DRAFT EROSION PROTECTION ANALYSIS AND DESIGN

San Francisquito Creek - Hwy 101 to SF Bay ***February 2013***

Reviewed by: Su Mishra, P.E, Ph.D. and Brian Doeing, P.E.

Prepared by: Elizabeth Mesbah, P.E.

Purpose

HDR is responsible for providing plans, specifications, and engineering support for levee design improvements along San Francisquito Creek (SF Creek) from Hwy 101 to the San Francisco Bay. An evaluation of the proposed channel conditions has been completed. Due to significant changes in channel geometry and increased peak flows per the design, it has been determined that erosion protection is needed along portions of SF Creek levees and floodwalls. This Technical Memorandum (TM) summarizes the design analysis and proposed erosion protection.

Methodology

Hydraulic model results, existing soils information, and historical channel information has been evaluated to determine whether erosion protection will be needed along SF Creek proposed levees and floodwalls. See Attachment A, General Site Plan, for overview of proposed channel configuration.

The US Army Corps of Engineer's (USACE) Engineer Manual (EM) 1110-02-1601 Table 2-5 suggested maximum permissible mean channel velocities was used as guidance to determine the permissible velocity range for the existing soils type. The existing soils type found along SF Creek is fine sand with an assumed permissible velocity of 2.0 ft/s. The proposed levee fill material is composed of soil with higher clay content, and therefore, a higher permissible velocity of 5 ft/s is assumed. See Attachment B for USACE's permissible velocity table. At locations where velocities were found higher than the permissible mean channel velocities, erosion protection and/or additional monitoring has been recommended.

Various peak flows and starting water surface conditions were evaluated to determine which event may produce the greatest scour prone conditions. The worst case condition, which produced the highest channel velocities, was selected to size rock slope protection (RSP). It was determined that the 100-year design flood run with a downstream control at normal depth produced the highest channel and overbank velocities. The 100-year design flood run at normal depth represents a 100-year design flood occurring at a low tide, i.e. no backwater is present in

SF Creek or the Faber Tract. Note, the top of levee and floodwall elevations are being designed considering the 100-year design flood run during a 100-year tidal event with an addition 26 inches to account for sea level rise. This design condition was not used to compute erosion protection because channel velocities are significantly reduced when tidal backwater is present.

Historical aerial imagery dating back to as far as 1948 was also reviewed for channel behavior. It is unknown how flow profiles have changed over these years, however, the general channel alignment has remained fairly constant. The reference, *Fluvial Forms and Processes*, by Knighton, was considered when evaluating the tendency of channel meandering. It has been estimated that SF Creek is in the transition area between meandering and straight channels which indicates a relatively low risk of meander migration into the floodwalls or levees in the system. This assumption, however, does not decrease the amount of operations, maintenance, and monitoring that should be performed as part of the maintenance plan.

Stringent operations and maintenance (O&M) procedures are included in the O&M Plan to ensure the channel performs as designed. Additional riprap protection may need to be installed at a later time if erosion begins to occur at other locations along SF Creek. O&M inspections are critical to identify problem locations before they cause significant failures.

Analysis

Due to the unique proposed channel configuration of the San Francisquito Creek, multiple erosion protection design methods were considered. These design methods include:

- Toe Scour for Depth of Revetment
  - Purpose: To compute depth of toe scour if revetment was not installed. Results of this computation are used to determine the depth of revetment required to protect against toe scour.
  - Equation used for computation: Bureau of Reclamation's Computing Degradation and Local Scour (Neill, Lacey, and Blench for channel scour during peak flood flows) equation
- Revetment Toe Protection Design
  - Purpose: To compute dimensions of launchable placed along an expected erosion area at an elevation above the zone of attack
  - Equation used for computation: USACE's EM 1110-2-1601, Section 3-11
- Bank Revetment Design
  - Purpose: To appropriately size rock slope protection based upon channel conditions.
  - Equation used for computation: USACE Engineer Manual No. 1110-2-1601 (Maynord et al. (1989) and Maynord (1990) equation

- Geotextile Design
  - Purpose: To appropriately select and design filter material to be placed underneath rock slope protection.
  - Equation used for computation: Federal Highway Administration's (FHWA) Hydraulic Engineering Circular (HEC) 23 Design Guide 16 (Cistin-Ziems) method
- Overtopping Flow Riprap Design
  - Purpose: To compute depth and extents of rock slope protection when flow is allowed to overtop the terraced mitigation area from project left levee Stations 73+50 to STA 68+00.
  - Equation used for computation: NCHRP Report 568 (Mishra, 1998) equation
- Wave Attack Riprap Design
  - Purpose: To compute depth and extents of rock slope protection when existing levee from project left levee Stations 25+00 to 16+00 is impacted by wind and wave runup.
  - Equation used for computation: FHWA's HEC-23 Design Guide 17 (Hudson Method and Pilarczyk Method) equations
- Bed Scour at Vertical Drop Structure
  - Purpose: To estimate potential extents of scour due to a vertical drop structure located within the Palo Alto Pump Station channel
  - Equation used for computation: FHWA's HEC-23 Design Guide 3 (Pemberton and Lara 1984) equation

Toe Scour for Depth of Revetment

River channel scour due to the 100-year design flood has been calculated for SF Creek. The Bureau of Reclamation's *Computing Degradation and Local Scour* report, dated January 1984 was used as technical guidance. Three empirical equations, Neill, Lacey, and Blench, were used to compute general scour depth throughout the project reach. Equations and input parameters are shown below.

Equation 1. Neill Equation for Channel Scour

$$d_f = d_i \left(\frac{q_f}{q_i}\right)^m$$

Where,

d_f = Scoured depth below design flood water level, ft
d_i = Average depth at bankfull discharge in incised reach, ft
q_f = Design flood discharge per unit width, ft^3/s / ft
q_i = Bankfull discharge in incised reach per unit width, ft^3/s / ft
m = Exponent, 0.67 for sand

Scour depth results are then multiplied by the Z factor, where Z = 0.6 for moderate bend and 0.5 for straight channel.

Equation 2. Lacey Equation for Channel Scour

$$d_m = 0.47 \left(\frac{Q}{f}\right)^{1/3}$$

Where,

d_m = Mean depth at design discharge, ft
Q = Design discharge, ft^3/s
f = Lacey's silt factor

Scour depth results are then multiplied by the Z factor, where Z = 0.5 for moderate bend and 0.25 for straight channel.

Equation 3. Blench Equation for Channel Scour

$$d_{fo} = \frac{q_f^{\ 2/3}}{F_{bo}^{\ 1/3}}$$

Where,

D_{fo} = Depth for zero bed sediment transport, ft
q_f = Design flood discharge per unit width, ft^3/s / ft
F_{bo} = Blench's zero bed factor, ft/s^2

Scour depth results are then multiplied by the Z factor, where Z = 0.6 for moderate bend and 0.6 for straight channel.

Results

Results from three different methods indicate that a range of 6 to 10 feet of scour may occur on a moderate bend while 3 to 10 feet of scour may occur on a straight channel if no erosion protection was installed. For the riprap protection around a moderate bend in the channel, a depth of 7 feet of erosion protection will be constructed. For the riprap protection through a straight section of the channel, a depth of 6 feet of erosion protection will be provided. Table 1 below summarizes the computation results while hand calculations are included in Attachment C. Attachment D illustrates how SF Creek was segmented and analyzed into straight channels and bends. In order to protect from the scour, a combination of placed and self launching riprap revetment has been recommended.

Table 1 Channel Scour Results during Peak Flood Event

Channel Type	Neill Equation Depth of Scour (ft)	Lacey Equation Depth of Scour (ft)	Blench Equation Depth of Scour (ft)
Moderate Bend	9.0	6.3	9.8
Straight Channel	7.5	3.2	9.8

Revetment Toe Protection

The USACE's Engineering Manual (EM) 1110-2-1601 was used to design required launchable toe protection. Launchable stone is defined as stone that is placed along expected erosion areas at an elevation above the zone of attack. As the attack and resulting erosion occur below the stone, the stone is undermined and rolls/slides down the slope, stopping the erosion.

For gradual scour in regular bendways, the height of the stone section before launching ranged from 2.5 to 4.0 times the bank protection thickness. Bank protection thickness ranged from 2 to 3 feet. To account for the stone lost during launching, stone volume increased by 25% was used assuming dry installation.

Results

Since two bank protection thicknesses were used, different dimensions of revetment toe protection were used in the design. Also different configurations were used for floodwall and levee protection. Attachment D includes revetment toe protection calculations and assumptions.

Bank Revetment

The USACE's program CHANLPRO Version 2.0, was used to size riprap revetment for the entire project length. CHANLPRO computes a recommended rock size gradation using USACE's Engineering Manual 1110-2-1601, equation 3-3, (Maynord et al. (1989) and Maynord (1990)), as shown below. Bank revetment was evaluated by segmenting SF Creek into multiple bends and straight channels. Attachment E illustrates how the channel was segmented and analyzed.

Equation 4 Compute Bank Revetment Size

$$D_{30} = S_f C_s C_V C_T d^{-0.25} \left(\frac{V_{ss}}{\sqrt{K_1 g (s-1)}} \right)^{2.5}$$

Where,

S_f = safety factor, 1.3 has been assumed for this study
C_s = stability coefficient for incipient failure, 0.3 for angular rock
C_v = vertical velocity distribution coefficient
C_T = thickness coefficient
d = local depth of flow
s = specific gravity of riprap, 165 lb/ft^3
V_{ss} = local side slope corrected velocity
K_1= side slope correction factor
g = acceleration of gravity, 32.2 ft/s^2

Results

CHANLPRO model input parameters were taken from the hydraulic HEC-RAS model as shown in Table 2. Standard USACE ETL gradations were used to compute preliminary sizes and thicknesses of protection.

Table 2 CHNLPRO Model Input Parameters

Location	Location HEC-RAS River STA	Return Period (yr)	Channel Discharge (ft3/s)	Average Channel Velocity (ft/s)	Vss[5] (ft/s)	Average Flow Depth (ft)	Average Main Channel Top Width (ft)	Radius of Curvature	Side Slope (xH:1V)
Bend 1	7696.158 – 7167.645	100 yr	9300	6.9	9.1	15.4[4]	224	270	Floodwall[1]
Straight 1	7068.691 – 6963.459	100 yr	9300	8.0	--	14.4[4]	202	--	Floodwall[1]
Bend 2	6890.463 – 6317.478	100 yr	9300	7.8	9.4	14.4[4]	172	412	Floodwall[1]
Bend 3	6100.918 – 5303.36	100 yr	9300	7.2	9.0	13.9[4]	215	488	Floodwall[1]
Bend 4	5003.749 – 4404.458	100 yr	9300	6.4	9.6	11.5[3]	260	826	3
Bend 5	4404.458 – 4200.504	100 yr	9300	6.6	9.5	11.3[3]	260	886	3
Bend 6	4200.504 – 4001.593	100 yr	9300	7.1	11.3	11.2[3]	260	589	3
Bend 7	3800.303 – 2201.506	100 yr	9400	7.9	12.4	10.1[3]	280	721	3
Straight 2	2002.523 – 1000.179	100 yr	9400	7.6	--	8.2[3]	Faber Tract[2]	--	2

[1] 4H:1V was used for computation at floodwalls.
[2] Top width is fairly unlimited due to Faber Tract located on northeast side of SF Creek.
[3] For side slopes in natural channels, the local flow depth is measured at 80% of the total average depth.
[4] Full flow depth used for floodwall section .
[5] Vss factor, which is the local depth averaged velocity, is used for all bends computations.

Results using the USACE ETL gradations are shown in Table 3. Attachment F contains the CHANLPRO model output results.

Table 3 Computed Maximum Allowable Particle Sizes for Bank Revetment using USACE ETL Gradations

Location	USACE ETL Gradation	Median Particle Diameter (in)	D15 (in)		D50 (in)		D100 (in)		Thickness (in)
			Min	Max	Min	Max	Min	Max	
Bend 1	2	6	4.8	6.3	7.0	8.0	8.8	12.0	12.0
Straight 1	1	4	3.6	4.8	5.3	6.0	6.6	9.0	9.0
Bend 2	2	6	4.8	6.3	7.0	8.0	8.8	12.0	12.0
Bend 3	2	6	4.8	6.3	7.0	8.0	8.8	12.0	12.0
Bend 4	3	8	6.0	7.9	8.8	10.0	11.1	15.0	15.0
Bend 5	3	8	6.0	7.9	8.8	10.0	11.1	15.0	15.0
Bend 6	4	10	7.1	9.5	10.5	12.0	13.3	18	24.0
Bend 7	7	16	10.7	14.3	15.8	18.0	19.9	27.0	27.0
Straight 2	1	4	3.6	4.8	5.3	6.0	6.6	9.0	9.0

Once preliminary sizes were computed, local bay area quarries were contacted to determine common and available gradations in the surrounding project area. Caltrans ¼-Ton rock was identified at multiple bay area quarries and satisfied the required gradation requirements. In addition, ¼-Ton rock is large enough to deter theft of riprap protection. The ¼-Ton rock gradation is provided in Table 4 below. Recommended locations and depths are provided in Table 5 below. Attachment G illustrates the design details for rock placement along the proposed levee slope or floodwall.

Table 4 Recommended Gradation Curve for ¼-Ton Rock

Rock Size (weight, lb)	Typical Gradation (%>)	Specification
1000 lb	0	0-5
500 lb	65	50-100
75 lb	98	90-100

Recommended locations and depths of 1/4 –Ton riprap protection are provided in Table 5 below. Attachment G illustrates the design details for rock placement along the proposed levee slope and floodwall.

At Bend 4, it was identified that the existing low flow channel encroaching into the levee toe. The low flow channel will be shifted over to the center of the proposed channel. Since significant bench width between the channel and levee toe will be provided, riprap protection is not recommended around Bend 4. If the low flow channel begins to meander back near the levee toe, riprap protection should be considered.

Table 5 Recommended Maximum Allowable Particle Sizes for Bank Revetment using proposed ¼-Ton Rock Gradation

Location	Riprap Placement Stationing and Location	User Specified Gradation	D30 (in)	D100 (in)	D85/D15	Thickness (in)
			Min	Max		
Bend 1	76+00 – 73+86 (Left Floodwall) 75+20 – 68+00 (Right Floodwall)	¼ - Ton	12.6	24.0	2.1	24.0
Straight 1	72+00 – 68+00 (Left Terraced Slope) 68+00 – 67+00 (Left Floodwall)	¼ - Ton	12.6	24.0	2.1	24.0
Bend 2	Riprap is not recommended at this time due to the wide bench present on both sides of the low flow channel and overbank velocities less than 5 ft/s. If the channel begins to meander near the levee toe, riprap should be considered.					
Bend 3	58+50 – 50+00 (Left Floodwall) 50+00 – 47+25 (Left Floodwall / Levee Transition)	¼ - Ton	12.6	24.0	2.1	24.0
Bend 4	55+00 – 46+25 (Right Floodwall / Levee Transition and Right Levee)	Riprap is not recommended at this time due to the proposed low flow channel shift as well as the wide bench now present on both sides. If the channel begins to meander near the levee toe, riprap should be considered.				
Bend 5	Riprap is not recommended at this time due to the wide bench present on both sides of the low flow channel and overbank velocities less than 5 ft/s. If the channel begins to meander near the levee toe, riprap should be considered.					
Bend 6	Riprap is not recommended at this time due to the wide bench present on both sides of the low flow channel and overbank velocities less than 5 ft/s. If the channel begins to meander near the levee toe, riprap should be considered.					
Bend 7	32+00 - 29+53 (Right Floodwall) 29+53 – 26+25 (Right Levee and Backside of peninsula) 29+50 – 25+00 (Left Levee) 32+50 – 27+75 (Friendship Bridge Abutment Island)	¼ - Ton	12.6	24.0	2.1	36.0
Straight 2[2]	25+00 – 16+00 (Left Levee)	¼ - Ton	12.6	24.0	2.1	24.0

Geotextile Design

A granular filter layer was first considered, however, the span between the bank revetment and the native soil would require a minimum of three filter layers at 0.5 feet in thickness. Due to limited channel cross sectional area, a geotextile filter fabric is recommended. The filter layer was selected by applying the Federal Highway Administration's (FHWA) Hydraulic Engineering Circular (HEC) 23 Design Guide 16.

Limited geotechnical information located within the channel was used to determine the getextile for soil retention. Figure 16.3 of HEC-23 Design Guide 16 was followed and included in Attachment H. Boring data is limited within the channel, however, from the available information it can be assumed that the native soil is more than 30% clay with a d_{30} less than 0.002 mm.

Results

The geotextile recommended for placement beneath the rock slope protection shall be a non-woven geotextile product equivalent to TenCate Mirafi 1120N or approved equal. This geotextile selected satisfied all recommended tests and allowable values for geotextile propperties per Table 16.2 of the HEC-23 Design Guide 16. Design calculations have been included in Attachment H.

Overtopping Flow

When flow overtops an embankment, such as the terraced mitigation area downstream of the Palo Alto Pumping Plant, locally high velocities can occur at the downstream shoulder of the levee crest. When the tailwater is low relative to the crest of the levee slope, the flow will continue to accelerate along the downstream slope causing erosion of the terraced 3H:1V side slope. This condition will be present when peak flows begin to recede and the water surface elevation begins to drop. Overtopping flow, using NCHRP Report 568 (Mishra, 1998) equation, was analyzed at the terraced area immediately downstream of the Palo Alto pumping plant from left bank STA 73+50 to STA 68+00 (HEC-RAS STA 7418 – 6963), as shown below.

Equation 5 Overtopping Flow Interstitial Velocity

$$V_i = 2.48\sqrt{g d_{50}}\left(\frac{S^{0.58}}{C_u^{2.22}}\right)$$

where V_i = Interstitial velocity, ft/s (m/s)
g = Acceleration due to gravity, 32.2 ft/s^2 (9.81 m/s^2)
d_{50} = Particle size for which 50% is finer by weight, ft (m)
S = Slope of the embankment, ft/ft (m/m)
C_u = Coefficient of uniformity of the riprap, d_{60}/d_{10}

Equation 6 Overtopping Flow D_{50}

$$d_{50} = \frac{K_u q_f^{0.52}}{C_u^{0.25} S^{0.75}}\left(\frac{\sin\alpha}{(S_g\cos\alpha - 1)(\cos\alpha\tan\phi - \sin\alpha)}\right)^{1.11}$$

where d_{50} = Particle size for which 50% is finer by weight, ft (m)
K_u = 0.525 for English units
0.55 for SI units
q_f = Unit discharge at failure, ft^3/s/ft (m^3/s/m)
C_u = Coefficient of uniformity of the riprap, d_{60}/d_{10}
S = Slope of the embankment, ft/ft (m/m)
S_g = Specific gravity of the riprap
α = Slope of the embankment, degrees
ϕ = Angle of repose of the riprap, degrees

Results

Hand calculations using the equations above were used to compute a recommended riprap size considering overtopping. Hand calculations for evaluating overtopping have been included in Attachment I. Hand calculations determined that the computed rock size considering overtopping was smaller than what was sized for standard bank revetment computed for Bend 1 and Straight 1. Therefore, the bank revetment computed rock size is recommended for the 3H:1V side slope at the terraced mitigation area as shown in Table 5.

Rock Slope Protection from Wave Attack

Due to the proposed project, the existing left levee from Stations 25+00 to 16+00 will now be opened to wind and wave runup action made possible by the degrade of the right levee. HDR has conducted a basic wind and wave analysis to approximate wind and wave run-up. Back-up calculations have been including in Attachment J. It was originally anticipated that the USACE's Shoreline Study would be available for information; however, the document has still not yet been released. FEMA is also independently developing a South Bay Study which will include recommended tidal elevations. Since these two studies are not yet available, HDR went ahead and conducted a basic wind and wave analysis to approximate wind and wave run-up as well as input parameter to FHWA's HEC-23 Design Guideline No. 17 (Hudson Method and Pilarczyk Method) equations, as shown below.

Equation 7 Hudson Method for Wave Attack W_{50}

$$W_{50} = \frac{\gamma_r H^3 (\tan\theta)}{K_d (S_r - S_w)^3}$$

Equation 8 Hudson Method for Wave Attack d_{50}

$$d_{50} = \sqrt[3]{\frac{W_{50}}{0.85\gamma_r}}$$

where:

W_{50}	=	Weight of the median riprap particle size, lb (kg)
γ_r	=	Unit weight of riprap, lb/ft^3 (kg/m^3)
H	=	Design wave height, ft (m) (Note: Minimum recommended value for use with the Hudson equation is the 10 percent wave, $H_{0.10} = 1.27H_s$)
K_d	=	Empirical coefficient equal to 2.2 for riprap
S_r	=	Specific gravity of riprap
S_w	=	Specific gravity of water (1.0 for fresh water, 1.03 for seawater)
θ	=	Angle of slope inclination

Equation 9 Pilarczyk Method for Wave Attack ξ

$$\xi = \frac{\tan\theta}{\sqrt{H_s / L_o}} = \tan\theta \frac{K_u T}{\sqrt{H_s}}$$

Equation 10 Pilarczyk Method for Wave Attack d_{50}

$$d_{50} \geq \frac{2}{3}\left(\frac{H_s \xi^{0.5}}{1.64 \cos\theta}\right)$$

where:

ξ	=	Dimensionless breaker parameter
θ	=	Angle of slope inclination
L_o	=	Wave length, ft (m)
H_s	=	Significant wave height, ft (m)
T	=	Wave period, sec
K_u	=	Coefficient equal to 2.25 for wave height in ft, and 1.25 for wave height in m

Results

Rock slope protection is recommended along the existing left levee from Stations 25+00 to 16+00 to protect levee from wind and wave runup action now made possible by the degrade of the right levee. It is extremely important to note that the addition of rock slope protection on the waterside slope of the existing levee will not improve the levee integrity. The placement of rock slope protection is only to reduce wind wave run up on the existing levee slope, and does not protect levee integrity in overtopping flows. This segment of levee does not provide adequate freeboard above the design water surface elevation and is irregular in slope and size. It is assumed that this levee is not geotechnically sound and most likely has seepage and stability issues. HDR has been informed that this levee segment will be part of a future SFCJPA project to be repaired in the near future. The proposed riprap protection may be modified as part of the future project.

Hand calculations for evaluating wave attack have been included in Attachment J. Hand calculations determined that the computed rock size considering wave attack across the Faber Tract hitting the existing levee at the near the Palo Alto Airport was slightly larger than the recommended rock size computed for bank revetment for Straight 2. Therefore, the recommended rock size is based upon the wave attack calculations. The riprap to be placed will not be keyed down into the existing levee, but placed on top as shown on Attachment G illustrating the proposed riprap placement detail. The levee slope will be cleared and grubbed before a filter layer and riprap be placed.

Bed Scour for Vertical Drop Structures

A concrete vertical drop structure is location in a small channel stemming from the Palo Alto Pump Station into SF Creek. This concrete vertical drop structure has been installed to hold the channel invert; however, a scour hole is now forming on the downstream side of the structure. As-builts for the concrete drop structure show that the structure extends 4 feet deep. Stage-discharge information from the pumping plant is unknown. Figure 1 below provides a photograph of the vertical drop structure downstream of the Palo Alto Pumping Plant.

