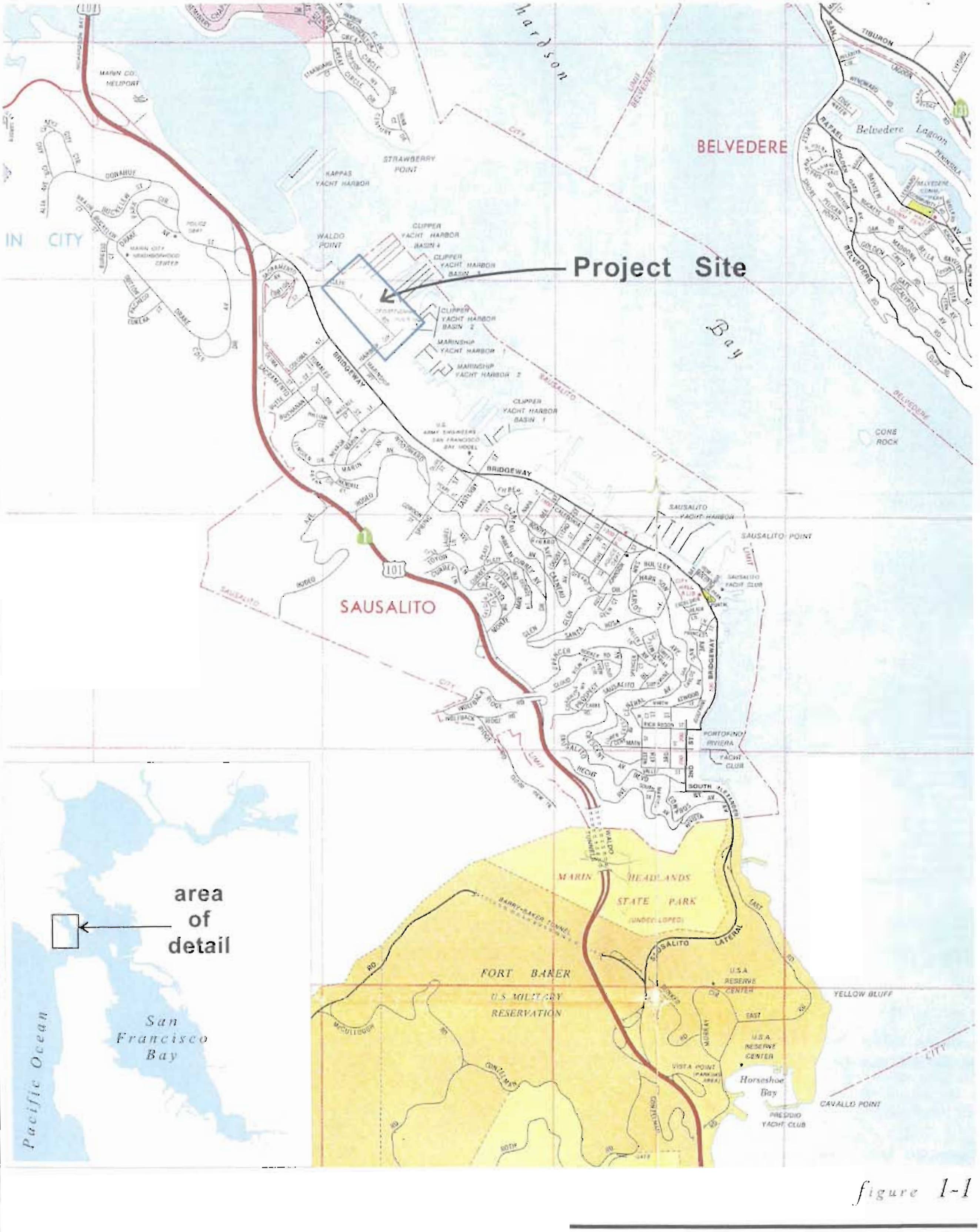

figure 1-1

Sausalito Flood Study

Site Vicinity Map

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PWA

2. REVIEW OF PRIOR STUDIES AND PROJECT DATA

2.1 PRIOR STUDIES

Prior studies in the area have been performed to assess flood impacts of raising site grades on nearby property and the flood plain. PWA has reviewed these prior studies on flood control and mitigation and their findings related to the present study. In 1998, PWA conducted a preliminary hydrologic evaluation of impacts to flooding potential by proposed development at 401-413 Coloma Street and 310 Gate 5 Road, Sausalito (PWA, 1998 a). In this study, the potential flood increase on the adjacent property due to raising ground elevations of the subject property was considered. More specifically, three events with a return period of 100 years or higher were considered: (1) 100-year tide with 10-year run-off event, (2) average annual extreme tide with 100-year run-off event, and (3) Bay water level (equal to lowest existing elevation on site) with 100-year run-off event. The flood storage vs. elevation curves were developed in this process, and the potential increase in flood levels due to filling the property was found to be on the order of hundredths of a foot, with or without allowing flooding under the building at 310 Gate 5 Road. Additional analyses were performed subsequently with updated topographic information (PWA, 1998 d) for more tide and runoff event combinations with different return periods.

Potential strategies to alleviate flooding problems in the Gate 5 Road area were also discussed in a prior PWA report (PWA, 1998 b). Specifically, installation of a tide gate to reduce flooding was analyzed. This study was performed with more recent topographic information (1998 topo). This option will be further discussed as an alternative in the present study.

The following table shows the tide and runoff combinations considered in a previous study and the resulting flood elevations assuming that the buildings are flooded (PWA, 1998 c).

Table 2-1. Summary of Hundred Year Flood Events and Existing Flood Elevations

100-year Event	Tide (feet MLLW)	Runoff (cu. ft)	Existing Flood Elevation (feet MLLW)
100-year tide, 10-year runoff	8.7	107,000	9.0
Average annual extreme tide and 100-year runoff	7.6	176,000	8.4

Runoff values shown in Table 2-1 had been obtained by the Rational method and a simple triangular hydrograph. The estimated flood elevations with runoff are somewhat conservative since the total runoff was added on top of the tide. Greater total runoff volumes are possible for a given event, based on a longer duration but lower intensity rainfall event (Section 5.2). Tides at Sausalito are described in the next section.

2.2 DATA REVIEW

Tidal datums for Sausalito (9414806 tide gage) were available from NOS and NOAA web sites, which are shown in Table 2-2.

Table 2-2. Tide Statistics at Sausalito (Tidal epoch 1960 -1978)

	Elevation (feet)	
	Relative to MLLW	**Relative to NGVD29**
100-year tide	8.70	5.90
10-year tide	8.20	5.40
MHHW	5.67	2.90
MHW	5.08	2.31
MTL	3.10	0.33
NGVD1929	2.77	0.00
MLW	1.13	-1.64
MLLW	0.0	-2.77

Note: 100-year and 10-year estimates are from USACE (1984) for tide gage 4806 without correction for sea level rise since the estimates were made.

The following data were also reviewed in the study:

1. 1998 Aerial topographic survey map with 1-foot contours.
2. FEMA Flood Insurance Study (FIS), Flood Insurance Rate Map (FIRM) and related hydrologic and hydrographic data.
3. Grade changes since 1998.
4. Storm drain capacity.
5. Coloma Street outfall survey.
6. FEMA regulations on proposed changes to the floodplain.
7. Sea level rise and land subsidence data.

2.3 HISTORICAL FLOODING

Extreme floods were reported in 1983 and 1998 during the severe El Niño winter storms. High winds and heavy rains combined with abnormally high tides caused flooding in many areas along San Francisco Bay, including Sausalito. Figure 2-1 shows two photos depicting the extreme flooding and waves at Sausalito and south of Sausalito at Fort Point, in February 1998. The tides related to this extreme event are shown in Figure 2-2 measured at the Presidio tide gauge in San Francisco. The measured tide of approximately 8.7 feet MLLW was more than 2-feet higher than the predicted tide, on February 06, 1998 between 6:00 a.m. and 9:00 a.m. In addition, the water levels were continuously higher than predicted by about 0.7 ft throughout the winter months. Figure 2-3 shows the predicted and measured tides during November 1997 to February 1998.

A huge wave breaks over the seawall at Fort Point under the Golden Gate Bridge and crashes onto a parked car on February 1, 1998. Throughout the following week, high winds and heavy rains combined with abnormally high tides to wreak havoc in the San Francisco Bay region. Inset photo shows a worker hauling sandbags through floodwaters in Sausalito, north of the Golden Gate Bridge, on February 7. U.S. Geological Survey scientists have shown that these extreme conditions were the direct result of the 1997—98 El Niño atmospheric phenomenon. (Photos by Lea Suzuki and Vince Maggiora / copyright San Francisco Chronicle