Figure 1 - Photograph of Vertical Drop Structure Downstream of Palo Alto Pumping Plant

Hand calculations have been performed using FHWA's HEC-23 Design Guide 3 (Pemberton and Lara 1984) equation to estimate potential local scour depths using estimated data inputs. The equation is shown below.

Equation 11 Scour Depth from Drop Structure

$$d_s = K_u H_t^{0.225} q^{0.54} - d_m$$

where:

d_s	=	local scour depth for a free overfall, measured from the streambed downstream of the drop, ft (m)
q	=	discharge per unit width, cfs/ft (m^3/s/m)
H_t	=	total drop in head, measured from the upstream to the downstream energy grade line, ft (m)
d_m,Y_d	=	tailwater depth, ft (m)
K_u	=	1.32 (English)
K_u	=	1.90 (SI)

Results

Since pumping plant release data is unknown, a range of estimate unit discharges have been assumed to compute estimated scour depths. Computation results are included in Table 6 below with backup calculations in Attachment K.

Table 6 Recommended Gradation Curve for Riprap Layer

Assumed Pumping Plant releases (cfs)	Measured channel width across structure (ft)	Computed Scour Depth (ft)
100 cfs	20	2.0
500 cfs	20	7.6

HDR recommends that the vertical drop structure should be monitored and repaired through the existing maintenance program and should not be included as part of construction. According to the as-builts, the structure is very close to becoming undermined. If the structure is undermined, significant erosion may occur upstream of the structure heading towards the pumping plant. The repairing agency may want to consider installing a second structure downstream to decreasing the drop elevation. It also may be worth considering additional rock slope protection below the structures.

HDR is not anticipating the proposed floodwall alignment and access ramp to increase erosion in the pumping plant channel, however, both the channel and SF Creek will need to be heavily monitored after the first few storms to see how the channel responds. Fairly dense mitigation plantings located on the slope of the channel may provide some additional protection from erosion. The dense vegetation may negate the need for riprap in this location. This will be determined through monitoring and maintenance.

Summary

Rock slope protection is recommended at multiple locations along the channel to protect against various forms of erosion as well as around the Friendship Bridge Island and Peninsula. See Attachment G for 95% plan sheets illustrating locations where rock slope protection is recommended.

Since Friendship Bridge and Boardwalk span the entire channel without active channel flow area reduction from abutments, the computation of abutment scour was not necessary. The large abutment "island" to remain in the middle of the channel has been treated as a levee and will be protected with placed riprap around the entire diameter of in channel island extending below the MHHW bench design surface to the required depth of riprap computed. Although the channel flow is split around the in channel island, the overall channel width is expanding headed downstream from a channel width of 185 feet to 240 feet through the bridge structure. Due to the increase in channel width traveling downstream through the bridge structure, contraction scour was not computed.

The piers to be installed for Friendship Boardwalk has been designed to withstand pier scour. The Boardwalk should be monitored for any signs of scour. Note scour at piers can be difficult to identify since pier scour usually will fill back in with sediment after flood water recedes. Therefore a survey of the bridge location may be recommended to ensure bridge is not experiencing any movement after large flooding events.

Additional riprap protection may need to be installed at a later time if erosion begins to occur at other locations along SF Creek. Stringent O&M procedures are included in the O&M Plan to ensure the channel performs as designed.

References

Federal Highway Administration's (FHWA) Hydraulic Engineering Circular (HEC) 23 – Bridge Scour and Stream Instability Countermeasures, September 2009.

National Cooperative Highway Research Program (NCHRP) Report 568 – Riprap Design Criteria, Recommended Specifications, and Quality Control, 2006.

USACE Engineer Manual No. 1110-2-1601 – Hydraulic Design of Flood control Channels, June 1994.

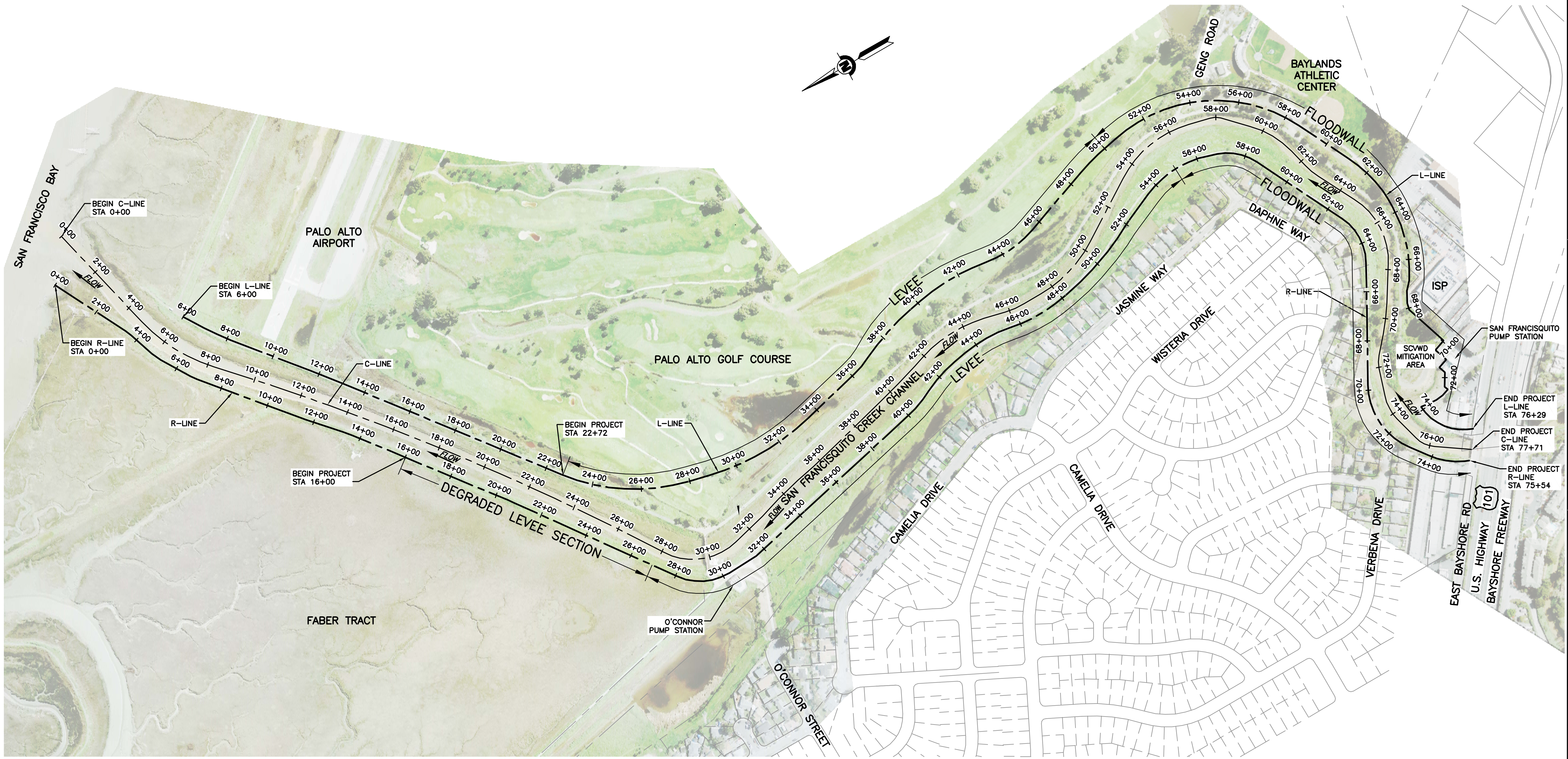
USACE Engineering Technical Letter No. 1110-2-571 – April 2009

Attachments

Attachment A – General Site Plan
Attachment B –USACE's EM 1110-02-1601 Suggested Max. Permissible Mean Channel Velocities Table
Attachment C – Toe Scour Hand Calculations
Attachment D – Revetment Toe Protection Hand Calculations
Attachment E – Bend and Segment Bank Revetment Analysis Location
Attachment F – CHNLPRO Model Results for Bank Revetment
Attachment G – Final Design Detailed Cross Sections for Rock Placement
Attachment H – Geotextile Design Hand Calculations
Attachment I – Overtopping Design Hand Calculations
Attachment J – Wind and Wave Runup and Wave Attack Hand Calculations
Attachment K –Vertical Drop Hand Calculations

Attachment A

General Site Plan



GENERAL SITE PLAN

SCALE: 1"=200'

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HDR Engineering, Inc.

DATE 07-13-12
DESIGN L. JONES
DRAWN H. SUAREZ
CHECKED B. JOHNSON

ENGINEERING CERTIFICATION
REGISTERED PROFESSIONAL ENGINEER SERGIO E. JIMENEZ C 055698 Exp. 12-31-12 CIVIL STATE OF CALIFORNIA
PRELIMINARY
PROJECT ENGINEER DATE

SAN FRANCISQUITO CREEK JOINT POWERS AUTHORITY
ACCEPTED BY DISTRICT
PROJECT ENGINEER DATE

PROJECT NAME AND SHEET DESCRIPTION:
SAN FRANCISQUITO CREEK EARLY IMPLEMENTATION PROJECT EAST BAYSHORE RD. TO S.F. BAY
GENERAL SITE PLAN

SCALE 1" = 200'
VERIFY SCALES 0 1"
BAR IS ONE INCH ON ORIGINAL DRAWING IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PROJECT NUMBER 26284002
SHEET CODE: G-4
SHEET NUMBER: 4 OF 94

Attachment B

US Army Corps of Engineer's (USACE) Engineer Manual (EM) 1110-02-1601 Suggested Maximum Permissible Mean Channel Velocities Table

critical scour velocities is given by the Task Committee on Preparation of Sedimentation Manual (1966). Table 2-5 gives a set of permissible velocities that can be used as a guide to design nonscouring flood control channels. Lane (1955) presents curves showing permissible channel shear stress to be used for design, and the Soil Conservation Service (1954) presents information on grass-lined channels. Departures from suggested permissible velocity or shear values should be based on reliable field experience or laboratory tests. Channels whose velocities and/or shear exceed permissible values will require paving or bank revetment. The permissible values of velocity and/or shear should be determined so that damage exceeding normal maintenance will not result from any flood that could be reasonably expected to occur during the service life of the channel.

Table 2-5
Suggested Maximum Permissible Mean Channel Velocities

Channel Material	Mean Channel Velocity, fps
Fine Sand	2.0
Coarse Sand	4.0
Fine Gravel[1]	6.0
Earth	
Sandy Silt	2.0
Silt Clay	3.5
Clay	6.0
Grass-lined Earth (slopes less than 5%)[2]	
Bermuda Grass	
Sandy Silt	6.0
Silt Clay	8.0
Kentucky Blue Grass	
Sandy Silt	5.0
Silt Clay	7.0
Poor Rock (usually sedimentary)	10.0
Soft Sandstone	8.0
Soft Shale	3.5
Good Rock (usually igneous or hard metamorphic)	20.0

Notes:
1. For particles larger than fine gravel (about 20 millimetres (mm) = 3/4 in.), see Plates 29 and 30.
2. Keep velocities less than 5.0 fps unless good cover and proper maintenance can be obtained.

Attachment C

Toe Scour Hand Calculations

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Project: SFCJPA		Computed: EKM	Date: 8/20/12
Subject: Toe Scour for ~~Depth~~		Checked:	Date:
Task: of revetment		Page: 1	of: 2
Job #:		No:	

Reference: BOR Computing Degradation and Local Scour
Channel Scour during peak flood flows
(Toe scour for depth of revetment)
Results for a Moderate Bend

p.34

Neill

$$d_f = d_i\left(\frac{q_f}{q_i}\right)^m$$

d_f = Scour depth below design floodwater level

q_f = Design flood discharge per unit width

$$= \frac{9300 \text{ cfs}}{200 \text{ ft}} \; ft^3/s$$

$q_f = 46.5 \; ft^3/s$

$d_i = 3.1 ft$

$m = 0.67$

$$q_i = \frac{200 \text{ cfs}}{45 \text{ ft}}$$

$q_i = 4.44 \; ft^2/s$

$$d_f = 3.1\left(\frac{46.5}{4.44}\right)^{0.67}$$

$d_f = 14.96$

$d_s = Z d_f$

Moderate Bend $Z = 0.6$

$d_s = (0.6)(14.96)$

$\boxed{d_s = 9.0 ft}$

Lacey

$$d_m = 0.47\left(\frac{Q}{f}\right)^{1/3}$$

$Q = 9300$ cfs

$D_m = 0.075$ mm

$f = 1.76(D_m)^{1/2}$

$f = 1.76(0.075)^{1/2}$

$f = 0.48$

$$d_m = 0.47\left(\frac{9300 \text{ cfs}}{0.48}\right)^{1/3}$$

$d_m = 12.62 ft$

$d_s = Z d_m$

moderate Bend $= 0.5$

$d_s = (0.5)(12.62)$

$\boxed{d_s = 6.3 ft}$

Blench

$$d_{fo} = \frac{q_f^{2/3}}{F_{bo}^{1/3}}$$

$q_f = 46.5 \; ft^3/s$

$F_{bo} = 1.6$ (figure 9)

$$d_{fo} = \frac{(46.5)^{2/3}}{(0.5)^{1/3}}$$

$d_{fo} = 16.29 ft$

$d_s = Z d_{fo}$

moderate Bend $= 0.6$

$d_s = (0.6)(16.29)$

$\boxed{d_s = 9.8 ft}$

Results Summary for Moderate Bend

Neill Eqn, ds = 9.0 ft
Lacey Eqn, ds = 6.3 ft
Blench Eqn, ds = 9.8 ft

$\boxed{\text{avg} = 8.3 \text{ ft}}$ computed scour during peak flood flows at a moderate bend

Results Summary for Straight Channel

Neill	LACEY	Blench
Z = 0.5 for straight reach	Z = 0.25 for straight reach	Z = 0.6 for straight reach
ds = (0.5)(14.96)	ds = (0.25)(12.62)	ds = (0.6)(16.26)
$\boxed{\text{ds} = 7.5 \text{ ft}}$	$\boxed{\text{ds} = 3.2 \text{ ft}}$	$\boxed{\text{ds} = 9.8 \text{ ft}}$

Neill Eqn, ds = 7.5 ft
Lacey Eqn, ds = 3.2 ft
Blench Eqn, ds = 9.8 ft

$\boxed{\text{average} = 6.8 \text{ ft}}$ computed scour during peak flood flows at a moderate bend

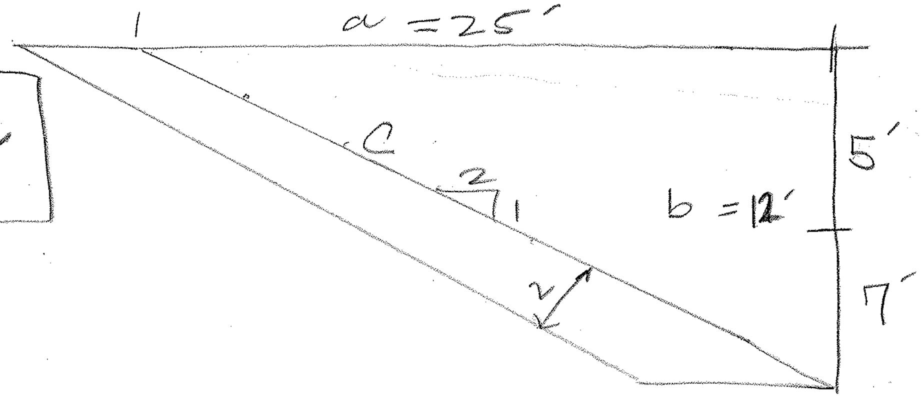
Attachment D

Revetment Toe Protection Hand Calculations

Rock Slope Key Design

Guidance EM 1110-2-1601

When $t = 2'$



$$a^2 + b^2 = c^2$$

$$25^2 + 12^2 = c^2$$

$$27.7' = c$$

$$A = 2 \times 27.7 = 55.5 \text{ ft}^2$$

assuming a dry install,

Total Volume $= (1.25)(\text{Thick})(12)(\sqrt{5})$

$(1.25)(2)(12)(\sqrt{5})$

Total Volume required for key $= \boxed{67.1 \text{ ft}^3}$ ← Key must be sized to hold same volume

---

When $t = 3'$ $\quad A = 3' \times 22.7 = 68.1 \text{ ft}^2$

total volume $= (1.25)(3)(12)(\sqrt{5})$

total volume required for key $= \boxed{100.6 \text{ ft}^3}$ ← Key must be sized to hold same volume

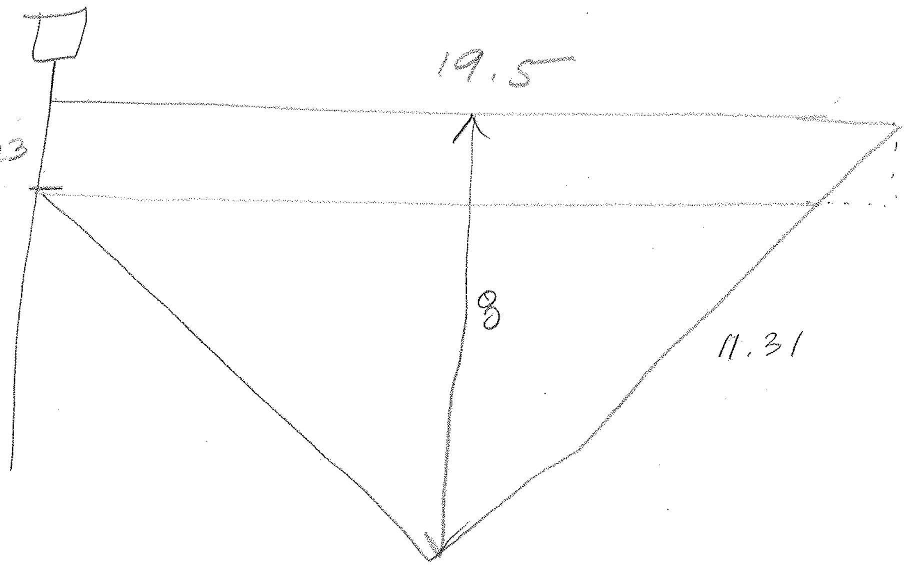
Project: SFCJPA | Computed: EKM | Date: 9/25/12

Subject: Revetment toe Protection | Checked: | Date:

Task: | Page: 2 | of: 4

Job #: | No:

For Floodwall



$area = 19.5 \times 2.23 = 43.5\ ft^2$

$+$

$(5.77)(19.5)(1/2) = 56.25\ ft$

total area = 99.75 ft

Since $99.75\ ft^3 > 67.1\ ft^3$ Key Sized appropriately

Project: SFCJPA		Computed: EKM	Date: 9/25/12
Subject: Revetment Toe Protection		Checked:	Date:
Task:		Page: 3	of: 4
Job #:		No:	

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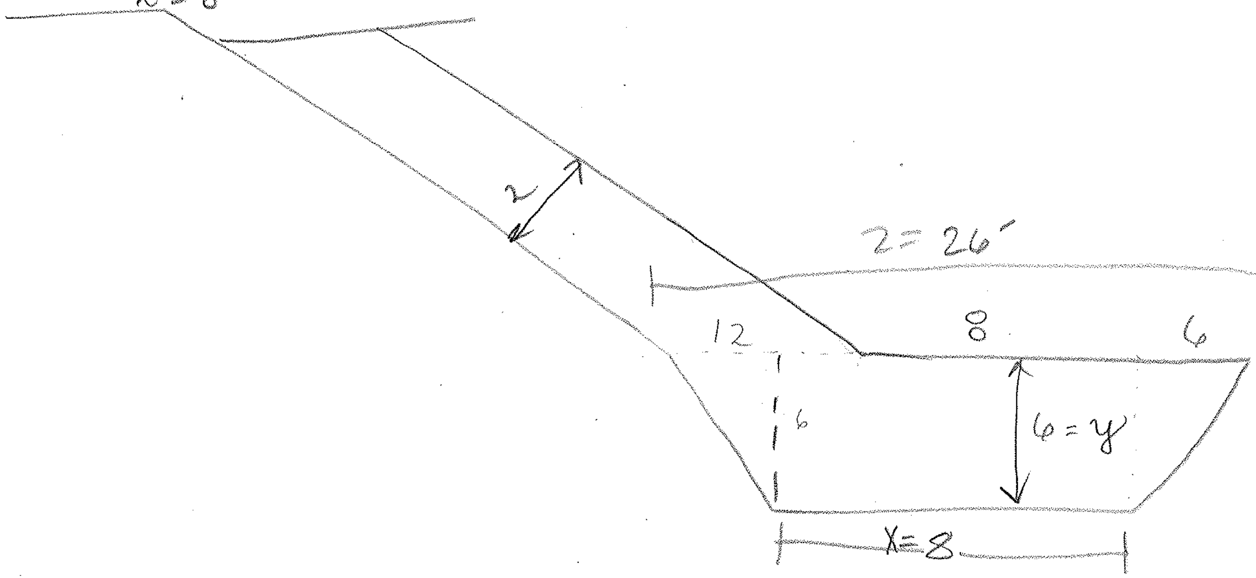
When $t = 2'$
$y = 6'$
$x = 8'$



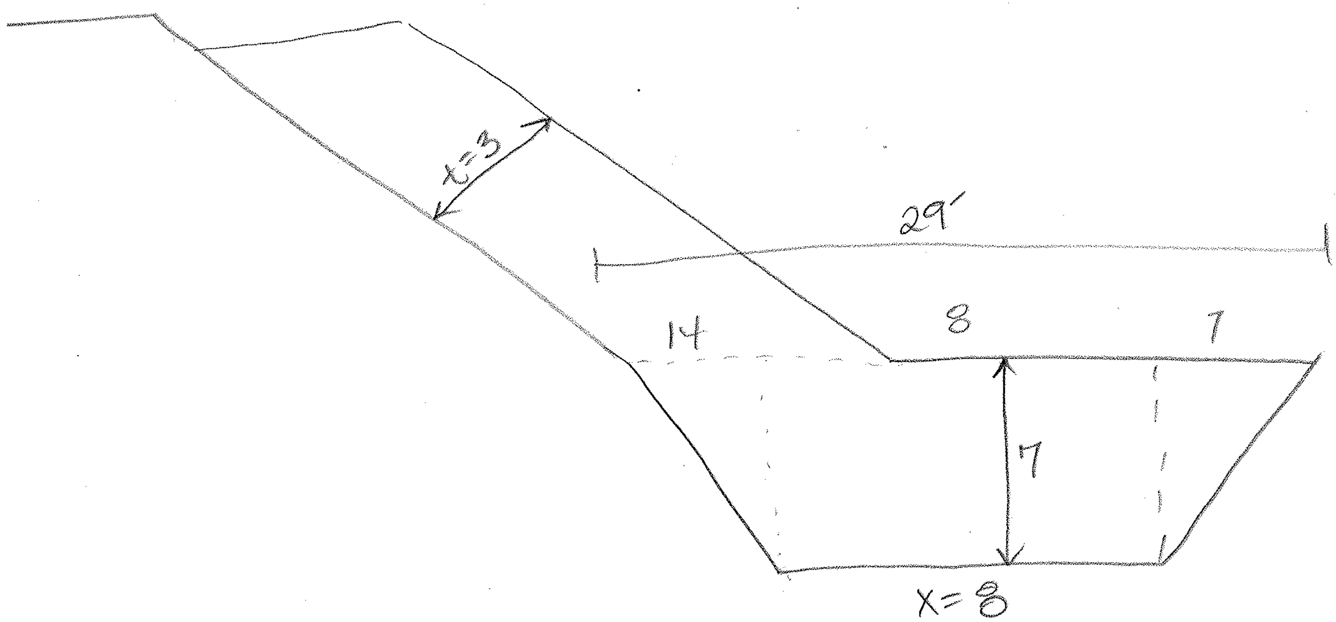
$$Area = \frac{1}{2}(6)\left(8 + (12+8+6)\right)$$

$$Area = 102\ ft^2$$

Since $102 ft^3 > 67.1\ ft^3$ Key sized appropriately

Project: SFCJPA		Computed: EKM	Date: 9/25/12
Subject: Revetment Toe Protection		Checked:	Date:
Task:		Page: 4	of: 4
Job #:		No:	

When $t = 3'$
$y = 7'$
$x = 8'$

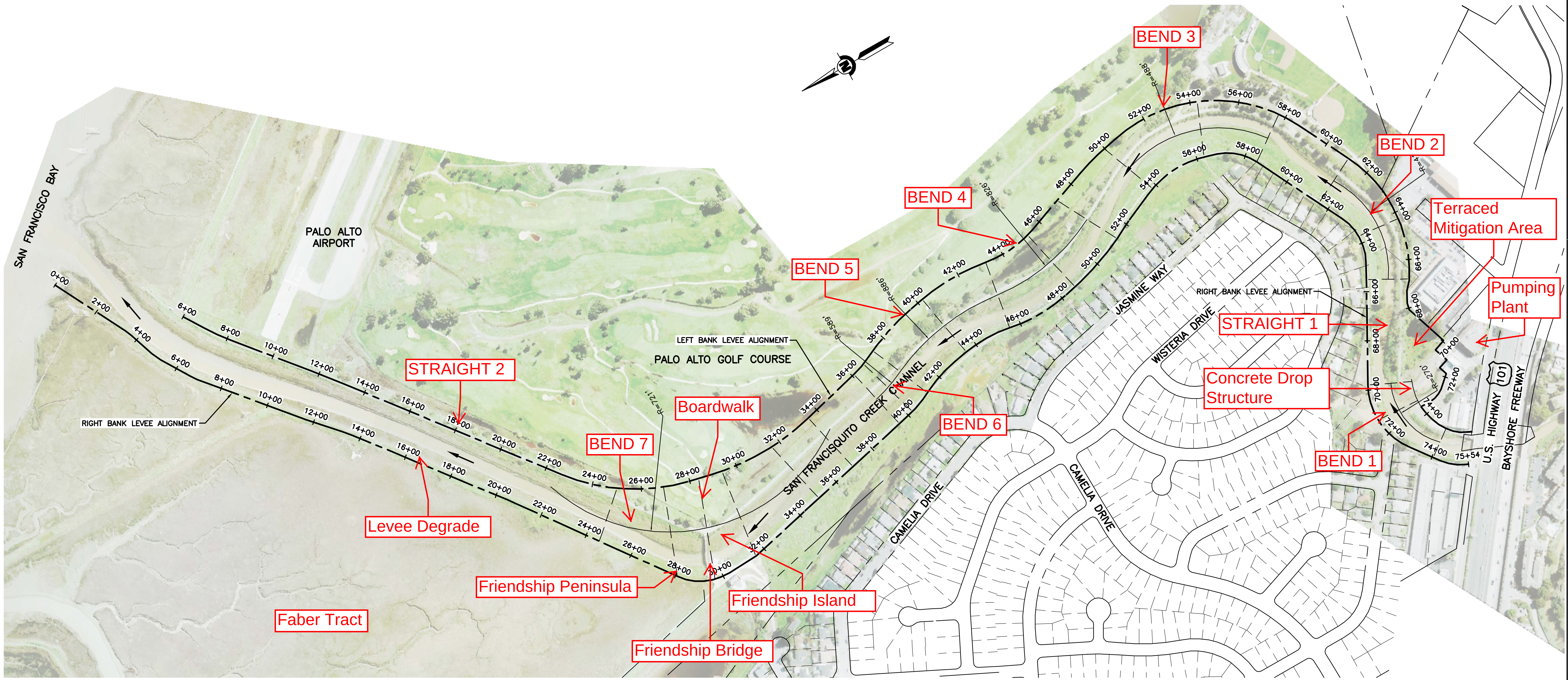


$$\text{Area} = \frac{1}{2}(7)(8 + 29) = 129.5' \ ft^2$$

Since $129.5\ ft^3 > 101\ ft^3$ Key sized appropriately

Attachment E

Bend and Segment Bank Revetment Analysis Location



GENERAL SITE PLAN

SCALE: 1"=200'

REV	DESCRIPTION	DATE	APPR.
	PRELIMINARY 90% 04-XX-2012		

HDR
HDR Engineering, Inc.

DATE 03-02-12
DESIGN L. JONES
DRAWN H. SUAREZ
CHECKED B. JOHNSON

ENGINEERING CERTIFICATION
REGISTERED PROFESSIONAL ENGINEER SERGIO E. JIMENEZ C 60598 Exp. 12-31-12 CIVIL STATE OF CALIFORNIA
PRELIMINARY
PROJECT ENGINEER DATE

SAN FRANCISQUITO CREEK
JOINT POWERS AUTHORITY
ACCEPTED BY DISTRICT
PROJECT ENGINEER DATE

PROJECT NAME AND SHEET DESCRIPTION:
SAN FRANCISQUITO CREEK
FLOOD PROTECTION CAPITAL PROJECT
EAST BAYSHORE RD. TO S.F. BAY
GENERAL SITE PLAN

SCALE 1" = 200'
VERIFY SCALES
BAR IS ONE INCH ON ORIGINAL DRAWING IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PROJECT NUMBER 10284008
SHEET CODE: G-004
SHEET NUMBER: 4 OF 103

DOCUMENT NUMBER: SFC_LP-G-1028-XXXXXXX

Attachment F

CHNLPRO Model Results for Bank Revetment

Bend 1 9/14/12 ETL Gradation

PROGRAM OUTPUT FOR A CHANNEL WITH A KNOWN LOCAL
DEPTH AVERAGED VELOCITY, BENDWAY
INPUT PARAMETERS

SPECIFIC WEIGHT OF STONE,PCF	165.0
MINIMUM CENTER LINE BEND RADIUS,FT	270.0
WATER SURFACE WIDTH,FT	224.0
LOCAL FLOW DEPTH,FT	15.4
CHANNEL SIDE SLOPE,1 VER: 4.00 HORZ	
LOCAL DEPTH AVG VELOCITY,FPS	9.10
SIDE SLOPE CORRECTION FACTOR K1	1.00
CORRECTION FOR VELOCITY PROFILE IN BEND	1.22
RIPRAP DESIGN SAFETY FACTOR	1.30

SELECTED STABLE GRADATIONS
ETL GRADATION

NAME	COMPUTED D30 FT	D30(MIN) FT	D100(MAX) IN	D85/D15	N=THICKNESS/ D100(MAX)	CT	THICKNESS IN
1	.37	.37	9.00	1.70	1.60	.88	14.4
2	.42	.48	12.00	1.70	1.00	1.00	12.0

D100(MAX) IN	LIMITS OF STONE WEIGHT,LB FOR PERCENT LIGHTER BY WEIGHT 100		50		15		D30(MIN) FT	D90(MIN) FT
9.00	36	15	11	7	5	2	.37	.53
12.00	86	35	26	17	13	5	.48	.70

EQUIVALENT SPHERICAL DIAMETERS IN INCHES

D100(MAX)	D100(MIN)	D50(MAX)	D50(MIN)	D15(MAX)	D15(MIN)
9.0	6.6	6.0	5.3	4.8	3.6
12.0	8.8	8.0	7.0	6.3	4.8

PROGRAM OUTPUT FOR A CHANNEL WITH A KNOWN LOCAL
DEPTH AVERAGED VELOCITY, BENDWAY
INPUT PARAMETERS

SPECIFIC WEIGHT OF STONE,PCF	165.0
MINIMUM CENTER LINE BEND RADIUS,FT	412.0
WATER SURFACE WIDTH,FT	172.0
LOCAL FLOW DEPTH,FT	14.4
CHANNEL SIDE SLOPE,1 VER: 4.00 HORZ	
LOCAL DEPTH AVG VELOCITY,FPS	9.40
SIDE SLOPE CORRECTION FACTOR K1	1.00
CORRECTION FOR VELOCITY PROFILE IN BEND	1.21
RIPRAP DESIGN SAFETY FACTOR	1.30

SELECTED STABLE GRADATIONS
ETL GRADATION

NAME	COMPUTED D30 FT	D30(MIN) FT	D100(MAX) IN	D85/D15	N=THICKNESS/ D100(MAX)	CT	THICKNESS IN
1		.37	9.00	1.70	NOT STABLE		
2	.46	.48	12.00	1.70	1.00	1.00	12.0

D100(MAX) IN	LIMITS OF STONE WEIGHT,LB FOR PERCENT LIGHTER BY WEIGHT 100		50		15		D30(MIN) FT	D90(MIN) FT
12.00	86	35	26	17	13	5	.48	.70

EQUIVALENT SPHERICAL DIAMETERS IN INCHES

D100(MAX)	D100(MIN)	D50(MAX)	D50(MIN)	D15(MAX)	D15(MIN)
12.0	8.8	8.0	7.0	6.3	4.8

PROGRAM OUTPUT FOR A CHANNEL WITH A KNOWN LOCAL
DEPTH AVERAGED VELOCITY, BENDWAY
INPUT PARAMETERS

SPECIFIC WEIGHT OF STONE,PCF 165.0
MINIMUM CENTER LINE BEND RADIUS,FT 488.0
WATER SURFACE WIDTH,FT 215.0
LOCAL FLOW DEPTH,FT 13.9
CHANNEL SIDE SLOPE,1 VER: 4.00 HORZ
LOCAL DEPTH AVG VELOCITY,FPS 9.00
SIDE SLOPE CORRECTION FACTOR K1 1.00
CORRECTION FOR VELOCITY PROFILE IN BEND 1.21
RIPRAP DESIGN SAFETY FACTOR 1.30

SELECTED STABLE GRADATIONS
ETL GRADATION

NAME	COMPUTED D30 FT	D30(MIN) FT	D100(MAX) IN	D85/D15	N=THICKNESS/ D100(MAX)	CT	THICKNESS IN
1	.37	.37	9.00	1.70	1.54	.89	13.8
2	.42	.48	12.00	1.70	1.00	1.00	12.0

D100(MAX) IN	LIMITS OF STONE WEIGHT,LB FOR PERCENT LIGHTER BY WEIGHT 100		50		15		D30(MIN) FT	D90(MIN) FT
9.00	36	15	11	7	5	2	.37	.53
12.00	86	35	26	17	13	5	.48	.70

EQUIVALENT SPHERICAL DIAMETERS IN INCHES

D100(MAX)	D100(MIN)	D50(MAX)	D50(MIN)	D15(MAX)	D15(MIN)
9.0	6.6	6.0	5.3	4.8	3.6
12.0	8.8	8.0	7.0	6.3	4.8

PROGRAM OUTPUT FOR A CHANNEL WITH A KNOWN LOCAL
DEPTH AVERAGED VELOCITY, BENDWAY
INPUT PARAMETERS

SPECIFIC WEIGHT OF STONE,PCF	165.0
MINIMUM CENTER LINE BEND RADIUS,FT	826.0
WATER SURFACE WIDTH,FT	260.0
LOCAL FLOW DEPTH,FT	6.9
CHANNEL SIDE SLOPE,1 VER: 3.00 HORZ	
LOCAL DEPTH AVG VELOCITY,FPS	9.60
SIDE SLOPE CORRECTION FACTOR K1	.99
CORRECTION FOR VELOCITY PROFILE IN BEND	1.18
RIPRAP DESIGN SAFETY FACTOR	1.30

SELECTED STABLE GRADATIONS
ETL GRADATION

NAME	COMPUTED D30 FT	D30(MIN) FT	D100(MAX) IN	D85/D15	N=THICKNESS/ D100(MAX)	CT	THICKNESS IN
2		.48	12.00	1.70	NOT STABLE		
3	.58	.61	15.00	1.70	1.00	1.00	15.0

D100(MAX) IN	LIMITS OF STONE WEIGHT,LB FOR PERCENT LIGHTER BY WEIGHT 100		50		15		D30(MIN) FT	D90(MIN) FT
15.00	169	67	50	34	25	11	.61	.88

EQUIVALENT SPHERICAL DIAMETERS IN INCHES

D100(MAX)	D100(MIN)	D50(MAX)	D50(MIN)	D15(MAX)	D15(MIN)
15.0	11.1	10.0	8.8	7.9	6.0

PROGRAM OUTPUT FOR A CHANNEL WITH A KNOWN LOCAL
DEPTH AVERAGED VELOCITY, BENDWAY
INPUT PARAMETERS

SPECIFIC WEIGHT OF STONE,PCF	165.0
MINIMUM CENTER LINE BEND RADIUS,FT	886.0
WATER SURFACE WIDTH,FT	260.0
LOCAL FLOW DEPTH,FT	6.6
CHANNEL SIDE SLOPE,1 VER: 3.00 HORZ	
LOCAL DEPTH AVG VELOCITY,FPS	9.50
SIDE SLOPE CORRECTION FACTOR K1	.99
CORRECTION FOR VELOCITY PROFILE IN BEND	1.18
RIPRAP DESIGN SAFETY FACTOR	1.30

SELECTED STABLE GRADATIONS
ETL GRADATION

NAME	COMPUTED D30 FT	D30(MIN) FT	D100(MAX) IN	D85/D15	N=THICKNESS/ D100(MAX)	CT	THICKNESS IN
1		.37	9.00	1.70	NOT STABLE		
2	.48	.48	12.00	1.70	1.89	.84	22.7
3	.57	.61	15.00	1.70	1.00	1.00	15.0

D100(MAX) IN	LIMITS OF STONE WEIGHT,LB FOR PERCENT LIGHTER BY WEIGHT 100		50		15		D30(MIN) FT	D90(MIN) FT
12.00	86	35	26	17	13	5	.48	.70
15.00	169	67	50	34	25	11	.61	.88

EQUIVALENT SPHERICAL DIAMETERS IN INCHES

D100(MAX)	D100(MIN)	D50(MAX)	D50(MIN)	D15(MAX)	D15(MIN)
12.0	8.8	8.0	7.0	6.3	4.8
15.0	11.1	10.0	8.8	7.9	6.0

PROGRAM OUTPUT FOR A CHANNEL WITH A KNOWN LOCAL
DEPTH AVERAGED VELOCITY, BENDWAY
INPUT PARAMETERS

SPECIFIC WEIGHT OF STONE,PCF	165.0
MINIMUM CENTER LINE BEND RADIUS,FT	589.0
WATER SURFACE WIDTH,FT	260.0
LOCAL FLOW DEPTH,FT	6.6
CHANNEL SIDE SLOPE,1 VER: 3.00 HORZ	
LOCAL DEPTH AVG VELOCITY,FPS	11.30
SIDE SLOPE CORRECTION FACTOR K1	.99
CORRECTION FOR VELOCITY PROFILE IN BEND	1.21
RIPRAP DESIGN SAFETY FACTOR	1.30

SELECTED STABLE GRADATIONS
ETL GRADATION

NAME	COMPUTED D30 FT	D30(MIN) FT	D100(MAX) IN	D85/D15	N=THICKNESS/ D100(MAX)	CT	THICKNESS IN
4		.73	18.00	1.70	NOT STABLE		
5	.85	.85	21.00	1.70	1.25	.94	26.2
6	.90	.97	24.00	1.70	1.00	1.00	24.0

D100(MAX) IN	LIMITS OF STONE WEIGHT,LB FOR PERCENT LIGHTER BY WEIGHT 100		50		15		D30(MIN) FT	D90(MIN) FT
21.00	463	185	137	93	69	29	.85	1.23
24.00	691	276	205	138	102	43	.97	1.40

EQUIVALENT SPHERICAL DIAMETERS IN INCHES

D100(MAX)	D100(MIN)	D50(MAX)	D50(MIN)	D15(MAX)	D15(MIN)
21.0	15.5	14.0	12.3	11.1	8.3
24.0	17.7	16.0	14.0	12.7	9.5

PROGRAM OUTPUT FOR A CHANNEL WITH A KNOWN LOCAL
DEPTH AVERAGED VELOCITY, BENDWAY
INPUT PARAMETERS

SPECIFIC WEIGHT OF STONE,PCF	165.0
MINIMUM CENTER LINE BEND RADIUS,FT	721.0
WATER SURFACE WIDTH,FT	280.0
LOCAL FLOW DEPTH,FT	5.2
CHANNEL SIDE SLOPE,1 VER: 3.00 HORZ	
LOCAL DEPTH AVG VELOCITY,FPS	12.40
SIDE SLOPE CORRECTION FACTOR K1	.99
CORRECTION FOR VELOCITY PROFILE IN BEND	1.20
RIPRAP DESIGN SAFETY FACTOR	1.30

SELECTED STABLE GRADATIONS
ETL GRADATION

NAME	COMPUTED D30 FT	D30(MIN) FT	D100(MAX) IN	D85/D15	N=THICKNESS/ D100(MAX)	CT	THICKNESS IN
6		.97	24.00	1.70	NOT STABLE		
7	1.10	1.10	27.00	1.70	1.36	.92	36.8
8	1.20	1.22	30.00	1.70	1.00	1.00	30.0

D100(MAX) IN	LIMITS OF STONE WEIGHT,LB FOR PERCENT LIGHTER BY WEIGHT 100		50		15		D30(MIN) FT	D90(MIN) FT
27.00	984	394	291	197	146	62	1.10	1.59
30.00	1350	540	400	270	200	84	1.22	1.77

EQUIVALENT SPHERICAL DIAMETERS IN INCHES

D100(MAX)	D100(MIN)	D50(MAX)	D50(MIN)	D15(MAX)	D15(MIN)
27.0	19.9	18.0	15.8	14.3	10.7
30.0	22.1	20.0	17.5	15.9	11.9

PROGRAM OUTPUT FOR A NATURAL CHANNEL SIDE SLOPE RIPRAP, STRAIGHT REACH

INPUT PARAMETERS

SPECIFIC WEIGHT OF STONE,PCF	165.0
LOCAL FLOW DEPTH,FT	14.4
CHANNEL SIDE SLOPE,1 VER: 4.00 HORZ	
AVERAGE CHANNEL VELOCITY,FPS	8.00
COMPUTED LOCAL DEPTH AVG VEL,FPS	8.00
(LOCAL VELOCITY)/(AVG CHANNEL VEL)	1.00
SIDE SLOPE CORRECTION FACTOR K1	1.00
CORRECTION FOR VELOCITY PROFILE IN BEND	1.00
RIPRAP DESIGN SAFETY FACTOR	1.30

SELECTED STABLE GRADATIONS
ETL GRADATION

NAME	COMPUTED D30 FT	D30(MIN) FT	D100(MAX) IN	D85/D15	N=THICKNESS/ D100(MAX)	CT	THICKNESS IN
1	.25	.37	9.00	1.70	1.00	1.00	9.0

D100(MAX) IN	LIMITS OF STONE WEIGHT,LB FOR PERCENT LIGHTER BY WEIGHT 100		50		15		D30(MIN) FT	D90(MIN) FT
9.00	36	15	11	7	5	2	.37	.53

EQUIVALENT SPHERICAL DIAMETERS IN INCHES

D100(MAX)	D100(MIN)	D50(MAX)	D50(MIN)	D15(MAX)	D15(MIN)
9.0	6.6	6.0	5.3	4.8	3.6

PROGRAM OUTPUT FOR A NATURAL CHANNEL SIDE SLOPE RIPRAP, STRAIGHT REACH

INPUT PARAMETERS

SPECIFIC WEIGHT OF STONE,PCF	165.0
LOCAL FLOW DEPTH,FT	4.0
CHANNEL SIDE SLOPE,1 VER: 2.00 HORZ	
AVERAGE CHANNEL VELOCITY,FPS	7.60
COMPUTED LOCAL DEPTH AVG VEL,FPS	7.60
(LOCAL VELOCITY)/(AVG CHANNEL VEL)	1.00
SIDE SLOPE CORRECTION FACTOR K1	.88
CORRECTION FOR VELOCITY PROFILE IN BEND	1.00
RIPRAP DESIGN SAFETY FACTOR	1.30

SELECTED STABLE GRADATIONS

ETL GRADATION

NAME	COMPUTED D30 FT	D30(MIN) FT	D100(MAX) IN	D85/D15	N=THICKNESS/ D100(MAX)	CT	THICKNESS IN
1	.36	.37	9.00	1.70	1.00	1.00	9.0

D100(MAX) IN	LIMITS OF STONE WEIGHT,LB FOR PERCENT LIGHTER BY WEIGHT 100		50		15		D30(MIN) FT	D90(MIN) FT
9.00	36	15	11	7	5	2	.37	.53

EQUIVALENT SPHERICAL DIAMETERS IN INCHES

D100(MAX)	D100(MIN)	D50(MAX)	D50(MIN)	D15(MAX)	D15(MIN)
9.0	6.6	6.0	5.3	4.8	3.6

PROGRAM OUTPUT FOR A CHANNEL WITH A KNOWN LOCAL
DEPTH AVERAGED VELOCITY, BENDWAY
INPUT PARAMETERS

SPECIFIC WEIGHT OF STONE,PCF	165.0
MINIMUM CENTER LINE BEND RADIUS,FT	270.0
WATER SURFACE WIDTH,FT	224.0
LOCAL FLOW DEPTH,FT	15.4
CHANNEL SIDE SLOPE,1 VER: 4.00 HORZ	
LOCAL DEPTH AVG VELOCITY,FPS	9.10
SIDE SLOPE CORRECTION FACTOR K1	1.00
CORRECTION FOR VELOCITY PROFILE IN BEND	1.22
RIPRAP DESIGN SAFETY FACTOR	1.30

SELECTED STABLE GRADATIONS
ALTERNATE GRADATION

NAME	COMPUTED D30 FT	D30(MIN) FT	D100(MAX) IN	D85/D15	N=THICKNESS/ D100(MAX)	CT	THICKNESS IN
1/4-TON-#1	.42	1.05	24.00	2.10	1.00	1.00	24.0

PROGRAM OUTPUT FOR A CHANNEL WITH A KNOWN LOCAL
DEPTH AVERAGED VELOCITY, BENDWAY
INPUT PARAMETERS

SPECIFIC WEIGHT OF STONE,PCF	165.0
MINIMUM CENTER LINE BEND RADIUS,FT	412.0
WATER SURFACE WIDTH,FT	172.0
LOCAL FLOW DEPTH,FT	14.4
CHANNEL SIDE SLOPE,1 VER: 4.00 HORZ	
LOCAL DEPTH AVG VELOCITY,FPS	9.40
SIDE SLOPE CORRECTION FACTOR K1	1.00
CORRECTION FOR VELOCITY PROFILE IN BEND	1.21
RIPRAP DESIGN SAFETY FACTOR	1.30