Source: Internet / San Francisco Chronicle

figure 2-1

Sausalito Flood Study

Pictures of Flooding and Waves / Feb1998

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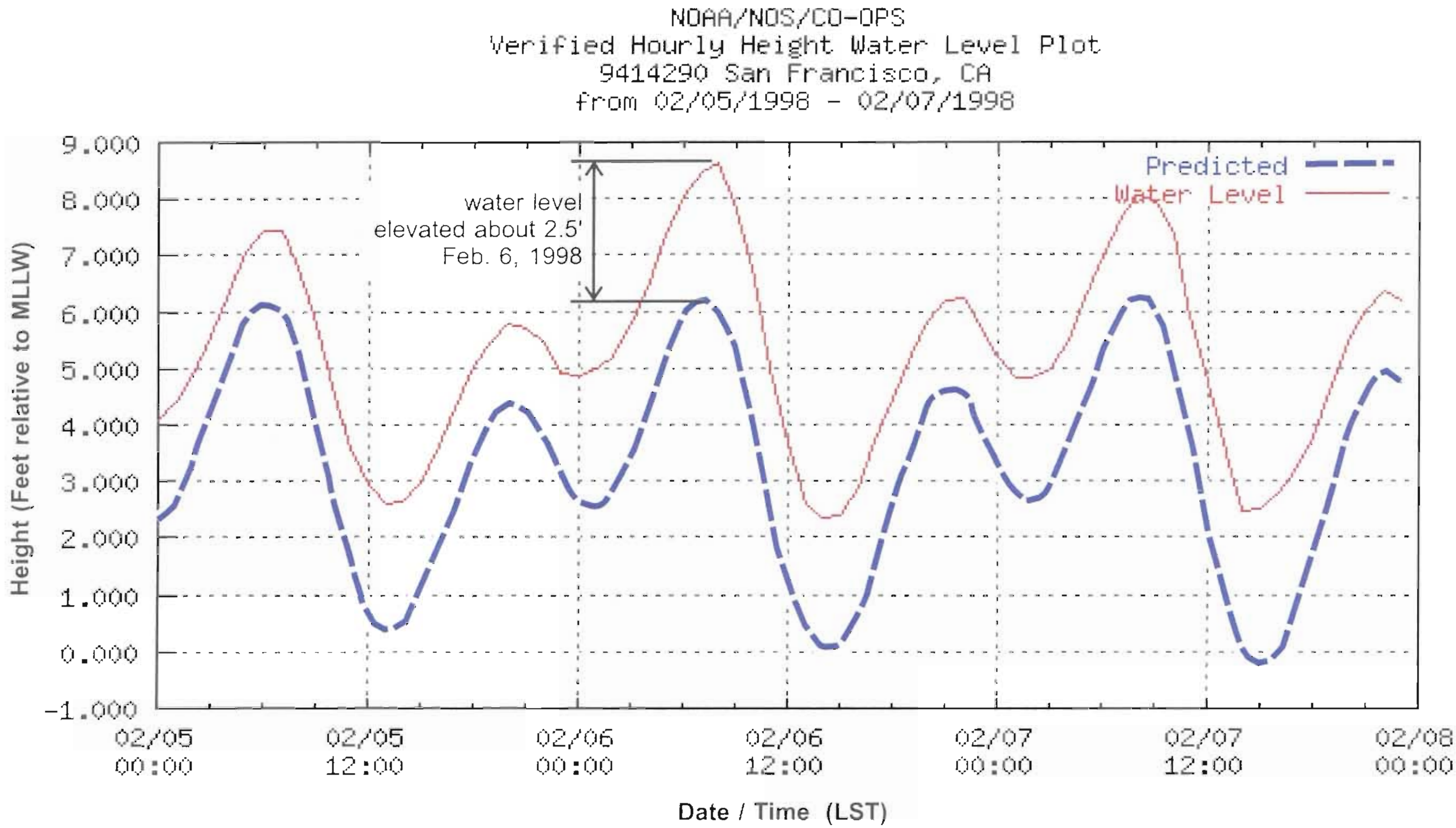

figure 2-2

Sausalito Flood Study

Extreme High Tides during February 1998

Source: NOAA/NOS/CO-OPS

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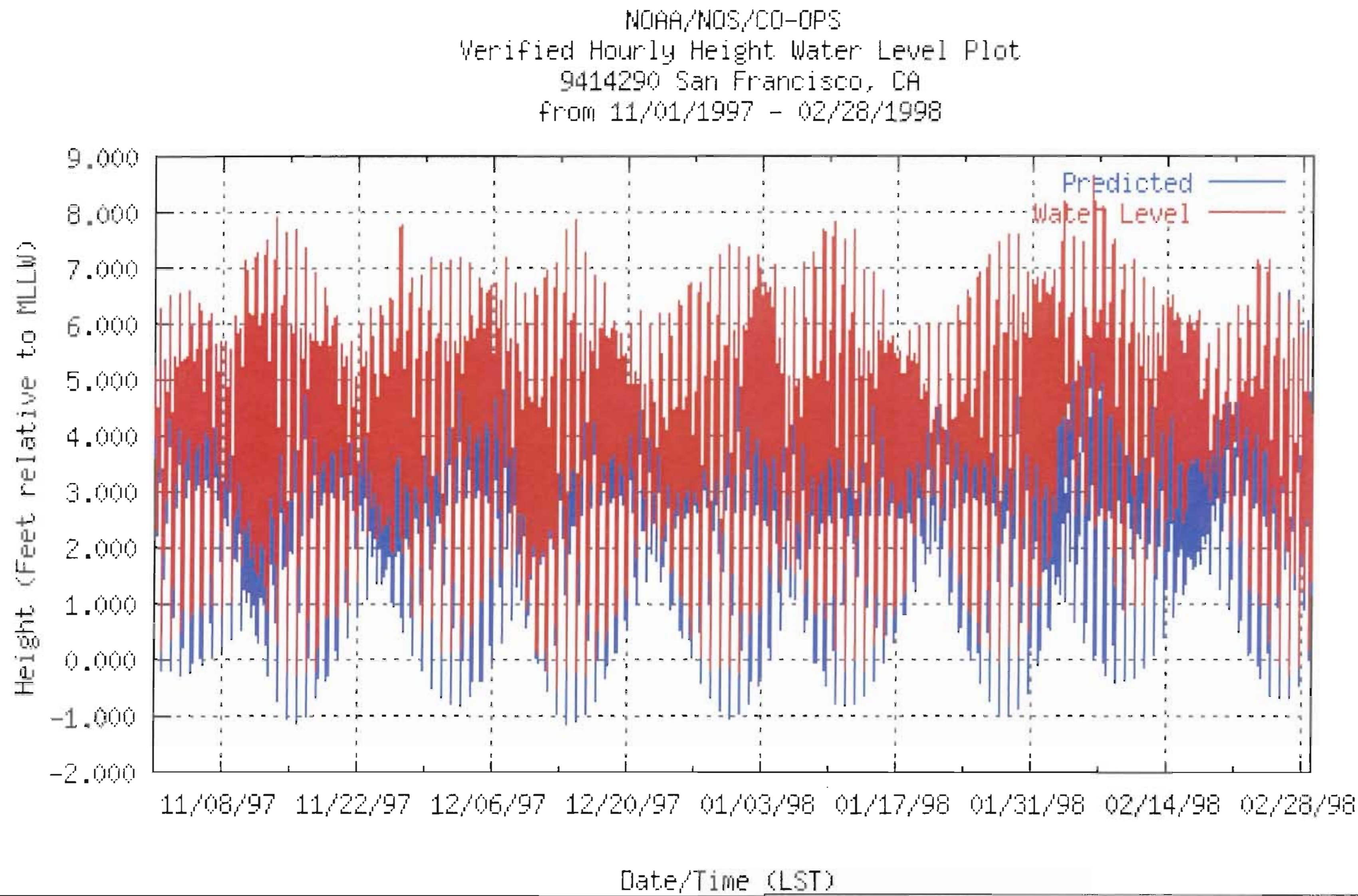

Source: NOAA/NOS/CO-OPS

figure 2-3

Sausalito Flood Study

Extreme High Tides during El Nino 1997/1998

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3. FEMA FLOOD STUDY AND REGULATIONS

3.1 REVIEW

3.1.1 Flood Insurance Study (FIS) and Flood Insurance Rate Map (FIRM)

The FIS and FIRM of September 1980 are the most recent FEMA flood studies. The base flood elevation is 6.2 feet (6 feet to the nearest whole foot) for the study area (FIS, 1980). According to the FIS, this elevation is the higher value of the maximum tides or the tsunami runup. The Gate 5 Road shown in the FIRM (1980) is within the flood boundary defined as a Special Flood Hazard Area (zone V1). Associated FEMA hydrologic and hydrographic data were reviewed, but these documents could not be used to derive the flood zone limits used in the FIS. FEMA (1993) states that V zones are defined from approximate studies while VE zones are defined from more detailed studies, which implies that the current FIS (1980) is an approximate study.

In the current methodology, VE zones are Coastal High Hazard Areas where wave action and/or high velocity water can cause structural damage in the 100-year flood. Primarily defined by (1) the area where 3-foot or greater wave heights could occur, (2) the area where the eroded ground profile is 3 feet or more below the representative runup elevation, and (3) the entire primary frontal dune (FEMA, 1995). VE zones can be extended some distance landward from this point to account for likely wave impacts through the flood's course (about 30 feet or more).

Given that the site grades are less than +9 feet MLLW elevation (lower than the Base Flood Elevation [BFE]), it is likely that a more detailed study would result in an A-zone designation for much of the area presently mapped as V zone. A zone is defined as areas of inundation by the 100-year flood, including wave heights less than 3 feet and runup elevations less than 3 feet above the ground. Accordingly, with current standards, the flood zones may be different from those shown on the FIRM of 1980.

3.1.2 Regulations

As part of the agreement for making flood insurance available in a community, the National Flood Insurance Program (NFIP) requires the community to adopt floodplain management ordinances containing certain minimum requirements intended to reduce future flood losses. The community is also responsible for submitting data to FEMA reflecting revised flood hazard information so that NFIP maps can be revised as appropriate. This will allow risk premium rates and floodplain management requirements to be based on current data (FEMA, 1999). Accordingly a Conditional Letter of Map Revision (CLOMR) may be applied, which is a letter from FEMA commenting on whether a proposed project, if built as proposed, would justify a map revision.

3.2 RECOMMENDATIONS

In addition to changes to the floodplain resulting from the proposed flood control elements, the hydrology changes can be incorporated in a map revision. The FIS and FIRM do not include data related to the more recent extreme El Niño flooding events of 1983 and 1998 or other more recent data. Also, it is not clear if the Special Flood Hazard Areas defined as V1 zones in the FIRM, are consistent with current FEMA methods (FEMA, 1995). A Flood Insurance Study (FIS) that takes into account more recent data and methods is recommended. This may result in changes to the zones defined for insurance purposes.

4. FLOOD STORAGE - ELEVATION CURVES

Flood-storage elevation curves for the Gate 5 Road area were generated in 1998 as part of an earlier study (PWA, 1998 c). These curves were to be updated to include more recent grade changes and the consequent changes to the flood storage-elevation curves. Figure 4-1 shows the boundaries of the watershed considered in the flood storage elevation assessment. We note, however, that at high flood elevations there is a possibility for the floodwater to flow into adjacent areas (specially to the south of Harbor Drive) through low points on the watershed. Therefore, the boundaries shown on Figure 4-1 are approximate.

There has been one major filling in the area south of Harbor Drive at the Clipper Yacht Harbor. The parking lot south of Harbor Drive was raised from approximately +5 ft NGVD to +8 ft NGVD. The filled area was not considered in the evaluations of flood storage in the earlier study (PWA, 1998 c); therefore, it will not be included in the Flood Storage-Elevation curve. The Flood Storage-Elevation curve for the existing condition is shown in Figure 4-2.