SELECTED STABLE GRADATIONS
ALTERNATE GRADATION

NAME	COMPUTED D30 FT	D30(MIN) FT	D100(MAX) IN	D85/D15	N=THICKNESS/ D100(MAX)	CT	THICKNESS IN
1/4-TON-#1	.46	1.05	24.00	2.10	1.00	1.00	24.0

PROGRAM OUTPUT FOR A CHANNEL WITH A KNOWN LOCAL
DEPTH AVERAGED VELOCITY, BENDWAY
INPUT PARAMETERS

SPECIFIC WEIGHT OF STONE,PCF 165.0
MINIMUM CENTER LINE BEND RADIUS,FT 488.0
WATER SURFACE WIDTH,FT 215.0
LOCAL FLOW DEPTH,FT 13.9
CHANNEL SIDE SLOPE,1 VER: 4.00 HORZ
LOCAL DEPTH AVG VELOCITY,FPS 9.00
SIDE SLOPE CORRECTION FACTOR K1 1.00
CORRECTION FOR VELOCITY PROFILE IN BEND 1.21
RIPRAP DESIGN SAFETY FACTOR 1.30

SELECTED STABLE GRADATIONS
ALTERNATE GRADATION

NAME	COMPUTED D30 FT	D30(MIN) FT	D100(MAX) IN	D85/D15	N=THICKNESS/ D100(MAX)	CT	THICKNESS IN
1/4-TON-#1	.42	1.05	24.00	2.10	1.00	1.00	24.0

PROGRAM OUTPUT FOR A CHANNEL WITH A KNOWN LOCAL
DEPTH AVERAGED VELOCITY, BENDWAY
INPUT PARAMETERS

SPECIFIC WEIGHT OF STONE,PCF	165.0
MINIMUM CENTER LINE BEND RADIUS,FT	826.0
WATER SURFACE WIDTH,FT	260.0
LOCAL FLOW DEPTH,FT	6.9
CHANNEL SIDE SLOPE,1 VER: 3.00 HORZ	
LOCAL DEPTH AVG VELOCITY,FPS	9.60
SIDE SLOPE CORRECTION FACTOR K1	.99
CORRECTION FOR VELOCITY PROFILE IN BEND	1.18
RIPRAP DESIGN SAFETY FACTOR	1.30

SELECTED STABLE GRADATIONS
ALTERNATE GRADATION

NAME	COMPUTED D30 FT	D30(MIN) FT	D100(MAX) IN	D85/D15	N=THICKNESS/ D100(MAX)	CT	THICKNESS IN
1/4-TON-#1	.58	1.05	24.00	2.10	1.00	1.00	24.0

PROGRAM OUTPUT FOR A CHANNEL WITH A KNOWN LOCAL
DEPTH AVERAGED VELOCITY, BENDWAY
INPUT PARAMETERS

SPECIFIC WEIGHT OF STONE,PCF	165.0
MINIMUM CENTER LINE BEND RADIUS,FT	886.0
WATER SURFACE WIDTH,FT	260.0
LOCAL FLOW DEPTH,FT	6.6
CHANNEL SIDE SLOPE,1 VER: 3.00 HORZ	
LOCAL DEPTH AVG VELOCITY,FPS	9.50
SIDE SLOPE CORRECTION FACTOR K1	.99
CORRECTION FOR VELOCITY PROFILE IN BEND	1.18
RIPRAP DESIGN SAFETY FACTOR	1.30

SELECTED STABLE GRADATIONS
ALTERNATE GRADATION

NAME	COMPUTED D30 FT	D30(MIN) FT	D100(MAX) IN	D85/D15	N=THICKNESS/ D100(MAX)	CT	THICKNESS IN
1/4-TON-#1	.57	1.05	24.00	2.10	1.00	1.00	24.0

PROGRAM OUTPUT FOR A CHANNEL WITH A KNOWN LOCAL
DEPTH AVERAGED VELOCITY, BENDWAY
INPUT PARAMETERS

SPECIFIC WEIGHT OF STONE,PCF	165.0
MINIMUM CENTER LINE BEND RADIUS,FT	589.0
WATER SURFACE WIDTH,FT	260.0
LOCAL FLOW DEPTH,FT	6.6
CHANNEL SIDE SLOPE,1 VER: 3.00 HORZ	
LOCAL DEPTH AVG VELOCITY,FPS	11.30
SIDE SLOPE CORRECTION FACTOR K1	.99
CORRECTION FOR VELOCITY PROFILE IN BEND	1.21
RIPRAP DESIGN SAFETY FACTOR	1.30

SELECTED STABLE GRADATIONS
ALTERNATE GRADATION

NAME	COMPUTED D30 FT	D30(MIN) FT	D100(MAX) IN	D85/D15	N=THICKNESS/ D100(MAX)	CT	THICKNESS IN
1/4-TON-#1	.90	1.05	24.00	2.10	1.00	1.00	24.0

PROGRAM OUTPUT FOR A CHANNEL WITH A KNOWN LOCAL
DEPTH AVERAGED VELOCITY, BENDWAY
INPUT PARAMETERS

SPECIFIC WEIGHT OF STONE,PCF	165.0
MINIMUM CENTER LINE BEND RADIUS,FT	721.0
WATER SURFACE WIDTH,FT	280.0
LOCAL FLOW DEPTH,FT	5.2
CHANNEL SIDE SLOPE,1 VER: 3.00 HORZ	
LOCAL DEPTH AVG VELOCITY,FPS	12.40
SIDE SLOPE CORRECTION FACTOR K1	.99
CORRECTION FOR VELOCITY PROFILE IN BEND	1.20
RIPRAP DESIGN SAFETY FACTOR	1.30

SELECTED STABLE GRADATIONS
ALTERNATE GRADATION

NAME	COMPUTED D30 FT	D30(MIN) FT	D100(MAX) IN	D85/D15	N=THICKNESS/ D100(MAX)	CT	THICKNESS IN
1/4-TON-#1	1.05	1.05	24.00	2.10	1.46	.88	35.1

PROGRAM OUTPUT FOR A CHANNEL WITH A KNOWN LOCAL
DEPTH AVERAGED VELOCITY, STRAIGHT REACH
INPUT PARAMETERS

SPECIFIC WEIGHT OF STONE,PCF	165.0
LOCAL FLOW DEPTH,FT	14.4
CHANNEL SIDE SLOPE,1 VER: 4.00 HORZ	
LOCAL DEPTH AVG VELOCITY,FPS	8.00
SIDE SLOPE CORRECTION FACTOR K1	1.00
CORRECTION FOR VELOCITY PROFILE IN BEND	1.00
RIPRAP DESIGN SAFETY FACTOR	1.30

SELECTED STABLE GRADATIONS
ALTERNATE GRADATION

NAME	COMPUTED D30 FT	D30(MIN) FT	D100(MAX) IN	D85/D15	N=THICKNESS/ D100(MAX)	CT	THICKNESS IN
1/4-TON-#1	.25	1.05	24.00	2.10	1.00	1.00	24.0

PROGRAM OUTPUT FOR A CHANNEL WITH A KNOWN LOCAL
DEPTH AVERAGED VELOCITY, STRAIGHT REACH
INPUT PARAMETERS

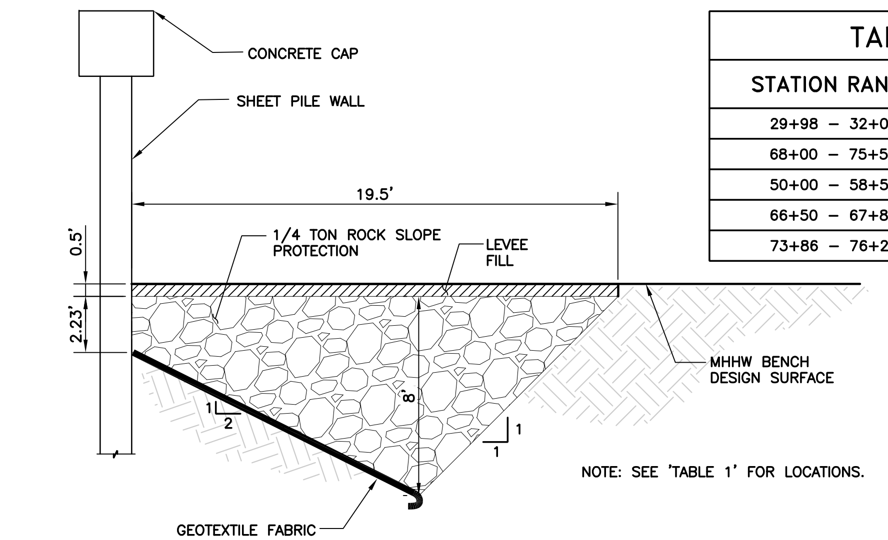
SPECIFIC WEIGHT OF STONE,PCF	165.0
LOCAL FLOW DEPTH,FT	4.0
CHANNEL SIDE SLOPE,1 VER: 2.00 HORZ	
LOCAL DEPTH AVG VELOCITY,FPS	7.60
SIDE SLOPE CORRECTION FACTOR K1	.88
CORRECTION FOR VELOCITY PROFILE IN BEND	1.00
RIPRAP DESIGN SAFETY FACTOR	1.30

SELECTED STABLE GRADATIONS
ALTERNATE GRADATION

NAME	COMPUTED D30 FT	D30(MIN) FT	D100(MAX) IN	D85/D15	N=THICKNESS/ D100(MAX)	CT	THICKNESS IN
1/4-TON-#1	.36	1.05	24.00	2.10	1.00	1.00	24.0

Attachment G

Final Design Detailed Cross Sections for Rock Placement

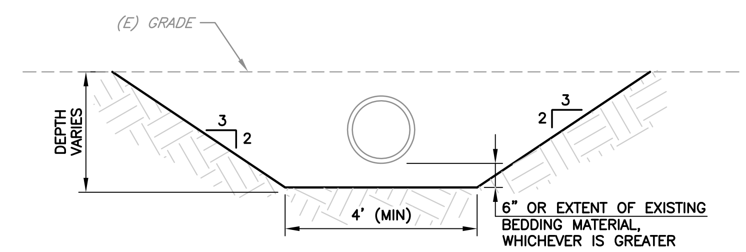


NOTE: SEE 'TABLE 1' FOR LOCATIONS.

SECTION A (C-25) ROCK SLOPE PROTECTION AT FLOOD WALL
NTS

TABLE 1

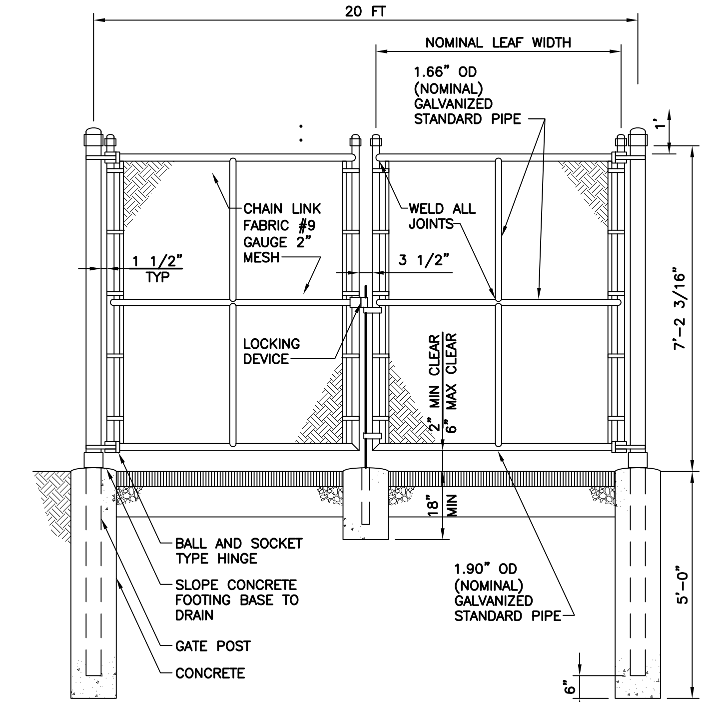
STATION RANGE	ALIGNMENT
29+98 – 32+00	R-LINE
68+00 – 75+54	R-LINE
50+00 – 58+50	L-LINE
66+50 – 67+80	L-LINE
73+86 – 76+29	L-LINE



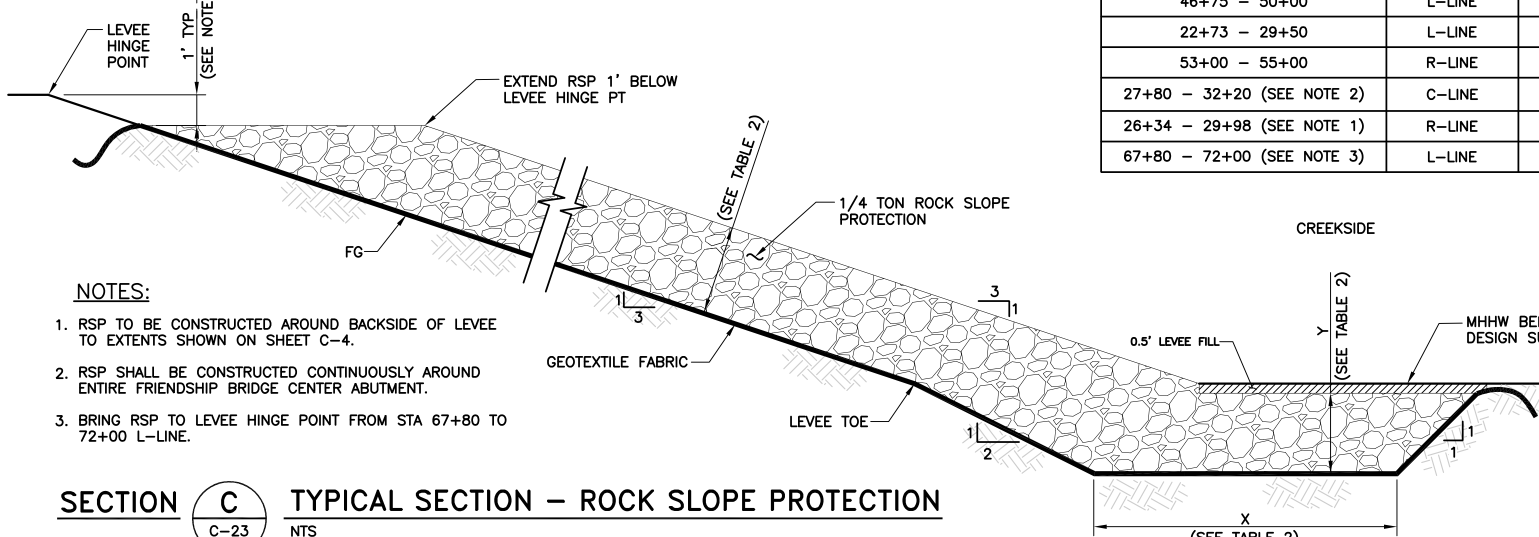
SECTION B TYPICAL EXISTING PIPE REMOVAL EXCAVATION DETAIL
NTS

TABLE 2

STATION RANGE	ALIGNMENT	X (FT)	Y (FT)	Z (FT)
46+75 – 50+00	L-LINE	8	6	2
22+73 – 29+50	L-LINE	8	6	2
53+00 – 55+00	R-LINE	8	6	2
27+80 – 32+20 (SEE NOTE 2)	C-LINE	8	7	3
26+34 – 29+98 (SEE NOTE 1)	R-LINE	8	7	3
67+80 – 72+00 (SEE NOTE 3)	L-LINE	8	6	2



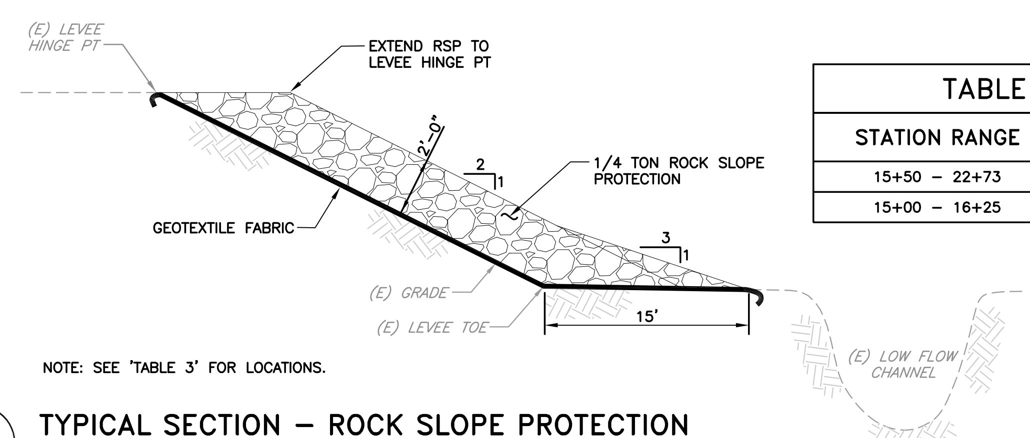
DETAIL 1 (C-17) 20 FOOT WIDE GATE
NTS



NOTES:

1. RSP TO BE CONSTRUCTED AROUND BACKSIDE OF LEVEE TO EXTENTS SHOWN ON SHEET C-4.
2. RSP SHALL BE CONSTRUCTED CONTINUOUSLY AROUND ENTIRE FRIENDSHIP BRIDGE CENTER ABUTMENT.
3. BRING RSP TO LEVEE HINGE POINT FROM STA 67+80 TO 72+00 L-LINE.

SECTION C (C-23) TYPICAL SECTION – ROCK SLOPE PROTECTION
NTS

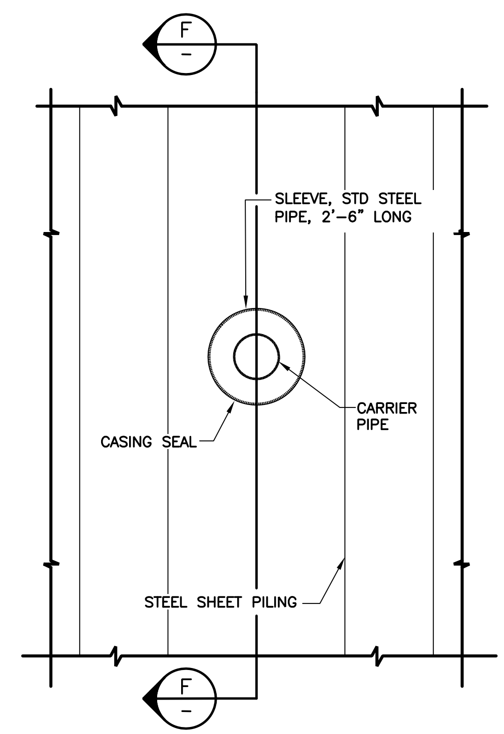


NOTE: SEE 'TABLE 3' FOR LOCATIONS.

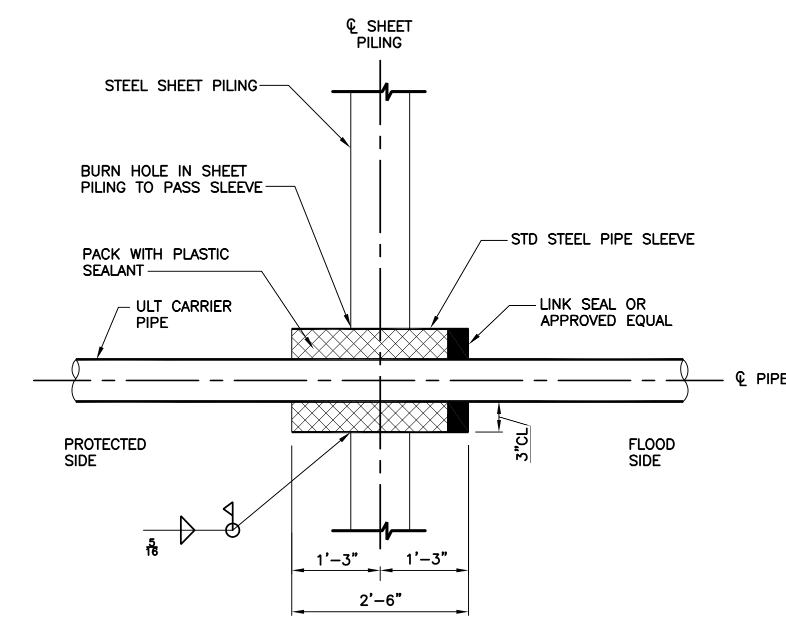
SECTION E (C-12) TYPICAL SECTION – ROCK SLOPE PROTECTION
NTS

TABLE 3

STATION RANGE	ALIGNMENT
15+50 – 22+73	L-LINE
15+00 – 16+25	R-LINE



SECTION D (C-39) PIPE PENETRATION OPENING
NTS



SECTION F
NTS

REV	DESCRIPTION	DATE	APPR.
	95% PRELIMINARY 11-14-2012		

HDR Engineering, Inc.

DATE 09-30-12
DESIGN S. JIMENEZ
DRAWN H. SUAREZ
CHECKED P. HRADILEK

ENGINEERING CERTIFICATION
REGISTERED PROFESSIONAL ENGINEER SERGIO E. JIMENEZ C 055698 Exp. 12-31-12 CIVIL STATE OF CALIFORNIA
PRELIMINARY
PROJECT ENGINEER DATE

SAN FRANCISQUITO CREEK JOINT POWERS AUTHORITY
ACCEPTED BY DISTRICT
PROJECT ENGINEER DATE

PROJECT NAME AND SHEET DESCRIPTION:

SAN FRANCISQUITO CREEK FLOOD REDUCTION, ECOSYSTEM RESTORATION, & RECREATION PROJECT

TYPICAL SECTIONS AND DETAILS

SCALE: NOT TO SCALE
VERIFY SCALES: 0 1" BAR IS ONE INCH ON ORIGINAL DRAWING IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PROJECT NUMBER 130806
SHEET CODE: C-30
SHEET NUMBER: 43 OF 107

DOCUMENT NUMBER: SFC_LP-C-1028-XXXXXXX

Attachment H

Geotextile Design Hand Calculations

Using HEC-23, DG16

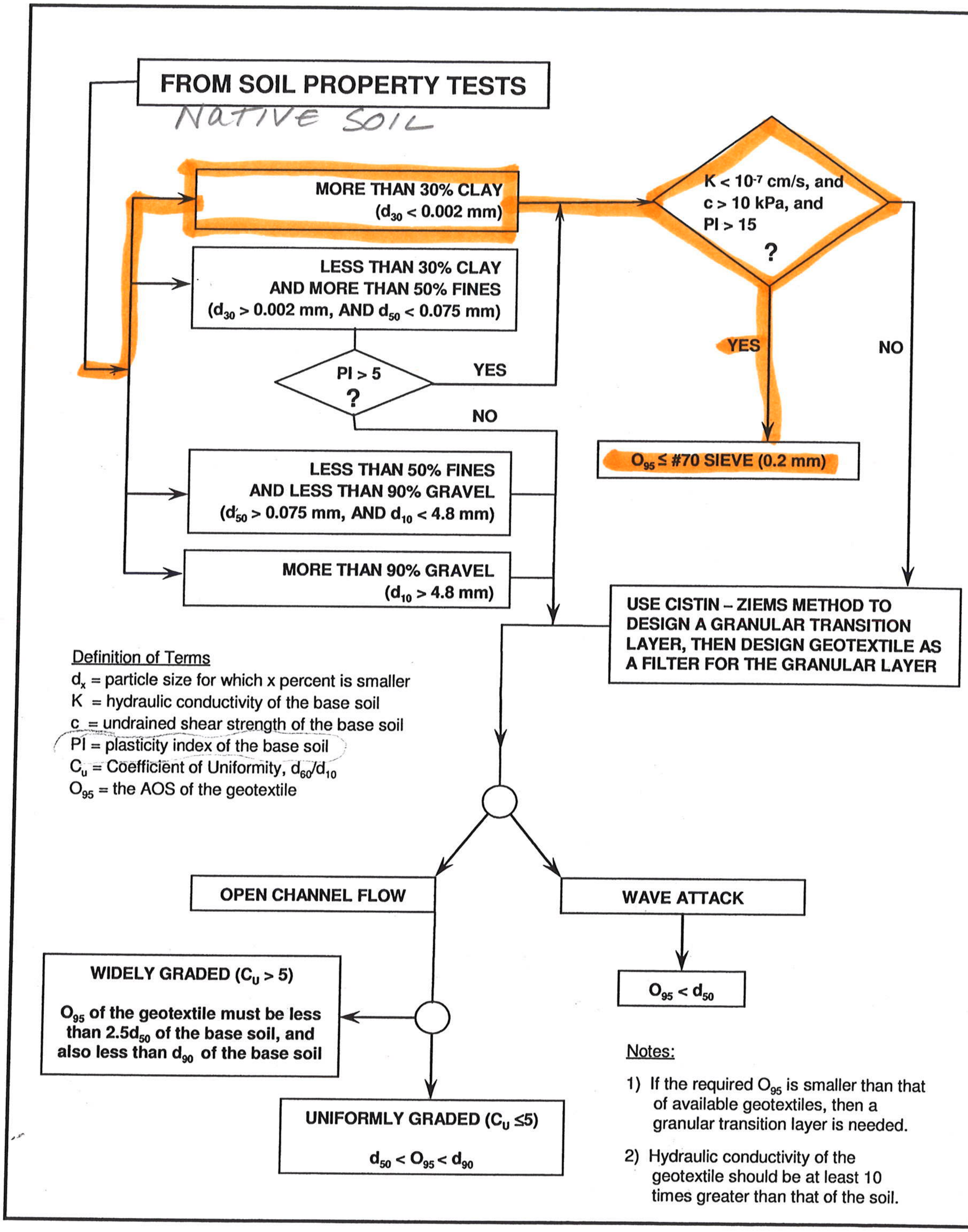


Figure 16.3. Geotextile selection for soil retention (modified from NCHRP Report 593).