4.1 INFILTRATION TEST OF THE COLOMA DRAIN OUTFALL SYSTEM

An infiltration test was performed on August 14, 2002 to test for leaks in the storm drain system and to re-evaluate the actual storm drain capacity. Leaks in the system allow ground water in the soil to enter the drainage system. The test was performed at a low tide elevation by installing a temporary barrier and cutting off the rising tide from the storm drain near the bayward end of the 265 Gate 5 Road property. Details of the test were communicated to the City via a memo, which is shown in Appendix A. The measured change in water level was 0.12 inches/hour. Based on the estimated free water surface area and assuming there was no seepage of water through the temporary barrier, the corresponding volume of groundwater infiltration calculated for 6 hours is 313 cubic feet.

The test was performed successfully, and the results indicate that the rate of infiltration is not very significant. Therefore, the infiltration test results indicate that the tide gate installation is operating effectively. The total storm drain capacity of the Coloma storm drainage system is about 31,363 cubic feet (PWA, 1998 c). The infiltration volume of 313 cubic feet during a tide-cycle (6 hours, between high tide and low tide) is approximately 1% of the total storage capacity of the system. The net storage capacity after considering infiltration is about 31,000 cubic feet.

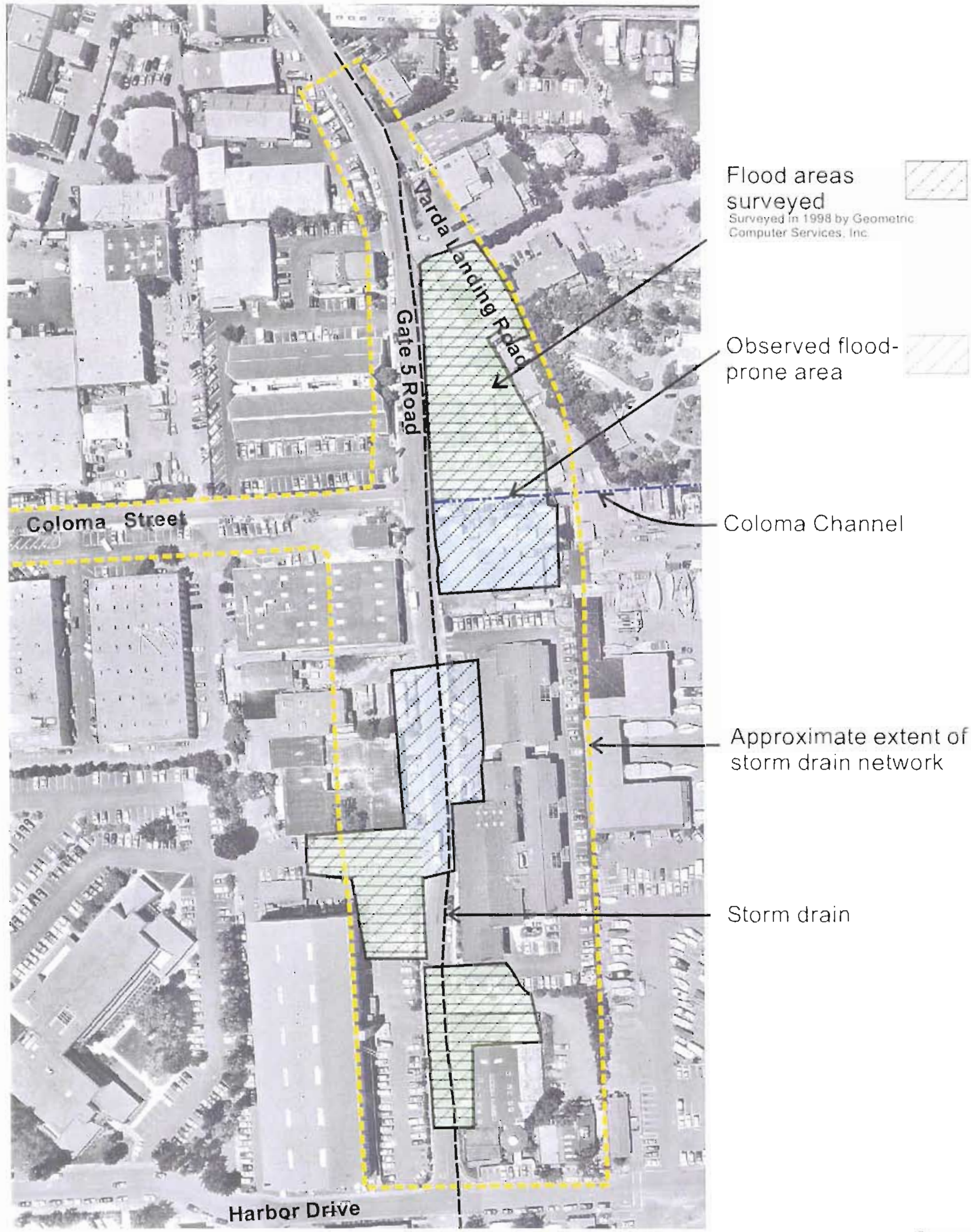
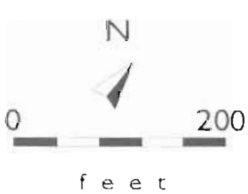


figure 4-1

Sausalito Flood Study

Existing Storm Drain Network & Watershed Area for Flood Storage



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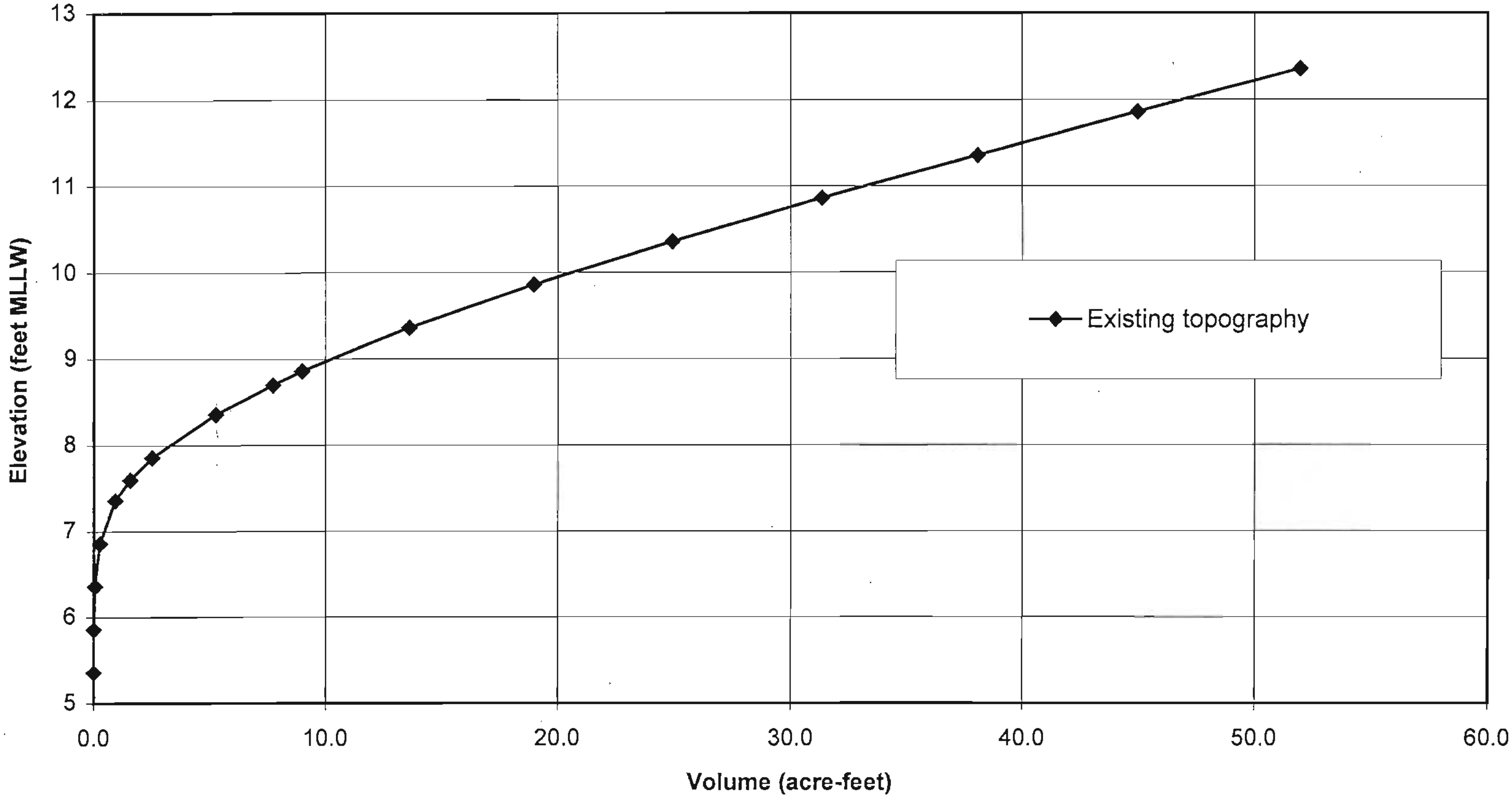

Source: PWA (1998 c)

figure 4-2

Sausalito Flood Study

Flood Storage Elevation Curve

(buildings not flooded)

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5. ANALYSIS OF FLOOD CONTROL ELEMENTS

Flooding in the Gate 5 Road up to Harbor Drive is caused by the interaction between tidally driven flooding and rainfall runoff-driven flooding, or by high tide flooding alone. Three flood control elements were selected to deal with the flooding problem:

1. One-way tide gate (flap gate) installed in the Coloma Channel, at the outfall or at the bay end of 265 Gate 5 Road Property,
2. Pump station to pump excess runoff in to the bay, and,
3. Levee system to prevent overtopping of extreme bay tides.

These elements are discussed in detail below.

Figure 5-1 shows a schematic diagram of the flood protection scenarios. There are 3 different scenarios for flood protection using the above flood control elements:

1. When the bay still water level (SWL) < Elevation A, a levee does not help. One-way flood-gate controls the tide. Runoff can initially be stored in the storm drain system. For higher runoff events, the pump station is necessary to pump excess runoff into the bay.
2. When the bay SWL > Elevation A, a levee is needed to prevent direct coastal flooding. Elevation B is the lowest elevation to prevent overtopping of water due to extreme tides and waves.
3. When an extreme tide and a high runoff event occurs at the same time, all of the above flood control elements are needed.