Project: SFCJPA	Computed: EKM	Date: 10/2/12
Subject: Geotextile Design	Checked:	Date:
Task:	Page:	of:
Job #:	No:	

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Using HEC-23, DG16

assuming Elongation > 50%

Parameter	1120N	FW700	HEC-23 Requirements
Grab Strength (lbs)	300	370-250	>200
Tear Strength (lbs)	115	(Fail) 100-60	>110
Puncture Strength (lbs)	800	950	>110
AOS (mm)	100 (0.15 mm)	70 (0.212 mm)	≤ #70 Sieve
Hydraulic Conductivity (cm/s)	4.41 $\frac{cm}{s}$	1.22 $\frac{cm}{s}$	10x Native Soil K
Type	needlepunched nonwoven	woven monofilament	
Min. Open Area, %	N/A	4.0	≥ 4.0%
Mass per Unit Area oz/yd²	13 $\frac{oz}{yd^2}$	(FAIL) 6.2	≥ 12 $\frac{oz}{yd^2}$

Conclusion: Tencate Mirafi 1120N or an equivalent geotextile will be recommended.

Selected Geotextile

TENCATE

Mirafi®

Drainage

Separation

Mirafi® 1120N

Mirafi® 1120N is a needlepunched nonwoven geotextile composed of polypropylene fibers, which are formed into a stable network such that the fibers retain their relative position. Mirafi® 1120N is inert to biological degradation and resists naturally encountered chemicals, alkalis, and acids.

Mechanical Properties	Test Method	Unit	Minimum Average Roll Value	
			MD	CD
Grab Tensile Strength	ASTM D4632	lbs (N)	300 (1335)	300 (1335)
Grab Tensile Elongation	ASTM D4632	%	50	50
Trapezoid Tear Strength	ASTM D4533	lbs (N)	115 (512)	115 (512)
CBR Puncture Strength	ASTM D6241	lbs (N)	800 (3560)	
Apparent Opening Size (AOS)[1]	ASTM D4751	U.S. Sieve (mm)	100 (0.15)	
Permittivity	ASTM D4491	sec^{-1}	0.8	
Flow Rate	ASTM D4491	$gal/min/ft^2$ ($l/min/m^2$)	65 (2648)	
UV Resistance (at 500 hours)	ASTM D4355	% strength retained	70	

[1] ASTM D4751: AOS is a Maximum Opening Diameter Value

Physical Properties	Unit	Typical Value
Roll Dimensions (width x length)	ft (m)	15 x 300 (4.5 x 91)
Roll Area	yd^2 (m^2)	500 (418)
Estimated Roll Weight	lb (kg)	404 (183)

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FGS000357
ETQR24

TENCATE
Mirafi®

FILTRATION SEPARATION

Mirafi® FW700

Mirafi® FW700 geotextile is composed of high-tenacity monofilament polypropylene yarns, which are woven into a stable network such that the yarns retain their relative position. Mirafi® FW700 geotextile is inert to biological degradation and resists naturally encountered chemicals, alkalis, and acids. NTPEP No. GTX-08-04-18

Mechanical Properties	Test Method	Unit	Minimum Average Roll Value	
			MD	CD
Wide Width Tensile Strength	ASTM D4595	lbs/in (kN/m)	225 (39.4)	145 (25.4)
Grab Tensile Strength	ASTM D4632	lbs (N)	370 (1647)	250 (1113)
Grab Tensile Elongation	ASTM D4632	%	15	15
Trapezoid Tear Strength	ASTM D4533	lbs (N)	100 (445)	60 (267)
CBR Puncture Strength	ASTM D6241	lbs (N)	950 (4228)	
Apparent Opening Size (AOS)[1]	ASTM D4751	U.S. Sieve (mm)	70 (0.212)	
Percent Open Area	COE-02215	%	4	
Permittivity	ASTM D4491	sec^{-1}	0.28	
Permeability	ASTM D4491	cm/sec	0.01	
Flow Rate	ASTM D4491	$gal/min/ft^2$ ($l/min/m^2$)	18 (733)	
UV Resistance (at 500 hours)	ASTM D4355	% strength retained	90	

[1] ASTM D4751: AOS is a Maximum Opening Diameter Value

Physical Properties	Unit	Typical Value
Mass/Unit Area (ASTM D5261)	oz/yd^2 (g/m^2)	6.2 (210)
Thickness (ASTM D5199)	mils (mm)	15 (0.4)
Roll Dimensions (width x length)	ft (m)	12 x 300 (3.7 x 91)
Roll Area	yd^2 (m^2)	400 (334)
Estimated Roll Weight	lbs (kg)	164 (74)

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FGS000054
ETQR31

Attachment I

Overtopping Design Hand Calculations

Project: SFCJPA		Computed: EKM	Date: 4/25/12
Subject: Riprap for Overtopping		Checked:	Date:
Task: Flow		Page: 1	of: 4
Job #: 130806-028		No:	

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Procedure: Sizing the riprap when flows overtop mitigation terrace area RAS stations 7418.67 - 6963.459

Criteria: For slopes steeper than 4H:1V, the method requires that all flow is contained within the thickness of the riprap layer (interstitial flow).

Data:

Q, total discharge (over lip) = 14,400 cfs (sum Q right bank)
L, embankment overtopping length = 480 ft
q_f, unit discharge = 2.0 cfs/ft
C, weir coefficient = 2.84 $ft^{0.5}/s$
K_u, riprap sizing equation coeff = 0.525 $s^{0.52}/ft^{0.04}$
Manning-Strickler coeff = 0.034
S, slope = 0.333 ft/ft (3H:1V)
α, slope angle = 18.43°
Cu, d_{60}/d_{10} = 2.1 starting assumption
ϕ, angle of repose = 42° (assume)
S_g = specific gravity of riprap, 2.65
n = porosity = 0.45 (assume)

Calculations:

Step 1: Check overtopping depth using weir eqn

$$Q = CLH^{1.5} \longrightarrow H = \left(\frac{Q}{CL}\right)^{2/3} = H = 5.6 \text{ ft per RAS results}$$

Step 2: Compute smallest possible rock size (d_{50})

$$d_{50} = \frac{K_u q_f^{0.52}}{C_u^{0.25} S^{0.75}}\left[\frac{\sin\alpha}{(S_g\cos\alpha - 1)(\cos\alpha\tan\phi - \sin\alpha)}\right]^{1.11}$$

$$d_{50} = \frac{(0.525)(2)^{0.52}}{(2.1)^{0.25}(0.333)^{0.75}}\left[\frac{\sin(18.43)}{(2.65)(\cos 18.43 - 1)((\cos 18.43)(\tan 42) - \sin 18.43)}\right]$$

$$d_{50} = (1.42)\left[\frac{0.316146}{(1.5141)(0.53808)}\right]^{1.11}$$

$d_{50} = 0.50$ ft (0.75 ft)

Step 3: Select Class II riprap (nominal size = 9 in)
(Class I (nominal size = 6 in) was not selected since computed minimal d_{50} = 6 in)

Step 4: Compute the interstitial velocity and avg velocity

$$V_i = 2.48\sqrt{g d_{50}}\ \frac{S^{0.58}}{C_u^{2.22}}$$

$$V_i = 2.48\sqrt{(32.2)(0.75)}\left(\frac{(0.333)^{0.58}}{(2.1)^{2.22}}\right)$$

$$V_i = (2.48)(4.914)(0.10179)$$

$V_i = 1.24$ ft/s

$V_{avg} = \eta V_i = (0.45)(1.24) = 0.558$ ft/s
(porosity, η)

Step 5: Compute thickness as if all the flow were through riprap

$$t = \frac{q_f}{V_{avg}} = \frac{2.0}{0.558} = 3.58 \text{ ft}$$

Check $t > 2(d_{50}) \rightarrow 3.58 \text{ ft} > (2)(0.75)$
1.5

Check $S = 0.333 > 0.25$
Must increase riprap size to next gradation class

Step 6: Reselect Class III riprap, nominal $d_{50} = 1 ft$

$$V_i = 2.48 \sqrt{g d_{50}} \left(\frac{S^{0.58}}{C_u^{2.22}} \right)$$

$$V_i = 2.48(\sqrt{(32.2)(1 ft)}) \frac{(0.333)^{0.58}}{(2.1)^{2.22}}$$

$$V_i = 1.43 \ ft/s$$

$$V_{avg} = \eta V_i = 0.45(1.43 ft/s) = 0.6446 \ ft/s$$

Step 7: Compute thickness (depth)

$$t = \frac{q_f}{V_{avg}} = \frac{2.0}{0.6446} = 3.1 \ ft$$

Check since $t = 3.1 ft > 2(d_{50}) \rightarrow$ increase size

Step 8: Reselect Class IV riprap, nominal $d_{50} = 15 in$ (1.25 ft)

$V_i = 1.60$ ft/s

$V_{avg} = 0.72$ ft/s

$t = 2.78$ ft

Check since $t = 2.78 ft > (2(1.25)) \rightarrow$ increase size

Step 9 Reselect Class V riprap, nominal $d_{50} = 18$ in (1.5 ft)

$V_i = 1.75$ ft/s

$V_{avg} = 0.79$ ft/s

$t = 2.53$ ft

check since $t = 2.53 ft < (2(1.5))$ OK!

$2.53 ft < 3$

Project:		Computed: EKM	Date: 4/25/12
Subject:	overtopping flow	Checked:	Date:
Task:		Page: 4	of: 4
Job #:		No:	

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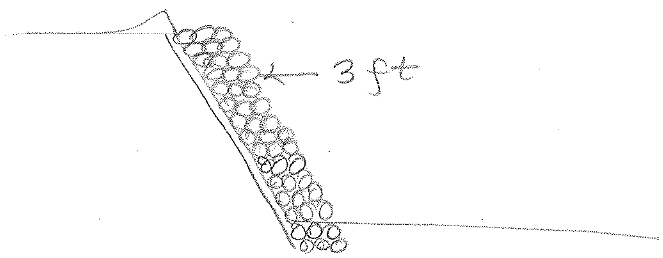
Summary of Results

$2 \times d_{50}$ = Recommended thickness

$2 \times 1.5\,ft = \boxed{3\ ft}$



Attachment J

Wind and Wave Runup and Wave Attack Hand Calculations

HDR ONE COMPANY *Many Solutions*	Project: SFCJPA FLOOD IMPROVEMENTS	Computed: DJH	Date: 4/16/12
	Subject: SAN FRANCISQUITO CREEK	Checked: LNA	Date: 4/16/12
	Task: WAVE RUNUP ANALYSIS	Page: 1	of: 17
	Job #: 130806 (DEPT 241)	No:	

PRELIMINARY

- SEE ATTACHED SITE MAPS. TASK WAS TO CALCULATE WAVE RUNUP ALONG NW FACE OF LEVEE WHERE SAN FRANCISQUITO CREEK MEETS SAN FRANCISCO BAY. CONSIDERED WAVES PROPAGATING FROM THE OPEN BAY, AND ACROSS THE MARSH AT THE FABER TRACT. APPLY 100YR WIND AND WATER LEVEL.

- FETCH — SEE SHEET ~~2~~ 3. APPLIED 4 MILE FETCH TO THE NORTH. ✓

- BATHYMETRY — REFERENCE EMAIL FROM LIBBY MESBAH DATED 4/13/12. SEE IDEALIZED 1-D BATHY TRANSECT ON SHEET 6. IGNORE NON-ACCREDITED LEVEES. ON FABER TRACT.

- WIND — APPLY ASCE 07 (SEE SHEET 8). $U_{10\,MIN} \simeq 63$ MPH. ✓

- WAVES
  - FOR WAVE GENERATED IN OPEN BAY, APPLY ACES (SHEET 9).

    $H_{mo} = 3.9'$ (4.1') ; $T_p = 3.5$ SEC ✓ — DOES NOT ACCOUNT FOR 2D WAVE EFFECTS, DIFFRACTION, REFRACTION, SHOALING, REFLECTION, ETC.
  - WAVE WILL SHOAL AS IT PROPAGATES THROUGH SHALLOWER WATER ABOVE MARSH APPLY OWEN'S (1980) BREAKER INDEX (SHEET 10).

$$h_s \simeq 12.9 - 7.5 = 5.4' \text{ (OVER MARSH)} ✓$$

$$\frac{h_s}{gT^2} = \frac{5.4}{(32.2)(3.5)^2} = 0.014 ✓$$

$$\Rightarrow \gamma_{\beta r} \simeq 0.59 \text{ FOR 1:100 SLOPE}, \quad \simeq 0.53 \text{ FOR 0 SLOPE} \quad \Big\} \text{ USE } 0.55 ✓$$

$$\therefore \quad H_{sb} \simeq \gamma_{\beta r} h_s = 0.55(5.4') = \underline{\underline{3.0'}} ✓$$

SINCE $H_{sb} < H_{mo} = 3.9'$, H_{sb} CONTROLS AS DESIGN VALUE

- SURF SIMILARITY PARAMETER:

$$\xi_{om} = \frac{m}{\sqrt{H_o / L_o}} = \frac{\tan\alpha}{\sqrt{H_o / \left(\frac{g}{2\pi} T_m^2\right)}} ✓$$

$$T_m \simeq 0.87\, T_p = 0.87(3.5) = 3.0 \text{ SEC} \rightarrow$$

FEMA guidelines use $T_{m-1.0}$

$$T_{m-1.0} = \frac{T_p}{1.1} = 3.2$$

$$\xi_{om} = \frac{1/3}{\sqrt{\dfrac{3.0}{\frac{32.2\,(3.2)^2}{2\pi}}}} = 1.39$$

$\xi = 1.37$

Should use $T_{m-1,0}$ to be conservative?

Project:	Computed:	Date:
Subject:	Checked: LNA	Date: 4/16/12
Task:	Page: 2	of:
Job #:	No:	

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- WAVE RUNUP

APPLY FEMA GUIDELINES FOR COASTAL FLOOD HAZARD ANALYSIS AND MAPPING FOR THE PACIFIC COAST OF THE U.S. (FINAL DRAFT DATED NOV. 2004, SECTION D.4.5 REVISED JAN 2005) — SEE SHEETS 11-13.

FOR $\gamma_b \xi_{om} < 1.8$, $R_{2\%} = H_{mo} (1.77 \gamma_r \gamma_b \gamma_\beta \gamma_p \xi_{om})$

~.77

γ_r = REDUCTION FACTOR FOR SURFACE ROUGHNESS

$\simeq$ 1.0 FOR GRASS

$\simeq$ 0.55 FOR RIPRAP

γ_b = REDUCTION FACTOR FOR INFLUENCE OF BERM = 1.0 (NO BERM)

γ_β = REDUCTION FACTOR FOR INFLUENCE OF ANGLED WAVE ATTACK

$= 1 - 0.0022(45°) = 0.90$

γ_p = REDUCTION FACTOR FOR STRUCTURE PERMEABILITY = 1.0 (IMPERMEABLE)

$\Rightarrow R_{2\%} = (3.0)(1.77)(1.0)(1.0)(0.90)(1.0)(1.39) = \boxed{6.6' \text{ GRASS}}$

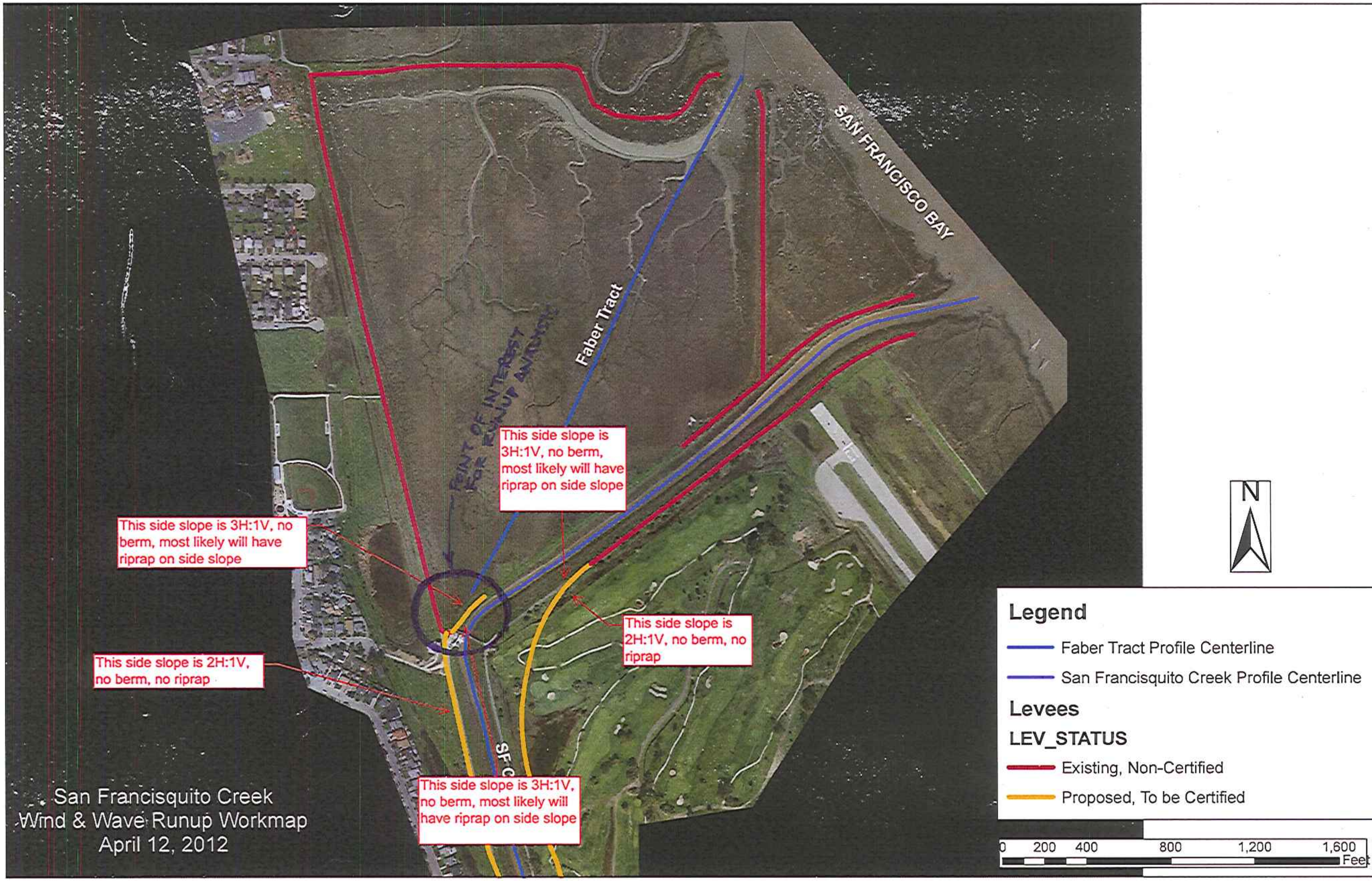
$= (3.0)(1.77)(0.55)(1.0)(0.90)(1.0)(1.39) = \boxed{3.7' \text{ RIPRAP}}$

For $\xi = 1.39$ $R_{2\%\,GRASS} = 6.64'$

$R_{2\%,RIPRAP} = 3.65'$

* NOTE – METHODOLOGY APPLIED INCLUDED SEVERAL CONSERVATIVE SIMPLIFYING ASSUMPTIONS. A MORE RIGOROUS ANALYSIS CAN BE APPLIED IF REDUCTION OF CONSERVATISM IS NEEDED.

SITE LOCATION AND FETCH APPLIED FOR WAVE RUNUP ANALYSIS.


SAN FRANCISCO BAY
Faber Tract
POINT OF INTEREST FOR RUNUP ANALYSIS
This side slope is 3H:1V, no berm, most likely will have riprap on side slope
This side slope is 3H:1V, no berm, most likely will have riprap on side slope
This side slope is 2H:1V, no berm, no riprap
This side slope is 2H:1V, no berm, no riprap
This side slope is 3H:1V, no berm, most likely will have riprap on side slope
SF C
N
Legend
Faber Tract Profile Centerline
San Francisquito Creek Profile Centerline
Levees
LEV_STATUS
Existing, Non-Certified
Proposed, To be Certified
0 200 400 800 1,200 1,600 Feet
San Francisquito Creek
Wind & Wave Runup Workmap
April 12, 2012

4

5

Heilman, Daniel

From: Mesbah, Elizabeth K.
Sent: Friday, April 13, 2012 12:32 PM
To: Heilman, Daniel
Subject: RE: SFC Wind/Wave
Attachments: 2012-04-13 Schematic Profile San Fran Creek markup.pdf

Hi Dan,

See attached profile for elevation markups. I am not exactly sure how to find the SF Bay elevation near our project site. I found a couple of websites, but not exactly sure how to use data in relation to all the different datums. I can keep digging if that is a critical piece. Just let me know. We would prefer one of your team members to perform QC on the analysis. We don't have anyone who is knowledgeable enough to perform a thorough QC here.

http://pubs.usgs.gov/of/2007/1169/
http://www.charts.noaa.gov/OnLineViewer/18651.shtml BAY BATHYMETRY

From: Heilman, Daniel
Sent: Friday, April 13, 2012 7:26 AM
To: Mesbah, Elizabeth K.
Subject: RE: SFC Wind/Wave

Libby – I'm not sure what's represented by the two profiles you sent. The attached sketch shows the information I need. Can you please provide?

Also, unless you had something else in mind, I'll apply the 100 yr windspeed from ASCE-07 for the wind-wave calculation.

From: Mesbah, Elizabeth K.
Sent: Thursday, April 12, 2012 6:44 PM
To: Heilman, Daniel
Cc: Jimenez, Sergio; Jones, Lance
Subject: SFC Wind/Wave

Hi Dan,

Please find the attached workmap and water surface profiles illustrating conditions at the confluence of San Francisquito Creek and the San Francisco Bay. I've confirmed with the PM, Serge Jimenez, that we would like to confirm that the wind and wave runup is contained within the levee freeboard using FEMA's criteria and methodology. In addition to calculations we would also like to request a short description of procedure and assumptions. We are anticipating that this analysis will take limited effort, 8-10 hours. Let me know if you anticipate this to be a larger effort and we can discuss our options.

Project assumptions:

- Design Still water elevation equal to 12.52' at the SF Bay (assumes 100-Year tidal plus 26" of Sea Level Rise in conjunction with 100-Year riverine flood flow)
- WSE profile shows a very slight increase in elevation when it reaches the proposed levees.
- Proposed levees will provide 100-Year level of protection, existing levees do not provide 100-Year level of protection and will be part of another project to restore.
- Top of levee elevations have been set by using the design water surface profile developed using a HEC-RAS hydraulic model and adding 3 feet of freeboard (4 feet at structures)

6

I will be in all day tomorrow. Give me a call or email me letting me know what additional information I need to provide.

Thanks.

Libby

ELIZABETH MESBAH, PE

HDR Engineering, Inc.
Water Resources Project Engineer

2365 Iron Point Road, Suite 300 | Folsom, CA 95630
916.817.4700 | direct: 916.817.4913 | cell: 916.317.7034
elizabeth.mesbah@hdrinc.com | hdrinc.com

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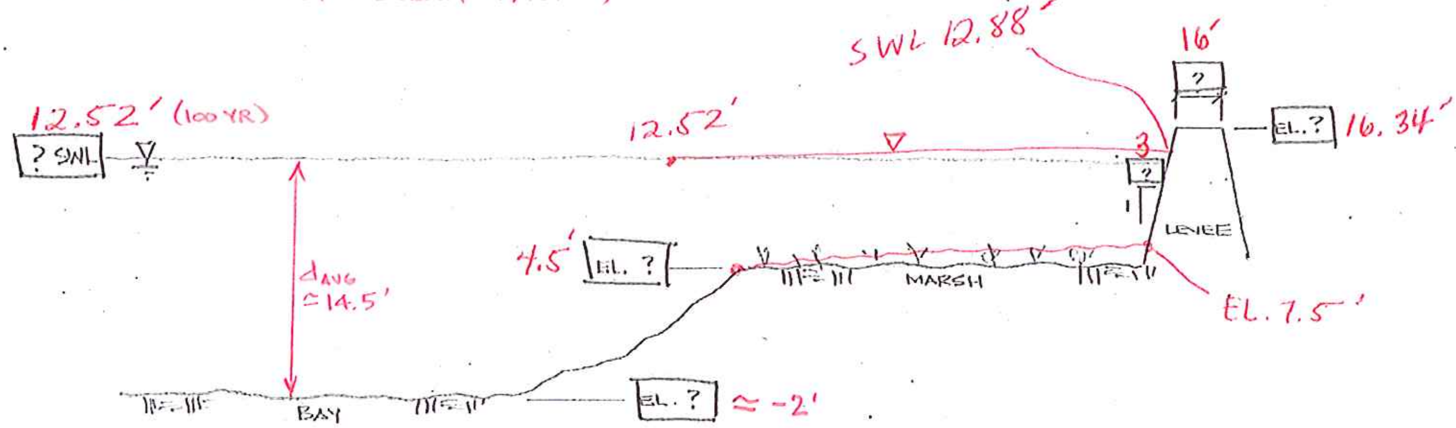
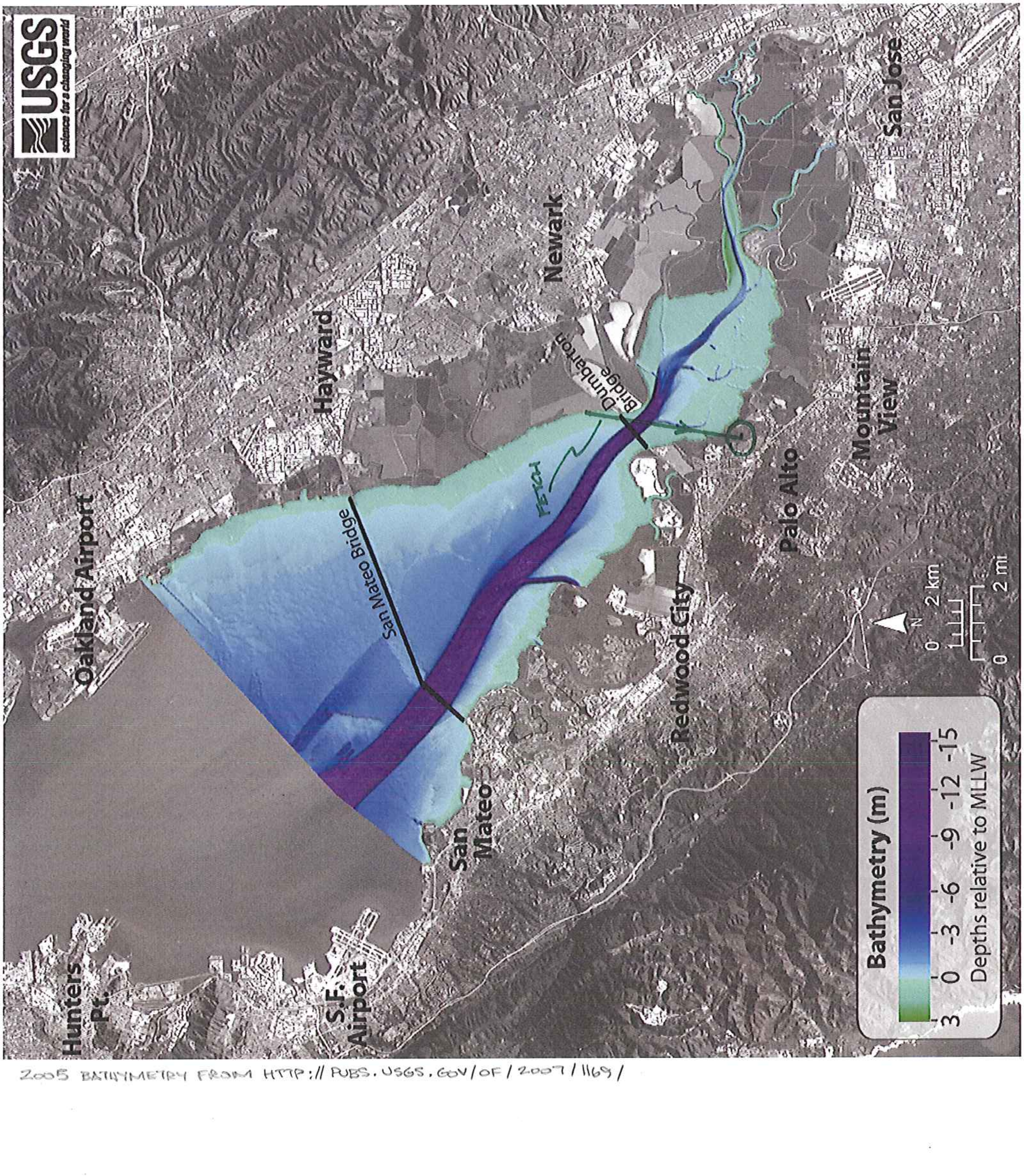
Project:		Computed:	Date:
Subject:		Checked:	Date:
Task:		Page:	of:
Job #:		No:	

All elevations provided are in NAVD88
(PROVIDED BY LIBBY MESBAH 4/13/12)

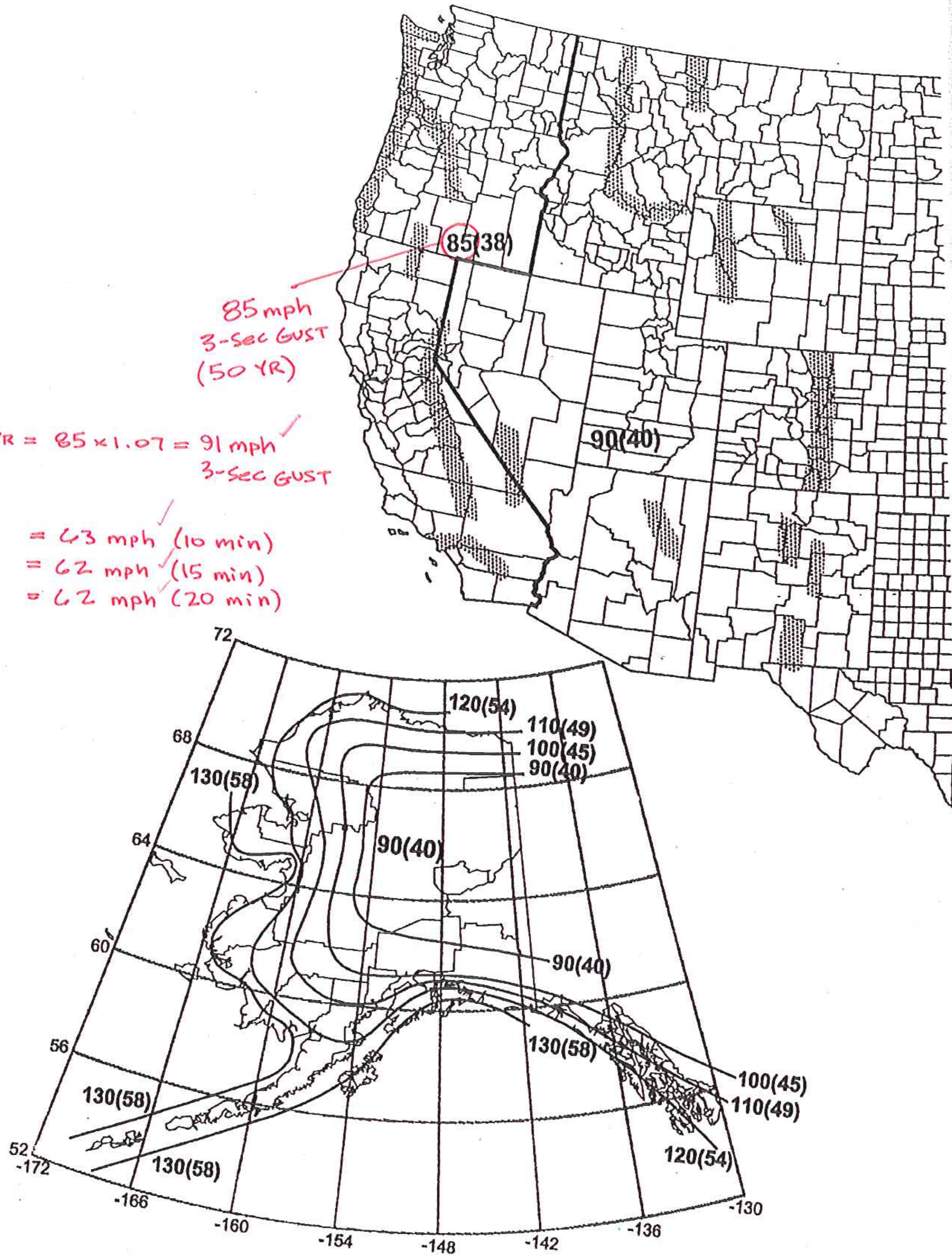


NEEDED INFORMATION:

- SWL
- REPRESENTATIVE/AVG BAY BOTTOM ELEV.
- " MARSH ELEV.
- LEVEE SLOPE, CREST ELEV., AND CREST WIDTH



2005 BATHYMETRY FROM HTTP://PUBS.USGS.GOV/OF/2007/1169/



85 mph
3-Sec GUST
(50 YR)

100 YR = 85 × 1.07 = 91 mph
3-Sec GUST

= 63 mph (10 min)
= 62 mph (15 min)
= 62 mph (20 min)

FIGURE 6-1 BASIC WIND SPEED

TABLE C6-6 PROBABILITY OF EXCEEDING DESIGN WIND SPEED DURING REFERENCE PERIOD

Annual Probability P_a	Reference (Exposure) Period, *n* (years)					
	1	5	10	25	50	100
0.04 (1/25)	0.04	0.18	0.34	0.64	0.87	0.98
0.02 (1/50)	0.02	0.10	0.18	0.40	0.64	0.87
0.01 (1/100)	0.01	0.05	0.10	0.22	0.40	0.64
0.005 (1/200)	0.005	0.02	0.05	0.10	0.22	0.39

TABLE C6-7 CONVERSION FACTORS FOR OTHER MEAN RECURRENCE INTERVALS

MRI (years)	Peak Gust Wind Speed, V (mph) m/s		
	Continental U.S.		
	V = 85–100 (38–45 m/s)	V > 100 (hurricane) (44.7 m/s)	Alaska
500	1.23	1.23	1.18
200	1.14	1.14	1.12
100	1.07	1.07	1.06
50	1.00	1.00	1.00
25	0.93	0.88	0.94
10	0.84	0.74 (76 mph min.) (33.9 m/s)	0.87
5	0.78	0.66 (70 mph min.) (31.3 m/s)	0.81

Note: Conversion factors for the column "V > 100 (hurricane)" are approximate. For the MRI = 50 as shown, the actual return period, as represented by the design wind speed map in Fig. 6-1, varies from 50 to approximately 90 years. For an MRI = 500, the conversion factor is theoretically "exact" as shown.

11

In order to determine the inshore significant wave height, H_{si}, with respect to depth limiting, the following should be adopted:

$$\begin{aligned} &\text{if } H_{sb} < H_s \text{ then } H_{si} = H_{sb} \\ &\text{otherwise if } H_{sb} > H_s \text{ then } H_{si} = H \end{aligned} \tag{7.8}$$

where H_s is the significant wave height in deep water.

The prediction of depth-limited wave heights can be uncertain. Where the design conditionis limited by depth it may be appropriate to use more reliable methods such as the method by Goda (1975) or those recommended by Allsop *et al* (1998).

Other effects

Waves in shallow water can also be modified by other processes. Reflections from structures can sometimes modify incoming waves. Reflections are most severe from smooth, impermeable vertical or near vertical structures. In such cases, reflections may cause severe damage in the form of scour if there is a beach at the toe of the structure. Reflections are significantly less for sloping structures, particularly if permeable or with a high roughness, which provides significant energy dissipation.

Waves can also be modified by currents. Generally, a following current will 'stretch' the waves, reducing height and increasing wave length. Conversely, an opposing current will increase wave height and cause a reduction in wave length. A change of current speed normal to the direction of wave propagation can cause reflection. The effects of currents usually balance out over a tidal cycle.

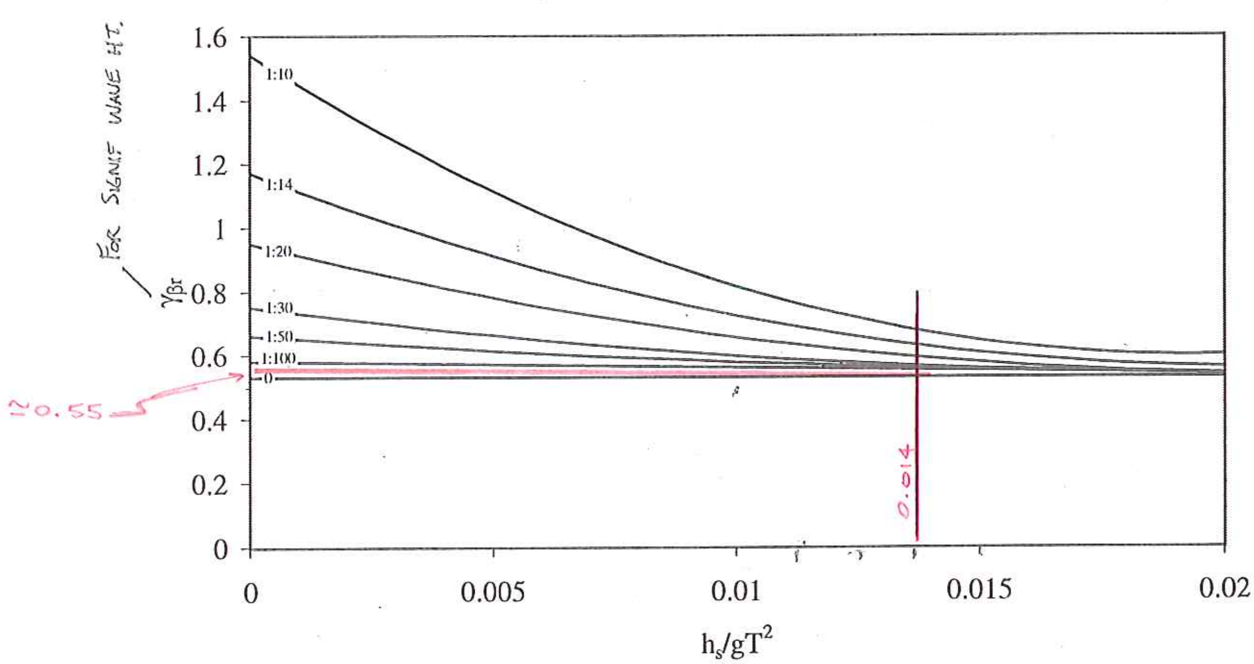


Figure 7.4 *Breaker index, after Owen (1980)*

ACES OUTPUT

Case: San Francisquito Creek 100 yr Wind

Windspeed Adjustment and Wave Growth

Breaking criteria 0.780

Item	Value	Units
El of Observed Wind (Zobs)	33.00	feet
Observed Wind Speed (Uobs)	63.00	mph
Air Sea Temp. Diff. (dT)	0.00	deg F
Dur of Observed Wind (DurO)	10.00	min
Dur of Final Wind (DurF)	10.00	min
Lat. of Observation (LAT)	37.50	deg
Results		
Wind Fetch Length (F)	4.00	MILES
Avg Fetch Depth (d)	14.50	feet
Eq Neutral Wind Speed (Ue)	56.68	mph
Adjusted Wind Speed (Ua)	88.69	mph
Wave Height (Hmo)	4.06	feet
Wave Period (Tp)	3.53	sec

VALUES IN OPEN BAY

Wind Obs Type	Wind Fetch Options
Overwater	Shallow openwater

Wave Growth: Shallow

Final Draft Guidelines for
Coastal Flood Hazard Analysis and Mapping
for the
Pacific Coast of the United States

Prepared for:

FEMA

A Joint Project by
FEMA Region IX, FEMA Region X, FEMA Headquarters

FEMA Study Contractor:

northwest hydraulic consultants, inc.

Final Draft Prepared November 2004, Section D.4.5 Revised January 2005

runup and ξ_{om} for values of ξ_{om} below approximately 2. For values of ξ_{om} above approximately 2, only the van der Meer method is applicable. Moreover, due to its long period of availability and wide international acceptance, the van der Meer relationship (also referred to as the TAW runup methodology) is recommended here. The Mapping Partner shall characterize the wave conditions in terms of ξ_{om} and be aware of the runup predictions provided by the various methods available in the general literature.

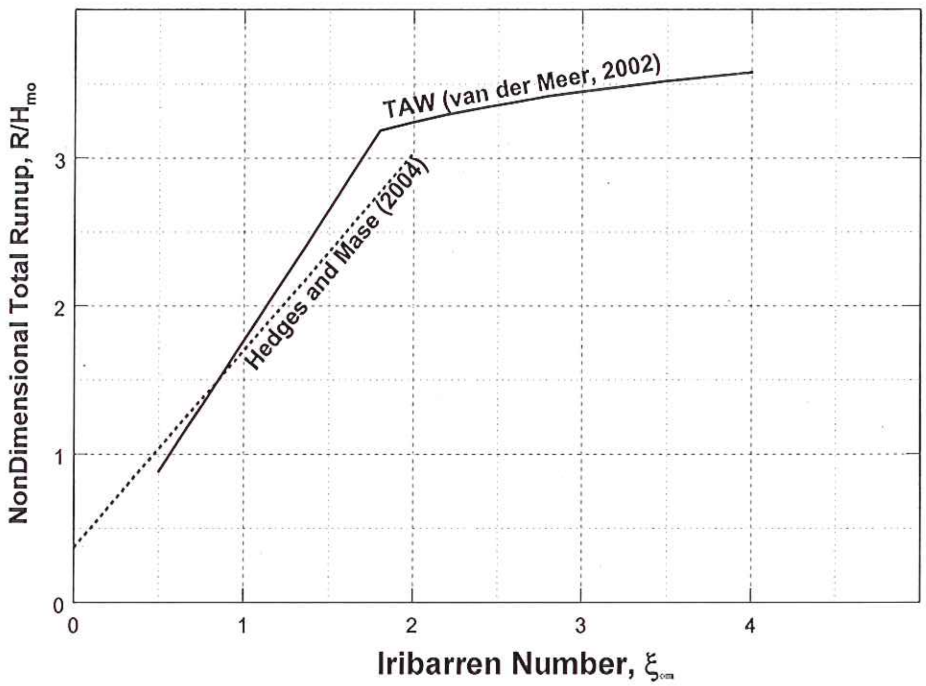


Figure D.4.5-7. Non-dimensional Total Runup vs. Iribarren Number

The general form of the wave runup equation recommended for use is (modified from van der Meer, 2002):

$$R = H_{mo}\begin{Bmatrix} 1.77\gamma_r\gamma_b\gamma_\beta\gamma_P\xi_{om} & 0.5 \le \gamma_b\xi_{om} < 1.8 \\ \gamma_r\gamma_b\gamma_\beta\gamma_P\left(4.3 - \dfrac{1.6}{\sqrt{\xi_{om}}}\right) & 1.8 \le \gamma_b\xi_{om} \end{Bmatrix} \qquad \text{(D.4.5-19)}$$

where:

R is the 2% runup = $2\,\sigma_2$

H_{mo} = spectral significant wave height at the structure toe

γ_r = reduction factor for influence of surface roughness

γ_b = reduction factor for influence of berm

γ_β = reduction factor for influence of angled wave attack

γ_P = reduction factor for influence of structure permeability

Equations for quantifying the γ parameters are presented in Table D.4.5-3. The reference water level at the toe of the barrier for runup calculations is DWL2%. Additionally, because some wave setup influence is present in the laboratory tests that led to Equation D.4.5-19, the following adjustments are made to the calculation procedure for cases of runup on barriers.