Contours of the 1998 topographic map indicate that the elevation A is about +7.0 feet MLLW. The elevation B is +9.0 feet MLLW, which is the base flood elevation given in the FIS.

5.1 ONE-WAY TIDE GATE AT THE COLOMA OUTFALL

A one-way tide gate could inhibit bay water from entering the storm drain system at high tide, thereby freeing storage space for runoff water to be stored in the drainage system until the tide recedes. The storm drain capacity had been estimated in an earlier study (PWA, 1998 c). The reduction in storage capacity was estimated by a ground water infiltration test (Section 4), which showed less than 1% reduction in storage due to groundwater infiltration. The reduced net storage available is about 31,000 cubic feet. This translates to 29% of the 10-year and 18%of the 100-year runoff volumes calculated by the Rational method (see Table 2-1 for runoff volumes). Longer duration storms can yield higher total runoff volumes but result in lower peak runoff rates (see Section 5.2).

In the present study, two possible locations are identified for the installation of a one-way tide gate as shown in Figure 5-2. The tide gate location T1 is at the end of 265, Gate 5 Road property. The owner of the property at 265 Gate 5 Road had shown interest in this option to mitigate for flood storage losses when raising the site grade. The other location, T2, is at the outfall near the bay. The second option maximizes storage for storm water allowing an additional 14,400 cubic feet of storage.

In the 1998 PWA study (1998 c), a tide gate design was considered. A schematic plan and elevation view is shown in Figure 5-3. In this conceptual design, several tide gates in a single headwall was proposed. A schematic of the tide gate head wall is shown in Figure 5-4.

Based on preliminary analysis, the construction cost of the tide-gate is estimated to be about $100,000 and includes:

- Dewatering and sand bagging to allow placement of pre-cast vault,
- Earth work excavation and disposal, shoring,
- Pile driving,
- Pre-cast vault,
- Tide gates,
- Landscaping and fencing, and
- Debris rack.

In addition, maintenance would be required for proper functioning of the tide gate.

5.2 PUMP STATION

A storm water pump system is often designed to discharge the estimated peak flow rate from runoff. The peak flow rate was estimated by the Rational method using the equation:

$$Q_p = \Sigma\, CIA,$$

where C=Runoff Coefficient, I = Rainfall Intensity and A= Watershed Area.
The peak flows for 100-year and 10-year return periods were calculated as 366 cfs and 222 cfs. Assuming a triangular hydrograph and a time of concentration of 10 minutes, total runoff volumes were estimated for the above two events as 176,000 and 107,000 cubic feet, respectively.

Storm drains can contain about 31,000 ft^3 of runoff; therefore, the required pumping volume is only 145,000 ft^3 of runoff for the 100-year event. An average pumping rate of 150 cfs was calculated assuming the pumping time is the same as the base time of the triangular hydrograph (16 minutes). Conditions were also checked for longer duration storms. Precipitation frequency data were obtained from the NOAA Atlas 2 web site (NOAA, 1973). Precipitation for 100-year / 6-hour storm and 10-year / 6-hour storm are 2.58 inches and 1.8 inches, respectively. Storm water runoff volume estimated using the Curve Number method are 313,200 ft^3 and 185,600 ft^3 for the 100-year and 10-year / 6-hour storms. Although these volumes are higher than those from the Rational method, these volumes are discharged over longer periods. Therefore, the above pumping rates calculated from the Rational method will govern the pump design criteria.

Pump selection includes the size, type, and number of pumps. Vertical propeller or submersible pumps can be used. Pump sizes are usually selected to provide multiple pumps rather than a single pump of appropriate size. Smaller pumps are cheaper, and with multiple pumps, the loss of one will not shut down the entire pump station (Hydraulic Design Manual, 2002). The volume of 145,000 ft^3 can be pumped using four pumps of average pumping capacity of 39.3 cfs, or a combination of pumps of different capacity.

A pump station typically includes pumps, motors, power sources, controls, structures, wet well sumps, and discharge conduits. Pump motors are usually three-phase electric motors. The specific voltage selected depends on the power available from the utility and on what pump-motor combinations are commercially available. The size of each motor depends on the pump size, flow rate, pressure head, and duty cycle (Hydraulic Design Manual, 2002).

Depending on availability, a redundant, secondary electrical service feed from a different electrical substation can provide regular power if the primary service power is interrupted. Every pump station should have an on-site standby electrical generator regardless of the presence of redundant utility power because the type of storm that makes a pump station necessary is also the type of storm that interrupts utility power (Hydraulic Design Manual, 2002).

Control circuitry includes the flood level at which the pump station will be activated, sequence of operation, activation of the standby generator when necessary, deactivation when the flood event has passed, and operation of any night security lighting. Controls may also include automatic communication with a central office on the station's status regarding water levels, pump readiness, utility electrical power, standby generator fuel level, security, or other central office concerns (Hydraulic Design Manual, 2002).

The structure should meet requirements for public safety, local extreme weather conditions, site security, and maintenance operation. In addition, aesthetics and the ability to accommodate potential future expansion have to be considered (Hydraulic Design Manual, 2002).

The wet well sump receives the inflow of storm water prior to pumping. The wet well sump needs to be designed with provisions for screening trash and other debris associated with storm water and convenient access for the removal of accumulated debris and silt (Hydraulic Design Manual, 2002).

A rough cost estimate for the pump station and all the pumps would be approximately $1,000,000.

5.3 LEVEE

At extreme tides, the tide gate and the pump station are inadequate to prevent flooding. The 100-year base flood extends to +9.0 ft MLLW in vertical extent, and up to the +9.0 ft MLLW contour in horizontal extent. The water would inundate up to and including most of Gate 5 Road; therefore, a levee is necessary to prevent land inundation by extreme bay floods. An earth levee is proposed to protect the Gate 5 Road and the vicinity from flooding due to these extreme tides. Depending on the property owners' interest to protect their property, the levee alignment can be varied. The levee alignment also depends on the location of the one-way tide gate, to be effective in flood protection. Figure 5-2 indicates two possible levee alignments considered for further analyses and cost estimation purposes. However, the alignment north of Coloma Street can be varied between the two orientations L1 and L2, as shown on Figure 5-2.

The levee will connect to the higher ground at Harbor Drive near the first cross road west of Gate 5 Road and continue toward the bay along Harbor Drive and turn north at the Clipper Yacht Harbor and continue toward Coloma Street Channel. Continuation of the flood control elements north of Coloma Street has several options. We discuss four alternatives below.

Alternative 1

Path L1 assumes that the tide gate is in the T1 location. With this alternative, the levee crosses over to the north bank and continues along the L1 path. Channel storage between T1 and T2 is lost in this alternative.

Alternative 2

In Alternative 2, the tide gate is located at T1 with the levee path at L2. This is same as Alternative 1, except that the levee continues along the north side of Coloma Channel until L2 is met and continues along L2.

Alternative 3

In this alternative, the tide gate is located at T2 with the levee path at L1. For this alternative, the top of the channel has to be sealed to prevent overtopped bay water from entering the storm drain. The advantage over Alternatives 1 and 2 is that there is more storage for storm water. The levee can cross the channel near T1 and continue along L1, or the levee can continue toward the bay on the south side of Coloma Channel and cross over at the intersection of L1 and the Coloma Channel and continue along L1. In the former path, the channel has to be sealed up to T1, while in the latter path, the channel needs to be sealed only up to the intersection of L1 and the Coloma Channel.

Alternative 4

In Alternative 4, the tide gate is located at T2 with the levee path at L2. If the levee continues on the south side of Coloma Channel, it will cross over at T2 and continue along L2. It is also possible for the levee to cross over at T1 and continue along the north side of channel and then along path L2. For the latter method to be successful, the top of channel has to be sealed up to T1 (where the levee crosses over).
The levee has to be connected to a higher ground at its north end.

5.3.1 Key Parameters for Conceptual Levee Design

Crest Elevation

Crest elevation is one of the main parameters in a levee design. In the present study, we have recognized the following factors that contribute to the crest elevation estimate:

1. 100-year base flood elevation,
2. wave runup on the levee,
3. consolidation of underlying soil due to overburden,

4. local settlement,
5. regional subsidence and sea level rise, and,
6. FEMA Free-board requirements.

Hundred-year base flood elevation (BFE) from the FIS (1980) is +9 feet MLLW. The FIS does not indicate if the local wave runup was included in the estimation of the 100-year BFE, but it is presumed that it was or that tsunami runup governed. We used three typical profiles to estimate wave runup with the levee in place. Profile locations are shown in Figure 5-5, and typical profiles for wave runup estimation are shown in Figure 5-6. Profiles L1A and L1B are two typical profiles for levee L1, while L2A is typical for levee L2.

Waves at Sausalito are governed by wind-waves rather than swells. Swells would not be apparent in the area of interest. Waves at Sausalito were studied at the Napa Street Pier (Moffatt & Nichol Engineers, 1991) and the 100-year return period wind-waves estimated in the study were approximately 3 ft high and of 5-second wave period. These waves had been obtained by wave hindcasting, using extreme wind speeds, durations, fetches from the north and the southeast, and at deeper water (at the Pier Head line). Farther shoreward, these waves will be refracted and shoaled. Further, the marinas and floats out in the water dissipate the waves; therefore, waves arriving at the shallow waters in the vicinity of the study area are much smaller.

Wave runup estimates were obtained by using FEMA approved RUNUP version 2.0 software. A range of wave heights and periods were used to calculate runup at a still water level (SWL) of +9.0 feet MLLW. Maximum runup was less than 1.0 ft for the profiles L1A and L1B respectively, and about 4 ft for the steep profile L2A. The high runup estimate for the L2A can be reduced if the levee is placed more landward. Actual runup would be less due to energy dissipation prior to reaching levees. For crest elevation determination, we used an average runup of 1.0 ft for L1 and 2.0 ft for L2, assuming the runup can be lowered by moving the levee landward.

The weight of the levee causes consolidation of the underlying soils. In the past, the project area has been subjected to fill placed over the pre-existing marshland. The Bay Mud deposits underlying the existing fill are generally soft and compressible. The rule of thumb for consolidation due to overburden is about 20% of the levee height (above the existing grade). Assuming a levee height of 6 feet above the existing grade, the consolidation is estimated to be about 1.2 feet. The actual magnitude of anticipated settlement should be adjusted based on the findings of a geotechnical investigation at the design phase.