Table D.4.5-3. Summary of γ Runup Reduction Factors

Runup Reduction Factor	Characteristic/Condition	Value of γ for Runup	
Roughness Reduction Factor, γ_r	Smooth Concrete, Asphalt and Smooth Block Revetment	$\gamma_r = 1.0$	(D.4.5-20)
	1 Layer of Rock With Diameter, *D*. $H_s / D = 1$ to 3.	$\gamma_r = 0.55$ to 0.60	
	2 or More Layers of Rock. $H_s / D = 1.5$ to 6.	$\gamma_r = 0.5$ to 0.55	
	Quadratic Blocks	$\gamma_r = 0.70$ to 0.95. See Table V-5-3 in CEM for greater detail	
Berm Section in Breakwater, γ_b, B = Berm Width, $\left(\frac{\pi d_h}{x}\right)$ in radians	Berm Present in Structure Cross-section. See Figure D.4.5-8 for Definitions of B, L_{berm}, and Other Parameters	$\gamma_b = 1 - \frac{B}{2L_{berm}}\left[1+\cos\left(\frac{\pi d_h}{x}\right)\right],\ 0.6 < \gamma_B < 1.0$ $x = \begin{cases} R \ if \ \frac{-R}{H_{mo}} \le \frac{d_h}{H_{mo}} \le 0 \\ 2H_{mo} \ if \ 0 \le \frac{d_h}{H_{mo}} \le 2 \end{cases}$ Minimum and maximum values of $\gamma_b = 0.6$ and 1.0, respectively	(D.4.5-21)
Wave Direction Factor, γ_β, β is in degrees and $= 0^o$ for normally incident waves	Long-Crested Waves	$\gamma_\beta = \begin{cases} 1.0,\ 0 < \lvert\beta\rvert < 10^o \\ \cos(\lvert\beta\rvert - 10^o),\ 10^o < \lvert\beta\rvert < 63^o \\ 0.63,\ \lvert\beta\rvert > 63^o \end{cases}$	(D.4.5-22)
	Short-Crested Waves	$1 - 0.0022\lvert\beta\rvert,\ \lvert\beta\rvert \le 80^o$ $1 - 0.0022\lvert 80\rvert,\ \lvert\beta\rvert \ge 80^o$	(D.4.5-23)
Porosity Factor, γ_P	Permeable Structure Core	$\gamma_P = 1.0,\ \xi_{om} < 3.3;\ \gamma_P = \frac{2.0}{1.17(\xi_{om})^{0.46}},\ \xi_{om} > 3.3$ and porosity = 0.5. for smaller porosities, proportion γ_P according to porosity . See Figure D.4.5-9 for definition of porosity	(D.4.5-24)

For a smooth impermeable structure of uniform slope with normally incident waves, each of the γ runup reduction factors is 1.0.

In calculating the Iribarren number to apply in Equation D.4.5-19, the Mapping Partner shall use Equation D.4.5-9 and replace H_o with H_{mo} and replace T with $T_{m-1.0}$ (the spectral wave period) in Equation D.4.5-10. H_{mo} and $T_{m-1.0}$ are calculated as:

$$H_{mo} = 4.0\sqrt{m_o} \qquad \text{(D.4.5-25)}$$

$$T_{m-1.0} = \frac{T_p}{1.1} \qquad \text{(D.4.5-26)}$$

where H_{mo} is the spectral significant wave height at the toe of the structure and T_p is the peak wave period. In deep water, H_{mo} is approximately the same as H_s, but in shallow water, H_{mo} is 10-15% smaller than the H_s obtained by zero up crossings (van der Meer, 2002). In many cases, waves are depth-limited at the toe of the structure and H_b can be substituted for H_{mo}, with H_b calculated using a breaker index of 0.78 unless the Mapping Partner can justify a different value. The breaker index can be calculated based on the bottom slope and wave steepness by several methods, as discussed in the CEM (USACE, 2003). As noted, the water depth at the toe of the structure shall include the static wave setup and the 2% dynamic wave setup, calculated with DIM. In terms of the Iribarren number, the TAW method is valid in the range of $0.5 < \xi_{om} <$ 8-10, and in terms of structure slope, the TAW method is valid between values of 1:8 to 1:1. The Iribarren number as described above is denoted ξ_{om} as indicated in Equation D.4.5-19.

Runup on structures is very dependent on the characteristics of the nearshore and structure geometries. Hence, better runup estimates may be possible with other runup equations for particular conditions. The Mapping Partner may use other runup methods based on an assessment that the selected equations are derived from data that better represent the actual profile geometry or wave conditions. See CEM (USACE, 2003) for a list of presently available methods and their ranges of applicability.

D.4.5.1.5.3 Special Cases—Runup from Smaller Waves

In some special cases, neither of the previously described methods (Subsection D.4.5.1.4, Setup and Runup Beaches: Description and Recommendations, or Subsection D.4.5.1.5 Runup on Barriers) is applicable. These special cases include steep slopes in the nearshore with large Iribarren numbers or conditions otherwise outside the range of data used to develop the total runup for natural beach methods. Also, use of the TAW method is questionable where the toe of a structure, or naturally steep profile such as a rocky bluff, is high relative to the water levels, limiting the local wave height and calculated runups to small values. In these cases, it is necessary to calculate runup with equations of the form of Equation D.4.5.1-19 and to avoid double inclusion of the setup as discussed in Subsections D.4.5.1.5.1 and D.4.5.1.5.2 and Table D.4.5-2 and to carry out the calculations at several locations across the surf zone using the average slope in the Iribarren number. With this approach, it is possible that calculations with the largest waves in a given sea condition may not produce the highest runup, but that the highest runup will be the result of waves breaking at an intermediate location within the breaking zone.

The recommended procedure is to consider a range of (smaller) wave heights inside the surf zone in runup calculations. For this approach, for all depths considered, the dynamic setup is reduced if the *Gamma* of interest exceeds 3.3 as described in Subsections D.4.5.1.5.1 and D.4.5.1.5.2 and Table D.4.5-2. For each depth considered, the static setup is calculated with Equation D.4.5-5 with the water level including the 2% dynamic wave setup replacing the depth, h, in that equation. With the 2% dynamic water level available, methods of calculating wave runup on barriers is applied and are described in greater detail below.

The concept of a range of calculated runup values is depicted schematically in Figure D.4.5-10 where an example transect and setup water surface profile are shown. Figure D.4.5-10 also shows the corresponding range of depth-limited breaking wave heights calculated based on a breaker index and plotted by breaker location on the shore transect. The Iribarren number was also calculated and plotted by breaker location in Figure D.4.5-10. The calculation of ξ at each location uses the deshoaled deepwater wave height corresponding to the breaker height, the deepwater wave length and the average slope calculated from the breaker point to the approximate runup limit. Note that this average slope (also called composite slope, as defined in the CEM [USACE, 2003] and SPM [USACE, 1984] increases with smaller waves because the breaker location approaches the steeper part of the transect near the shoreline. This increases the numerator in the ξ equation. Also, the wave height decreases with shallower depths, reducing the wave steepness in the denominator of the ξ equation. Hence, as plotted in Figure D.4.5-10, ξ increases as smaller waves closer to shore are examined, increasing the relative runup (R/H). However, because the wave height decreases, the runup value, R, reaches a maximum and then decreases.

The following specific steps are used to determine the highest wave runup caused by a range of wave heights in the surf zone:

1. Calculate, using DIM, the reduced 2% dynamic wave setup based on the *Gamma* of interest and Subsections D.4.5.1.5.1 and D.4.5.1.5.2 and Table D.4.5-2. Calculate the static wave setup based on Equation D.4.5-5 for the cross-shore location considered. Replace h in that equation with the sum of the still water depth at the location and the 2% dynamic wave setup.

2. Calculate the runup using the methods described earlier for runup on a barrier. This requires iteration for this location to determine the average slope based on the differences between the runup elevation and the profile elevation at the location and the associated cross-shore locations. Iterate until the runup converges for this location.

3. Repeat the runup calculations at different cross-shore locations until a maximum runup is determined.

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Project: SFCJPA FLOOD IMPROVEMENTS — Computed: LNA — Date: 4|23|2012

Subject: SAN FRANCISQUITO CREEK — Checked: EAR — Date: 4/24/2012

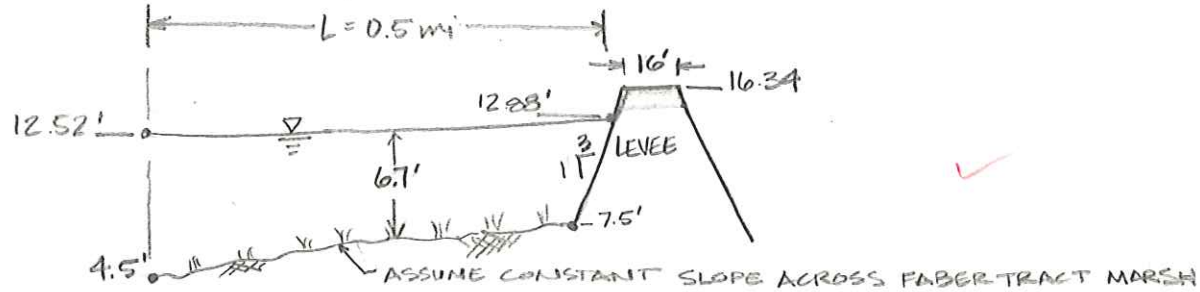
Task: WAVE RUNUP ANALYSIS — Page: 18 — of:

Job #: 130806 (DEPT 241) — No:

TASK

CALCULATE WAVE RUNUP ALONG NW FACE OF LEVEE WHERE SAN FRANCISQUITO CREEK MEETS SAN FRANCISCO BAY ("POINT OF INTEREST"), CONSIDER WAVES PROPAGATING ACROSS FABER TRACT FROM BREAKER LEVEES.

- FETCH = 0.5 MILES (SEE SHEET 19) ✓
- PROFILE OF BATHYMETRY (PROVIDED BY LIBBY MESBAH 4/13/12)



AVERAGE DEPTH ACROSS FETCH = 12.7' − 6.0' = 6.7'

- WIND – APPLY ASCE 07 (SEE SHEET 1) ✓

$U_{10} \cong 63$ MPH

- WAVES – APPLY ACES (SEE SHEET 21) FOR WAVES GENERATED OVER MARSH.

$H_{mo} = 1.7'$ ✓

$T_p = 1.9$ sec ✓

AT TOE OF LEVEE $h = 12.9 - 7.5 = 5.4'$ ✓

$\gamma_{br} = 0.55$ (SEE SHEET 1)

$H_b = 0.55(5.4) = 3.0'$, WAVE IS FETCH-LIMITED ✓

H_{mo} CONTROLS AS DESIGN WAVE

- CALCULATE SURF SIMILARITY PARAMETER –

$$\xi_{om} = \frac{\tan\alpha}{\sqrt{H_{mo}/\left(\frac{g}{2\pi}T_{m-1.0}^2\right)}} = \frac{(1/3)}{\sqrt{1.7'/\left(\frac{32.2}{2\pi}(1.7)^2\right)}} = 0.98$$

$$T_{m-1.0} = \frac{T_p}{1.1} = \frac{1.9}{1.1} = 1.7\,s$$

CONT'D →

Project:		Computed: LNA	Date: 4/23/2012
Subject:		Checked: EAR	Date: 4/24/2012
Task:		Page: 19	of:
Job #: 130806		No:	

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- CALCULATE RUNUP

FOR $\gamma_b \xi_{om} < 1.8$, $R_{2\%} = H_{mo}(1.77 \gamma_r \gamma_b \gamma_\beta \gamma_p \xi_{om})$

WHERE

γ_r = REDUCTION FACTOR FOR SURFACE ROUGHNESS

$\simeq 1.0$ GRASS

$\simeq 0.55$ FOR RIPRAP

γ_b = REDUCTION FACTOR FOR INFLUENCE OF BERM = 1.0 (NO BERM)

γ_β = REDUCTION FACTOR FOR INFLUENCE OF ANGLED WAVE ATTACK

$= 1 - 0.0022(45) = 0.90$

γ_p = REDUCTION FACTOR FOR STRUCTURE PERMEABILITY = 1.0 (IMPERMEABLE)

RUNUP FOR GRASS:

$R_{2\%,GRASS} = (1.7)(1.77)(1.0)(1.0)(0.90)(1.0)(0.98) = \boxed{2.65' \text{ (GRASS)}}$

RUNUP FOR RIPRAP:

$R_{2\%,RIPRAP} = (1.7)(1.77)(0.55)(1.0)(0.90)(1.0)(0.98) = \boxed{1.46' \text{ (RIPRAP)}}$

* METHODOLOGY DOES NOT ACCOUNT FOR 2D WAVE EFFECTS OR POTENTIAL WAVE ATTENUATION THROUGH MARSH, ∴ RUNUP VALUES ARE CONSERVATIVE ESTIMATES.

20

Heilman, Dan...

From: Mesbah, Elizabeth K.
Sent: Tuesday, April 17, 2012 5:22 PM
To: Heilman, Daniel
Subject: RE: SFC Wind/Wave
Attachments: Pages from 2012-04-16 SFCJPA San Fran Creek (Wave Runup Calcs).pdf; fetch_length_options.docx

Hi Dan,

Thanks for the calculations. This is an adequate level of documentation needed. I do have a couple of additional questions. Ideally, we would not like to see such great depths... This is probably why they constructed the breaker levees surrounding the Faber Tract in the first place. Although, I understand FEMA's criteria regarding assuming non-certifiable levees as failed, I think it might be important to know what the wind/wave runup is assuming the fetch length begins at the breaker levees or even waves entering through the small gap between breaker levees. I've drawn up some possible fetch lengths on the attached screen capture, not knowing the wind speed data. Do you think that this would be worthwhile? Might be easier to discuss via phone.

Other questions:

- Can we assume the same computed depths for wind/wave making its upstream into the main channel? I have identified the point of interest location on the attached figure.
- Is the ASCE07 wind data fairly conservative? ✓
- Would it be more appropriate to research if the Palo Alto Municipal Airport adjacent to the project site collects wind speed and directional data appropriate for our calculations?
- The assumed fetch length seems conservative. Are we being overly conservative?

Thanks!

Libby

From: Heilman, Daniel
Sent: Monday, April 16, 2012 1:50 PM
To: Mesbah, Elizabeth K.
Subject: RE: SFC Wind/Wave

Hi Libby,

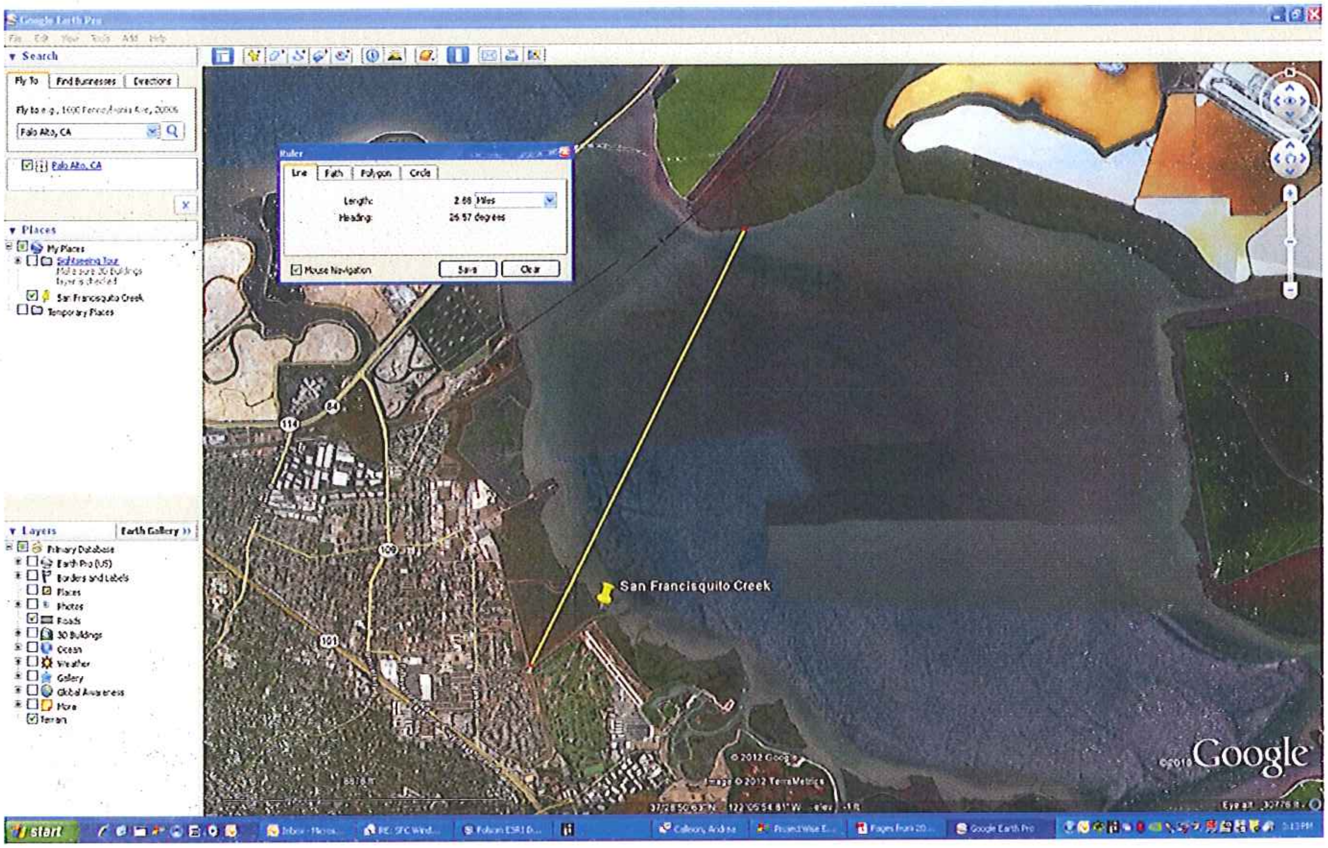
Attached are the wave runup calculations for both a grass and riprap slope. The relatively simple methods applied rely on a number of simplifying assumptions that add conservatism to the results. Based on these methods, for both the grass and riprap cases, the runup exceeds the proposed levee crest elevation (see bottom of Sheet 2), although for the riprap case it is by only about 3 inches.

Please let me know if you need a more formal write-up and I can summarize the analysis and results in a short memo. Also feel free to contact me if you have any questions or comments.

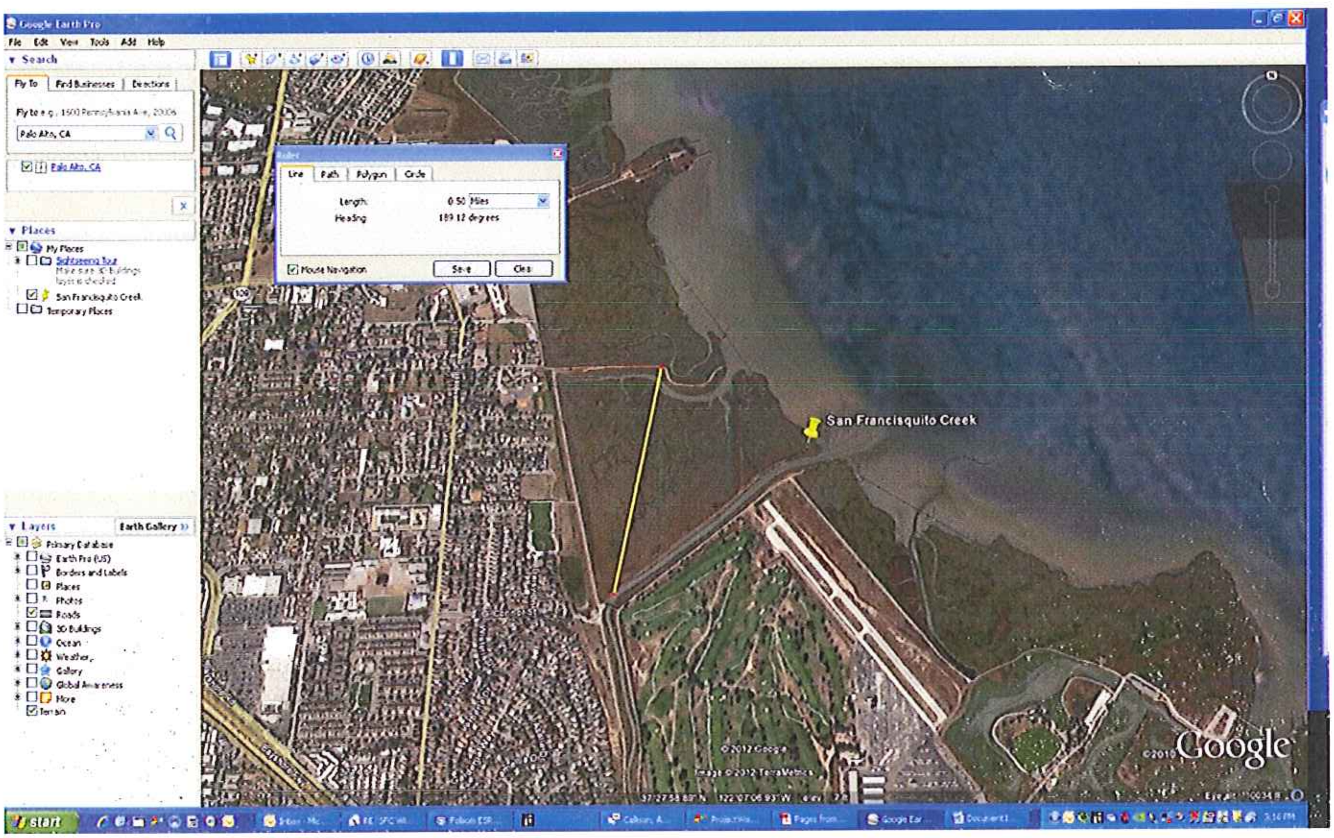
-Dan

From: Mesbah, Elizabeth K.
Sent: Friday, April 13, 2012 5:05 PM
To: Heilman, Daniel
Subject: RE: SFC Wind/Wave

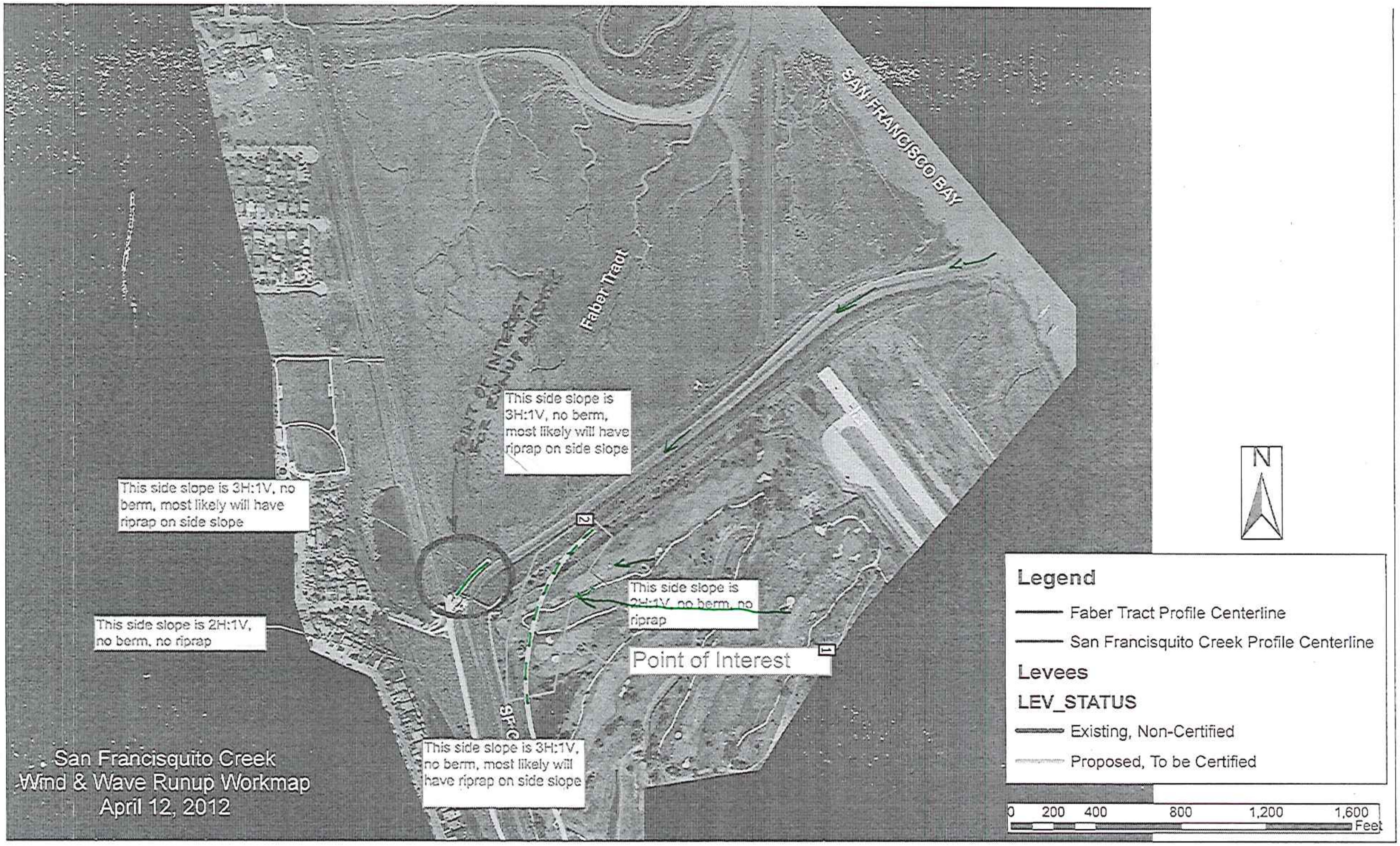
21



Google Earth Pro
Ruler
Length: 2.88 Miles
Heading: 26.57 degrees
San Francisquito Creek
Google



Google Earth Pro
Ruler
Length: 0.50 Miles
Heading: 189.12 degrees
San Francisquito Creek
Google


San Francisquito Creek
Wind & Wave Runup Workmap
April 12, 2012
SAN FRANCISCO BAY
Faber Tract
This side slope is 3H:1V, no berm, most likely will have riprap on side slope
This side slope is 3H:1V, no berm, most likely will have riprap on side slope
This side slope is 2H:1V, no berm, no riprap
This side slope is 2H:1V, no berm, no riprap
This side slope is 3H:1V, no berm, most likely will have riprap on side slope
Point of Interest
1
2
SFC
N
Legend
Faber Tract Profile Centerline
San Francisquito Creek Profile Centerline
Levees
LEV_STATUS
Existing, Non-Certified
Proposed, To be Certified
0 200 400 800 1,200 1,600 Feet

Project: San Fran Creek Runup
Group: San Fran Creek

Case: Avg Depth - Marsh

Windspeed Adjustment and Wave Growth

Breaking criteria 0.780

Item	Value	Units
El of Observed Wind (Zobs)	33.00	feet
Observed Wind Speed (Uobs)	63.00	mph
Air Sea Temp. Diff. (dT)	0.00	deg F
Dur of Observed Wind (DurO)	10.00	min
Dur of Final Wind (DurF)	10.00	min
Lat. of Observation (LAT)	37.50	deg
Results		
Wind Fetch Length (F)	0.50	MILES
Avg Fetch Depth (d)	6.70	feet
Eq Neutral Wind Speed (Ue)	56.68	mph
Adjusted Wind Speed (Ua)	88.69	mph
Wave Height (Hmo)	1.67	feet
Wave Period (Tp)	1.88	sec

Wind Obs Type	Wind Fetch Options
Overwater	Shallow openwater

Wave Growth: Shallow

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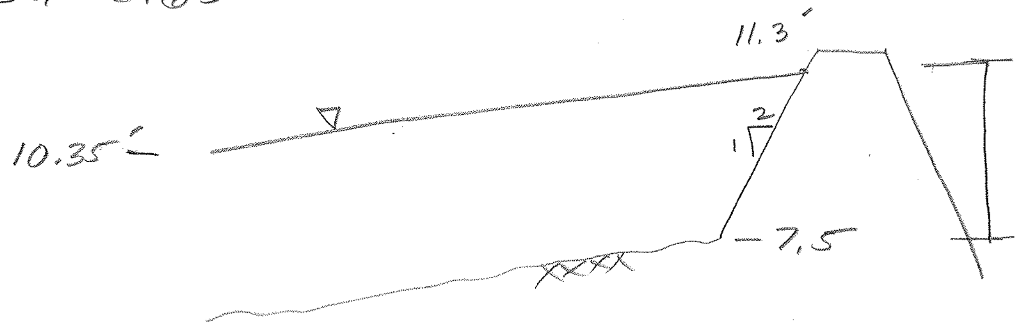
Project: SFCJPA		Computed: EKM	Date: 7/1/12
Subject: Riprap design for Wave Attack		Checked:	Date:
Task:		Page: 1	of: 3
Job #:		No:	

Riprap design for wave attack

Slope: 2H:1V
Significant wave height, H_s = 1.6'
Still water depth, h = 11.3 − 7.5 = 3.8'
Wave period, 1.9 sec
SG = 2.65



A) Hudson Equation

Step 1: Calculate the design wave $H_{0.10}$

$$H_{0.10} = 1.27 H_s = 1.27(1.6') = 2.03 \text{ ft}$$

Step 2: Calculate median stone weight W_{50}

$$W_{50} = \frac{\gamma_r H^3 \tan\theta}{K_d (S_r - 1)^3} = \frac{(2.65)(62.4 \text{ lb/ft}^3)(2.03 \text{ ft})^3 \times \tan(26.6)}{2.2(2.65 - 1.03)^3}$$

$$W_{50} = 74 \text{ lbs}$$

$$d_{50} = \sqrt[3]{\frac{W_{50}}{0.85\gamma_r}} = \sqrt[3]{\frac{74 \text{ lbs}}{0.85(2.65)(62.4 \text{ lb/ft}^3)}}$$

d_{50} = 0.81 ft → Class III

B.) Pilarczyk Equation

Step 1. Calc the breaker parameter

$$\xi = \frac{\tan\theta}{\sqrt{H_s/L_o}} = \tan\theta \frac{K_u T}{\sqrt{H_s}}$$

$$= \tan 26.6 \frac{2.26\,(1.9\,\text{sec})}{\sqrt{2.03}}$$

$$\xi = 1.5$$

Step 2. Calc min allowable median stone size d_{50}

$$d_{50} \geq \frac{2}{3}\left(\frac{H_s \xi^b}{1.64 \cos\theta}\right)$$

$$= \frac{2}{3} \frac{(2.03)(1.5)^{0.5}}{1.64 \cos(26.6)}$$

$d_{50} = 1.13$ ft $\longrightarrow$ Class IV

C. Layout Specifics

Step 1. Determine Wave runup

$R_u = 1.6 H_s (r \xi)$

$= (1.6)(1.6)(0.55)(1.5)$

$R_u = 2.1$ ft

- Check upper limit of $R_u = 3.2(r H_s)$

$= (3.2)(0.55)(1.6)$

$R_u = 2.82$

- Therefore use $R_u = 2.1$ ft

Step 2. Determine verical height of riprap above the toe of slope

Vertical height = (Still water depth)	3.8
+ (Wave height)	+ 1.6
+ (Runup)	+ 2.1
+ (Freeboard)	+ 2.0
	9.5'

Step 3. Determine min thickness of riprap

Selected class → Class IV nominal d_{50} = (15 in) 1.25 ft

- minimum thickness of riprap layer

$t_{min} = 2(d_{50})$ or d_{100}, whichever is greater

✓ $2(1.25) = 2.5$ ft $= d_{100} = 30$ in

Project: SFCJPA FLOOD IMPROVEMENTS	Computed: DJH	Date: 6/12/2012	
Subject: SAN FRANCISQUITO CREEK	Checked: EAR	Date: 6/13/12	
Task: WAVE RUNUP ANALYSIS	Page: 19b	of:	
Job #: 130806 (DEPT 244)	No:		

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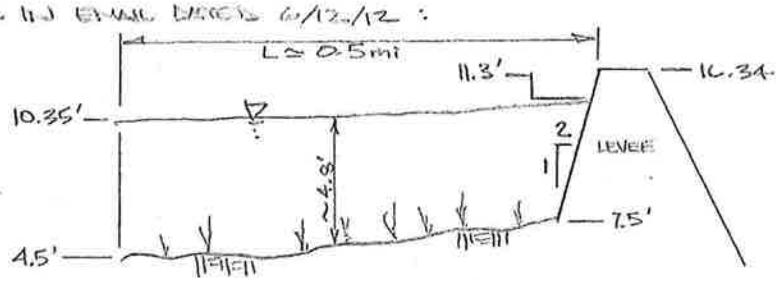
TASK

REPEAT CALCULATION SHOWN ON PAGE 18 USING MODIFIED PROFILE AS PROVIDED BY LANCE JONES IN EMAIL DATED 6/12/12 :



ASSUME $d_{AVG} \simeq 10.8 - 6 = 4.8'$

WAVES : SEE SHEET 21 b

$H_{mo} = 1.6'$

$T_p = 1.9$ SEC , $T_m \simeq \frac{T_p}{1.1} = \frac{1.9}{1.1} = 1.75$ 1.73

AT TOE OF LEVEE $h = 11.3 - 7.5 = 3.8'$

$\gamma_{br} = 0.55$ (SEE SHEET 1)

$H_b = 0.55(3.8) = 2.1'$, WAVE IS FETCH-LIMITED

H_{mo} CONTROLS

SURF SIMILARITY PARAMETER :

$$\xi_{om} = \frac{1/2}{\sqrt{1.6 \Big/ \left[\frac{32.2}{2\pi}(1.75)^2\right]}} = 1.6$$

1.5 w/ $T_m = 1.73$

RUNUP :

$R_{2\%,GRASS} = (1.6)(1.77)(1.0)(1.0)(0.90)(1.0)(1.6) = \boxed{4.1' \text{ (GRASS)}}$ EL = 11.3 + 4.1 = 15.4'

3.8' w/ $\xi_{om} = 1.5$ 15.1' w/ 3.8'

$R_{2\%,RIPRAP} = (1.6)(1.77)(0.55)(1.0)(0.90)(1.0)(1.6) = \boxed{2.2' \text{ (RIPRAP)}}$ EL = 11.3 + 2.2 = 13.5'

2.1' w/ $\xi_{om} = 1.5$ 13.4' w/ 2.1'

6/12/2012 ACES WAVE CALC — MODIFIED CONDITION (REF P. 19b) 21b

Case: Avg Depth - Marsh

Windspeed Adjustment and Wave Growth

Breaking criteria 0.780

Item	Value	Units
El of Observed Wind (Zobs)	33.00	feet
Observed Wind Speed (Uobs)	63.00	mph
Air Sea Temp. Diff. (dT)	0.00	deg F
Dur of Observed Wind (DurO)	10.00	min
Dur of Final Wind (DurF)	10.00	min
Lat. of Observation (LAT)	37.50	deg

Wind Obs Type	Wind Fetch Options
Overwater	Shallow openwater

Results

Item	Value	Units
Wind Fetch Length (F)	0.50	MILES
Avg Fetch Depth (d)	4.80	feet
Eq Neutral Wind Speed (Ue)	56.68	mph
Adjusted Wind Speed (Ua)	88.69	mph
Wave Height (Hmo)	1.56	feet
Wave Period (Tp)	1.85	sec

Wave Growth: Shallow

Attachment K

Vertical Drop Hand Calculations

Project: SF Creek		Computed: EKM	Date: 7/31/12
Subject: Bed Scour for Drop		Checked:	Date:
Task: Structures		Page: 1	of: 1
Job #:		No:	

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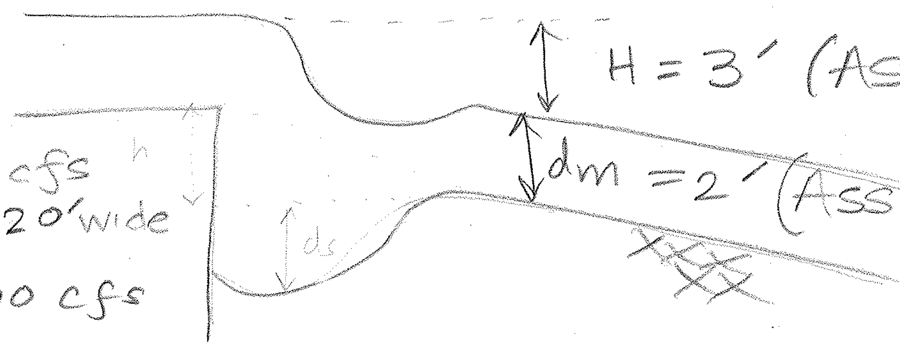
Bed Scour for Vertical Drop Structures
Using HEC-23 - DG 3



$H = 3'$ (Assume)

$d_m = 2'$ (Assume)

$q_1 = 100$ cfs per 20' wide

$q_2 = 500$ cfs

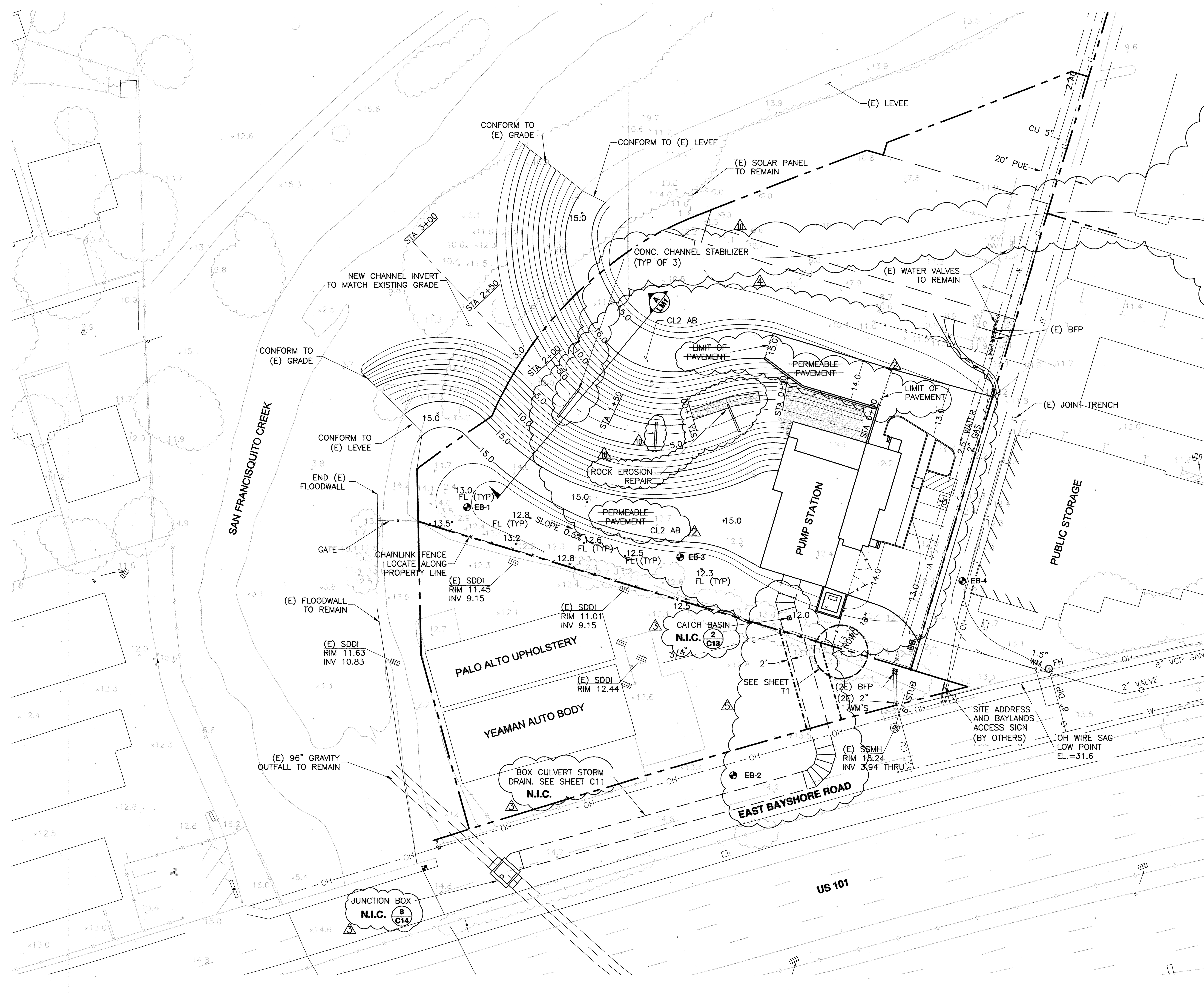
Trial 1: (100cfs) $d_s = k_u H_t^{0.225} q^{0.54} - d_m$

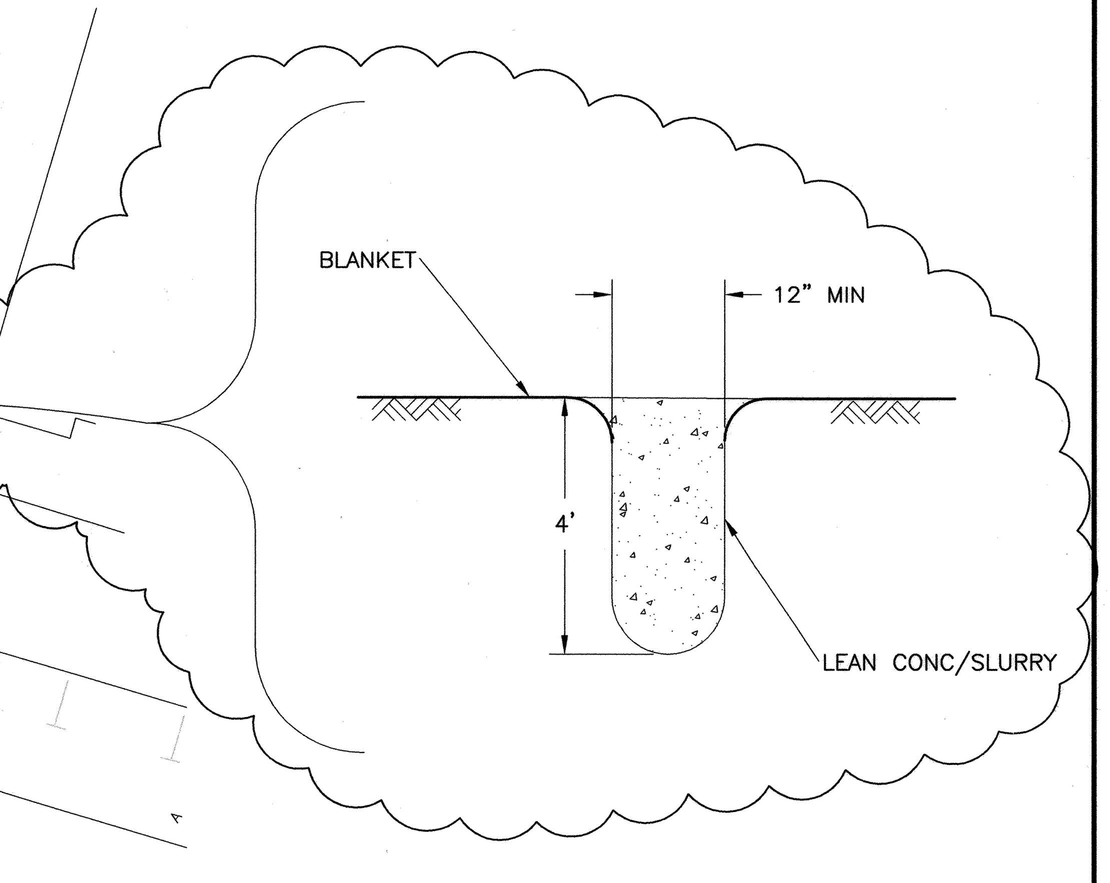
$$d_s = 1.32\,(3')^{0.225}\left(\frac{100\ \frac{ft^3}{s}}{20\ ft}\right)^{0.54} - 2\ ft$$

$$\boxed{d_s = 2.0\ ft}$$

Trial 2: (500cfs) $d_s = 1.32\,(3')^{0.225}\left(\dfrac{500\ cfs}{20\ ft}\right)^{0.54} - 2\ ft$

$$\boxed{d_s = 7.6\ ft}$$





LEGEND

EB-1 LOCATION OF EXPLORATORY BORING (SEE GEOTECHNICAL REPORT)

SCVWD BENCHMARK BM-362 (BRASS DISK) EL 15.07 FEET NGVD, PROJECT DATUM

RECORD DRAWINGS BASED ON AS-BUILT INFORMATION PROVIDED BY PROJECT CONTRACTOR

30 0 30 60

SCALE: 1 INCH = 30 FEET

NO.	DATE	REVISIONS	BY	CHK'D	APR
6	10/01/10	RECORD DRAWINGS			
5	12/14/07	CULVERT REALIGNMENT	EDS	CDA	
4	12/14/07	STATION FIELD ADJUSTMENT	EDS	CDA	
3	9/12/07	C.O. #1	NJL	CDA	
2	8/21/07	SCVWD PERMIT	NJL	CDA	

REGISTERED PROFESSIONAL ENGINEER CHARLES D. ANDERSON No. 43776 Exp. 6-30-07 CIVIL STATE OF CALIFORNIA

SD-06102

SAN FRANCISQUITO CREEK STORM WATER PUMP STATION

CITY OF PALO ALTO INCORPORATED CALIFORNIA

APPROVED FOR THE CITY OF PALO ALTO:

Joe Teresi

PROJECT MANAGER

11/10/10

DATE

Schaaf & Wheeler
CONSULTING CIVIL ENGINEERS
100 N. WINCHESTER BLVD, STE. 200
SANTA CLARA, CA 95050
(408) 246-4848

	BY	DATE
DRAWN	NJL	5/25/07
CHECKED	LMC	5/25/07
REVIEWED	CDA	5/25/07

SITE IMPROVEMENT PLAN

CITY OF PALO ALTO CALIFORNIA

SHEET 5 OF 74 SHEETS

Scale : AS SHOWN

VERIFY SCALE

BAR IS ONE INCH ON ORIGINAL DRAWING

0 1"

IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY.

SHEET NO.

C5

DWG. NO. SHT. 5 OF 75 SHTS. FILE NO. 102-0603-005

J:\JOBS\PALO01-SanFrancisquito\Drawings\100% PLANS\RECORD DRAWINGS\Sheet-5R4.dwg, 10/27/2010 4:24:11 PM, 1:1