In addition to consolidation due to overburden, there is also local settlement, even in the absence of a levee. An average rate of settlement is estimated as 0.5 inches/year, after consulting property owners in the area and a range of estimates obtained from repeated surveys (W.B. Clausen Structural Engineers, 1988; pers. communication with Holt Greene). The rates vary form 0.4 inches/year to 0.9 inches/year at different locations. For the crest elevation determination, we estimate 0.83 ft of settlement will occur over 20 years, assuming a rate of settlement of 0.5 inches/year.

Sea level rise (SLR) is another important factor that needs to be addressed in determining the levee crest elevation. San Francisco Bay Conservation and Development Commission (SFBCDC, 1987) reports a 0.0039 ft /year rise at the Presidio gage. This estimate is low compared to the recent estimates of future SLR in the Intergovernmental Panel on Climate Change (IPCC, 2001). The global sea level rise from 1990 to 2100 is plotted in Figure 11.12 of IPCC (2001). Using this figure, we estimated a mean sea level rise of 0.2 ft over the next 20 years. The actual water level increase due to SLR from the time of base flood determination (1980) to the present also needs to be included. In total, we estimate SLR from 1980 to 2020 to be about 1.0 ft.

FEMA requires at least 3 feet of freeboard above the BFE to be recognized as a flood control element for flood insurance purposes. Accordingly, the as-built crest elevation would need to be at least +12 feet MLLW. The constructed levee crest elevation will become relatively lower over time due to (1) the consolidation of soil directly below the earth levee, (2) local subsidence of the Gate 5 area, and (3) sea level rise. In 20 years, a total change of about 3 feet is estimated.

Wave runup on the levees are less than the actual free-board available, as shown in Table 5-1. Therefore, the crest of Levees L1 and L2 were both estimated at 12 ft MLLW. These elevations are used in the cost estimate.

Table 5-1. Crest Elevation Determination

Crest Elevation	Levee L1 (feet)	Levee L2 (feet)
For FEMA Flood Insurance Benefits		
BFE	9.0 MLLW	9.0 MLLW
Free-board Requirement-FEMA	3.0	3.0
Minimum Crest Elevation	12.0 MLLW	12.0 MLLW
From Present Analysis – up to 20 years		
BFE	9.0 MLLW	9.0 MLLW
[1]Consolidation of Underlying Soil	1.2	1.2
[2]Local Settlement	0.83	0.83
[3]Sea Level Rise	1.0	1.0
Free-board due to 1, 2, and 3	3.0	3.0
Crest Elevation	12.0 feet MLLW	12.0 feet MLLW
Wave Runup	1.0 < 3' of free-board	2.0 < 3' of free-board

Side Slopes and Crest Width

In the absence of geotechnical and other analysis, a prismatic levee cross-section with relatively flat side slopes (4 horizontal : 1 vertical) has been assumed. However, with subsequent design investigation and analysis, alternative levee configurations may be considered. The relatively conservative prismatic cross section used for this estimate will allow design optimization without significantly increasing quantities.

We have considered a preliminary levee cross-section shown in Figure 5-7 to produce an engineer's estimate. Existing elevations vary along the length. Cost estimates are based on the cross-section shown in Figure 5-7 and the existing ground elevation.

Additional fill needs to be placed in the future and maintained at the FEMA required crest elevation of at least 12 ft MLLW. Although a crest width of 10 to 12 feet is adequate, widening the crest to 15 feet provides easy access for future maintenance and allows addition of fill to the crest without having to make changes to the sides of the levee to increase the crest elevation. The other aspects of levee design such as embankment geotechnical stability, settlement, liquefaction, maintenance, and inspection are not covered in the present conceptual design and need to be addressed in the design phase. Erosion of the levee and slope protection has not been considered at this phase.

Fill volumes were calculated by dividing the total length of the levee into segments of equal existing elevation and then summing up the fill volumes for all segments. The levee alignment L1 has a total length of 2,125 feet and a fill volume of 11,400 cubic yards (in place) is estimated for this configuration. The levee L2 alignment has a total length of 2,500 feet, and an insitu fill volume of 13,900 cubic yards is estimated. The cost of the earth work based on a unit cost of $50 per cubic yard are $570,000 for levee L1 and $700,000 for levee L2.

5.4 INCREASING FLOOD STORAGE BY DETENTION BASIN

Flood control elements discussed so far discharge (pump) floodwater above the capacity of the storm drains back into the bay at the same rate of inflow. An alternative is to store the rainfall runoff in a detention (storage) basin until the tide recedes and then discharge it into the bay. This method is appropriate when the problem of flooding is mainly due to extreme runoff. At the project site, a detention basin alone is insufficient because flooding also occurs due to high tides. Flood proofing in this case can be achieved with a combination of a levee, tide gate, and a detention basin. The main advantage of a detention basin over a pump station is the lower operating cost.

5.4.1 Detention Basin Sizing

Detention basin volume is based on the runoff volumes calculated for the 10-year and the 100-year rainfall events. With the tide gate option, the volume required to be stored in a detention basin is approximately 1.75 to 3.3 acre-ft for the above rainfall events. The net runoff volume can be then stored in a below grade storage. The lower elevation of the detention basin is limited by the invert elevation of the inlet drain to the Coloma outfall. This elevation is approximately 0 MLLW. Therefore, the elevation range for detention basin is +6 MLLW to 0 MLLW. The following table shows potential combinations of storage areas and corresponding elevations for the storage basin. It is also noted that the runoff volumes considered were based on short duration high intensity storms and that the volumes may be higher for longer duration storms of the same return periods.

Table 5-2. Potential Detention Basin Areas and Heights

Event	Storage Needed* (acre-ft)	Land Area (ft)	Height of Storage** (ft)	Elevation (ft MLLW)
10-year runoff	1.75	1.0	1.75	6 to 4.25
	1.75	0.5	3.5	6 to 2.5
100-year runoff	3.3	1.0	3.3	6 to 2.7
	3.3	0.5	6.7	6 to –0.7

Note: * Storage needed – in addition to the channel storage
** Height of storage – assuming vertical sides

5.5 EVALUATION OF FLOOD CONTROL ELEMENTS

Four flood control elements have been identified and described in a conceptual manner:

- Tide Gate – Blocks frequent high tide inundation and allows runoff storage in the storm drain system;
- Levee – Blocks inundation from overland flow during very high bay water levels;
- Detention Basin – Provides storage for rainfall runoff during high tides;
- Pump Station – Pumps water from a detention basin to the bay providing flood protection during severe combined high tide and high rainfall runoff events.

These elements are characterized in Table 5-3. The most efficient flood control strategy will involve a combination of these elements. The elements can be constructed in phases once a flood control strategy is identified. Due to the frequency of tidal flooding and relatively low initial cost, the tide gate is given the highest priority. The levee is the next highest priority, although a floodwall in raised grades that function like a levee may be preferred. A detention basin to store runoff during high tides is the element with the next highest priority. Finally, a pump station may be used to allow a smaller detention basin and to provide greater flood protection. The optimum combination of elements, geometries and construction phasing can be developed with community input, consultation with FEMA, consideration of funding, and further hydrologic evaluation. The engineer's estimates of construction costs are approximate owing to the conceptual level of element descriptions, and do not include real estate costs. These estimates are intended to provide 'ballpark' values for comparison between elements.

Table 5-3. Comparison of Flood Control Elements

Element	Protection Extent		Cost (Approx. Relative)
	Tide	Runoff	
Tide gate	Up to Elev. A (about +7' MLLW)	Up to 2 to less than 10 year	About $100,000
Detention basin	Up to Elev. A (about +7' MLLW)	Up to 10 year +	Not determined
Pump station	Up to Elev. A (about +7' MLLW)	Up to 100 year	About $1,000,000
Levee / Flood wall	Up to Elev. B (+9' MLLW)	Up to 100 year	About $1,000,000

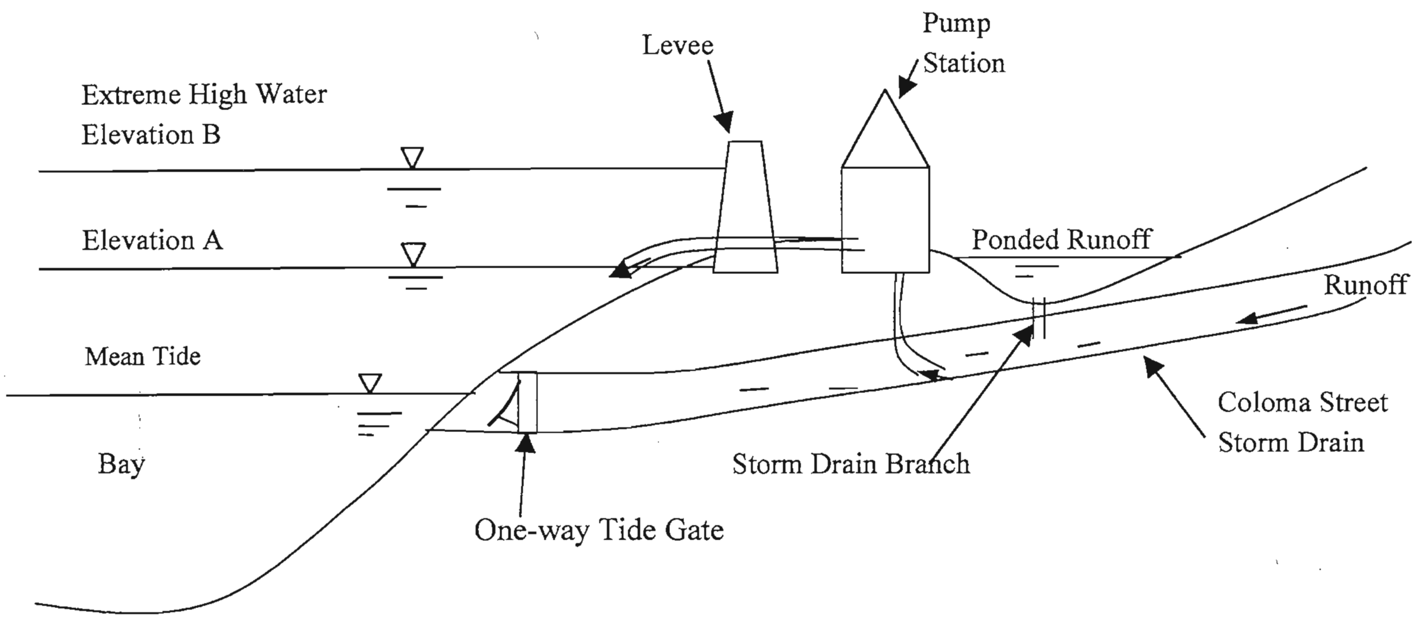


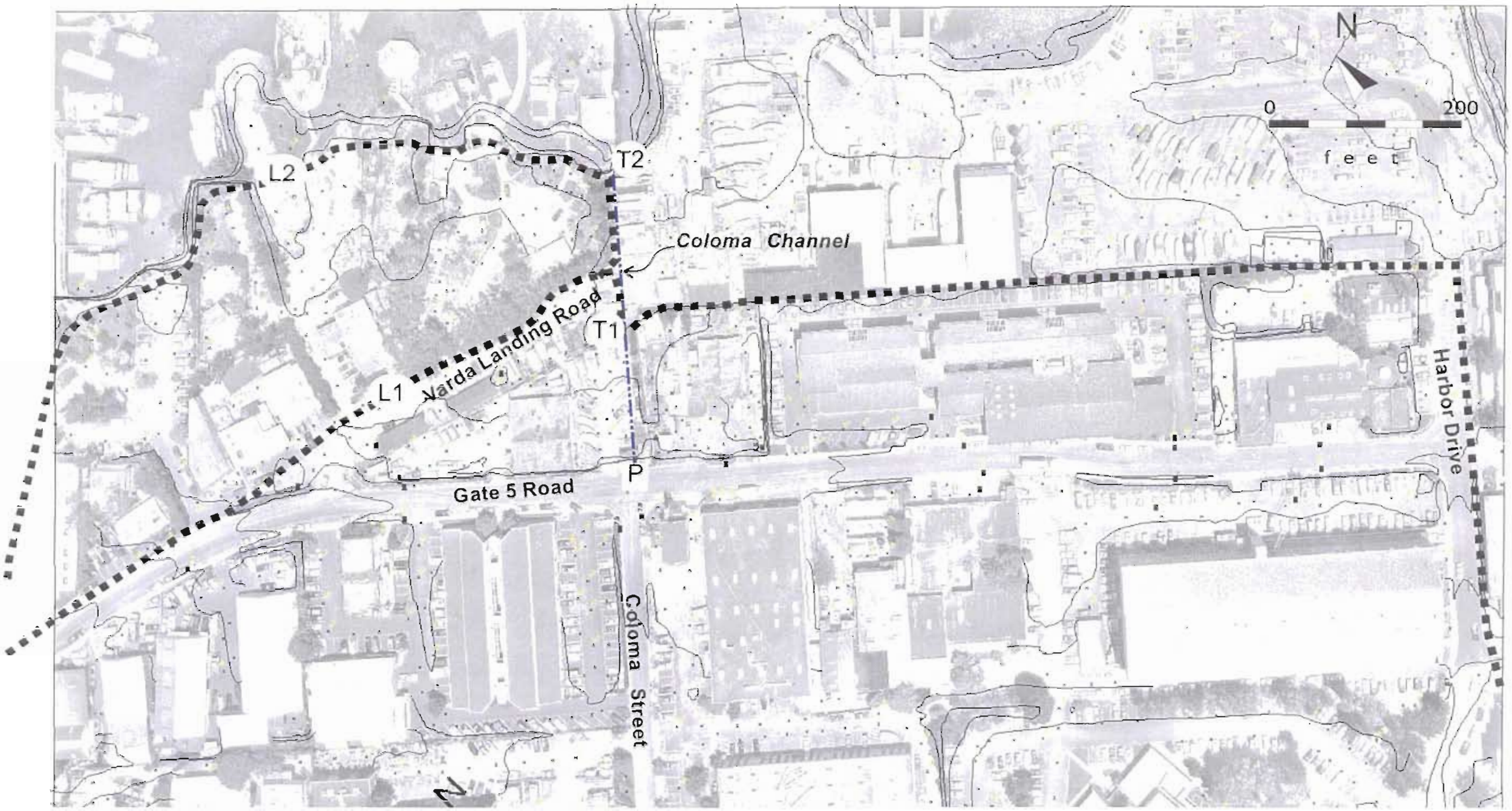
figure 5-1

Sausalito Flood Study

Flood Protection Scenarios

PWA

PWA #: 1598



T2 = tide gates

P = pump station

L1 = levees

Note: Levee location can be varied between L1 and L2

figure 5-2

Sausalito Flood Study

Proposed Flood Control Element Layout

Source: Aerial photo 1968 from City of Sausalito
Topo Map 1998 from Geomatric Computer Services, Inc.

PWA

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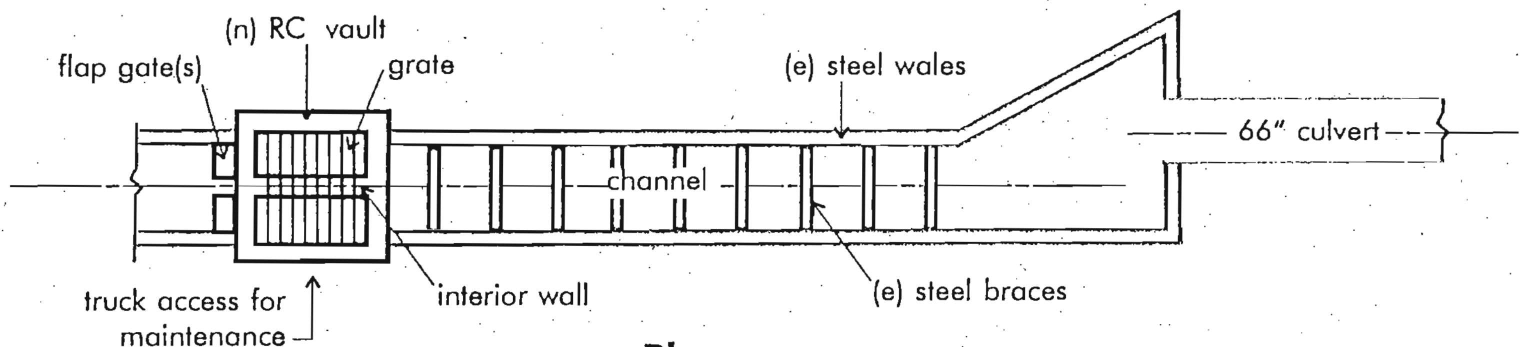
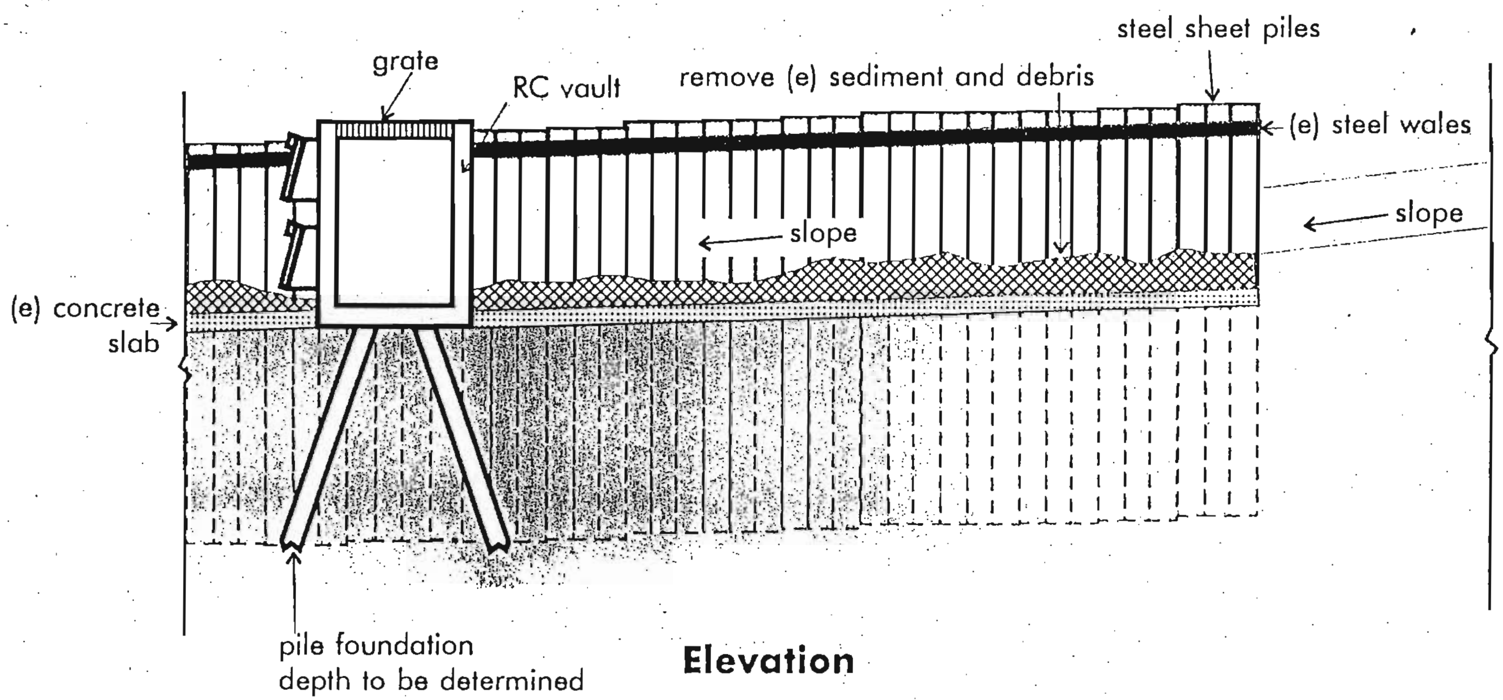


Plan



Elevation

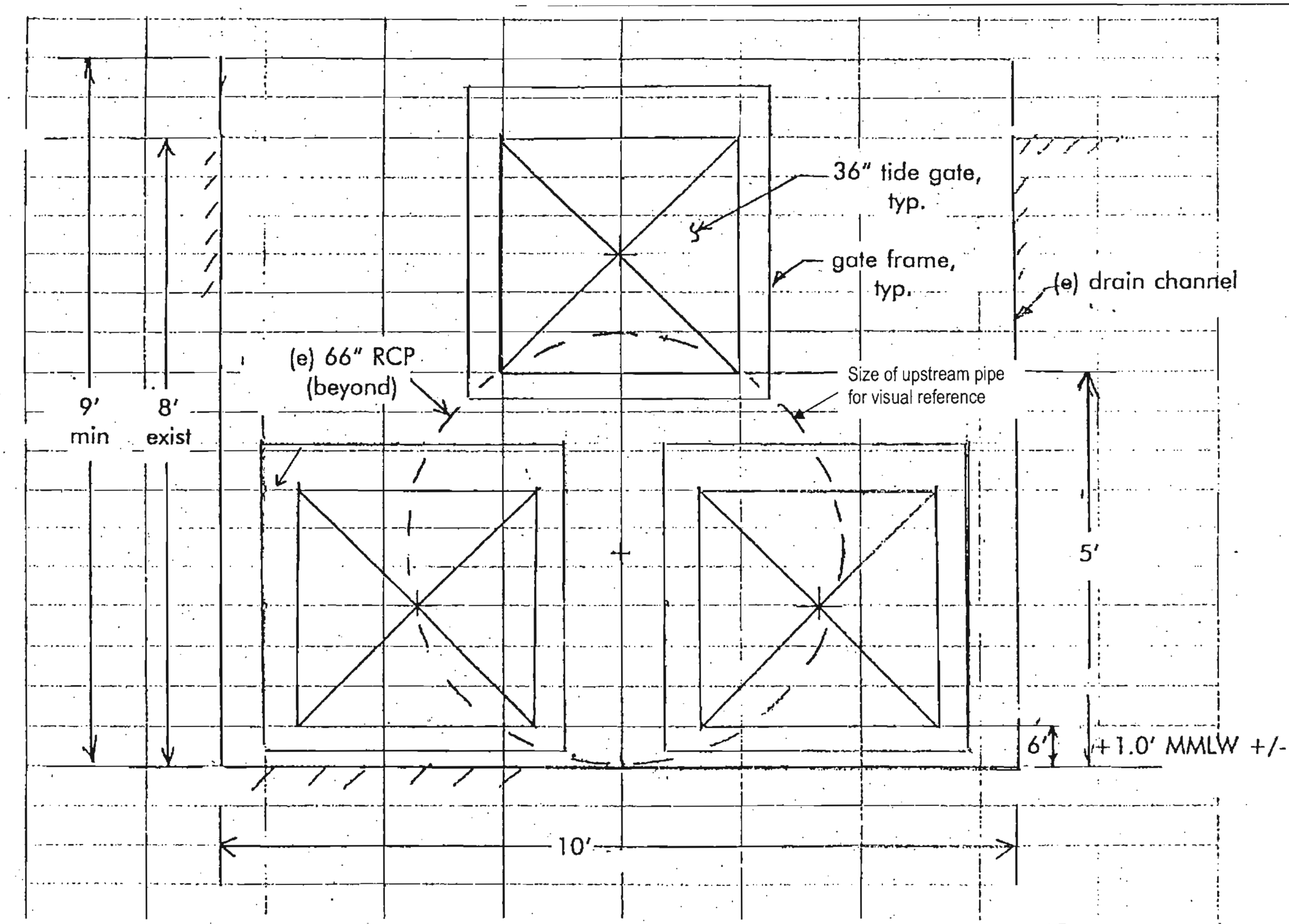
note: safety fence, access paving, and debis rack not shown. Existing lid and structures over channel not shown.

figure 5-3

Sausalito Flood Study

Schematic of New Tide Gate
(Source: PWA # 1251 Tide Gate)

PWA



Elevation

approximate scale: 1" = 2'

figure 5-4

Sausalito Flood Study

Tide Gate Headwall

(Source: PWA # 1251 Tide Gate)

PWA

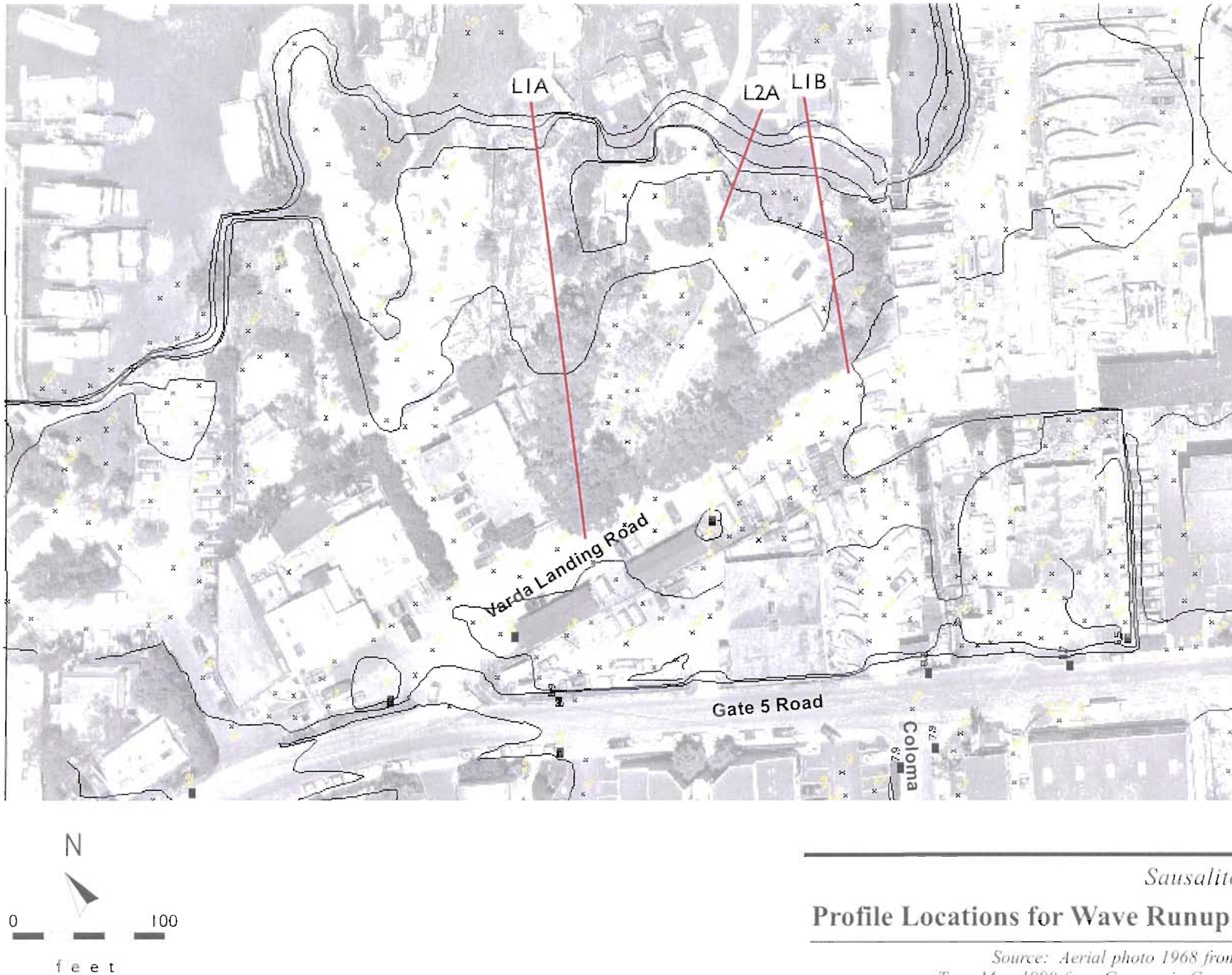


figure 5-5

Sausalito Flood Study

Profile Locations for Wave Runup Estimation

Source: Aerial photo 1968 from City of Sausalito
Topo Map 1998 from Geomatric Computer Services, Inc.

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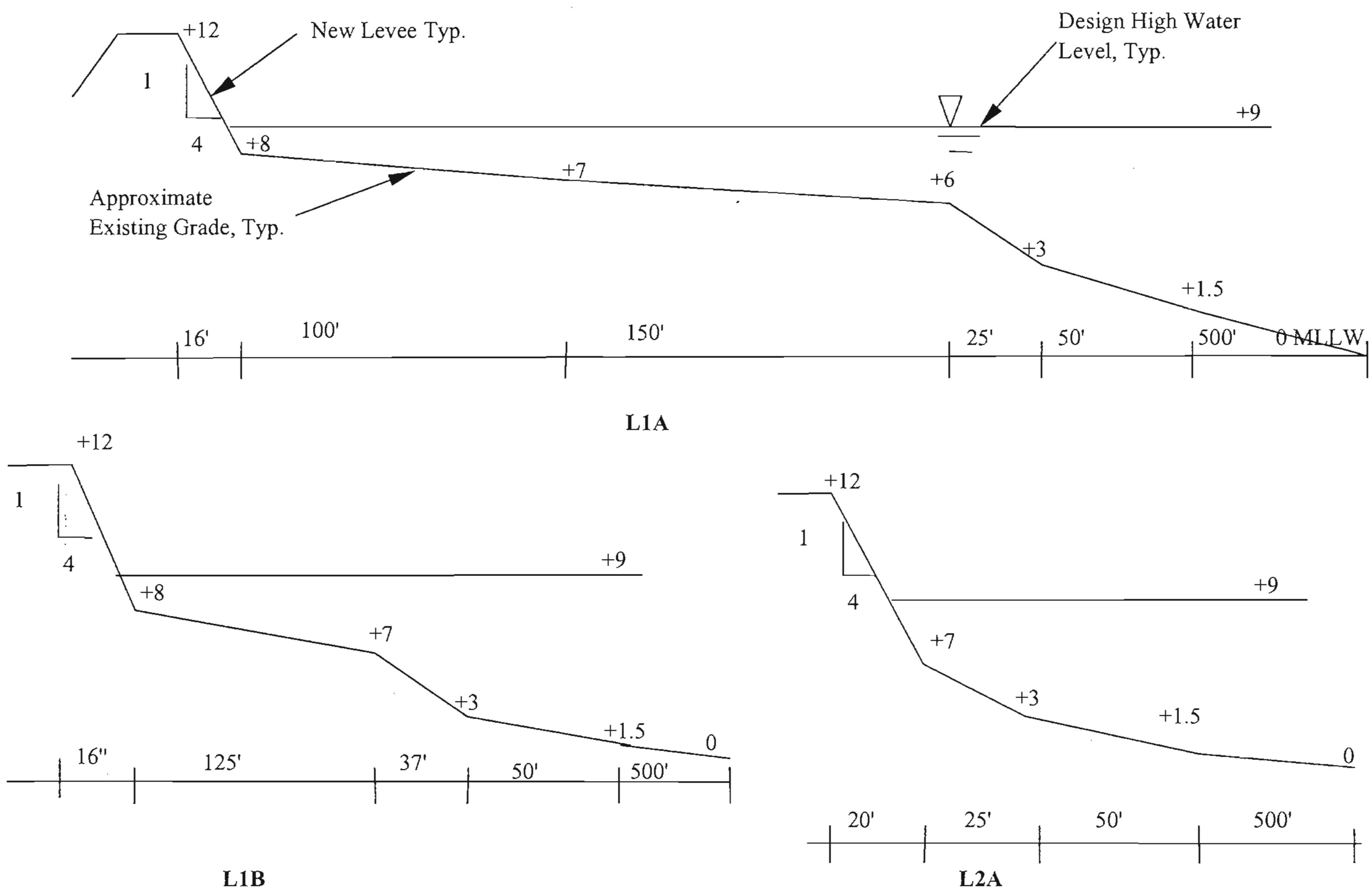
PWA



note: 1.Figures not to scale.
2. Datum : MLLW
3. Units: feet

figure 5-6

Sausalito Flood Study

Typical Profiles for Wave Runup Estimation

PWA

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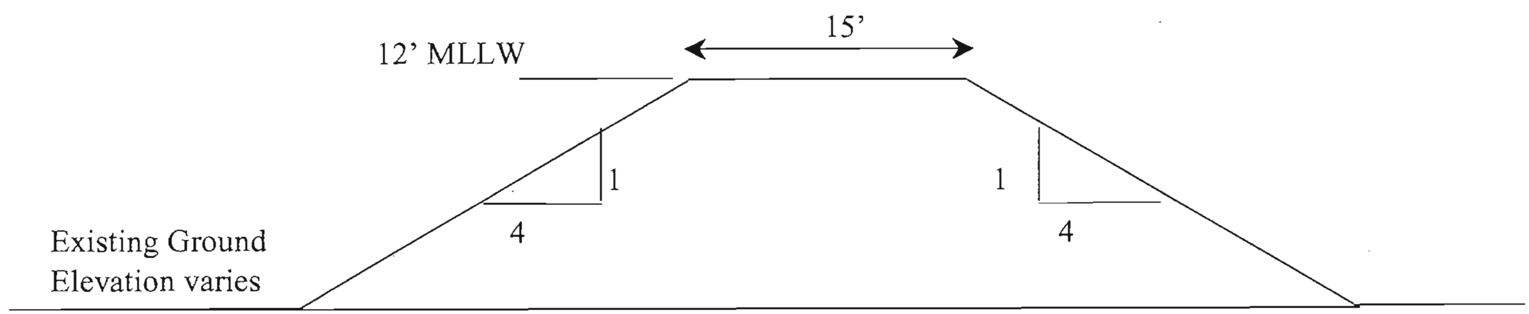

Levee Configuration

figure 5-7

Sausalito Flood Study

Preliminary Levee Cross-Section

PWA

PWA #: 1598

6. FUTURE WORK

A meeting was held at the city of Sausalito on 9/26/02 to present the flood analysis discussed in this report to the property owners and the city engineers. Some of the property owners expressed their desire to raise their property at their expense as an alternative means of inland protection. This is a surrogate for the levee adjoining their property. In that case, a levee is required in the gaps that are not raised by the owners, in order to provide effective protection to the landward property from bay flooding. The previously mentioned property owners showed interest in raising their property above the BFE but not up an additional three feet to the elevation that would qualify the fill as a levee for FEMA flood insurance purposes. The installation of a one-way tide gate, a pump station, and/or a detention basin can be considered as mitigation measures for filling and installation of the levee in the floodplain as well as a coordinated flood control strategy for the Gate 5 Road community.

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8. LIST OF PREPARERS

This report was prepared by the following PWA staff:

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APPENDIX A

Memo on Infiltration

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MEMORANDUM
via FAX

DATE: September 03, 2002

TO: Gordon Sweeny, City Engineer, City of Sausalito; (415) 289-4167
Page(s): 6

FROM: Carmela Chandrasekera, PWA

RE: **Sausalito Flood Study –Infiltration Test**
PWA Ref. #: 1598

Infiltration test for checking leaks in the Coloma Street storm drain system.

As part of the Sausalito flood study an infiltration test was performed to estimate the infiltration rate of ground water into the storm drain system at Coloma Street. A one-way tide gate at the Coloma outfall is one of the flood control elements being studied in the project. The purpose of the tide gate is to prevent bay water from entering the storm drain system at high tide, which would allow runoff water to be stored in the system until the tide recedes. The success of this method is largely dependant on a leak free drainage system.

An infiltration test was performed on August 14, 2002 to test for leaks in the storm drain system and to re-evaluate the actual storm drain capacity. The leaks in the system allow ground water in the soil to enter the drainage system. The test was performed at a low tide elevation by installing a temporary barrier and cutting off the rising tide from the storm drain, near bayward end of the 265 Gate 5 Road property. This temporary barrier was installed by the property owner by stacking sand-bags and covering with a plastic cover to seal water seepage. The rise in water level was recorded every 15 minutes, at a location about 25 feet upstream of the tide barrier for about two hours to estimate the inflow (infiltration) rate.

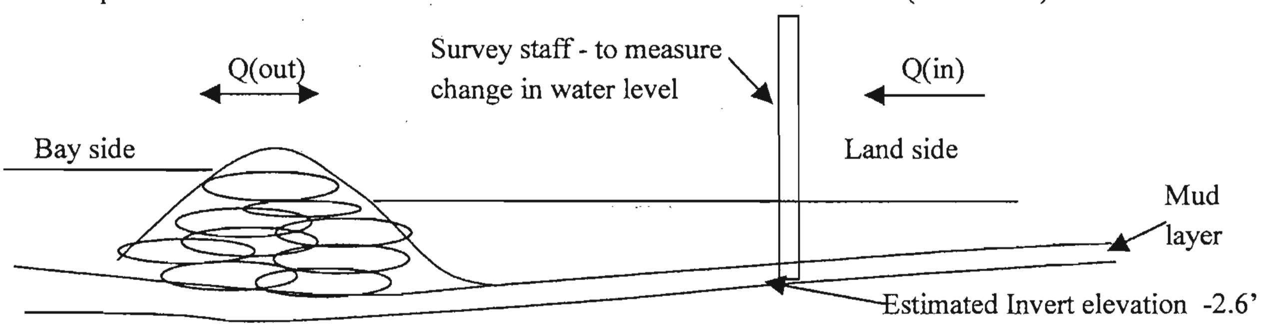

Figure 1 – Schematic of the test setup.

Figure 1 shows a schematic of the test setup. The predicted tides at the Sausalito, Corps of Engineers Dock on the day of the test is shown in the appendix (Nautical Software). The low tide was 1.5 feet MLLW at 10.30 am. The tide was cutoff from the outfall channel between 10.00 am and 10.15 am. and the measurement was continued till 12.30 pm. After 12.00 noon there were visible signs of leakage through the barrier and the water level measurements after 12.00 noon were not included in the estimation of the infiltration rate. The difference in the water surface elevation upstream and down stream of the barrier was visually estimated to be approximately 0.5 ft. The predicted tide change at the Corps of Engineers Dock during the period of measurement is 0.4 ft, (see the appendix) which is close to the visual observation.

The invert level of the channel was estimated using two methods 1) using the tide elevation and water depth and 2) using the site grade from the topo maps and the depth to the bottom of channel from the site grade. The invert calculated form these two methods is between –2.6 to –3.3 ft NGVD. In addition, about 6 inches of mud covers the bottom of the channel. This estimated invert elevation is lower than the invert elevations shown on Coloma Street storm drain extension drawings (Hoffman and Albritton Engineers and Planners, 1981) by about 2 feet.

In Figure 1, Q(in) represents ground water seeping into the drain system through leaks in side walls and any flow from the upstream. The flow from upstream is minimum due to the dry condition (no runoff). Q(out) represents any seepage through the temporary barrier. The change in volume of water in the upstream (V) = Q(in)-Q(out) and using the sign convention - flow towards the bay as positive (+). The change in water level (h) is proportional to Q(in)-Q(out). In the rising tide, seepage if any through the barrier is from the bay side to the land side and Q(out) is negative according to the sign convention. In the absence of tide and Q(out)=0, the increase in water level was assumed to be solely due to ground water infiltration from the sides. The measured change in water level was 0.12 inches/hour. Based on the estimated free water surface area and assuming Q(out) =0, the corresponding volume of infiltration calculated for 6 hours is 313 cubic feet. The test indicates that the ground water infiltration is approximately equal to or less than this volume since the test was performed on a rising tide, and if there was water leaking, the actual ground water infiltration would be lower than this limit. Figures 2 and 3 show pictures of tide barrier installation and the infiltration test setup.

Figure 2 - Temporary tide barrier installation before the infiltration test.

Figure 3 - Measurement of change in water level upstream of the temporary tide barrier.

The test was performed successfully and the results indicate that the rate of infiltration is not very significant. The ground water level at the time of experiment is presumably higher than the water level at the Coloma storm drain. If there were significant infiltration, water seepage through the sheet piling would have been clearly visible. Even though the sheet piles were wet, rapid water seepage was not observed. Similarly, at high tide, sheet piling is likely to act as a barrier and keep the ground water infiltration into the storm drain system at insignificant rates. Therefore the infiltration test results support the tide gate installation as effective. Total storm drain capacity of this drainage system is about 31,363 cubic feet (PWA, 1998). The volume of infiltration of 313 cubic feet between high tide and low tide is approximately 1% of the total storage capacity of the storm drain system. The net storage capacity after considering infiltration is about 31,000 cubic feet.

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